HIGHWAY SAFETY MANUAL

PART A PREFACE TO THE FIRST EDITION

A.1. Purpose of the HSM A-1
A.2. The Need For The HSM A-1
A.3. The History of The First Edition of the HSM A-1
A.4. Considerations and Cautions When Using the HSM A-2
A.5. Future Editions of the HSM A-3

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PART A PREFACE TO THE HSM

A.1. PURPOSE OF THE HSM

The Highway Safety Manual (HSM) is a resource that provides safety knowledge and tools in a useful form to facilitate improved decision-making based on safety performance. The focus of the HSM is to provide quantitative information for decision making. The HSM assembles currently available information and methodologies on measuring, estimating and evaluating roadways in terms of crash frequency (number of crashes per year) and crash severity (level of injuries due to crashes). The HSM presents tools and methodologies for consideration of "safety" across the range of highway activities: planning, programming, project development, construction, operations and maintenance. The purpose is to convey present knowledge regarding highway safety information for use by a broad array of transportation professionals.

A.2. THE NEED FOR THE HSM

Prior to this edition of the HSM, transportation professionals did not have a single national resource for quantitative information about crash analysis and evaluation. The HSM begins to fill this gap, providing transportation professionals with current knowledge, techniques and methodologies to: estimate future crash frequency and severity and identify and evaluate options to reduce crash frequency and severity.

In addition to using descriptive methods in better ways, the HSM permits use of predictive methodologies that improve and expand the use of crash estimation methods to new and alternative design or conditions in past or future periods. The more statistically-rigorous predictive methods in the HSM reduce the vulnerability of historical crash based methods to random variations of crash data and provides a means to estimate crashes based on geometry, operating characteristics, and traffic volumes. These techniques provide an opportunity to: 1) improve the reliability of common activities, such as screening a network for sites at which to reduce crashes and 2) expand analysis to include assessments of new or alternative geometric and operational characteristics.

A.3. THE HISTORY OF THE FIRST EDITION OF THE HSM

A special conference session was held at the annual meeting of the Transportation Research Board (TRB) in January 1999 on the subject of predicting highway safety impacts of highway design and operation. The session participants concluded that one reason for a lack of quantitative safety emphasis in decision-making is the absence of a single authoritative document to use for quantitatively estimating "safety". In December of 1999, a workshop was held under sponsorship of eight TRB committees, and funded by FHWA, for the purpose of determining the need for, nature of, and feasibility of producing a Highway Safety Manual. An initial outline and plan for a HSM was produced. This led to the formation of a TRB Task Force for the Development of a Highway Safety Manual in May of 2000. It was under the direction of this Task Force of volunteers that this edition was produced. The Task Force formed several sub-committees to oversee various research and development aspects of the task. They also employed independent review groups to assess research results before proceeding with final preparation of materials. The majority of the research and development was funded by the NCHRP, with significant supplementary funding and research support provided by the FHWA.

Finally, AASHTO formed a HSM Task Force in TBA to monitor and participate in the final stages of the development of this edition.

A.4. CONSIDERATIONS AND CAUTIONS WHEN USING THE HSM

The HSM provides analytical tools based upon accepted knowledge, methods, and processes, in a form that is usable by transportation professionals.

The HSM will be used by individuals with a variety of professional and technical backgrounds, including engineering, planning, field operations, enforcement, and education. They will come to the HSM with different levels of understanding of the fundamentals of roadway safety. *Chapter 1 Introduction and Overview* provides key information and the context for understanding how to apply and integrate safety analysis related to the common activities within highway planning, design, and operations. The HSM includes traditional "safety" analysis techniques and also applies recent developments in crash estimation and evaluation methodologies. A majority of the analytical techniques are new; it is important to fully understand the material presented in *Chapter 2 Human Factors* and *Chapter 3 Fundamentals*, to understand reasons for development and use of these techniques.

Because the HSM does not account for jurisdiction-specific differences, it contains calibration techniques to modify tools for local use. This is necessary because of differences in factors, such as driver populations, local roadway and roadside conditions, traffic composition, typical geometrics, and traffic control measures. There are also variations in how each state or jurisdiction reports crashes and manages crash data. *Chapter 3 Fundamentals* discusses this topic and others related to the reliability of crash data. Calibration does not make the crash data uniform across states. Similarly, applying the HSM outside the United States and Canada should be done with caution. The models and research findings presented in this document may not be applicable in other countries as the roadway systems, driver training and behavior, and crash frequencies and severity patterns may be widely different. At a minimum, techniques presented in the HSM should be properly calibrated.

The HSM is not a legal standard of care as to the information contained herein. Instead, the HSM provides analytical tools and techniques for quantifying the potential effects of decisions made in planning, design, operations, and maintenance. There is no such thing as absolute "safety." There is risk in all highway transportation. A universal objective is to reduce the number and severity of crashes within the limits of available resources, science, technology, and legislatively mandated priorities. The information in the HSM is provided to assist agencies in their effort to integrate safety into their decision-making processes. The HSM is not intended to be a substitute for the exercise of sound engineering judgment. No standard of conduct or any duty toward the public or any person shall be created or imposed by the publication and use or nonuse of the HSM.

As a resource, the HSM does not supersede publications such as the Manual on Uniform Traffic Control Devices (MUTCD), American Association of State Highway Transportation Official's (AASHTO) "Green Book" *A Policy on Geometric Design of Highways and Streets*, or other AASHTO and agency guidelines, manuals, and policies. If conflicts arise between these publications and the HSM, the previously established publications should be given the weight they would otherwise be entitled, if in accordance with sound engineering judgment. The HSM may provide needed justification for an exception from previously established publications.

A.5. FUTURE EDITIONS OF THE HSM

This first edition of the HSM provides the most current and accepted knowledge and practices relating to roadway safety management. The TRB and AASHTO HSM Task Forces recognize that knowledge and methods of analysis are evolving and improving with new research and lessons learned in practice.

The evolution in professional practice and knowledge will be influenced by this first edition of the HSM because it introduces new methods, techniques, and information to transportation professionals. The knowledge-base will also continue to grow and to enhance transportation professionals' understanding of how decisions related to planning, design, operations, and maintenance affect crash frequency and severity. The transportation profession will continue to take the opportunity to learn more about the relationships between crash occurrences on various types of facilities and the corresponding geometry and operational characteristics of those facilities that may affect crash frequency and severity. This will be facilitated as agencies improve the processes used to collect and maintain data for: crashes, roadway geometry, traffic volumes, land uses, and many other useful data to assess the roadway environment and context in which crashes are occurring. These and/or other potential enhancements in analysis techniques and knowledge will be reflected in future editions of the HSM.

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PART A— INTRODUCTION AND FUNDAMENTALS

CHAPTER 1—INTRODUCTION AND OVERVIEW

1.1. Purpose and Intended Audience....1-1
1.2. Advancement in Safety Knowledge....1-1
1.3. Applications....1-2
1.4. Scope and Organization....1-3
1.4.1. Relationship Among Parts of the HSM....1-5
1.4.2. Activities Beyond the Scope of the HSM....1-6
1.5. Relating the HSM to the Project Development Process....1-6
1.5.1. Defining the Project Development Process....1-6
1.5.2. Connecting the HSM to the Project Development Process....1-7
1.6. Relating Activities and Projects to the HSM....1-9
1.7. Summary....1-11
1.8. References....1-12

EXHIBITS

Exhibit 1-1: Organization of the Highway Safety Manual 1-5
Exhibit 1-2: Relating the Project Development Process to the HSM 1-8
Exhibit 1-3: General Project Types and Activities and the HSM 1-10

CHAPTER 1 – INTRODUCTION AND OVERVIEW

1.1. PURPOSE AND INTENDED AUDIENCE

The Highway Safety Manual (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. There is no such thing as absolute safety. There is risk in all highway transportation. A universal objective is to reduce the number and severity of crashes within the limits of available resources, science, and technology, while meeting legislatively mandated priorities. The information in the HSM is provided to assist agencies in their effort to integrate safety into their decision-making processes. Specifically, the HSM is written for practitioners at the state, county, metropolitan planning organization (MPO), or local level. The HSM's intended users have an understanding of the transportation safety field through experience, education, or both. This knowledge base includes:

The Highway Safety Manual (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.

- Familiarity with the general principles and practice of transportation safety;
- Familiarity with basic statistical procedures and interpretation of results; and,
- Suitable competence to exercise sound traffic safety and operational engineering judgment.

The users and professionals described above include, but are not limited to, transportation planners, highway designers, traffic engineers, and other transportation professionals who make discretionary road planning, design and operational decisions. The HSM is intended to be a resource document that is used nationwide to help transportation professionals conduct safety analyses in a technically sound and consistent manner thereby improving decisions made based on safety performance.

Documentation used, developed, compiled or collected for analyses conducted in connection with the HSM may be protected under Federal law (23 USC 409). The HSM is neither intended to be, nor does it establish, a legal standard of care for users or professionals as to the information contained herein. No standard of conduct or any duty toward the public or any person shall be created or imposed by the publication and use or nonuse of the HSM.

The HSM is not a legal standard of care for users.

The HSM does not supersede publications such as the USDOT FHWA's Manual on Uniform Traffic Control Devices (MUTCD); Association of American State Highway Transportation Officials' (AASHTO) "Green Book" titled *A Policy on Geometric Design of Highways and Streets*; or other AASHTO and agency guidelines, manuals and policies. If conflicts arise between these publications and the HSM, the previously established publications should be given the weight they would otherwise be entitled, if in accordance with sound engineering judgment. The HSM may provide needed justification for an exception from previously established publications.

The HSM does not supersede existing publications.

1.2. ADVANCEMENT IN SAFETY KNOWLEDGE

The new techniques and knowledge in the HSM reflect the evolution in safety analysis from descriptive methods to quantitative, predictive analyses (the gray box below further explains the differences between descriptive and predictive method). Information throughout the HSM highlights the strengths and limitations of the methods presented. While these predictive analyses are quantitatively and statistically valid, they do not exactly predict a certain outcome at a particular location. Moreover, they can not be applied without the exercise of sound engineering judgment.

Descriptive Analyses and Quantitative Predictive Analyses

What are descriptive analyses?
Traditional descriptive analyses includes methods such as frequency, crash rate, and equivalent property damage only (EPDO), which summarize in different forms the history of crash occurrence, type and/or severity at a site.

What are quantitative predictive analyses?
Quantitative predictive analyses are used to calculate an expected number and severity of crashes at sites with similar geometric and operational characteristics for existing conditions, future conditions and/or roadway design alternatives.

What is the difference?
Descriptive analyses focus on summarizing and quantifying information about crashes that have occurred at a site (i.e. summarizing historic crash data in different forms). Predictive analyses focus on estimating the expected average number and severity of crashes at sites with similar geometric and operational characteristics. The expected and predicted number of crashes by severity can be used for comparisons among different design alternatives.

Section 1.3 provides an overview of the applications of the HSM.

1.3. APPLICATIONS

The HSM can be used to:

- Identify sites with the most potential for crash frequency or severity reduction;
- Identify factors contributing to crashes and associated potential countermeasures to address these issues;
- Conduct economic appraisals of improvements and prioritize projects;
- 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; and
- Estimate potential effects on crash frequency and severity of planning, design, operations, and policy decisions.

These applications are used to consider projects and activities related not only to safety but also those intended to improve other aspects of the roadway, such as capacity, pedestrian amenities and transit service. The HSM provides an opportunity to consider safety quantitatively along with other typical transportation performance measures.

1.4. SCOPE AND ORGANIZATION

The emphasis of the HSM is on quantifying the safety effects of decisions in planning, design, operations, and maintenance through the use of analytical methods. The first edition does not address issues such as driver education, law enforcement, and vehicle safety, although it is recognized that these are important considerations within the broad topic of improving highway safety.

The HSM is organized into the following four parts:

- Part A - Introduction, Human Factors, and Fundamentals
- Part B - Roadway Safety Management Process
- Part C - Predictive Method
- Part D - Accident Modification Factors

Part A Introduction, Human Factors and Fundamentals

Part A Chapter 2 Human Factors and Chapter 3 Fundamentals provide basic information needed to understand how to apply the HSM.

Part A describes the purpose and scope of the HSM. It explains the relationship of the HSM to planning, design, operations, and maintenance activities. *Part A* also presents an overview of human factors principles for road safety, and fundamentals of the processes and tools described in the HSM. Content in *Chapter 3 Fundamentals* provides background information needed prior to applying the predictive method, accident modification factors, or evaluation methods provided in the HSM. This content is the basis for the material in *Parts B, C*, and *D*. The chapters in *Part A* are:

- Chapter 1 – Introduction and Overview
- Chapter 2 – Human Factors
- Chapter 3 - Fundamentals

Part B Roadway Safety Management Process

Part B (Chapters 4 through 9) presents the roadway safety management process including tools for conducting network screening analyses.

Part B presents the steps that can be used to monitor, and reduce crash frequency and severity on existing roadway networks. It includes methods useful for identifying improvement sites, diagnosis, countermeasure selection, economic appraisal, project prioritization and effectiveness evaluation. The chapters in *Part B* are:

- Chapter 4 – Network Screening
- Chapter 5 – Diagnosis
- Chapter 6 – Select Countermeasures
- Chapter 7 – Economic Appraisal
- Chapter 8 – Prioritize Projects
- Chapter 9 – Safety Effectiveness Evaluation

Part C (Chapters 10 through 12) presents the predictive method for estimating expected average crashes on two-lane rural highways, multilane rural highways, and urban and suburban arterials.

Part C Predictive Method

Part C of the HSM provides a predictive method for estimating expected average crash frequency of a network, facility or individual site. The estimate can be made for existing conditions, alternative conditions, or proposed new roadways. The predictive method is applied to a given time period, traffic volume, and constant geometric design characteristics of the roadway. The *Part C* predictive method is most applicable when developing and assessing multiple solutions for a specific location. For example, a roadway project that is considering varying cross-section alternatives could use *Part C* to assess the expected average crash frequency of each alternative. *Part C* can also be used as a source for safety performance functions (SPFs).

The chapters in *Part C* provide the prediction method for the following facility types;

- Chapter 10 – Rural Two-Lane Roads (Segments and Intersections)
- Chapter 11 – Rural Multilane Highways (Segments and Intersections)
- Chapter 12 – Urban and Suburban Arterials (Segments and Intersections)

Future editions of the HSM will expand the material included in *Part C* to include information applicable to additional types of roadway facilities.

Part D (Chapters 13 through 17) provides AMFs related roadway segments, intersections, interchanges, special facilities, and roadway networks.

Part D Accident Modification Factors

Part D summarizes the effects of various treatments such as geometric and operational modifications at a site. Some of the effects are quantified as accident modification factors (AMFs). AMFs quantify the change in expected average crash frequency as a result of modifications to a site.

The AMFs in *Part D Accident Modification Factors* can be used as a resource for methods and calculations presented in *Chapter 6 Select Countermeasures, Chapter 7 Economic Appraisal,* and chapters in *Part C Predictive Method*. Some *Part D* AMFs are used in the *Part C Predictive Method*. However, not all AMFs presented in *Part D* apply to the predictive models in *Part C*. AMFs in general can be used to test alternative design options.

The chapters in *Part D* are organized by site type as:

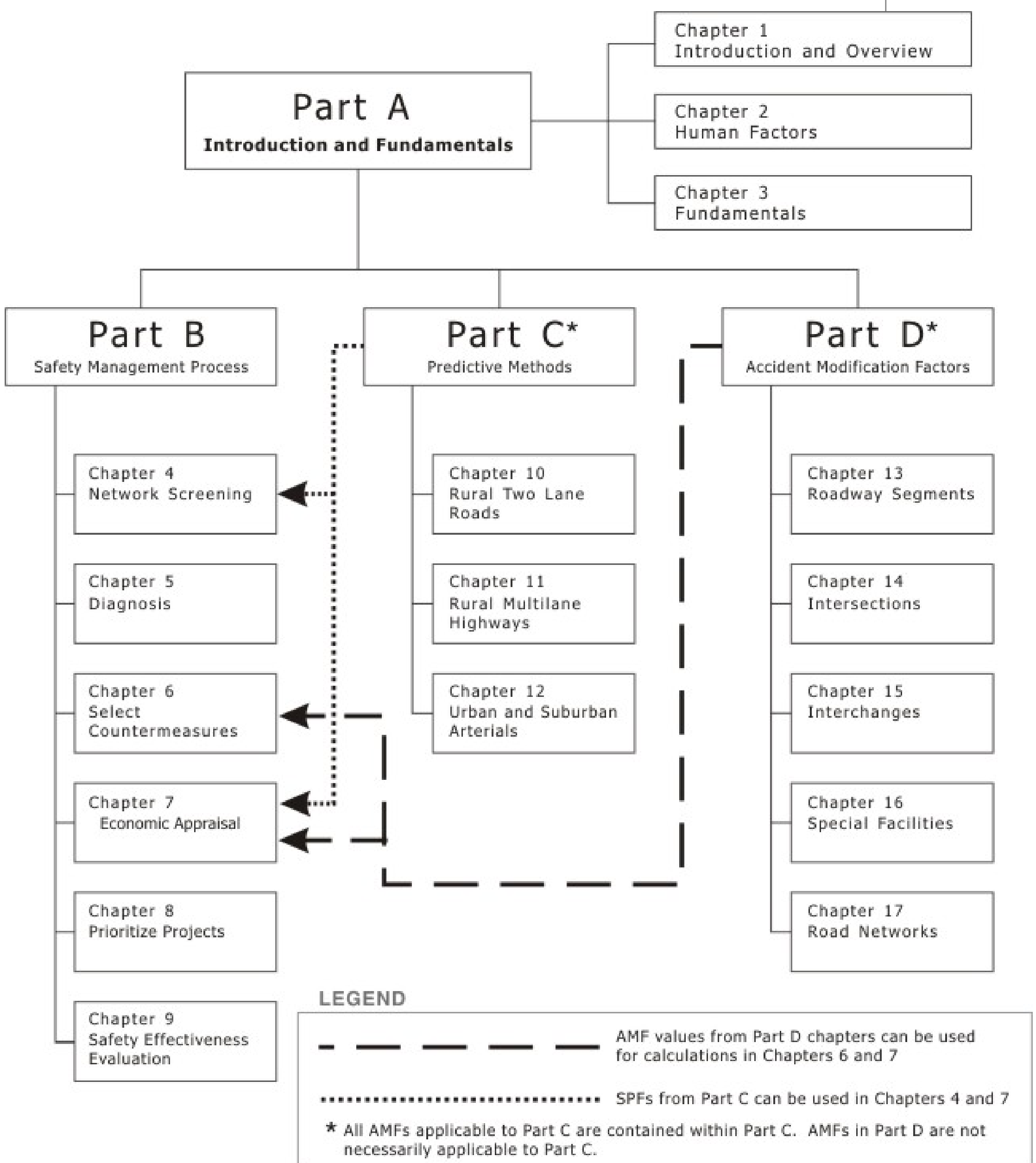
- Chapter 13 – Roadway Segments
- Chapter 14 – Intersections
- Chapter 15 – Interchanges
- Chapter 16 – Special Facilities
- Chapter 17 – Road Networks

Each chapter includes exhibits summarizing the treatments and available AMFs. The appendix to each chapter contains the treatments for which AMFs are not available but general trends are known (e.g. increase or decrease in crash occurrence), and the treatments whose crash effects are unknown. Similar to *Part C*, it is envisioned that the material included in *Part D* will be expanded in future editions of the HSM.

1.4.1. Relationship Among Parts of the HSM

Exhibit 1-1 illustrates the relationship among the four parts of the HSM and how the associated chapters within each part relate to one another.

Exhibit 1-1: Organization of the Highway Safety Manual



Part A is the foundation for the remaining information in the HSM. This part presents fundamental knowledge useful throughout the manual. *Parts B, C,* and *D* can be used in any order following *Part A* depending on the purpose of the project or analysis. The chapters within each part can also be used in an order most applicable to a specific project rather than working through each chapter in order. The dashed line connecting *Part C* with *Chapters 4* and *7* denotes that the safety performance functions in *Part C* can be calibrated and applied in *Chapters 4* and *7*. The dashed line connecting *Part D* with *Chapters 6* and *7* denotes that the accident modification factors in *Part D* are used for calculations in *Chapters 6* and *7*.

1.4.2. Activities Beyond the Scope of the HSM

The HSM focuses on engineering analyses and treatments. Crashes may also be reduced through enforcement and education programs.

The procedures in the HSM support engineering analysis and decision making to reduce crash frequency and/or severity on a roadway network. In general, crash reduction may also be achieved by considering:

- Enforcement
- Education for road users
- Improving incident response and emergency medical services (EMS)
- Improving vehicle safety performance

Enforcement of traffic laws, compliance with driving under the influence laws, the proper use of passenger restraints, driver education and other safety-related legislative efforts–along with infrastructure improvements–contribute to a roadway's safety performance. Although education, enforcement, and emergency medical services are not addressed in the HSM, these are also important factors in reducing crashes and crash severity.

1.5. RELATING THE HSM TO THE PROJECT DEVELOPMENT PROCESS

The following defines a generalized project development process for the purpose of explaining the connection between planning, design, construction, operations, and maintenance activities and the HSM. This section further provides example applications of the HSM within the generalized project development process illustrating how to integrate the HSM into various types of projects and activities.

1.5.1. Defining the Project Development Process

For the purposes of the HSM, the project development process consists of:
- System Planning
- Project Planning
- Preliminary Design, Final Design, and Construction
- Operations and Maintenance

The phrase and concept of the "project development process" was framed and is documented by AASHTO in *A Guide for Achieving Flexibility in Highway Design* and the Federal Highway Administration's (FHWA) *Flexibility in Highway Design*.[1,2] The process was developed as a means to discuss the typical stages of a project from planning to post-construction operations and maintenance activities. It is applicable to all projects including those influenced by other processes, policies, and/or legislation (e.g National Environmental Policy Act (NEPA), Context Sensitive Solutions).

There are minor differences in how AASHTO and FHWA have documented the process; however, for the purpose of the HSM a generalized project development process is:

- System Planning
    - Assess the system needs and identify projects/studies that address these needs.
    - Program projects based on the system needs and available funding.
- Project Planning

    - Within a specific project, identify project issues and alternative solutions to address those issues.
    - Assess the alternatives based on safety, traffic operations, environmental impacts, right-of-way impacts, cost and any other project specific performance measures.
    - Determine preferred alternative.
- Preliminary Design, Final Design, and Construction
    - Develop preliminary and final design plans for the preferred alternative.
    - Evaluate how the project-specific performance measures are impacted by design changes.
    - Construct final design.
- Operations and Maintenance
    - Monitor existing operations with the goal of maintaining acceptable conditions balancing safety, mobility and access.
    - Modify the existing roadway network as necessary to maintain and improve operations.
    - Evaluate the effectiveness of improvements that have been implemented.

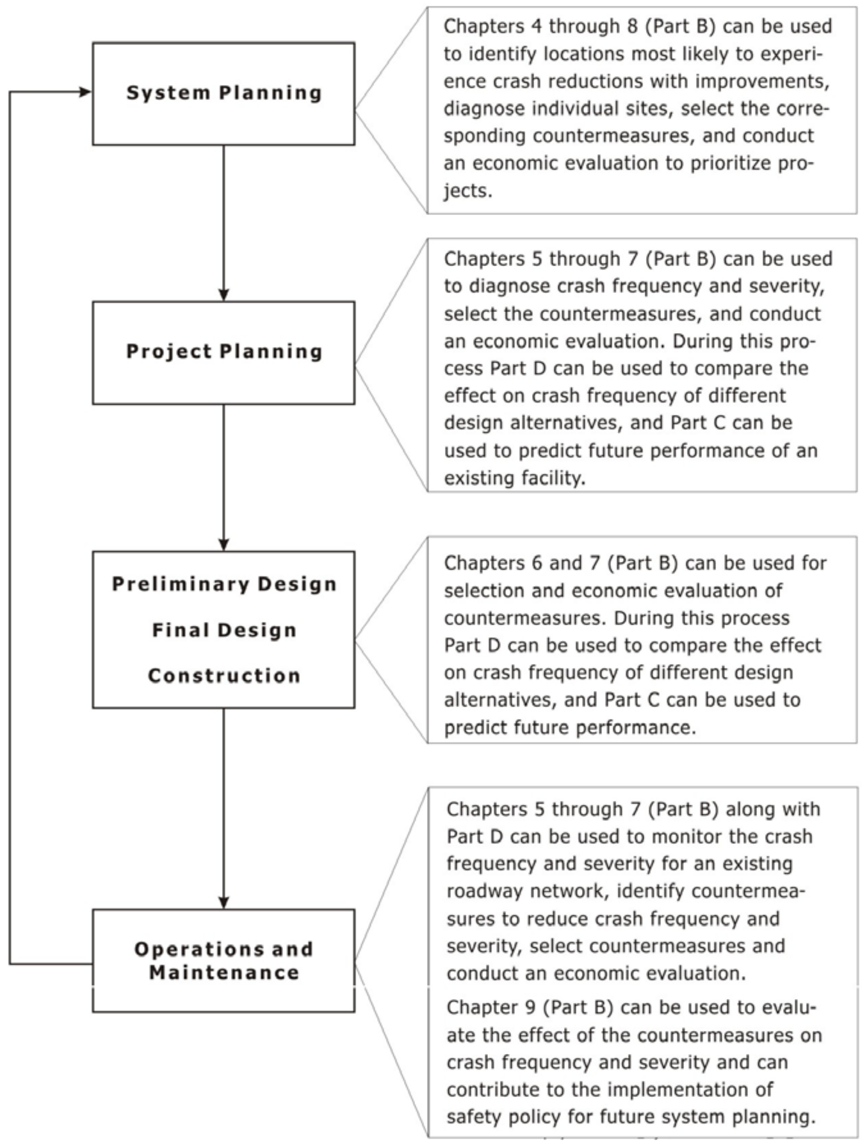
Other processes, policies, and/or legislation that influence a project's form and scope often include activities that fall within this generalized process.

1.5.2. Connecting the HSM to the Project Development Process

Exhibit 1-2 illustrates how planning, design, construction, operations and maintenance activities relate to the HSM. Specific information about how to apply individual chapters in the HSM is provided in the *Parts B*, *C*, and *D Introduction and Applications Guidance*. The left side of the exhibit depicts the overall project development process. The right side describes how the HSM is used within each stage of the project development process. The text following Exhibit 1-2 further explains the relationship between the project development process and the HSM.

Sections 1.5.2 and 1.6 provide examples of how the HSM can support typical activities within the project development process.

Exhibit 1-2: Relating the Project Development Process to the HSM



System planning is the first stage of the project development process and it is the stage in which network infrastructure priorities are identified and assessed. This stage is an opportunity to identify system safety priorities and to integrate safety with other project types (e.g. corridor studies, streetscape enhancements). *Chapter 4 Network Screening* is used to identify sites most likely to benefit from safety improvements. *Chapter 5 Diagnosis* can be used to identify crash patterns to be targeted for improvement at each site. *Chapter 6 Select Countermeasures* can be used to identify the factors contributing to observed crash patterns and to select corresponding countermeasures. *Chapters 7 Economic Appraisal* and *Chapter 8 Prioritize Projects* are used to prioritize expenditures and ensure the largest crash reductions from improvements throughout the system.

During the **project planning** stage, project alternatives are developed and analyzed to enhance a specific performance measure or a set of performance measures, such as, capacity, multimodal amenities, transit service, and safety at a particular site. Each alternative is evaluated across multiple performance measures, which can include weighing project costs versus project benefits. These projects can include extensive redesign or design of new facilities (e.g. introducing a couplet system, altering the base number of lanes on an existing roadway, and other changes that would substantially change the operational characteristics of the site). The result of this stage is a preferred design alternative carried forward into preliminary design. *Chapters 5 Diagnosis* can be used to identify crash patterns to be targeted for improvement during project planning. *Chapter 6 Select Countermeasures* is used to identify the factors contributing to observed crash patterns and to evaluate countermeasures. *Chapters 7 Economic Appraisal* can be used to conduct an economic appraisal of countermeasures as part of the overall project costs. The chapters within *Part D* are a resource to compare the safety implications of different design alternatives, and the Chapters in *Part C* can be used to predict future safety performance of the alternatives

The **preliminary design, final design, and construction** stage of the project development process includes design iterations and reviews at 30-percent complete, 60-percent complete, 90-percent complete, and 100-percent complete design plans. Through the design reviews and iterations, there is a potential for modifications to the preferred design. As modifications to the preferred design are made, the potential crash effects of those changes can be assessed to confirm that the changes are consistent with the ultimate project goal and intent. *Chapter 6 Select Countermeasures* and *Chapters 7 Economic Appraisal* can be used during preliminary design to select countermeasures and conduct an economic appraisal of the design options. Chapters in *Parts C* and *D* are a resource to estimate crash frequencies for different design alternatives.

Activities related to **operations and maintenance** focus on evaluating existing roadway network performance; identifying opportunities for near-term improvements to the system; implementing improvements to the existing network; and evaluating the effectiveness of past projects. These activities can be conducted from a safety perspective using *Chapters 5 Diagnosis* to identify crash patterns at an existing location, and *Chapter 6 Select Countermeasures* and *Chapters 7 Economic Appraisal* to select and appraise countermeasures. Throughout this process *Part D* serves as a resource for AMFs. *Chapter 9 Safety Effectiveness Evaluation* provides methods to conduct a safety effectiveness evaluation of countermeasures. This can contribute to the implementation or modification of safety policy, and design criteria for future transportation system planning.

1.6. RELATING ACTIVITIES AND PROJECTS TO THE HSM

Examples of how to integrate the HSM into typical project types or activities required by state or federal legislation (e.g. Highway Safety Improvement Program - HSIP, Strategic Highway Safety Plan - SHSP) are summarized in Exhibit 1-3.

322 **Exhibit 1-3: General Project Types and Activities and the HSM**

Project Development Process Stage	Activity or Project Type	Opportunity to Apply the HSM
System Planning	Long-Range Transportation Plans	**Part B, Chapters 4-8** – Identify sites most likely to benefit from a safety improvement. This information could be used to identify projects for safety funding and opportunities to incorporate safety into previously funded projects or studies.
System Planning/Project Planning	Highway Safety Improvement Program (HSIP)	**Part B**, **Chapters 4-8** – Identify a state's top locations most likely to benefit from safety improvements. Identify crash patterns, contributing factors, and countermeasures most likely to reduce crashes. Evaluate the economic validity of individual projects and prioritize projects across a system.
System Planning/Project Planning	Corridor Study	**Part B Chapters 4-8** – Identify sites most likely to benefit from a safety improvement, diagnose crash patterns, evaluate countermeasures and economic implications, and identify project priorities. **Parts C and D** – Assess the safety performance of design alternatives related to change in roadway cross-section, alignment and intersection configuration or operations.
Project Planning/Preliminary Design	Context Sensitive Design/Solutions Projects (Includes Developing and Assessing Multiple Design Alternatives)	**Parts C and D** – Assess the safety performance of design alternatives based on their geometric and operational characteristics. The results of these methods can be used to help reach a preferred alternative that balances multiple performance measures.
Project Planning/Preliminary Design	Designing a New Network Connection or Facility	**Part B Chapters 5-7** – Diagnose expected average crash frequency for similar locations, consider countermeasures and conduct an economic evaluation of design alternatives. **Parts C and D** – Assess the safety performance of design alternatives related to change in roadway cross-section, alignment and intersection configuration or operations. This information can be used to select a preferred alternative that balances multiple performance measures.
Preliminary Design, Final Design/Operations and Maintenance	Widening an Existing Roadway	**Part C** – Assess the change in crashes that may be attributed to different design alternatives for widening an existing roadway. **Part D, Chapter 13** - Assess the change in crashes from changing roadway cross section.
Operations and Maintenance	Signal Timing or Phase Modifications	**Part D, Chapter 14** – Assess the effects that signal timing adjustments can have at individual intersections.
Operations and Maintenance	Adding Lanes to an Existing Intersection	**Part D, Chapter 14** – Assess the effects that modifying lane configurations can have on safety.
Operations and Maintenance	Developing an On-Street Parking Management Plan	**Part D, Chapter 13** – Assess the effects that the presence or absence of on-street parking has on the expected number of crashes for a roadway segment. It can also be used to assess the safety effects of different types of on-street parking.
System Planning/Operations and Maintenance	Traffic Impact Study	**Part B** – Identify sites most likely to benefit from a safety improvement and identify ways to improve safety as part of other mitigations. **Part D, Chapter 13 and 14** – Identify the effects that mitigations to roadway segments (Ch 13) and intersections (Ch 14) may have on safety.

1.7. SUMMARY

The HSM contains specific analysis procedures that facilitate integrating safety into roadway planning, design, operations and maintenance decisions based on crash frequency. The following parts and chapters of the HSM present information, processes and procedures that are tools to help improve safety decision-making and knowledge. The HSM consists of the four parts shown below:

- Part A provides an introduction to the HSM along with fundamental knowledge;
- Part B discusses the roadway safety improvement and evaluation process;
- Part C contains the predictive method for rural two-lane highways, rural multilane highways, and urban and suburban arterials; and
- Part D summarizes accident modification factors for planning, geometric, and operational elements.

Future editions of the HSM will continue to reflect the evolution in highway safety knowledge and analysis techniques being developed.

1.8. REFERENCES

1. AASHTO. *Achieving Flexibility in Highway Design*. American Association of State Highway and Transportation Officials, Washington, D.C., 2004
2. FHWA. *Flexibility in Highway Design*. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

PART A— INTRODUCTION AND FUNDAMENTALS

CHAPTER 2—HUMAN FACTORS

2.1. Introduction: The Role of Human Factors in Road Safety 2-1
2.2. Driving Task Model 2-1
2.3. Driver Characteristics and Limitations 2-2
2.3.1. Attention and Information Processing 2-2
2.3.2. Vision 2-4
2.3.3. Perception-Reaction Time 2-9
2.3.4. Speed Choice 2-11
2.4. Positive Guidance 2-13
2.5. Impacts of Road Design on the Driver 2-14
2.5.1. Intersections and Access Points 2-14
2.5.2. Interchanges 2-17
2.5.3. Divided, Controlled-Access Mainline 2-18
2.5.4. Undivided Roadways 2-19
2.6. Summary: Human Factors and the HSM 2-20
2.7. References 2-22

EXHIBITS

Exhibit 2-1: Driving Task Hierarchy 2-2
Exhibit 2-2: Example Scenarios of Driver Overload 2-3
Exhibit 2-3: Area of Accurate Vision in the Eye.. 2-6
Exhibit 2-4: Relative Visibility of Target Object as Viewed with Peripheral Vision 2-7
Exhibit 2-5: Relationship Between Viewing Distance and Image Size 2-8
Exhibit 2-6: Perceived Risk of an Accident and Speed..2-13

CHAPTER 2 HUMAN FACTORS

The purpose of this chapter is to introduce the core elements of human factors that affect the interaction of drivers and roadways. With an understanding of how drivers interact with the roadway, there is more potential for roadways to be designed and constructed in a manner that minimizes human error and associated crashes.

This chapter is intended to support the application of knowledge presented in *Parts B*, *C*, and *D*. It does not contain specific design guidance, as that is not the purpose of the Highway Safety Manual (HSM). For more detailed discussion of human factors and roadway elements, the reader is referred to NCHRP Report 600: Human Factors Guidelines for Road Systems.[6]

2.1. INTRODUCTION: THE ROLE OF HUMAN FACTORS IN ROAD SAFETY

The interdisciplinary study of human factors applies knowledge from the human sciences such as psychology, physiology, and kinesiology to the design of systems, tasks, and environments for effective and safe use. The goal of human factors is to reduce the probability and consequences of human error within systems, and associated injuries and fatalities, by designing with respect to human characteristics and limitations.

The goal of human factors is to reduce human error within systems, and associated injuries and fatalities, by designing with respect to human characteristics and limitations.

Drivers make frequent mistakes because of human physical, perceptual, and cognitive limitations. These errors may not result in crashes because drivers compensate for other drivers' errors or because the circumstances are forgiving (e.g., there is room to maneuver and avoid a crash). Near misses, or conflicts, are vastly more frequent than crashes. One study found a conflict-to-crash ratio of about 2,000 to 1 at urban intersections.[28]

In transportation, driver error is a significant contributing factor in most crashes.[41] For example, drivers can make errors of judgment concerning closing speed, gap acceptance, curve negotiation, and appropriate speeds to approach intersections. In-vehicle and roadway distractions, driver inattentiveness, and driver weariness can lead to errors. A driver can also be overloaded by the information processing required to carry out multiple tasks simultaneously, which may lead to error. To reduce their information load, drivers rely on a-priori knowledge, based on learned patterns of response; therefore, they are more likely to make mistakes when their expectations are not met. In addition to unintentional errors, drivers sometimes deliberately violate traffic control devices and laws.

Chapter 3, Section 3.2.4 provides a discussion of the interactions among drivers, vehicles, and roadway crashes.

2.2. DRIVING TASK MODEL

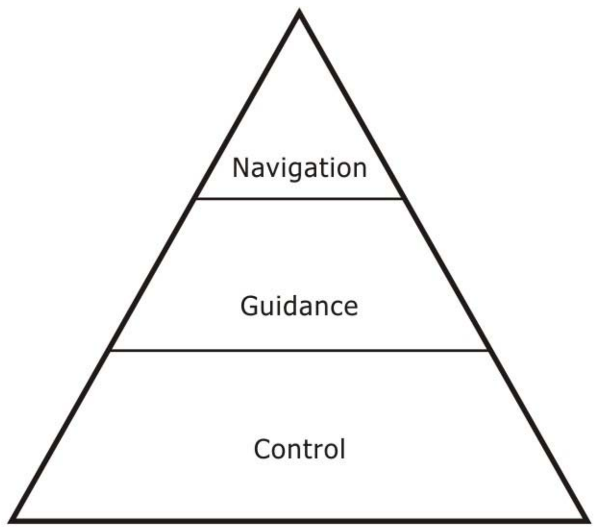
Driving comprises many sub-tasks, some of which must be performed simultaneously. The three major sub-tasks are:

- Control: Keeping the vehicle at a desired speed and heading within the lane;
- Guidance: Interacting with other vehicles (following, passing, merging, etc.) by maintaining a safe following distance and by following markings, traffic control signs, and signals; and,
- Navigation: Following a path from origin to destination by reading guide signs and using landmarks.[23]

Each of these major sub-tasks involves observing different information sources and various levels of decision-making. The relationship between the sub-tasks can be illustrated in a hierarchical form, as shown in Exhibit 2-1. The hierarchical relationship is based on the complexity and primacy of each subtask to the overall driving task. The navigation task is the most complex of the subtasks, while the control sub-task forms the basis for conducting the other driving tasks.

The driving task includes: control, guidance, and navigation.

Exhibit 2-1: Driving Task Hierarchy



Adapted from Alexander and Lunenfeld.(1)

A successful driving experience requires smooth integration of the three tasks, with driver attention being switched from one to another task as appropriate for the circumstances. This can be achieved when high workload in the sub-tasks of control, guidance, and navigation does not happen simultaneously.

2.3. DRIVER CHARACTERISTICS AND LIMITATIONS

This section outlines basic driver capabilities and limitations in performing the driving tasks which can influence safety. Topics include driver attention and information processing ability, vision capability, perception-response time, and speed choice.

2.3.1. Attention and Information Processing

Driver attention and ability to process information is limited. These limitations can create difficulties because driving requires the division of attention between control tasks, guidance tasks, and navigational tasks. While attention can be switched rapidly from one information source to another, drivers only attend well to one source at a time. For example, drivers can only extract a small proportion of the available information from the road scene. It has been estimated that more than one billion units of information, each equivalent to the answer to a single yes or no question, are directed at the sensory system in one second.(25) On average, humans are expected to consciously recognize only 16 units of information in one second.

To account for limited information processing capacity while driving, drivers subconsciously determine acceptable information loads they can manage. When drivers' acceptable incoming information load is exceeded, they tend to neglect other information based on level of importance. As with decision making of any sort, error is possible during this process. A driver may neglect a piece of information that turns out to be critical, while another less-important piece of information was retained.

Scenarios illustrating circumstances in which drivers might be overloaded with information are described in Exhibit 2-2. Each may increase the probability of driver error given human information processing limitations.

Overload of information or distractions can increase probability of driver error.

Exhibit 2-2: Example Scenarios of Driver Overload

Scenario	Example
High demands from more than one information source	Merging into a high-volume, high-speed freeway traffic stream from a high-speed interchange ramp
The need to make a complex decision quickly	Stop or go on a yellow signal close to the stop line
The need to take in large quantities of information at one time	An overhead sign with multiple panels, while driving in an unfamiliar place

As shown in Exhibit 2-2, traffic conditions and operational situations can overload the user in many ways. Roadway design considerations for reducing driver workload are:

- Presenting information in a consistent manner to maintain appropriate workload;
- Presenting information sequentially, rather than all at once, for each of the control, guidance, and navigation tasks; and,
- Providing clues to help drivers prioritize the most important information to assist them in reducing their workload by shedding extraneous tasks.

In addition to information processing limitations, drivers' attention is not fully within their conscious control. For drivers with some degree of experience, driving is a highly automated task. That is, driving can be, and often is, performed while the driver is engaged in thinking about other matters. Most drivers, especially on a familiar route, have experienced the phenomenon of becoming aware that they have not been paying attention during the last few miles of driving. The less demanding the driving task, the more likely it is that the driver's attention will wander, either through internal preoccupation or through engaging in non-driving tasks. Factors such as increased traffic congestion and increased societal pressure to be productive could also contribute to distracted drivers and inattention. Inattention may result in inadvertent movements out of the lane, or failure to detect a stop sign, a traffic signal, or a vehicle or pedestrian on a conflicting path at an intersection.

Driver Expectation

One way to accommodate for human information processing limitations is to design roadway environments in accordance with driver expectations. When drivers can rely on past experience to assist with control, guidance, or navigation tasks there is less to process because they only need to process new information. Drivers develop

Designing facilities consistent with driver expectations simplifies the driving task.

both long- and short-term expectancies. Examples of long-term expectancies that an unfamiliar driver will bring to a new section of roadway include:

- Upcoming freeway exits will be on the right-hand side of the road;
- When a minor and a major road cross, the stop control will be on the road that appears to be the minor road;
- When approaching an intersection, drivers must be in the left lane to make a left turn at the cross street; and,
- A continuous through lane (on a freeway or arterial) will not end at an interchange or intersection junction.

Examples of short-term expectancies include:

- After driving a few miles on a gently winding roadway, upcoming curves will continue to be gentle;
- After traveling at a relatively high speed for some considerable distance, drivers expect the road ahead will be designed to accommodate the same speed; and,
- After driving at a consistent speed on well-timed, coordinated signalized arterial corridors drivers may not anticipate a location that operates at a different cycle length.

2.3.2. Vision

The majority of driver information is visual information.

Approximately 90 percent of the information that drivers use is visual.[17] While visual acuity is the most familiar aspect of vision related to driving, numerous other aspects are equally important. The following aspects of driver vision are described in this section:

- **Visual Acuity** – The ability to see details at a distance;
- **Contrast Sensitivity** – The ability to detect slight differences in luminance (brightness of light) between an object and its background;
- **Peripheral Vision** – The ability to detect objects that are outside of the area of most accurate vision within the eye;
- **Movement in Depth** – The ability to estimate the speed of another vehicle by the rate of change of visual angle of the vehicle created at the eye ; and,
- **Visual Search –** The ability to search the rapidly changing road scene to collect road information.

Visual Acuity

Visual acuity determines how well drivers can see details at a distance. It is important for guidance and navigation tasks, which require reading signs and identifying potential objects ahead.

Under ideal conditions, in daylight, with high contrast text (black on white), and unlimited time, a person with a visual acuity of 20/20, considered "normal vision," can just read letters that subtend an angle of 5 minutes of arc. A person with 20/40 vision needs letters that subtend twice this angle, or 10 minutes of arc. With respect to traffic signs, a person with 20/20 vision can just barely read letters that are 1 inch tall at 57 feet, and letters that are 2 inches tall at 114 feet and so on. A person with 20/40 vision would need letters of twice this height to read them at the same distances. Given that actual driving conditions often vary from the ideal conditions listed above and driver vision varies with age, driver acuity is often assumed to be less than 57 feet per inch of letter height for fonts used on highway guide signs.(24)

Contrast Sensitivity

Contrast sensitivity is often recognized as having a greater impact on crash occurrence than visual acuity. Contrast sensitivity is the ability to detect small differences in luminance (brightness of light) between an object and the background. The lower the luminance of the targeted object, the more contrast is required to see the object. The target object could be a curb, debris on the road, or a pedestrian.

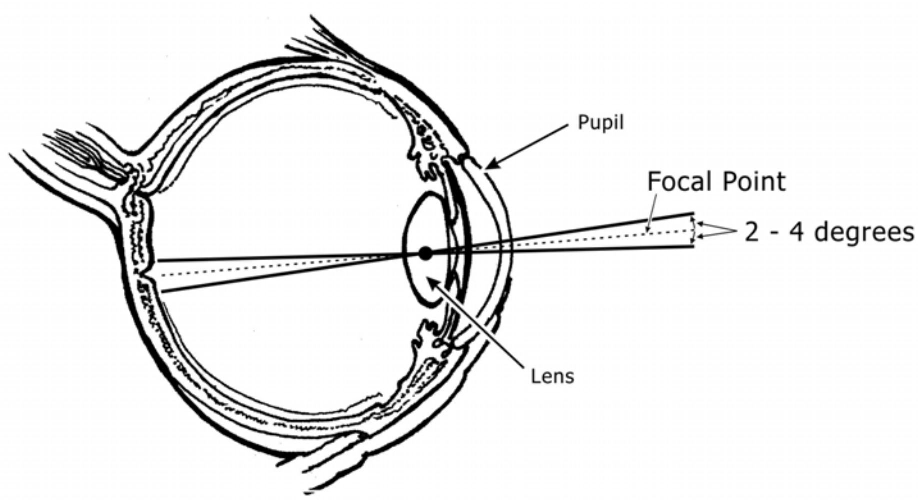
Good visual acuity does not necessarily imply good contrast sensitivity. For people with standard visual acuity of 20/20, the distance at which non-reflective objects are detected at night can vary by a factor of 5 to 1.(31) Drivers with normal vision but poor contrast sensitivity may have to get very close to a low-contrast target before detecting it. Experimental studies show that even alerted subjects can come as close as 30 feet before detecting a pedestrian in dark clothing standing on the left side of the road.(24) In general, pedestrians tend to overestimate their own visibility to drivers at night. On average, drivers see pedestrians at half the distance at which pedestrians think they can be seen.(3) This may result in pedestrians stepping out to cross a street while assuming that drivers have seen them, surprising drivers, and leading to a crash or near-miss event.

Key aspects of vision are acuity, contrast sensitivity, peripheral vision, movement in depth, and visual search.

Peripheral Vision

The visual field of human eyes is large: approximately 55 degrees above the horizontal, 70 degrees below the horizontal, 90 degrees to the left and 90 degrees to the right. However, only a small area of the visual field allows accurate vision. This area of accurate vision includes a cone of about two to four degrees from the focal point (see Exhibit 2-3). The lower-resolution visual field outside the area of accurate vision is referred to as peripheral vision. Although acuity is reduced, targets of interest can be detected in the low-resolution peripheral vision. Once detected, the eyes shift so that the target is seen using the area of the eye with the most accurate vision.

Exhibit 2-3: Area of Accurate Vision in the Eye

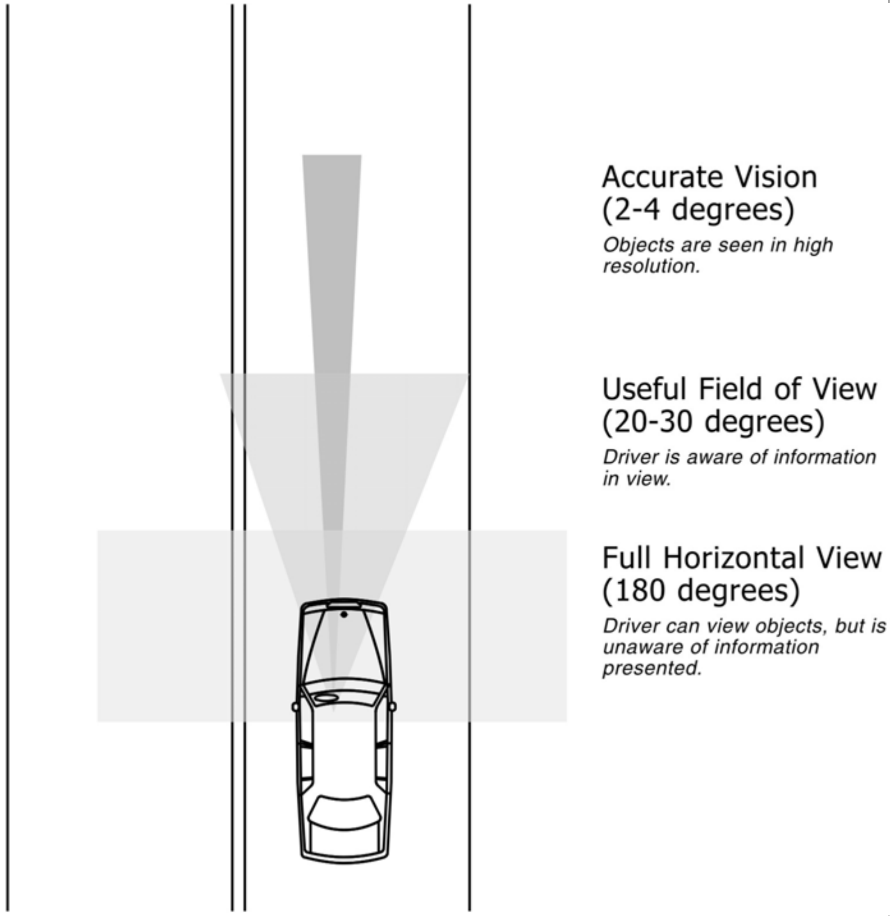


Targets that drivers need to detect in their peripheral vision include vehicles on an intersecting path, pedestrians, signs, and signals. In general, targets best detected by peripheral vision are objects that are closest to the focal point; that differ greatly from their backgrounds in terms of brightness, color, and texture; that are large; and that are moving. Studies show the majority of targets are noticed when located less than 10 to 15 degrees from the focal point and that even when targets are conspicuous, glances at angles over 30 degrees are rare.[8,39]

Target detection in peripheral vision is also dependent on demands placed on the driver. The more demanding the task, the narrower the "visual cone of awareness" or the "useful field of view," and the less likely the driver is to detect peripheral targets.

Exhibit 2-4 summarizes the driver's view and awareness of information as the field of view increases from the focal point. Targets are seen in high resolution within the central 2-4 degrees of the field of view. While carrying out the driving task, the driver is aware of information seen peripherally, within the central 20 to 30 degrees. The driver can physically see information over a 180-degree area, but is not aware of it while driving, unless motivated to direct his or her attention there.

Exhibit 2-4: Relative Visibility of Target Object as Viewed with Peripheral Vision



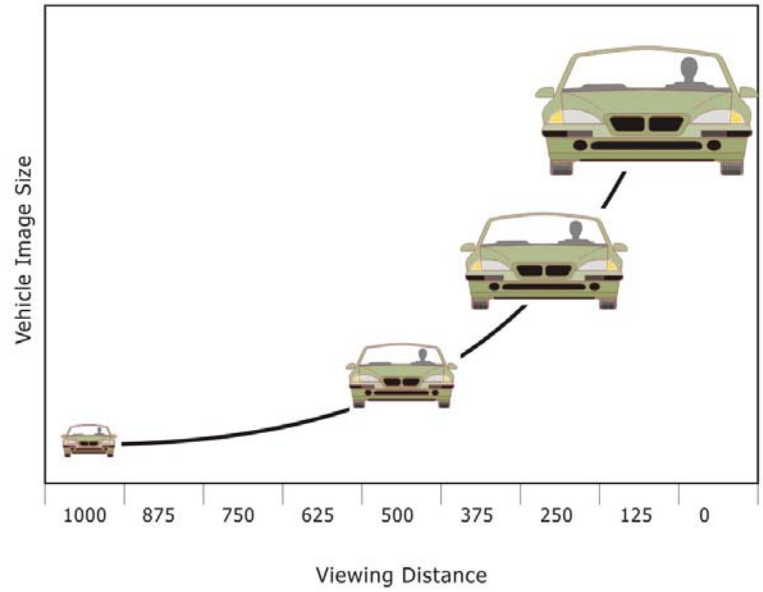
Movement in Depth

Numerous driving situations require drivers to estimate movement of vehicles based on the rate of change of visual angle created at the eye by the vehicle. These situations include safe following of a vehicle in traffic, selecting a safe gap on a two-way stop-controlled approach, and passing another vehicle with oncoming traffic and no passing lane.

The primary cue that drivers use to determine their closing speed to another vehicle is the rate of change of the image size. Exhibit 2-5 illustrates the relative change of the size of an image at different distances from a viewer.

Exhibit 2-5: Relationship Between Viewing Distance and Image Size

Drivers use the observed change in size of an object to estimate speed.



Adapted from Olson and Farber.(14)

Drivers have difficulty detecting the rate of closing speed due to the relatively small amount of change in the size of the vehicle that occurs per second when the vehicle is at a distance.

As shown in Exhibit 2-5, the relationship between viewing distance and image size is not a linear relationship. The fact that it is a non-linear relationship is likely the source of the difficulty drivers have in making accurate estimates of closing speed.

Drivers use the observed change in the size of a distant vehicle, measured by the rate of change of the visual angle occupied by the vehicle, to estimate the vehicle's travel speed. Drivers have difficulty detecting changes in vehicle speed over a long distance due to the relatively small amount of change in the size of the vehicle that occurs per second. This is particularly important in overtaking situations on two-lane roadways where drivers must be sensitive to the speed of oncoming vehicles. When the oncoming vehicle is at a distance at which a driver might pull out to overtake the vehicle in front, the size of that oncoming vehicle is changing gradually and the driver may not be able to distinguish whether the oncoming vehicle is traveling at a speed above or below that of average vehicles. In overtaking situations such as this, drivers have been shown to accept insufficient time gaps when passing in the face of high-speed vehicles, and to reject sufficient time gaps when passing in the face of other low-speed vehicles.(5,13)

Limitations in driver perception of closing speed may also lead to increased potential for rear-end crashes when drivers traveling at highway speeds approach stopped or slowing vehicles and misjudge the stopping distance available. This safety concern is compounded when drivers are not expecting this situation. One example is on a two-lane rural roadway where a left-turning driver must stop in the through lane to wait for an acceptable gap in opposing traffic. An approaching driver may not detect the stopped vehicle. In this circumstance the use of turn signals or visibility of brake lights may prove to be a crucial cue for determining that the vehicle is stopped and waiting to turn.

Visual Search

The driving task requires active search of the rapidly changing road scene, which requires rapid collection and absorption of road information. While the length of an eye fixation on a particular subject can be as short as 1/10 of a second for a simple task such as checking lane position, fixation on a complex subject can take up to 2 seconds.(35) By understanding where drivers fix their eyes while performing a particular driving task, information can be placed in the most effective location and format.

Studies using specialized cameras that record driver-eye movements have revealed how drivers distribute their attention amongst the various driving sub-tasks, and the very brief periods of time (fixations) drivers can allocate to any one target while moving. On an open road, study drivers fixated approximately 90 percent of the time within a 4-degree region vertically and horizontally from a point directly ahead of the driver.(26) Within this focused region, slightly more than 50-percent of all eye fixations occurred to the right side of the road where traffic signs are found. This indicates that driver visual search is fairly concentrated.

The visual search pattern changes when a driver is negotiating a horizontal curve as opposed to driving on a tangent. On tangent sections, drivers can gather both path and lateral position information by looking ahead. During curve negotiation, visual demand is essentially doubled, as the location of street sign and roadside information is displaced (to the left or to the right) from information about lane position. Eye movement studies show that drivers change their search behavior several seconds prior to the start of the curve. These findings suggest that advisory curve signs placed just prior to the beginning of the approach zone may reduce visual search challenges.(38)

Other road users, such as pedestrians and cyclists, also have a visual search task. Pedestrians can be observed to conduct a visual search if within three seconds of entering the vehicle path the head is turned toward the direction in which the vehicle would be coming from. The visual search varies with respect to the three types of threats: vehicles from behind, from the side, and ahead. Vehicles coming from behind require the greatest head movement and are searched for the least. These searches are conducted by only about 30 percent of pedestrians. Searches for vehicles coming from the side and from ahead are more frequent, and are conducted by approximately 50 and 60 percent of pedestrians, respectively. Interestingly between 8 and 25 percent of pedestrians at signalized downtown intersections without auditory signals do not look for threats.(42)

2.3.3. Perception-Reaction Time

Perception-reaction time (PRT) includes time to detect a target, process the information, decide on a response, and initiate a reaction. Although higher values such as 1.5 or 2.5 seconds are commonly used because it accommodates the vast percentage of drivers in most situations, it is important to note that PRT is not fixed. PRT depends on human elements discussed in previous sections, including information processing, driver alertness, driver expectations, and vision.

Perception reaction time is influenced by: detection time, decision time, and response time.

The following sections describe the components of perception-reaction time: detection, decision, and response.

Detection

The initiation of PRT begins with detection of an object or obstacle that may have potential to cause a crash. At this stage the driver does not know whether the observed object is truly something to be concerned with, and if so, the level of concern.

Detection can take a fraction of a second for an expected object or a highly conspicuous object placed where the driver is looking. However, at night an object which is located several degrees from the line of sight, and which is of low contrast compared to the background, may not be seen for many seconds. The object cannot be seen until the contrast of the object exceeds the threshold contrast sensitivity of the driver viewing it.

Failures in detection are most likely for objects that are:

- More than a few degrees from the driver's line of sight;
- Minimally contrasted with the background;
- Small in size;
- Seen in the presence of glare;
- Not moving; and,
- Unexpected and not being actively searched for by the driver.

Once an object or obstacle has been detected, the details of the object or obstacle must be determined in order to have enough information to make a decision. As discussed in the next section, identification will be delayed when the object being detected is unfamiliar and unexpected. For example, a low-bed, disabled tractor-trailer with inadequate reflectors blocking a highway at night will be unexpected and hard to identify.

Decision

Once an object or obstacle has been detected and enough information has been collected to identify it, a decision can be made as to what action to take.

Once an object or obstacle has been detected and enough information has been collected to identify it, a decision can be made as to what action to take. The decision does not involve any action, but rather is a mental process that takes what is known about the situation and determines how the driver will respond.

Decision time is highly dependent on circumstances that increase the complexity of a decision or require it be made immediately. Many decisions are made quickly when the response is obvious. For example, when the driver is a substantial distance from the intersection and the traffic light turns red, minimal time is needed to make the decision. If, on the other hand, the driver is close to the intersection and the traffic light turns yellow, there is a dilemma: is it possible to stop comfortably without risking being rear-ended by a following vehicle, or is it better to proceed through the intersection? The time to make this stop-or-go decision will be longer given that there are two reasonable options and more information to process.

Decision-making also takes more time when there is an inadequate amount of information or an excess amount. If the driver needs more information, they must search for it. On the other hand, if there is too much information the driver must sort through it to find the essential elements, which may result in unnecessary effort and time. Decision-making also takes more time when drivers have to determine the nature of unclear information, such as bits of reflection on a road at night. The bits of reflection may result from various sources, such as harmless debris or a stopped vehicle.

Response

When the information has been collected, processed, and a decision has been made, time is needed to respond physically. Response time is primarily a function of physical ability to act upon the decision and can vary with age, lifestyle (athletic, active, or sedentary), and alertness.

Perception-Reaction Times in Various Conditions

Various factors present in each unique driving situation affect driver perception-reaction time; therefore, it is not a fixed value. Guidance for a straight-forward detection situation comes from a study of "stopping-sight distance" perception-reaction times. The experiment was conducted in daylight while a driver was cresting a hill and looking at the road at the very moment an object partially blocking the road came into view without warning. The majority of drivers (85%) reacted within 1.3 seconds, and 95% of drivers reacted within 1.6 seconds.[30] In a more recent study which also examined drivers' response to unexpected objects entering the roadway, it was concluded that a perception-reaction time of approximately 2.0 sec seems to be inclusive of nearly all the subjects' responses under all conditions tested.[12]

However, the 2.0 second perception-reaction time may not be appropriate for application to a low contrast object seen at night. Although an object can be within the driver's line of sight for hundreds of feet, there may be insufficient light from low beam headlights, and insufficient contrast between the object and the background for a driver to see it. Perception-reaction time cannot be considered to start until the object has reached the level of visibility necessary for detection, which varies from driver to driver and is influenced by the driver's state of expectation. A driving simulator study found that drivers who were anticipating having to respond to pedestrian targets on the road edge took an average of 1.4 seconds to respond to a high contrast pedestrian, and 2.8 seconds to respond to a low contrast pedestrian, indicating a substantial impact of contrast on perception-reaction time.[34] Glare lengthened these perception-reaction times even further. It should be noted that subjects in experiments are abnormally alert, and real-world reaction times could be expected to be longer.

Perception reaction time is not fixed. It is influenced by many factors including: driver vision, conspicuity of objects, and the complexity of a situation.

As is clear from this discussion, perception-reaction time is not a fixed value. It is dependent on driver vision, conspicuity of a traffic control device or objects ahead, the complexity of the response required, and the urgency of that response.

2.3.4. Speed Choice

A central aspect of traffic safety is driver speed choice. While speed limits influence driver speed choice, these are not the only or the most important influences. Drivers select speed using perceptual and "road message" cues. Understanding these cues can help establish self-regulating speeds with minimal or no enforcement.

This section includes a summary of how perceptual and road message cues influence speed choice.

Perceptual Cues

A driver's main cue for speed choice comes from peripheral vision. In experiments where drivers are asked to estimate their travel speed with their peripheral vision blocked (only the central field of view can be used), the ability to estimate speed is poor. This is because the view changes very slowly in the center of a road scene. If, on the other hand, the central portion of the road scene is blocked out, and drivers are asked to estimate speed based on the peripheral view, drivers do much better.[36]

Streaming (or "optical flow") of information in peripheral vision is one of the greatest influences on drivers' estimates of speed. Consequently, if peripheral stimuli are close by, then drivers will feel they are going faster than if they encounter a wide-open situation. In one study, drivers were asked to drive at 60 mph with the speedometer covered. In an open-road situation, the average speed was 57 mph. After the same instructions, but along a tree-lined route, the average speed was 53 mph.[38] The researchers believe that the trees near the road provided peripheral stimulation, giving a sense of higher speed.

Road message cues include: flow of information in peripheral vision, noise level, speed adaptation, and road geometry.

Noise level is also an important cue for speed choice. Several studies examined how removing noise cues influenced travel speed. While drivers' ears were covered (with ear muffs) they were asked to travel at a particular speed. All drivers underestimated how fast they were going and drove 4 to 6 mph faster than when the usual sound cues were present.[11,10] With respect to lowering speeds, it has been counter-productive to progressively quiet the ride in cars and to provide smoother pavements.

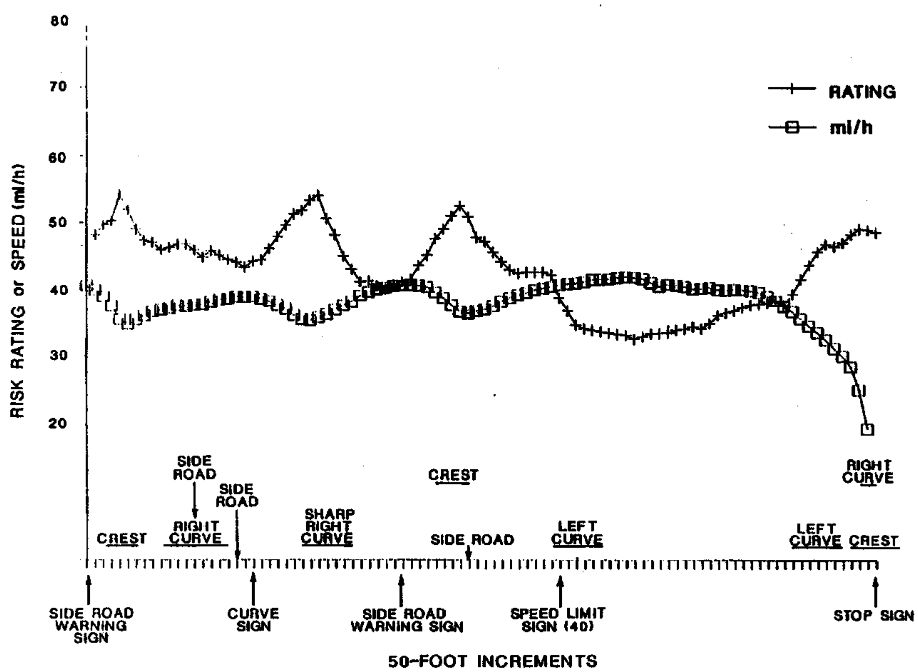
Another aspect of speed choice is speed adaptation. This is the experience of leaving a freeway after a long period of driving and having difficulty conforming to the speed limit on an arterial road. One study required subjects to drive for 20 miles on a freeway and then drop their speeds to 40 mph on an arterial road. The average speed on the arterial was 50 miles per hour.[37] This speed was higher than the requested speed despite the fact that these drivers were perfectly aware of the adaptation effect, told the researchers they knew this effect was happening, and tried to bring their speed down. The adaptation effect was shown to last up to five or six minutes after leaving a freeway, and to occur even after very short periods of high speed.[37] Various access management techniques, sign placement, and traffic calming devices may help to reduce speed adaptation effects.

Road Message Cues

Drivers may interpret the roadway environment as a whole to encourage fast or slow speeds depending on the effects of the geometry, terrain, or other roadway elements. Even though drivers may not have all the information for correctly assessing a safe speed, they respond to what they can see. Drivers tend to drive faster on a straight road with several lanes, wide shoulders, and a wide clear zone, than drivers on a narrow, winding road with no shoulders or a cliff on the side. For example, speeds on rural highway tangents are related to cross-section and other variables, such as the radius of the curve before and after the tangent, available sight distance, and general terrain.[33]

The difficulty of the driving task due to road geometry (e.g., sharp curves, narrow shoulders) strongly influences driver perception of risk and, in turn, driver speed. Exhibit 2-6 shows the relationship between risk perception, speed, various geometric elements, and control devices. These relationships were obtained from a study in which drivers travelled a section of roadway twice. Each time the speed of the vehicle was recorded. The first time test subjects travelled the roadway they drove the vehicle. The second time the test subjects travelled the roadway, there were passengers in the vehicle making continuous estimates of the risk of a crash.[33] As shown in Exhibit 2-6, where drivers perceived the accident risk to be greater (e.g., sharp curves, limited sight distance), they reduced their travel speed.

Exhibit 2-6: Perceived Risk of an Accident and Speed



Source: *Horizontal Alignment Design Consistency for Rural Two-lane Highways*, RD-94-034, FHWA.

Speed advisory plaques on curve warning signs appear to have little effect on curve approach speed, probably because drivers feel they have enough information from the roadway itself and select speed according to the appearance of the curve and its geometry. One study recorded the speeds of 40 drivers, unfamiliar with the route, on curves with and without speed plaques. Although driver eye movements were recorded and drivers were found to look at the warning sign, the presence of a speed plaque had no effect on drivers' selected speed.[22]

In contrast, a study of 36 arterial tangent sections found some influence of speed limit, but no influence of road design variables. The sections studied had speed limits that ranged from 25 to 55 mph. Speed limit accounted for 53 percent of the variance in speed, but factors such as alignment, cross-section, median presence, and roadside variables were not found to be statistically significantly related to operating speed.[21]

2.4. POSITIVE GUIDANCE

Positive guidance approach to road design considers driver limitations, expectations, and engineering principles.

Knowledge of human limitations in information processing and human reliance on expectation to compensate for those limitations in information processing, led to the "positive guidance" approach to highway design. This approach is based on a combination of human factors and traffic engineering principles.[18] The central principle is that road design that corresponds with driver limitations and expectations increases the likelihood of drivers responding to situations and information correctly and quickly. Conversely, when drivers are not provided with information in a timely fashion, when they are overloaded with information, or when their expectations are not met, slowed responses and errors may occur.

Design that conforms to long-term expectancies reduces the chance of driver error. For example, drivers expect that there are no traffic signals on freeways and freeway exits are on the right. If design conforms to those expectancies it reduces the risk of a crash. Short-term expectancies can also be impacted by design decisions. An example of a short-term expectation is that subsequent curves on a section of road are gradual, given that all previous curves were gradual.

With respect to traffic control devices, the positive guidance approach emphasizes assisting the driver with processing information accurately and quickly by considering:

- Primacy: Determine the placements of signs according to the importance of information, and avoid presenting the driver with information when and where the information is not essential.
- Spreading: Where all the information required by the driver cannot be placed on one sign or on a number of signs at one location, spread the signage along the road so that information is given in small chunks to reduce information load.
- Coding: Where possible, organize pieces of information into larger units. Color and shape coding of traffic signs accomplishes this organization by representing specific information about the message based on the color of the sign background and the shape of the sign panel (e.g., warning signs are yellow, regulatory signs are white).
- Redundancy: Say the same thing in more than one way. For example, the stop sign in North America has a unique shape and message, both of which convey the message to stop. A second example of redundancy is to give the same information by using two devices (e.g., “no passing” indicated with both signs and pavement markings).

2.5. IMPACTS OF ROAD DESIGN ON THE DRIVER

The influence of major road design elements, driving tasks, and human error on common crash types are summarized in section 2.5.

This section considers major road design elements, related driver tasks, and human errors associated with common crash types. It is not intended to be a comprehensive summary, but is intended to provide examples to help identify opportunities where human factors knowledge can be applied to improve design.

2.5.1. Intersections and Access Points

As discussed in Section 2.2, the driving task involves control, guidance, and navigation elements. At intersections, each of these elements presents challenges:

- Control: The path through the intersection is typically unmarked and may involve turning;
- Guidance: There are numerous potential conflicts with other vehicles, pedestrians, and cyclists on conflicting paths; and
- Navigation: Changes in direction are usually made at intersections, and road name signing can be difficult to locate and read in time to accomplish any required lane changes.

In the process of negotiating any intersection, drivers are required to:

- Detect the intersection;
- Identify signalization and appropriate paths;

- Search for vehicles, pedestrians, and bicyclists on a conflicting path;
- Assess adequacy of gaps for turning movements;
- Rapidly make a stop/go decision on the approach to a signalized intersection when in the decision zone; and,
- Successfully complete through or turning maneuvers.

Thus, intersections place high demands on drivers in terms of visual search, gap estimation, and decision-making requirements that increase the potential for error. Road crash statistics show that although intersections constitute a small portion of the highway network, about 50 percent of all urban crashes and 25 percent of rural crashes are related to intersections.[43] A study of the human factors contributing causes to crashes found that the most frequent type of error was "improper lookout," and that 74 percent of these errors occurred at intersections. In about half of the cases, drivers failed to look, and in about half of the cases, drivers "looked but did not see."[41,15]

Road crash statistics show that although intersections constitute a small portion of the highway network, about 50 percent of all urban crashes and 25 percent of rural crashes are related to intersections.[33]

Errors Leading to Rear-End and Sideswipe Crashes

Errors leading to rear-end and sideswipe crashes include the following:

- Assuming that the lead driver, once moving forward, will continue through the stop sign, but the lead driver stops due to late recognition that there is a vehicle or pedestrian on a conflicting path.
- Assuming that the lead driver will go through a green or yellow light, but the lead driver stops due to greater caution. Drivers following one another can make differing decisions in this "dilemma zone". As speed increases, the length of the dilemma zone increases. Additionally, as speed increases, the deceleration required is greater and the probability of a rear-end crash may also increase.
- Assuming that the lead driver will continue through a green or yellow light but the lead driver slows or stops due to a vehicle entering or exiting an access point just prior to the intersection, or a vehicle exiting an access point suddenly intruding into the lane, or a pedestrian crossing against a red light.
- Changing lanes to avoid a slowing or stopped vehicle, with inadequate search.
- Distracting situations that may lead to failure to detect slowing or stopping vehicles ahead. Distracting situations could include:
    - Preoccupation with personal thoughts,
    - Attention directed to non-driving tasks within the vehicle,
    - Distraction from the road by an object on the roadside, or
    - Anticipation of downstream traffic signal.

Errors Leading to Turning Crashes

Turning movements are often more demanding with respect to visual search, gap judgment, and path control than are through movements. Turning movements can lead to crashes at intersections or access points due to the following:

Turning movements at intersections may lead to crashes because of perceptual limitations, visual blockage, dilemma zones, and inadequate visual search.

- Perceptual limitations,
- Visual blockage,
- Permissive left-turn trap, and
- Inadequate visual search.

A description of these common errors that can lead to turning crashes at intersections is provided below.

Perceptual Limitations

Perceptual limitations in estimating closing vehicle speeds could lead to left-turning drivers selecting an inappropriate gap in oncoming traffic. Drivers turning left during a permissive green light may not realize that an oncoming vehicle is moving at high speed.

Visual Blockage

A visual blockage may limit visibility of an oncoming vehicle when making a turn at an intersection. About 40 percent of intersection crashes involve a view blockage.[41] Windshield pillars inside the vehicle, utility poles, commercial signs, and parked vehicles may block a driver's view of a pedestrian, bicyclist, or motorcycle on a conflicting path at a critical point during the brief glance that a driver may make in that direction. Visual blockages also occur where the offset of left-turn bays results in vehicles in the opposing left-turn lane blocking a left-turning driver's view of an oncoming through vehicle.

Permissive Left-turn Trap

On a high-volume road, drivers turning left on a permissive green light may be forced to wait for a yellow light to make their turn, at which time they come into conflict with oncoming drivers who continue through into a red light.

Inadequate Visual Search

Drivers turning right may concentrate their visual search only on vehicles coming from the left and fail to detect a bicyclist or pedestrian crossing from the right.[1] This is especially likely if drivers do not stop before turning right on red, and as a result give themselves less time to search both to the left and right.

Errors Leading to Angle Crashes

Angle crashes can occur due to:

- Delayed detection of an intersection (sign or signal) at which a stop is required;
- Delayed detection of crossing traffic by a driver who deliberately violates the sign or signal; or
- Inadequate search for crossing traffic or appropriate gaps.

Drivers may miss seeing a signal or stop sign because of inattention, or a combination of inattention and a lack of road message elements that would lead drivers to expect the need to stop. For example, visibility of the intersection pavement or the crossing traffic may be poor, or drivers may have had the right of way for some distance and the upcoming intersection does not look like a major road requiring a stop. In an urban area where signals are closely spaced, drivers may inadvertently attend to the signal beyond the signal they face. Drivers approaching at high speeds may become caught in the dilemma zone and continue through a red light.

Errors Leading to Crashes with Vulnerable Road Users

Pedestrian and bicycle crashes often result from inadequate search and lack of conspicuity. The inadequate search can be on the part of the driver, pedestrian, or bicyclist. In right-turning crashes, pedestrians and drivers have been found to be equally guilty of failure to search. In left-turning crashes, drivers are more frequently found at fault, likely because the left-turn task is more visually demanding than the right-turn task for the driver.[20]

Inadequate search and lack of conspicuity can cause pedestrian and bicycle crashes.

Examples of errors that may lead to pedestrian crashes include:

- Pedestrians crossing at traffic signals rely on the signal giving them the right of way, and fail to search adequately for turning traffic.[35]
- Pedestrians step into the path of a vehicle that is too close for the driver to have sufficient time to stop.

When accounting for perception-response time, a driver needs over 100 feet to stop when traveling at 30 mile per hour. Pedestrians are at risk because of the time required for drivers to respond and because of the energy involved in collisions, even at low speeds. Relatively small changes in speed can have a large impact on the severity of a pedestrian crash. A pedestrian hit at 40 mph has an 85-percent chance of being killed; at 30 mph the risk is reduced to 45 percent; at 20 mph the risk is reduced to 5 percent.[27]

A pedestrian hit at 40 mph has an 85- percent chance of being killed; at 30 mph the risk of being killed is reduced to 45 percent; at 20 mph the risk of being killed is reduced to 5 percent.[37]

Poor conspicuity, especially at night, greatly increases the risk of a pedestrian or bicyclist crash. Clothing is often dark, providing little contrast to the background. Although streetlighting helps drivers see pedestrians, streetlighting can create uneven patches of light and dark which makes pedestrians difficult to see at any distance.

2.5.2. Interchanges

At interchanges drivers can be traveling at high speeds, and at the same time can be faced with high demands in navigational, guidance, and control tasks. The number of crashes at interchanges as a result of driver error is influenced by the following elements of design:

The number of crashes around an interchange is influenced by: on-ramp/merge length, ramp spacing, sight distance, and off ramp radius.

- Entrance ramp/merge length,
- Distance between successive ramp terminals,
- Decision sight distance and guide signing, and
- Exit ramp design.

Entrance Ramp/Merge Length

If drivers entering a freeway are unable to accelerate to the speed of the traffic stream (e.g., due to acceleration lane length, the grade of the ramp, driver error, or heavy truck volumes), entering drivers will merge with the mainline at too slow a speed and may risk accepting an inadequate gap. Alternatively the freeway is congested or if mainline vehicles are tailgating, it may be difficult for drivers to find an appropriate gap into which to merge.

Distance Between Successive Ramp Terminals

If the next exit ramp is close to the entrance ramp, entering (accelerating) drivers will come into conflict with exiting (decelerating) drivers along the weaving section and crashes may increase.[40,16] Given the visual search required by both entering and exiting drivers, and the need to look away from the traffic immediately ahead in order to check for gaps in the adjacent lane, sideswipe and rear-end crashes can occur in weaving sections. Drivers may fail to detect slowing vehicles ahead, or vehicles changing lanes in the opposing direction, in time to avoid contact.

Decision Sight Distance and Guide Signing

Increased risk of error occurs in exit locations because drivers try to read signs, change lanes, and decelerate comfortably and safely. Drivers may try to complete all three tasks simultaneously thereby increasing their willingness to accept smaller gaps while changing lanes or to decelerate at greater than normal rates.

Exit Ramp Design

If the exit ramp radius is small and requires the exiting vehicle to decelerate more than expected, the speed adaptation effect discussed in the previous section can lead to insufficient speed reductions. Also, a tight exit ramp radius or an unusually long vehicle queue extending from the ramp terminal can potentially surprise drivers, leading to run-off-road and rear-end crashes.

2.5.3. Divided, Controlled-Access Mainline

Errors that can lead to crashes on a controlled-access mainline include: driver inattention and sleepiness, animal in the road, and slow-moving or stopped vehicles ahead.

Compared to intersections and interchanges, the driving task on a divided, controlled-access mainline is relatively undemanding with respect to control, guidance, and navigational tasks. This assumes that the mainline has paved shoulders, wide clear zones, and is outside the influence area of interchanges.

A description of each of these common errors and other factors that lead to crashes on divided, controlled-access mainline roadway sections is provided below.

Driver Inattention and Sleepiness

Low mental demand can lead to driver inattention and sleepiness, resulting in inadvertent (drift-over) lane departures. Sleepiness is strongly associated with time of day. It is particularly difficult for drivers to resist falling asleep in the early-morning hours (2 to 6 a.m.) and in the mid-afternoon. Sleepiness arises from the common practices of reduced sleep and working shifts. Sleepiness also results from alcohol and other drug use.[32] Shoulder-edge rumble strips are one example of a countermeasure that can be used to potentially reduce run-off-road crashes. They provide strong auditory and tactile feedback to drivers whose cars drift off the road because of inattention or impairment.

Slow-Moving or Stopped Vehicles Ahead

Mainline crashes can also occur when drivers encounter slow-moving or stopped vehicles which, except in congested traffic, are in a freeway through lane. Drivers' limitations in perceiving closing speed result in a short time to respond once the driver realizes the rapidity of the closure. Alternatively, drivers may be visually attending to the vehicle directly ahead of them and may not notice lane changes occurring beyond. If the lead driver is the first to encounter the stopped vehicle, realizes the situation just in time, and moves rapidly out of the lane, the stopped vehicle is uncovered at the last second, leaving the following driver with little time to respond.

Animals in the Road

Another common mainline crash type is with animals, particularly at night. Such crashes may occur because an animal enters the road immediately in front of the driver leaving little or no time for the driver to detect or avoid it. Low conspicuity of animals is also a problem. Given the similarity in coloring and reflectance between pedestrians and animals, the same driver limitations can be expected to apply to animals as to pedestrians in dark clothing. Based on data collected for pedestrian targets, the majority of drivers traveling at speeds much greater than 30 mph and with low-beam headlights would not be able to detect an animal in time to stop.[(4)]

2.5.4. Undivided Roadways

Undivided roadways vary greatly in design and therefore in driver workload and perceived risk. Some undivided roadways may have large-radius curves, mostly level grades, paved shoulders, and wide clear zones. On such roads, and in low levels of traffic, the driving task can be very undemanding, resulting in monotony and, in turn, possibly driver inattention and/or sleepiness. On the other hand, undivided roadways may be very challenging in design, with tight curves, steep grades, little or no shoulder, and no clear zone. In this case the driving task is considerably more demanding.

Errors that can lead to crashes on an undivided roadway includes: driver inattention and sleepiness, inadvertent movement into oncoming lane, driver speed choice, slow-moving or stopped vehicles ahead, and poor visibility of vulnerable road users or

Driver Inattention and Sleepiness

As described previously for the controlled-access mainline, inadvertent lane departures can result when drivers are inattentive, impaired by alcohol or drugs, or sleepy. On an undivided highway, these problems lead to run-off-road and head-on crashes. Rumble strips are effective in alerting drivers about to leave the lane, and have been shown to be effective in reducing run-off-road and cross-centerline crashes, respectively.[(7,9)]

Inadvertent Movement into Oncoming Lane

The vast majority of head-on crashes occur due to inadvertent movement into the oncoming lane. Contrary to some expectations, only about 4 percent of head-on crashes are related to overtaking.[(15)] Centerline rumble strips are very effective in reducing such crashes as they alert inattentive and sleepy drivers. Although overtaking crashes are infrequent, they have a much higher risk of injury and fatality than other crashes. As discussed previously, drivers are very limited in their ability to perceive their closing speed to oncoming traffic. They tend to select gaps based more on distance than on speed, leading to inadequate gaps when the oncoming vehicle is traveling substantially faster than the speed limit. Passing lanes and four-lane passing sections greatly alleviate driver workload and the risk of error involved in passing.

Driver Speed Choice

On roads with demanding geometry, driver speed choice when entering curves may be inappropriate, leading to run-off-road crashes. Treatments which improve delineation are often applied under the assumption that run-off-road crashes occur because the driver did not have adequate information about the direction of the road path. However, studies have not supported this assumption.[29]

Slow-Moving or Stopped Vehicles Ahead

For the controlled-access mainline, rear-end and sideswipe crashes occur when drivers encounter unexpected slowing or stopped vehicles and realize too late their closing speed.

Poor Visibility of Vulnerable Road Users or Animals

Vulnerable road user and animal crashes may occur due to low contrast with the background and drivers' inability to detect pedestrians, cyclists, or animals in time to stop.

2.6. SUMMARY: HUMAN FACTORS AND THE HSM

Integrating human factors considerations with other parts of the HSM can improve transportation planning and engineering decisions.

This chapter described the key factors of human behavior and ability that influence how drivers interact with the roadway. The core elements of the driving task were outlined and related to human ability so as to identify areas where humans may not always successfully complete the tasks. There is potential to reduce driver error and associated crashes by accounting for the following driver characteristics and limitations described in the chapter:

- Attention and information processing: Drivers can only process a limited amount of information and often rely on past experience to manage the amount of new information they must process while driving. Drivers can process information best when it is presented: in accordance with expectations; sequentially to maintain a consistent level of demand; and, in a way that it helps drivers prioritize the most essential information.

- Vision: Approximately 90 percent of the information used by a driver is obtained visually.[17] It is important that the information be presented in a way that considers the variability of driver visual capability such that users can see, comprehend, and respond to it appropriately.

- Perception-reaction time: The amount of time and distance needed by one driver to respond to a stimulus (e.g., hazard in road, traffic control device, or guide sign) depends on human elements, including information processing, driver alertness, driver expectations, and vision.

- Speed choice: Drivers use perceptual and road message cues to determine a speed they perceive to be safe. Information taken in through peripheral vision may lead drivers to speed up or slow down depending on the distance from the vehicle to the roadside objects.[38] Drivers may also drive faster than they realize after adapting to highway speeds and subsequently entering a lower-level facility.[37]

A combination of engineering and human factors knowledge can be applied through the positive guidance approach to road design. The positive guidance approach is based on the central principle that road design that corresponds with driver limitations and expectations increases the likelihood of drivers responding to situations and information correctly and quickly. When drivers are not provided or do not accept information in a timely fashion, when they are overloaded with information, or when their expectations are not met, slowed responses and errors may occur.

Human factors knowledge can be applied to all projects regardless of the project focus. *Parts B, C,* and *D* of the HSM provide specific guidance on the roadway safety management process, estimating safety effects of design alternatives, and predicting safety on different facilities. Applying human factors considerations to these activities can improve decision making and design considerations in analyzing and developing safer roads.

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PART A— INTRODUCTION AND FUNDAMENTALS

CHAPTER 3—FUNDAMENTALS

3.1. Chapter Introduction 3-3
3.2. Crashes as the Basis of Safety Analysis 3-3
3.2.1. Objective and Subjective Safety 3-4
3.2.2. Fundamental Definitions of Terms in the HSM 3-5
3.2.3. Crashes Are Rare and Random Events 3-7
3.2.4. Crash Contributing Factors 3-8
3.3. Data for Crash Estimation 3-11
3.3.1. Data Needed for Crash Analysis 3-11
3.3.2. Limitations of Observed Crash Data Accuracy 3-12
3.3.3. Limitations Due To Randomness and Change 3-14
3.4. Evolution of Crash Estimation Methods 3-17
3.4.1. Observed Crash Frequency and Crash Rate Methods 3-17
3.4.2. Indirect Safety Measures 3-19
3.4.3. Crash Estimation using Statistical Methods 3-19
3.4.4. Development and Content of the HSM Methods 3-20
3.5. Predictive Method in Part C of the HSM 3-21
3.5.1. Overview of the Part C Predictive Method 3-21
3.5.2. Safety Performance Functions 3-22
3.5.3. Accident Modification Factors 3-24
3.5.4. Calibration 3-30
3.5.5. Weighting using the Empirical Bayes Method 3-30
3.5.6. Limitations of Part C Predictive Method 3-31
3.6. Application of the HSM 3-32
3.7. Effectiveness Evaluation 3-33
3.7.1. Overview of Effectiveness Evaluation 3-33
3.7.2. Effectiveness Evaluation Study Types 3-33
3.8. Conclusions 3-35
3.9. References 3-36

EXHIBITS

Exhibit 3-1: Changes in Objective and Subjective Safety 3-5
Exhibit 3-2: Crashes are Rare and Random Events 3-8
Exhibit 3-3: Contributing Factors to Vehicle Crashes 3-9
Exhibit 3-4: Example Haddon Matrix for Identifying Contributing Factors 3-10
Exhibit 3-5: Variation in Short-Term Observed Crash Frequency 3-15
Exhibit 3-6 Regression-to-the-mean (RTM) and RTM Bias 3-16
Exhibit 3-9: Facility Types and Site Types included in Part C 3-23
Exhibit 3-10: Values for Determining Confidence Intervals using Standard Error 3-28

CHAPTER 3 FUNDAMENTALS

3.1. CHAPTER INTRODUCTION

This chapter introduces fundamentals for applying the HSM.

The purpose of this chapter is to introduce the fundamental concepts for understanding the roadway safety management techniques and crash estimation methods presented in subsequent chapters of the Highway Safety Manual (HSM).

In the HSM, crash frequency is the fundamental basis for safety analysis, selection of sites for treatment and evaluation of the effects of treatments. The overall aim of the HSM is to reduce crashes and crash severities through the comparison and evaluation of alternative treatments and design of roadways. A commensurate objective is to use limited safety funds in a cost effective manner.

This chapter presents the following concepts:

- An overview of the basic concepts relating to crash analysis, including definitions of key crash analysis terms, the difference between subjective and objective safety, factors that contribute to crashes and strategies to reduce crashes;
- Data for crash estimation and its limitations;
- A historical perspective of the evolution of crash estimation methods and the limitations their methods;
- An overview of the predictive method (*Part C*) and AMFs (*Parts C* and *D*);
- Application of the HSM; and
- The types of evaluation methods for determining the effectiveness of treatment types (*Part B*).

Users benefit by familiarizing themselves with the material in Chapter 3 in order to apply the HSM and understanding that engineering judgment is necessary to determine if and when the HSM procedures are appropriate.

3.2. CRASHES AS THE BASIS OF SAFETY ANALYSIS

Crash frequency is a fundamental quantitative performance measure in the HSM.

Crash frequency is used as a fundamental indicator of "safety" in the evaluation and estimation methods presented in the HSM. Where the term "safety" is used in the HSM, it refers to the crash frequency and/or crash severity and collision type for a specific time period, a given location, and a given set of geometric and operational conditions.

This section provides an overview of fundamental concepts relating to crashes and their use in the HSM:

- The difference between objective safety and subjective safety;
- The definition of a crash and other crash related terms;
- Crashes are rare and random events;
- Contributing factors influence crashes and can be addressed by a number of strategies;

- The HSM focuses on reducing crashes by changing the roadway/environment.

3.2.1. Objective and Subjective Safety

Section 3.2.1 presents objective and subjective safety concepts. The HSM focuses on objective safety.

The HSM focuses on how to estimate and evaluate the crash frequency and crash severity for a particular roadway network, facility or site, in a given period, and hence the focus is on "objective" safety. Objective safety refers to use of a quantitative measure which is independent of the observer. Crash frequency and severity are defined in Section 3.2.2.

In contrast, "subjective" safety concerns the perception of how safe a person feels on the transportation system. Assessment of subjective safety for the same site will vary between observers.

The traveling public, the transportation professional and the statisticians may all have diverse but valid opinions about whether a site is "safe" or "unsafe." Highway agencies draw information from each of these groups in determining policies and procedures which it will use to affect a change in crash frequency and/or severity among the road or highway system.

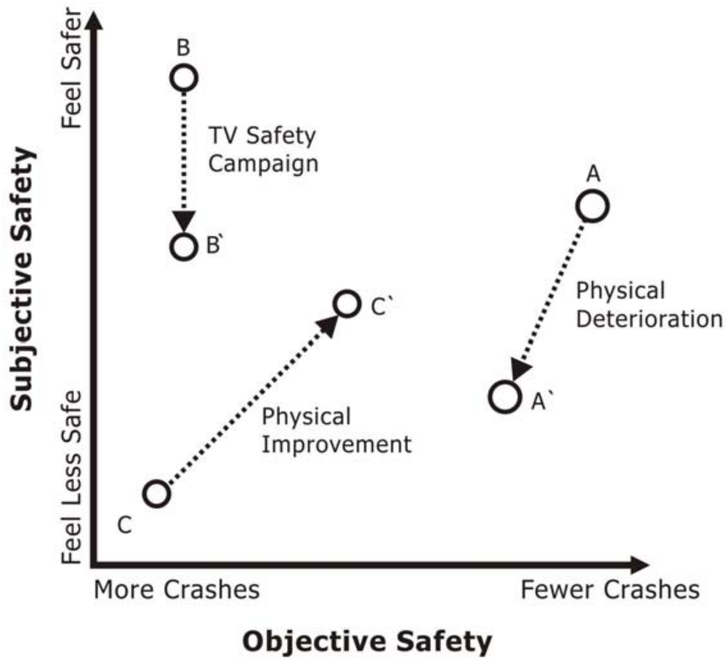
Exhibit 3-1 illustrates the difference between objective and subjective safety. Moving to the right on the horizontal axis of the graph conceptually shows an increase in objective safety (reduction in crashes). Moving up on the vertical axis conceptually shows an increase in subjective safety (i.e., increased perception of safety). In this exhibit, three examples illustrate the difference:

- The change between Points A to A′ represents a clear-cut deterioration in both objective and subjective safety. For example, removing lighting from an intersection may increase crashes and decrease the driver's perception of safety (at night).

- The change between Points B to B′ represents a reduction in the perception of safety on a transportation network, For example, as a result of a television campaign against aggressive driving, citizens may feel less secure on the roadways because of greater awareness of aggressive drivers. If the campaign is not effective in reducing crashes caused by aggressive driving, the decline in perceived safety occurs with no change in the number of crashes.

- The change from Point C to C′ represents a physical improvement to the roadway (such as the addition of left-turn lanes) that results in both a reduction in crashes and an increase in the subjective safety.

Exhibit 3-1: Changes in Objective and Subjective Safety



Source: NCHRP 17-27

3.2.2. Fundamental Definitions of Terms in the HSM

Definition of a Crash

In the HSM, a crash is defined as a set of events that result in injury or property damage, due to the collision of at least one motorized vehicle and may involve collision with another motorized vehicle, a bicyclist, a pedestrian or an object. The terms used in the HSM do not include crashes between cyclists and pedestrians, or vehicles on rails.(7) The terms "crash" and "accident" are used interchangeably throughout the HSM.

Definition of Crash Frequency

In the HSM, "crash frequency" is defined as the number of crashes occurring at a particular site, facility or network in a one-year period. Crash frequency is calculated according to Equation 3-1 and is measured in number of crashes per year.

$$Crash\ Frequency = \frac{Number\ of\ Crashes}{Period\ in\ Years} \qquad (3\text{-}1)$$

Section 3.2.2 provides fundamental definitions for using

Definition of Crash Estimation

"Crash estimation" refers to any methodology used to forecast or predict the crash frequency of:

- An existing roadway for existing conditions during a past or future period;

- An existing roadway for alternative conditions during a past or future period;
- A new roadway for given conditions for a future period.

The crash estimation method in *Part C* of the HSM is referred to as the "predictive method" and is used to estimate the "expected average crash frequency", which is defined below.

Definition of Predictive Method

The term "predictive method" refers to the methodology in *Part C* of the HSM that is used to estimate the "expected average crash frequency" of a site, facility or roadway under given geometric design, traffic volumes and for a specific period of time.

Definition of Expected Average Crash Frequency

The term "expected average crash frequency" is used in the HSM to describe the estimate of long-term average crash frequency of a site, facility or network under a given set of geometric design and traffic volumes in a given time period (in years).

As crashes are random events, the observed crash frequencies at a given site naturally fluctuate over time. Therefore, the observed crash frequency over a short period is not a reliable indictor of what average crash frequency is expected under the same conditions over a longer period of time.

If all conditions on a roadway could be controlled (e.g. fixed traffic volume, unchanged geometric design, etc), the long-term average crash frequency could be measured. However because it is rarely possible to achieve these constant conditions, the true long-term average crash frequency is unknown and must be estimated instead.

Definition of Crash Severity

Crashes vary in the level of injury or property damage. The American National Standard ANSI D16.1-1996 defines injury as "bodily harm to a person"[7]. The level of injury or property damage due to a crash is referred to in the HSM as "crash severity." While a crash may cause a number of injuries of varying severity, the term crash severity refers to the most severe injury caused by a crash.

Crash severity is often divided into categories according to the KABCO 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:

- K - Fatal injury: an injury that results in death;
- A - Incapacitating injury: any injury, other than a fatal injury, which prevents the injured person from walking, driving or normally continuing the activities the person was capable of performing before the injury occurred;
- B – Non-incapacitating evident injury: any injury, other than a fatal injury or an incapacitating injury, which is evident to observers at the scene of the accident in which the injury occurred;

- C - Possible injury: any injury reported or claimed which is not a fatal injury, incapacitating injury or non-incapacitating evident injury and includes claim of injuries not evident;
- O – No Injury/Property Damage Only (PDO).

While other scales for ranking crash severity exist, the KABCO scale is used in the HSM.

Definition of Crash Evaluation

In the HSM, "crash evaluation" refers to determining the effectiveness of a particular treatment or a treatment program after its implementation. Where the term effectiveness is used in the HSM, it refers to a change in the expected average crash frequency (or severity) for a site or project. Evaluation is based on comparing results obtained from crash estimation. Examples include:

- Evaluating a single application of a treatment to document its effectiveness;
- Evaluating a group of similar projects to document the effectiveness of those projects;
- Evaluating a group of similar projects for the specific purpose of quantifying the effectiveness of a countermeasure;
- Assessing the overall effectiveness of specific projects or countermeasures in comparison to their costs.

Crash evaluation is introduced in Section 3.7 and described in detail in *Chapter 9*.

3.2.3. Crashes Are Rare and Random Events

Crashes are rare and random events. By rare, it is implied that crashes represent only a very small proportion of the total number of events that occur on the transportation system. Random means that crashes occur as a function of a set of events influenced by several factors, which are partly deterministic (they can be controlled) and partly stochastic (random and unpredictable). An event refers to the movement of one or more vehicles and or pedestrians and cyclists on the transportation network.

Crashes are rare – They represent only a very small proportion of the total number of events that occur on the transportation system.

A crash is one possible outcome of a continuum of events on the transportation network during which the probability of a crash occurring may change from low risk to high risk. Crashes represent a very small proportion of the total events that occur on the transportation network. For example, for a crash to occur, two vehicles must arrive at the same point in space at the same time. However, arrival at the same time does not necessarily mean that a crash will occur. The drivers and vehicles have different properties (reaction times, braking efficiencies, visual capabilities, attentiveness, speed choice), which will determine whether or not a crash occurs.

Crashes are random - They occur as a function of a set of events influenced by several factors.

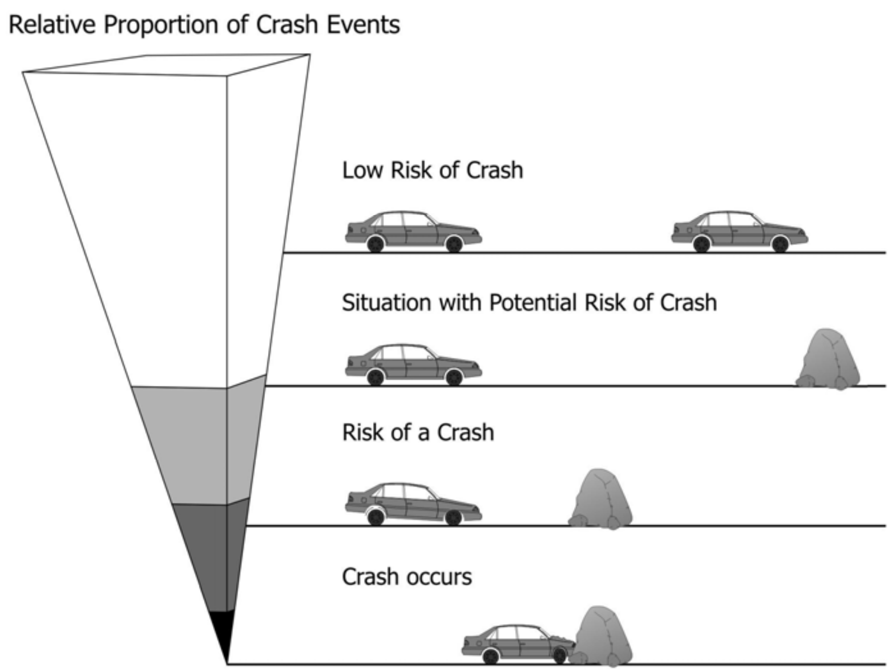
The continuum of events that may lead to crashes and the conceptual proportion of crash events to non-crash events are represented in Exhibit 3-2. For the vast majority of events(i.e. movement of one or more vehicles and or pedestrians and cyclists) in the transportation system, events occur with low risk of a crash (i.e., the probability of a crash occurring is very low for most events on the transportation network).

In a smaller number of events, the potential risk of a crash occurring increases, such as an unexpected change in traffic flow on a freeway, a person crossing a road, or an unexpected object is observed on the roadway. In the majority of these situations, the potential for a crash is avoided by a driver's advance action, such as slowing down, changing lanes, or sounding a horn.

In even fewer events, the risk of a crash occurring increases even more. For instance, if a driver is momentarily not paying attention, the probability of a crash occurring increases. However, the accident could still be avoided, for example by coming to an emergency stop. Finally, in only a very few events, a crash occurs. For instance, in the previous example, the driver may have not applied the brakes in time to avoid a collision.

Circumstances that lead to a crash in one event will not necessary lead to a crash in a similar event. This reflects the randomness that is inherent in crashes.

Exhibit 3-2: Crashes are Rare and Random Events



3.2.4. Crash Contributing Factors

Section 3.2.4 introduces crash contributing factors.

While it is common to refer to the "cause" of a crash, in reality, most crashes cannot be related to a singular causal event. Instead, crashes are the result of a convergence of a series of events that are influenced by a number of contributing factors (time of day, driver attentiveness, speed, vehicle condition, road design etc). These contributing factors influence the sequence of events (described above) before, during and after a crash.

- **Before-crash events** - reveal factors that contributed to the risk of a crash occurring, and how the crash may have been prevented. For example whether the brakes of one or both of the vehicles involved were worn;

- **During-crash events** – reveal factors that contributed to the crash severity and how engineering solutions or technological changes could reduce crash severity For example whether a car has airbags and if the airbag deployed correctly;

- **After-crash events** – reveal factors influencing the outcome of the crash and how damage and injury may have been reduced by improvements in emergency response and medical treatment For example the time and quality of emergency response to a crash.

Crashes have the following three general categories of contributing factors:

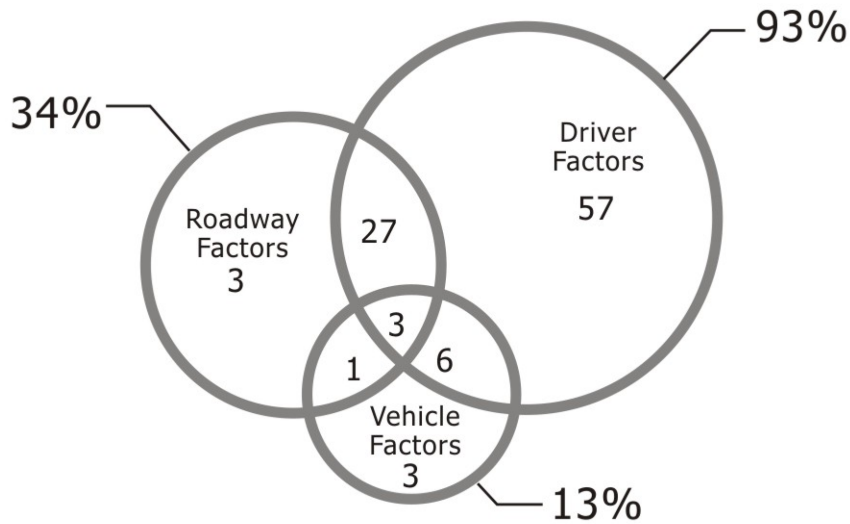
- **Human –** including age, judgment, driver skill, attention, fatigue, experience and sobriety;

- **Vehicle –** including design, manufacture and maintenance;

- **Roadway/Environment** – including geometric alignment, cross-section, traffic control devices, surface friction, grade, signage, weather, visibility.

By understanding these factors and how they might influence the sequence of events, crashes and crash severities can be reduced by implementing specific measures to target specific contributing factors. The relative contribution of these factors to crashes can assist with determining how to best allocate resources to reduce crashes. Research by Treat into the relative proportion of contributing factors is summarized in Exhibit 3-3[(10)]. The research was conducted in 1980 and therefore, the relative proportions are more informative than the actual values shown.

Exhibit 3-3: Contributing Factors to Vehicle Crashes



Source: Treat 1979

A framework for relating the series of events in a crash to the categories of crash contributing factors is the Haddon Matrix. Exhibit 3-4[(2)] provides an example of this matrix. The Haddon Matrix helps create order when determining which contributing factors influence a crash and which period of the crash the factors influence. The factors listed are not intended to be comprehensive; they are examples only.

The Haddon Matrix is a framework for identifying crash contributing factors.

Exhibit 3-4: Example Haddon Matrix for Identifying Contributing Factors

Period	Human Factors	Vehicle Factors	Roadway/Environment Factors
Before Crash Factors contributing to increased risk of crash	distraction, fatigue, inattention, poor judgment, age, cell phone use, deficient driving habits	worn tires, worn brakes	wet pavement, polished aggregate, steep downgrade, poorly coordinated signal system
During Crash Factors contributing to crash severity	vulnerability to injury, age, failure to wear a seat belt, driving speed, sobriety	bumper heights and energy adsorption, headrest design, airbag operations	pavement friction, grade, roadside environment
After Crash Factors contributing to crash outcome	age, gender	ease of removal of injured passengers	the time and quality of the emergency response, subsequent medical treatment

Considering the crash contributing factors and what period of a crash event they relate to supports the process of identifying appropriate crash reduction strategies. Some examples of how a reduction in crashes and crash severity may be achieved include:

- The behavior of humans;
- The condition of the roadway/environment;
- The design and maintenance of technology including vehicles, roadway and the environment technology;
- The provision of emergency medical treatment, medical treatment technology and post-crash rehabilitation;
- The exposure to travel, or level of transportation demand.

Strategies to influence the above and reduce crash and crash severity may include:

- **Design, Planning and Maintenance** - may reduce or eliminate crashes by improving and maintaining the transportation system, such as modifying signal phasing. Crash severity may also be reduced by selection of appropriate treatments (such as the use of median barriers to prevent head-on collisions).
- **Education** - may reduce crashes by influencing the behavior of humans including public awareness campaigns, driver training programs, and training of engineers and doctors.
- **Policy/Legislation** – may reduce crashes by influencing human behavior and design of roadway and vehicle technology. For example laws may prohibit cell phone use while driving, require minimum design standards, mandate use of helmets, and seatbelts.

- **Enforcement** – may reduce crashes by penalizing illegal behavior such as excessive speeding and drunken driving.

- **Technology Advances** – may reduce crashes and crash severity by minimizing the outcomes of a crash or preempting crashes from occurring altogether. For example, electronic stability control systems in vehicles improve the driver's ability to maintain control of a vehicle. The introduction of "Jaws of Life" tools (for removing injured persons from a vehicle) has reduced the time taken to provide emergency medical services.

- **Demand Management/Exposure reduction** – may reduce crashes by reducing the number of 'events' on the transportation system for which the risk of a crash may arise. For example, increasing the availability of mass transit reduces the number of passenger vehicles on the road and therefore a potential reduction in crash frequency may occur because of less exposure.

A direct relationship between individual contributing factors and particular strategies to reduce crashes does not exist. For example, in a head on crash on a two lane rural road in dry, well illuminated conditions, the roadway may not be considered as a contributing factor. However, the crash may have been prevented if the roadway was a divided road. Therefore while the roadway may not be listed as a contributing factor, changing the roadway design is one potential strategy to prevent similar accidents in the future.

While all of the above strategies play an important role in reducing crashes and crash severity, the majority of these strategies are beyond the scope of the HSM. The HSM focuses on the reduction of crashes and crash severity where it is believed that the roadway/environment is a contributing factor, either exclusively or through interactions with the vehicle and/or the driver.

3.3. DATA FOR CRASH ESTIMATION

This section describes the data that is typically collected and used for the purposes of crash analysis, and the limitations of observed crash data in the estimation of crashes and evaluation of crash reduction programs.

3.3.1. Data Needed for Crash Analysis

Accurate, detailed crash data, roadway or intersection inventory data, and traffic volume data are essential to undertake meaningful and statistically sound analyses. This data may include:

Typical data needs for crash analysis are: crash data, facility data, and traffic volume data.

- **Crash Data:** The data elements in a crash report describe the overall characteristics of the crash. While the specifics and level of detail of this data vary from state to state, in general, the most basic crash data consist of crash location, date and time, crash severity and collision type, and basic information about the roadway, vehicles and people involved.

- **Facility Data:** The roadway or intersection inventory data provide information about the physical characteristics of the accident site. The most basic roadway inventory data typically include roadway classification, number of lanes, length, and presence of medians and shoulder width. Intersection inventories typically include road names, area type, and traffic control and lane configurations.

- **Traffic Volume Data:** In most cases, the traffic volume data required for the methods in the HSM are annual average daily traffic (AADT). Some organizations may use ADT (average daily traffic) as precise data may not be available to determine AADT. If AADT data are unavailable, ADT can be used to estimate AADT. Other data that may be used for crash analysis includes intersection total entering vehicles (TEV), and vehicle-miles traveled (VMT) on a roadway segment, which is a measure of segment length and traffic volume. In some cases, additional volume data, such as pedestrian crossing counts or turning movement volumes, may be necessary.

The HSM Data Needs Guide[9] provides additional data information. In addition, in an effort to standardize databases related to crash analyses there are two guidelines published by FHWA: The Model Minimum Uniform Crash Criteria (MMUCC); and the Model Minimum Inventory of Roadway Elements (MMIRE). MMUCC (http://www.mmucc.us) is a set of voluntary guidelines to assist states in collecting consistent crash data. The goal of the MMUCC is that with standardized integrated databases, there can be consistent crash data analysis and transferability. MMIRE (http://www.mmire.org) provides guidance on what roadway inventory and traffic elements can be included in crash analysis, and proposes standardized coding for those elements. As with MMUCC, the goal of MMIRE is to provide transferability by standardizing database information.

3.3.2. Limitations of Observed Crash Data Accuracy

This section discusses the limitations of recording, reporting and measuring crash data with accuracy and consistency. These issues can introduce bias and affect crash estimation reliability in ways that are not easily addressed. These limitations are not specific to a particular crash analysis methodology and their implications require consideration regardless of the particular crash analysis methodology used.

Limitations of observed crash data include:

- Data quality and accuracy
- Crash reporting thresholds and the frequency-severity indeterminacy
- Differences in data collection methods and definitions used by jurisdictions

Limitations of typical crash data are summarized in Section 3.3.2.

Data Quality and Accuracy

Crash data are typically collected on standardized forms by trained police personnel and, in some states, by integrating information provided by citizens self-reporting PDO crashes. Not all crashes are reported, and not all reported crashes are recorded accurately. Errors may occur at any stage of the collection and recording of crash data and may be due to:

- **Data entry** - typographic errors;
- **Imprecise entry -** the use of general terms to describe a location;

- **Incorrect entry -** entry of road names, road surface, level of accident severity, vehicle types, impact description, etc.;
- **Incorrect training** -lack of training in use of collision codes;
- **Subjectivity -** Where data collection relies on the subjective opinion of an individual, inconsistency is likely. For example estimation of property damage thresholds, or excessive speed for conditions.

Crash Reporting Thresholds

Reported and recorded crashes are referred to as observed crash data in the HSM. One limitation on the accuracy of observed crash data is that all crashes are not reported. While a number of reasons for this may exist, a common reason is the use of minimum accident reporting thresholds.

Transportation agencies and jurisdictions typically use police accident reports as a source of observed crash records. In most states, crashes must be reported to police when damage is above a minimum dollar value threshold. This threshold varies between states. When thresholds change, the change in observed crash frequency does not necessarily represent a change in long term average crash frequency but rather creates a condition where comparisons between previous years can not be made.

To compensate for inflation, the minimum dollar value for accident reporting is periodically increased through legislation. Typically the increase is followed by a drop in the number of reported crashes. This decrease in reported crashes does not represent an increase in safety. It is important to be aware of crash reporting thresholds and to ensure that a change to reporting thresholds did not occur during the period of study under consideration.

Crash Reporting and the Frequency-Severity Indeterminacy

Not all reportable crashes are actually reported to police and therefore not all crashes are included in a crash database. In addition, studies indicate that crashes with greater severity are reported more reliably than crashes of lower severity. This situation creates an issue called frequency-severity indeterminancy, which represents the difficulty in determining if a change in the number of reported accidents is caused by an actual change in accidents, a shift in severity proportions, or a mixture of the two. It is important to recognize frequency-severity indeterminacy in measuring effectiveness of and selecting countermeasures. No quantitative tools currently exist to measure frequency-severity indeterminacy.

Differences between Crash Reporting Criteria of Jurisdictions

Differences exist between jurisdictions regarding how crashes are reported and classified. This especially affects the development of statistical models for different facility types using crash data from different jurisdictions, and the comparison or use of models across jurisdictions. Different definitions, criteria and methods of determining and measuring crash data may include:

- Crash reporting thresholds
- Definition of terms and criteria relating to crashes, traffic and geometric data

- Crash severity categories

Crash reporting thresholds were discussed above. Different definitions and terms relating to the three types of data (i.e. traffic volume, geometric design, and crash data) can create difficulties as it may be unclear whether the difference is limited to the terminology or whether the definitions and criteria for measuring a particular type of data is different. For example, most jurisdictions use annual average daily traffic (AADT) as an indicator of yearly traffic volume, others use average daily traffic (ADT).

Variation in crash severity terms can lead to difficulties in comparing data between states and development of models which are applicable to multiple states, for example, a fatal injury is defined by some agencies as “any injury that results in death within a specified period after the road vehicle accident in which the injury occurred. Typically the specified period is 30 days.”[7] In contrast, World Health Organization procedures, adopted for vital statistics reporting in the United States, use a 12-month limit. Simlarly, juridictions may use differing injury scales or have different severity classifications or groupings of classifications. These differences may lead to the inconsistencies in reported crash severity and the proportion of severe injury to fatalities across jursidictions.

Therefore, the count of reported crashes in a database is partial, may contain inaccurate or incomplete information, may not be uniform for all collision types and crash severities, may vary over time, and may differ from jurisdiction to jurisdiction.

3.3.3. Limitations Due To Randomness and Change

This section introduces regression to the mean concepts and issues associated with changes in site conditions (i.e., physical or traffic volume).

This section discusses the limitations associated with natural variations in crash data and the changes in site conditions. These are limitations due to inherent characteristics of the data itself , not limitations due to the method by which the data is collected or reported. If not considered and accounted for as possible, the limitations can introduce bias and affect crash data reliability in ways that are not easily accounted for. These limitations are not specific to a particular crash analysis methodology and their implications require consideration regardless of the particular crash analysis methodology being used.

Limitations due to randomness and changes include:

- Natural variability in crash frequency
- Regression-to-the-mean and regression-to-the-mean bias
- Variations in roadway characteristics
- Conflict between Crash Frequency Variability and Changing Site Conditions

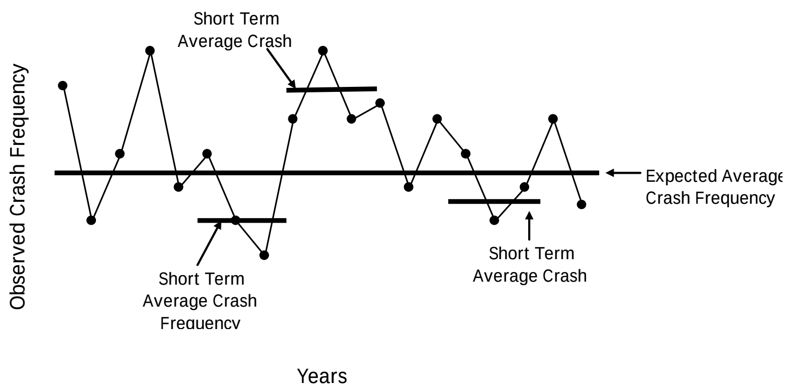
Natural Variability in Crash Frequency

Because crashes are random events, crash frequencies naturally fluctuate over time at any given site. The randomness of accident occurrence indicates that short-term crash frequencies alone are not a reliable estimator of long-term crash frequency. If a three-year period of crashes were used as the sample to estimate crash frequency, it would be difficult to know if this three-year period represents a typically high, average, or low crash frequency at the site.

This year-to-year variability in crash frequencies adversely affects crash estimation based on crash data collected over short periods. The short-term average crash frequency may vary significantly from the long-term average crash frequency. This effect is magnified at study locations with low crash frequencies where changes due to variability in crash frequencies represent an even larger fluctuation relative to the expected average crash frequency.

Exhibit 3-5 demonstrates the randomness of observed crash frequency, and limitation of estimating crash frequency based on short-term observations.

Exhibit 3-5: Variation in Short-Term Observed Crash Frequency



Regression-to-the-Mean and Regression-to-the-Mean Bias

The crash fluctuation over time makes it difficult to determine whether changes in the observed crash frequency are due to changes in site conditions or are due to natural fluctuations. When a period with a comparatively high crash frequency is observed, is statistically probable that the following period will be followed by a comparatively low crash frequency [8]. This tendency is known as regression-to-the-mean (RTM), and also applies to the high probability that a low crash frequency period will be followed by a high crash frequency period.

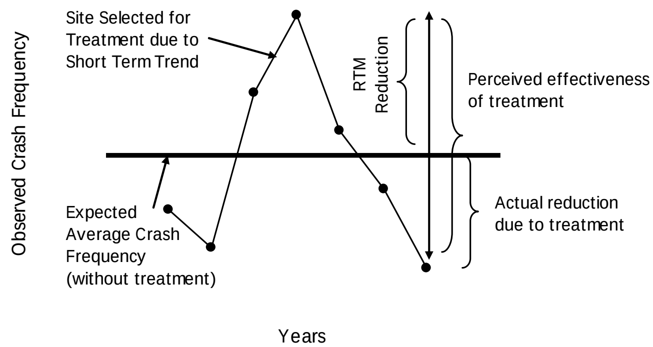
Chapter 4 and Part C of the HSM introduce crash estimation methods that address regression-to-the-mean.

Failure to account for the effects of RTM introduces the potential for "RTM bias", also known as "selection bias". Selection bias occurs when sites are selected for treatment based on short-term trends in observed crash frequency. For example, a site is selected for treatment based on a high observed crash frequency during a very short period of time (e.g. two years). However, the sites long-term crash frequency may actually be substantially lower and therefore the treatment may have been more cost effective at an alternate site. RTM bias can also result in the overestimation or underestimation of the effectiveness of a treatment (i.e., the change in expected average crash frequency). Without accounting for RTM bias, it is not possible to know if an observed reduction in crashes is due to the treatment or if it would have occurred without the modification.

The effect of RTM and RTM bias in evaluation of treatment effectiveness is shown on Exhibit 3-6. In this example, a site is selected for treatment based on its short term crash frequency trend over three years (which is trending upwards). Due to regression-to-the-mean, it is probable that the observed crash frequency will actually decrease (towards the expected average crash frequency) without any treatment. A treatment is applied, which has a beneficial effect (i.e., there is a reduction in crashes due to the treatment). However, if the reduction in crash frequency that would have occurred (due to RTM) without the treatment is ignored the effectiveness of the treatment is perceived to be greater than its actual effectiveness.

The effect of RTM bias is accounted for when treatment effectiveness (i.e., reduction in crash frequency or severity) and site selection is based on a long-term average crash frequency. Because of the short-term year-to-year variability in observed crash frequency, and consequences of not accounting for RTM bias, the HSM focuses on estimating of the ″expected average crash frequency″ as defined in section 3.2.4.

Exhibit 3-6 Regression-to-the-mean (RTM) and RTM Bias



Variations in Roadway Characteristics and Environment

A site's characteristics, such as traffic volume, weather, traffic control, land use and geometric design, are subject to change over time. Some conditions, such as traffic control or geometry changes at an intersection, are discrete events. Other characteristics, like traffic volume and weather, change on a continual basis.

The variation of site conditions over time makes it difficult to attribute changes in the expected average crash frequency to specific conditions. It also limits the number of years that can be included in a study. If longer time periods are studied (to improve the estimation of crash frequency and account for natural variability and RTM), it becomes likely that changes in conditions at the site occurred during the study period. One way to address this limitation is to estimate the expected average crash frequency for the specific conditions for each year in a study period. This is the predictive method applied in *Part C* of the HSM.

Variation in conditions also plays a role in evaluation of the effectiveness of a treatment. Changes in conditions between a "before" period and an "after" period may make it difficult to determine the actual effectiveness of a particular treatment. This may mean that a treatments effect may be over or under estimated, or unable to be determined. More information about this is included in *Chapter 9*.

Conflict between Crash Frequency Variability and Changing Site Conditions

The implications of crash frequency fluctuation and variation of site conditions are often in conflict. On one hand, the year-to-year fluctuation in crash frequencies tends toward acquiring more years of data to determine the expected average crash frequency. On the other hand, changes in site conditions can shorten the length of time for which crash frequencies are valid for considering averages. This push/pull relationship requires considerable judgment when undertaking large-scale analyses and using crash estimation procedures based on observed crash frequency. This limitation can be addressed by estimating the expected average crash frequency for the specific conditions for each year in a study period, which is the predictive method applied in *Part C* of the HSM.

3.4. EVOLUTION OF CRASH ESTIMATION METHODS

This section provides a brief overview of the evolution of crash estimation methods and their strengths and limitations. The development of new crash estimation methods is associated not only with increasing sophistication of the statistical techniques, but is also due to changes in the thinking about road safety. Additional information is included in Appendix A. The following crash estimation methods are discussed:

- Crash estimation using observed crash frequency and crash rates over a short-term period, and a long term period (e.g., more than 10 years);
- Indirect safety measures for identifying high crash locations. Indirect safety measures are also known as surrogate measures;
- Statistical analysis techniques (specifically the development of statistical regression models for estimation of crash frequency), and statistical methodologies to incorporate observed crash data to improve the reliability of crash estimation models.

3.4.1. Observed Crash Frequency and Crash Rate Methods

Crash frequency and crash rates are often used for crash estimation and evaluation of treatment effectiveness. In the HSM, the historic crash data on any facility (i.e., the number of recorded crashes in a given period) is referred to as the "observed crash frequency".

"Crash rate" is the number of crashes that occur at a given site during a certain time period in relation to a particular measure of exposure (e.g., per million vehicle miles of travel for a roadway segment or per million entering vehicles for an intersection). Crash rates may be interpreted as the probability (based on past events) of being involved in an accident per instance of the exposure measure. For example, if the crash rate on a roadway segment is one crash per one million vehicle miles per year, then a vehicle has a one-in-a-million chance of being in an accident for every mile traveled on that roadway segment. Crash rates are calculated according to Equation 3-2.

$$Crash\ Rate = \frac{Average\ Crash\ Frequency\ in\ a\ Period}{Exposure\ in\ Same\ Period} \tag{3-2}$$

Observed crash frequency and crash rates are often used as a tool to identify and prioritize sites in need of modifications, and for evaluation of the effectiveness of treatments. Typically, those sites with the highest crash rate or perhaps with rates higher than a certain threshold are analyzed in detail to identify potential modifications to reduce crashes. In addition, crash frequency and crash rate are often used in conjunction with other analysis techniques, such as reviewing crash records by year, collision type, crash severity, and/or environmental conditions to identify other apparent trends or patterns over time. Chapter 3 Appendix A.3 provides examples of crash estimation using historic crash data.

Advantages in the use of observed crash frequency and crash rates include:

- Understandability –observed crash frequency and rates are intuitive to most members of the public;
- Acceptance – it is intuitive for members of the public to assume that observed trends will continue to occur;
- Limited alternatives – in the absence of any other available methodology, observed crash frequency is the only available method of estimation.

Crash estimation methods based solely on historical crash data are subject to a number of limitations. These include the limitations associated with the collection of data described in section 3.3.2 and 3.3.3.

Also, the use of crash rate incorrectly assumes a linear relationship between crash frequency and the measure of exposure. Research has confirmed that while there are often strong relationships between crashes and many measures of exposure, these relationships are usually non-linear.[1,5,11]

A (theoretical) example which illustrates how crash rates can be misleading is to consider a rural two-lane two-way road with low traffic volumes with a very low observed crash frequency. Additional development may substantially increase the traffic volumes and consequently the number of crashes. However, it is likely that the crash rate may decline because the increased traffic volumes. For example the traffic volumes may increase threefold, but the observed crash frequency may only double, leading to a one third reduction in crash rate. If this change isn't accounted for, one might assume that the new development made the roadway safer.

Not accounting for the limitations described above may result in ineffective use of limited safety funding. Further, estimating crash conditions based solely on observed crash data limits crash estimation to the expected average crash frequency of an existing site where conditions (and traffic volumes) are likely to remain constant for a long-term period, which is rarely the case. This precludes the ability to estimate the expected average crash frequency for:

- The existing system under different geometric design or traffic volumes in the past (considering if a treatment had not been implemented) or in the future (in considering alternative treatment designs);
- Design alternatives of roadways that have not been constructed.

As the number of years of available crash data increases the risk of issues associated with regression-to-the-mean bias decrease. Therefore, in situations where crashes are extremely rare (e.g., at rail-grade crossings) observed crash frequency or crash rates may reliably estimate expected average crash frequency and therefore can be used as a comparative value for ranking (see Chapter 3 Appendix A.4 for further discussion).

Even when there have been limited changes at a site (e.g., traffic volume, land use, weather, driver demographics have remained constant) other limitations relating to changing contributing factors remain. For example the use of motorcycles may have increased across the network during the study period. An increase in observed motorcycle crashes at the site may be associated with the overall change in levels of motorcycle use across the network rather than in increase in motorcycle crashes at the specific site.

Agencies may be subject to reporting requirements which require provision of crash rate information. The evolution of crash estimation methods introduces new concepts with greater reliability than crash rates, and therefore the HSM does not focus on the use of crash rates. The techniques and methodologies presented in the HSM 1st Edition are relatively new to the field of transportation and will take time to become "best" practice. Therefore it is likely that agencies may continue to be subject to requirements to report crash rates in the near term.

3.4.2. Indirect Safety Measures

Indirect safety measures have also been applied to measure and monitor a site or a number of sites. Also known as surrogate safety measures, indirect safety measures provide a surrogate methodology when accident frequencies are not available because the roadway or facility is not yet in service or has only been in service for a short time; or when crash frequencies are low or have not been collected; or when a roadway or facility has significant unique features. The important added attraction of indirect safety measurements is that they may save having to wait for sufficient accidents to materialize before a problem is recognized and a remedy applied.

Past practices have mostly used two basic types of surrogate measures to use in place of observed crash frequency. These are:

- Surrogates based on events which are proximate to and usually precede the accident event. For example, at an intersection encroachment time, the time during which a turning vehicle infringes on the right of way of another vehicle may be used as a surrogate estimate.
- Surrogates that presume existence of a causal link to expected accident frequency. For example, proportion of occupants wearing seatbelts may be used as a surrogate for estimation of crash severities.

Conflict studies are another indirect measurement of safety. In these studies, direct observation of a site is conducted in order to examine "near-accidents" as an indirect measure of potential crash problems at a site. Because the HSM is focused on quantitative crash information, conflict studies are not included in the HSM.

The strength of indirect safety measures is that the data for analysis is more readily available. There is no need to wait for crashes to occur. The limitations of indirect safety measures include the often unproven relationship between the surrogate events and crash estimation. Chapter 3 Appendix D provides more detailed information about indirect safety measures.

3.4.3. Crash Estimation using Statistical Methods

Statistical models using regression analysis have been developed which address some of the limitations of other methods identified above. These models address RTM bias and also provide the ability to reliably estimate expected average crash frequency for not only existing roadway conditions, but also changes to existing conditions or a new roadway design prior to its construction and use.

As with all statistical methods used to make estimation, the reliability of the model is partially a function of how well the model fits the original data and partially a function of how well the model has been calibrated to local data. In addition to statistical models based on crash data from a range of similar sites, the reliability of crash estimation is improved when historic crash data for a specific site can be incorporated into the results of the model estimation.

A number of statistical methods exist for combining estimates of crashes from a statistical model with the estimate using observed crash frequency at a site or facility. These include:

- Empirical Bayes method (EB Method)
- Hierarchical Bayes method
- Full Bayes method

Jurisdictions may have the data and expertise to develop their own models and to implement these statistical methods. In the HSM, the EB Method is used as part of the predictive method described in *Part C*. A distinct advantage of the EB Method is that, once a calibrated model is developed for a particular site type, the method can be readily applied. The Hierarchical Bayes and Full Bayes method are not used in the HSM, and are not discussed within this manual.

3.4.4. Development and Content of the HSM Methods

Section 3.3 through 3.4.3 discuss the limitations related to the use of observed crash data in crash analysis and some of the various methods for crash estimation which have evolved as the field of crash estimation has matured. The HSM has been developed due to recognition amongst transportation professionals of the need to develop standardized quantitative methods for crash estimation and crash evaluation which address the limitations described in Section 3.3.

The HSM provides quantitative methods to reliably estimate crash frequencies and severities for a range of situations, and to provide related decision making tools to use within the road safety management process. *Part A* of the HSM provides an overview of Human Factors (in *Chapter 2*) and an introduction to the fundamental concepts used in the HSM (Chapter 3). *Part B* of the HSM focuses on methods to establish a comprehensive and continuous roadway safety management process. *Chapter 4* provides numerous performance measures for identifying sites which may respond to improvements. Some of these performance measures use concepts presented in the overview of the *Part C* predictive method presented below. *Chapters 5* through *8* present information about site crash diagnosis, selecting countermeasures, and prioritizing sites. *Chapter 9* presents methods for evaluating the effectiveness of improvements. Fundamentals of the *Chapter 9* concepts are presented in Section 3.7.

Part C of the HSM, overviewed in Section 3.5, presents the predictive method for estimating the expected average crash frequency for various roadway conditions. The material in this part of the HSM will be valuable in preliminary and final design processes.

Finally, *Part D* contains a variety of roadway treatments with accident modification factors (AMFs). The fundamentals of AMFs are described in Section 3.6, with more details provided in the *Part D Introduction and Applications Guidance.*

3.5. PREDICTIVE METHOD IN PART C OF THE HSM

3.5.1. Overview of the Part C Predictive Method

This section is intended to provide the user with a basic understanding of the predictive method found in Part C of the HSM. A complete overview of the method is provided in the Part C Introduction and Application Guidance. The detail method for specific facility types is described in *Chapter 10, 11* and *12* and the EB Method is explained fully in the *Part C* Appendix.

A detailed explanation of the steps for the HSM predictive method is in the Part C Introduction and Applications Guide.

The predictive method presented in *Part C* provides a structured methodology to estimate the expected average crash frequency (by total crashes, crash severity or collision type) of a site, facility or roadway network for a given time period, geometric design and traffic control features, and traffic volumes (AADT). The predictive method also allows for crash estimation in situations where no observed crash data is available or no predictive model is available.

The expected average crash frequency, $N_{expected}$, is estimated using a predictive model estimate of crash frequency, $N_{predicted}$ (referred to as the predicted average crash frequency) and, where available, observed crash frequency, $N_{observed}$. The basic elements of the predictive method are:

- Predictive model estimate of the average crash frequency for a specific site type. This is done using a statistical model developed from data for a number of similar sites. The model is adjusted to account for specific site conditions and local conditions;

- The use of the EB Method to combine the estimation from the statistical model with observed crash frequency at the specific site. A weighting factor is applied to the two estimates to reflect the model's statistical reliability. When observed crash data is not available or applicable, the EB Method does not apply.

Basic Elements of the Predictive Models in Part C

The predictive models in *Part C* of the HSM vary by facility and site type but all have the same basic elements:

- Safety Performance Functions (SPFs): statistical "base" models are used to estimate the average crash frequency for a facility type with specified base conditions.

- Accident Modification Factors (AMFs): AMFs are the ratio of the effectiveness of one condition in comparison to another condition. AMFs are multiplied with the crash frequency predicted by the SPF to account for the difference between site conditions and specified base conditions;

- Calibration factor (C): multiplied with the crash frequency predicted by the SPF to account for differences between the jurisdiction and time period for which the predictive models were developed and the jurisdiction and time period to which they are applied by HSM users.

While the functional form of the SPFs varies in the HSM, the predictive model to estimate the expected average crash frequency $N_{predicted}$, is generally calculated using Equation 3-3.

$$N_{predicted} = N_{SPF\ x} \times (AMF_{1x} \times AMF_{2x} \times \ldots \times AMF_{yx}) \times C_x \qquad (3\text{-}3)$$

Where,

$N_{predicted}$ = predictive model estimate of crash frequency for a specific year on site type *x* (crashes/year);

$N_{SPF\ x}$ = predicted average crash frequency determined for base conditions with the Safety Performance Function representing site type *x* (crashes/year);

AMF_{yx} = Accident Modification Factors specific to site type *x*;

C_x = Calibration Factor to adjust for local conditions for site type *x*.

The First Edition of the HSM provides a detailed predictive method for the following three facility types:

- Chapter 10: Rural Two-Lane Two-Way Roads;
- Chapter 11: Rural Multilane Highways;
- Chapter 12: Urban and Suburban Arterials.

This section presents the advantages of the HSM predictive method.

Advantages of the Predictive Method

Advantages of the predictive method are that:

- Regression-to-the-mean bias is addressed as the method concentrates on long-term expected average crash frequency rather than short-term observed crash frequency.
- Reliance on availability of limited crash data for any one site is reduced by incorporating predictive relationships based on data from many similar sites.
- The method accounts for the fundamentally nonlinear relationship between crash frequency and traffic volume.
- The SPFs in the HSM are based on the negative binomial distribution, which are better suited to modeling the high natural variability of crash data than traditional modeling techniques which are based on the normal distribution.

First time users of the HSM who wish to apply the predictive method are advised to read Section 3.5 (this section), read the Part C *Introduction and Applications Guidance*, and then select an appropriate facility type from *Chapter 10, 11*, or *12* for the roadway network, facility or site under consideration.

3.5.2. Safety Performance Functions

Safety Performance Functions (SPFs) are regression equations that estimate the average crash frequency for a specific site type (with specified base conditions) as a function of annual average daily traffic (AADT) and, in the case of roadway segments, the segment length (L). Base conditions are specified for each SPF and may include conditions such as lane width, presence or absence of lighting, presence of turn lanes etc. An example of a SPF (for roadway segments on rural two-lane highways) is shown in Equation 3-4.

$$N_{SPF\ rs} = (AADT) \times (L) \times (365) \times 10^{(-6)} \times e^{(-0.4865)} \tag{3-4}$$

Where,

$N_{spf\ rs}$ = estimate of predicted average crash frequency for SPF base conditions for a rural two-lane two-way roadway segment (described in Section 10.6) (crashes/year);

AADT = average annual daily traffic volume (vehicles per day) on roadway segment;

L = length of roadway segment (miles).

While the SPFs estimate the average crash frequency for all crashes, the predictive method provides procedures to separate the estimated crash frequency into components by crash severity levels and collision types (such as run-off-road or rear-end crashes). In most instances, this is accomplished with default distributions of crash severity level and/or collision type. As these distributions will vary between jurisdictions, the estimations will benefit from updates based on local crash severity and collision type data. This process is explained in the *Part C* Appendix. If sufficient experience exists within an agency, some agencies have chosen to use advanced statistical approaches that allow for prediction of changes by severity levels.[6]

The SPFs in the HSM have been developed for three facility types (rural two-lane two-way roads, rural multilane highways, and urban and suburban arterials), and for specific site types of each facility type (e.g. signalized intersections, unsignalized intersections, divided roadway segments and undivided roadway segments). The different facility types and site types for which SPFs are included in the HSM are summarized in Exhibit 3-9.

Exhibit 3.9 shows the Safety Performance Functions in Part C.

Exhibit 3-9: Facility Types and Site Types included in Part C

HSM Chapter	Undivided Roadway Segments	Divided Roadway Segments	Intersections: Stop Control on Minor Leg(s)		Intersections: Signalized	
			3-Leg	4-Leg	3-Leg	4-Leg
10 – Rural Two-Lane Roads	✓	-	✓	✓	-	✓
11 – Rural Multilane Highways	✓	✓	✓	✓	-	✓
12 – Urban and Suburban Arterial Highways	✓	✓	✓	✓	✓	✓

In order to apply a SPF the following information about the site under consideration is necessary:

- Basic geometric and geographic information of the site to determine the facility type and to determine whether a SPF is available for that facility and site type.
- Detailed geometric design and traffic control features conditions of the site to determine whether and how the site conditions vary from the SPF baseline conditions (the specific information required for each SPF is included in *Part C*.
- AADT information for estimation of past periods, or forecast estimates of AADT for estimation of future periods.

SPFs are developed through statistical multiple regression techniques using observed crash data collected over a number of years at sites with similar characteristics and covering a wide range of AADTs. The regression parameters of the SPFs are determined by assuming that crash frequencies follow a negative binomial distribution. The negative binomial distribution is an extension of the Poisson distribution, and is better suited than the Poisson distribution to modeling of crash data. The Poisson distribution is appropriate when the mean and the variance of the data are equal. For crash data, the variance typically exceeds the mean. Data for which the variance exceeds the mean are said to be overdispersed, and the negative binomial distribution is very well suited to modeling overdispersed data. The degree of overdispersion in a negative binomial model is represented by a statistical parameter, known as the *overdispersion parameter* that is estimated along with the coefficients of the regression equation. The larger the value of the overdispersion parameter, the more the crash data vary as compared to a Poisson distribution with the same mean. The overdispersion parameter is used to determine the value of a weight factor for use in the EB Method described in Section 3.5.5.

The SPFs in the HSM must be calibrated to local conditions as described in Section 3.5.4 below and in detail in the *Part C* Appendix. The derivation of SPFs through regression analysis is described in Chapter 3 Appendix B.

3.5.3. Accident Modification Factors

AMFs are the ratio of the expected average crash frequency of a site under one condition (such as a treatment) to the expected average crash frequency of the same site under a different condition. The different condition is often the base condition.

Accident Modification Factors (AMFs) represent the relative change in crash frequency due to a change in one specific condition (when all other conditions and site characteristics remain constant). AMFs are the ratio of the crash frequency of a site under two different conditions Therefore, an AMF may serve as an estimate of the effect of a particular geometric design or traffic control feature or the effectiveness of a particular treatment or condition.

AMFs are generally presented for the implementation of a particular treatment, also known as a countermeasure, intervention, action, or alternative design. Examples include illuminating an unlighted road segment, paving gravel shoulders, signalizing a stop-controlled intersection, or choosing a signal cycle time of 70 seconds instead of 80 seconds. AMFs have also been developed for conditions that are not associated with the roadway, but represent geographic or demographic conditions surrounding the site or with users of the site (e.g., the number of liquor outlets in proximity to the site).

Equation 3-5 shows the calculation of an AMF for the change in expected average crash frequency from site condition 'a' to site condition 'b'.[(3)]

$$AMF = \frac{\text{Expected average crash frequency with condition 'b'}}{\text{Expected average crash frequency with condition 'a'}} \qquad (3\text{-}5)$$

AMFs defined in this way for expected crashes can also be applied to comparison of predicted crashes between site condition 'a' and site condition 'b'.

Accident Modification Factor Examples

Example 1

Using a SPF for rural two-lane roadway segments, the expected average crash frequency for existing conditions is 10 injury crashes/year (assume observed data is not available). The base condition is the absence of automated speed enforcement. If automated speed enforcement were installed, the AMF for injury crashes is 0.83. Therefore, if there is no change to the site conditions other than implementation of automated speed enforcement, the estimate of expected average injury crash frequency is 0.83 x 10 = 8.3 crashes/year.

Example 2

The expected average crashes for an existing signalized intersection is estimated through application of the EB Method (using a SPF and observed crash frequency) to be 20 crashes/year. It is planned to replace the signalized intersection with a modern roundabout. The AMF for conversion of the base condition of an existing signalized intersection to a modern roundabout is 0.52. As no SPF is available for roundabouts, the project AMF is applied to the estimate for existing conditions. Therefore, after installation of a roundabout the expected average crash frequency, is estimated to be 0.52 x 20 = 10.4 crashes/year.

The values of AMFs in the HSM are determined for a specified set of base conditions. These base conditions serve the role of site condition 'a' in Equation 3-5. This allows comparison of treatment options against a specified reference condition. Under the base conditions (i.e., with no change in the conditions), the value of an AMF is 1.00. AMF values less than 1.00 indicate the alternative treatment reduces the estimated average crash frequency in comparison to the base condition. AMF values greater than 1.00 indicate the alternative treatment increases the estimated average crash frequency in comparison to the base condition. The relationship between an AMF and the expected percent change in crash frequency is shown in Equation 3-6.

$$\textit{Percent Reduction in Accidents} = 100 \times (1.00 - AMF) \qquad (3\text{-}6)$$

For example,

- If an AMF = 0.90 then the expected percent change is 100% × (1.00 - 0.90) = 10%, indicating a reduction in expected average crash frequency.
- If an AMF = 1.20 then the expected percent change is 100% × (1.00 - 1.20) = -20%, indicating an increase in expected average crash frequency.

The SPFs and AMFs used in the *Part C* predictive method for a given facility type use the same base conditions so that they are compatible.

Application of AMFs

Applications for AMFs include:

- Multiplying an AMF with a crash frequency for base conditions determined with a SPF to estimate predicted average crash frequency for an individual site, which may consist of existing conditions, alternative conditions or new site conditions. The AMFs are used to account for the difference between the base conditions and actual site conditions;
- Multiplying an AMF with the expected average crash frequency of an existing site that is being considered for treatment, when a site-specific SPF applicable to the treated site is not available. This estimates expected average crash frequency of the treated site. For example an AMF for a change in site type or conditions such as the change from an unsignalized intersection to a roundabout can be used if no SPF is available for the proposed site type or conditions;
- Multiplying an AMF with the observed crash frequency of an existing site that is being considered for treatment to estimate the change in expected average crash frequency due to application of a treatment, when a site-specific SPF applicable to the treated site is not available.

Application of an AMF will provide an estimate of the change in crashes due to a treatment. There will be variance in results at any particular location.

Applying Multiple AMFs

The predictive method assumes that AMFs can be multiplied together to estimate the combined effects of the respective elements or treatments. This approach assumes that the individual elements or treatments considered in the analysis are independent of one another. Limited research exists regarding the independence of individual treatments from one another.

AMFs are multiplicative even when a treatment can be implemented to various degrees such that a treatment is applied several times over. For example, a 4% grade can be decreased to 3%, 2%, and so on, or a 6-foot shoulder can be widened by 1-ft, 2-ft, and so on. When consecutive increments have the same degree of effect, Equation 3-7 can be applied to determine the treatment's cumulative effect.

$$AMF\ (for\ n\ increments) = [AMF(for\ one\ increment)]^{(n)} \qquad (3\text{-}7)$$

This relationship is also valid for non-integer values of n.

Applying Multiplicative Accident Modification Factors

Example 1

Treatment 'x' consists of providing a left-turn lane on both major-road approaches to an urban four-leg signalized intersection and treatment 'y' is permitting right-turn-on-red maneuvers. These treatments are to be implemented and it is assumed that their effects are independent of each other. An urban four-leg signalized intersection is expected to have 7.9 accidents/year. For treatment t_x, AMF_x = 0.81; for treatment t_y, AMF_y = 1.07.

What accident frequency is to be expected if treatment x and y are both implemented?

Answer to Example 1

Using Equation 3-7, expected accidents = 7.9 x 0.81 x 1.07 = 6.8 accidents/year.

Example 2

The AMF for single-vehicle run-off-road accidents for a 1% increase in grade is 1.04 regardless of whether the increase is from 1% to 2% or from 5% to 6%. What is the effect of increasing the grade from 2% to 4%?

Answer to Example 2

Using Equation 3-8, expected single-vehicle run-off-road accidents will increase by a factor of $1.04^{(4-2)} = 1.04^2$ =1.08 = 8% increase.

Multiplication of AMFs in Part C

In the *Part C* predictive method, a SPF estimate is multiplied by a series of AMFs to adjust the estimate of crash frequency from the base condition to the specific conditions present at a site. The AMFs are multiplicative because the effects of the features they represent are presumed to be independent. However, little research exists regarding the independence of these effects, but this is a reasonable assumption based on current knowledge. The use of observed crash frequency data in the EB Method can help to compensate for bias caused by lack of independence of the AMFs. As new research is completed, future HSM editions may be able to address the independence (or lack of independence) of these effects more fully.

Multiplication of AMFs in Part D

AMFs are also used in estimating the anticipated effects of proposed future treatments or countermeasures (e.g., in some of the methods discussed in Section C.8). The limited understanding of interrelationships between the various treatments presented in *Part D* requires consideration, especially when more than three AMFs are proposed. If AMFs are multiplied together, it is possible to overestimate the combined affect of multiple treatments when it is expected that more than one of the treatments may affect the same type of crash. The implementation of wider lanes and wider shoulders along a corridor is an example of a combined treatment where the independence of the individual treatments is unclear, because both treatments are expected to reduce the same crash types. When AMFs are multiplied, the practitioner accepts the assumption that the effects represented by the AMFs are independent of one another. Users should exercise engineering judgement to assess the interrelationship and/or independence of individual elements or treatments being considered for implementation.

Compatibility of Multiple AMFs

Engineering judgment is required to assess inter-relationships of AMFs and to assess the benefits of applying multiple AMFs.

Engineering judgment is also necessary in the use of combined AMFs where multiple treatments change the overall nature or character of the site; in this case, certain AMFs used in the analysis of the existing site conditions and the proposed treatment may not be compatible. An example of this concern is the installation of a roundabout at an urban two-way stop-controlled or signalized intersection. The procedure for estimating the crash frequency after installation of a roundabout (see *Chapter 12*) is to estimate the average crash frequency for the existing site conditions (as a SPF for roundabouts in currently unavailable) and then apply an AMF for a conventional intersection to roundabout conversion. Installing a roundabout changes the nature of the site so that other AMFs applicable to existing urban two-way stop-controlled or signalized intersections may no longer be relevant.

AMFs and Standard Error

The standard error is the standard deviation of the sample mean. The standard deviation is a measure of the spread of the sample data from the sample mean.

The standard error of an estimated value serves as a measure of the reliability of that estimate. The smaller the standard error, the more reliable (less error) the estimate becomes. All AMF values are estimates of the change in expected average crash frequency due to a change in one specific condition. Some AMFs in the HSM include a standard error, indicating the variability of the AMF estimation in relation to sample data values.

Standard error can also be used to calculate a confidence interval for the estimated change in expected average crash frequency. Confidence intervals can be calculated using Equation 3-8 and values from Exhibit 3-10.

$$CI\ (y\%) = AMF_x \pm SE_x \times MSE \qquad (3\text{-}8)$$

Where,

CI(y%) = the confidence interval for which it is y-percent probable that the true value of the AMF is within the interval;

AMF_x = Accident Modification Factor for condition x;

SE_x= Standard Error of the AMF_x;

MSE = Multiple of Standard Error (see Exhibit 3-10 for values).

Exhibit 3-10: Values for Determining Confidence Intervals using Standard Error

Desired Level of Confidence	Confidence Interval (probability that the true value is within the confidence interval)	Multiples of Standard Error (MSE) to use in Equation 3-8
Low	65-70%	1
Medium	95%	2
High	99.9%	3

AMF Confidence Intervals Using Standard Error

Situation

Roundabouts have been identified as a potential treatment to reduce the estimated average crash frequency for all crashes at a two-way stop-controlled intersection. Research has shown that the AMF for this treatment is 0.22 with a standard error of 0.07.

Confidence Intervals

The AMF estimates that installing a roundabout will reduce expected average crash frequency by 100 x (1 - 0.22) = 78%.

Using a Low Level of Confidence (65-70% probability) the estimated reduction at the site will be 78% ± 1 x 100 x 0.07%, or between 71% and 85%.

Using a High Level of Confidence (i.e., 99.9% probability) the estimated reduction at the site will be 78% ± 3 x 100 x 0.07%, or between 57% and 99%.

As can be seen in the above confidence interval estimates, the higher the level of confidence desired, the greater the range of estimated values.

The Chapter 3 Appendix C provides information of how an AMF and its standard error affect the probability that the AMF will achieve the estimated results.

AMFs in the HSM

AMF values in the HSM are either presented in text (typically where there are a limited range of options for a particular treatment), in formula (typically where treatment options are continuous variables) or in tabular form (where the AMF values vary by facility type, or are in discrete categories). Where AMFs are presented as a discrete value they are shown rounded to two decimal places. Where an AMF is determined using an equation or graph, it must also be rounded to two decimal places. A standard error is provided for some AMFs.

All AMFs in the HSM were selected by an inclusion process or from the results of an expert panel review. *Part D* contains all AMFs in the HSM, and the *Part D Introduction and Applications Guidance* chapter provides an overview of the AMF inclusion process and expert panel review process. All AMFs in *Part D* are presented with some combination of the following information:

Part D contains all AMFs in the HSM. The Part D Introduction and Applications Guidance chapter provides an overview of how the AMFs were developed.

- Base conditions, or when the AMF = 1.00;
- Setting and road type for which the AMF is applicable;
- AADT range in which the AMF is applicable;
- Accident type and severity addressed by the AMF;
- Quantitative value of the AMF;
- Standard error of the AMF;
- The source and studies on which the AMF value is based;
- The attributes of the original studies, if known.

This information presented for each AMF in *Part D* is important for proper application of the AMFs. AMFs in *Part C* are a subset of the *Part D* AMFs. The *Part C* AMFs have the same base conditions (i.e., AMF is 1.00 for base conditions) as their corresponding SPFs in *Part C*.

3.5.4. Calibration

The calibration procedure for the Part C predictive models is presented in the Appendix to Part C.

Crash frequencies, even for nominally similar roadway segments or intersections, can vary widely from one jurisdiction to another. Calibration is the process of adjusting the SPFs to reflect the differing crash frequencies between different jurisdictions. Calibration can be undertaken for a single state, or where appropriate, for a specific geographic region within a state.

Geographic regions may differ markedly in factors such as climate, animal population, driver populations, accident reporting threshold, and accident reporting practices. These variations may result in some jurisdictions experiencing different reported traffic accidents on a particular facility type than in other jurisdictions. In addition, some jurisdictions may have substantial variations in conditions between areas within the jurisdiction (e.g. snowy winter driving conditions in one part of the state and only wet winter driving conditions in another). Methods for calculating calibration factors for roadway segments C_r and intersections C_i are included in the *Part C* Appendix to allow highway agencies to adjust the SPF to match local conditions.

The calibration factors will have values greater than 1.0 for roadways that, on average, experience more accidents than the roadways used in developing the SPFs. The calibration factors for roadways that, on average, experience fewer accidents than the roadways used in the development of the SPF, will have values less than 1.0. The calibration procedures are presented in the Appendix to *Part C*.

Calibration factors provide one method of incorporating local data to improve estimated accident frequencies for individual agencies or locations. Several other default values used in the methodology, such as collision type distributions, can also be replaced with locally derived values. The derivation of values for these parameters is also addressed in the calibration procedure *Part C* Appendix A.1.

3.5.5. Weighting using the Empirical Bayes Method

The EB Method is presented in detail in the Part C Appendix.

Estimation of expected average crash frequency using only observed crash frequency or only estimation using a statistical model (such as the SPFs in *Part C*) may result in a reasonable estimate of crash frequency. However, as discussed in Section 3.4.3, the statistical reliability (the probability that the estimate is correct) is improved by combining observed crash frequency and the estimate of the average crash frequency from a predictive model. While a number of statistical methods exist that can compensate for the potential bias resulting from regression-to the mean, the predictive method in *Part C* uses the empirical Bayes method, herein referred to as the EB Method.

The EB Method uses a weight factor, which is a function of the SPF overdispersion parameter, to combine the two estimates into a weighted average.

The weighted adjustment is therefore dependant only on the variance of the SPF , and is not dependant on the validity of the observed crash data.

The EB Method is only applicable when both predicted and observed crash frequencies are available for the specific roadway network conditions for which the estimate is being made. It can be used to estimate expected average crash frequency for both past and future periods. The EB Method is applicable at either the site-specific level (where crashes can be assigned to a particular location) or the project specific level (where observed data may be known for a particular facility, but cannot be assigned to the site specific level). Where only a predicted or only observed crash data are available, the EB Method is not applicable (however the predictive method provides alternative estimation methods in these cases).

For an individual site the EB Method combines the observed crash frequency with the statistical model estimate using Equation 3-9:

$$N_{expected} = w \times N_{predicted} + (1 - w) \times N_{observed} \tag{3-9}$$

Where,

$N_{expected}$ = expected average crashes frequency for the study period.

$N_{predicted}$ = predicted average crash frequency predicted using a SPF for the study period under the given conditions.

w = weighted adjustment to be placed on the SPF prediction.

$N_{observed}$ = observed crash frequency at the site over the study period.

The weighted adjustment factor, w, is a function of the SPF's overdispersion parameter, k, and is calculated using Equation 3-10. The overdispersion parameter is of each SPF is stated in *Part C*.

$$w = \frac{1}{1 + k \times (\sum_{\substack{\text{all study} \\ \text{years}}} N_{Predicted})} \tag{3-10}$$

Where,

k = overdispersion parameter from the associated SPF.

As the value of the overdispersion parameter increases, the value of the weighted adjustment factor decreases. Thus, more emphasis is placed on the observed rather than the predicted crash frequency. When the data used to develop a model are greatly dispersed, the reliability of the resulting predicted crash frequency is likely to be lower. In this case, it is reasonable to place less weight on the predicted crash frequency and more weight on the observed crash frequency. On the other hand, when the data used to develop a model have little overdispersion, the reliability of the resulting SPF is likely to be higher. In this case, it is reasonable to place more weight on the predicted crash frequency and less weight on the observed crash frequency. A more detailed discussion of the EB Methods is presented in the Appendix to *Part C*.

3.5.6. Limitations of Part C Predictive Method

Limitations of the *Part C* predictive method are similar to all methodologies which include regression models: the estimations obtained are only as good as the quality of the model. Regression models do not necessarily always represent cause-and-effect relationships between crash frequency and the variables in the model. For this reason, the variables in the SPFs used in the HSM have been limited to AADT and roadway segment length, because the rationale for these variables having a cause-and-effect relationship to crash frequency is strong. SPFs are developed with observed crash data which, as previously described, has its own set of limitations.

SPFs vary in their ability to predict crash frequency; the SPFs used in the HSM are considered to be among the best available. SPFs are, by their nature, only directly representative of the sites that are used to develop them. Nevertheless, models developed in one jurisdiction are often applied in other jurisdictions. The calibration process provided in the *Part C* predictive method provides a method that agencies can use to adapt the SPFs to their own jurisdiction and to the time period for which they will be applied. Agencies with sufficient expertise may develop SPFs with data for their own jurisdiction for application in the *Part C* predictive method. Development of SPFs with local data is not a necessity for using the HSM. Guidance on development of SPFs using an agency's own data is presented in the *Part C Introduction and Applications Guidance*.

AMFs are used to adjust the crash frequencies predicted for base conditions to the actual site conditions. While multiple AMFs can be used in the predictive method, the interdependence of the effect of different treatment types on one another is not fully understood and engineering judgment is needed to assess when it is appropriate to use multiple AMFs (see Section 3.5.3).

3.6. APPLICATION OF THE HSM

The HSM provides methods for crash estimation for the purposes of making decisions relating to the design, planning, operation and maintenance of roadway networks.

These methods focus on the use of statistical methods in order to address the inherent randomness in crashes. Users do not need to have detailed knowledge of statistical analysis methods in order to understand and use the HSM. However, its use does require understanding of the following general principles:

- Observed crash frequency is an inherently random variable and it is not possible to predict the value for a specific period. The HSM estimates refer to the expected average crash frequency that would be observed if a site could be maintained under consistent conditions for a long-term period, which is rarely possible.
- Calibration of SPFs to local state conditions is an important step in the predictive method. Local and recent calibration factors may provide improved calibration.
- Engineering judgment is required in the use of all HSM procedures and methods, particularly selection and application of SPFs and AMFs to a given site condition.
- Errors and limitations exist in all crash data which affects both the observed crash data for a specific site and the models developed.
- Development of SPFs and AMFs requires understanding of statistical regression modeling and crash analysis techniques. The HSM does not provide sufficient detail and methodologies for users to develop their own SPFs or AMFs.

3.7. EFFECTIVENESS EVALUATION

3.7.1. Overview of Effectiveness Evaluation

Effectiveness evaluation is the process of developing quantitative estimates of the effect a treatment, project, or a group of projects has on expected average crash frequency. The effectiveness estimate for a project or treatment is a valuable piece of information for future decision-making and policy development. For instance, if a new type of treatment was installed at several pilot locations, the treatment's effectiveness evaluation can be used to determine if the treatment warrants application at additional locations.

Effectiveness evaluation may include:

- Evaluating a single project at a specific site to document the effectiveness of that specific project;
- Evaluating a group of similar projects to document the effectiveness of those projects;
- Evaluating a group of similar projects for the specific purpose of quantifying an AMF for a countermeasure;
- Assessing the overall effectiveness of specific types of projects or countermeasures in comparison to their costs.

Effectiveness evaluations may use several different types of performance measures, such as a percentage reduction in crash frequency, a shift in the proportions of crashes by collision type or severity level, an AMF for a treatment, or a comparison of the benefits achieved to the cost of a project or treatment.

As described in Section 3.3, various factors can limit the change in expected average crash frequency at a site or across a cross-section of sites that can be attributed to an implemented treatment. Regression-to-the-mean bias, as described in Section 3.3.3., can affect the perceived effectiveness (i.e., over or under estimate effectiveness) of a particular treatment if the study does not adequately account for the variability of observed crash data. This variability also necessitates acquiring a statistically valid sample size to validate the calculated effectiveness of the studied treatment.

Effectiveness evaluation techniques are presented in *Chapter 9*. The chapter presents statistical methods which provide improved estimates of the crash reduction benefits as compared to simple before-after studies. Simple before-after studies compare the count of crashes at a site before a modification to the count of crashes at a site after the modification to estimate the benefits of an improvement. This method relies on the (usually incorrect) assumption that site conditions have remained constant (e.g. weather, surrounding land use, driver demographics) and does not account for regression-to-the-mean bias. Discussion of the strengths and weaknesses of these methods are presented in *Chapter 9*.

Methods for safety effectiveness evaluation are presented in Chapter 9.

3.7.2. Effectiveness Evaluation Study Types

There are three basic study designs that can be used for effectiveness evaluations:

- Observational before/after studies
- Observational cross-sectional studies

- Experimental before/after studies

In observational studies, inferences are made from data observations for treatments that have been implemented in the normal course of the efforts to improve the road system. Treatments are not implemented specifically for evaluation. By contrast, experimental studies consider treatments that have been implemented specifically for evaluation of effectiveness. In experimental studies, sites that are potential candidates for improvement are randomly assigned to either a treatment group, at which the treatment of interest is implemented, or a comparison group, at which the treatment of interest is not implemented. Subsequent differences in crash frequency between the treatment and comparison groups can then be directly attributed to the treatment. Observational studies are much more common in road safety than experimental studies, because highway agencies operate with limited budgets and typically prioritize their projects based on benefits return. In this sense, random selection does not optimize investment selection and therefore agencies will typically not use this method, unless they are making system wide application of a countermeasure, such as rumble strips. For this reason, the focus of the HSM is on observational studies. The two types of observational studies are explained in further detail below.

Observational Before/After Studies

The scope of an observational before/after study is the evaluation of a treatment when the roadways or facilities are unchanged except for the implementation of the treatment. For example, the resurfacing of a roadway segment generally does not include changes to roadway geometry or other conditions. Similarly, the introduction of a seat belt law does not modify driver demography, travel patterns, vehicle performance or the road network. To conduct a before/after study, data are generally gathered from a group of roadways or facilities comparable in site characteristics where a treatment was implemented. Data are collected for specific time periods before and after the treatment was implemented. Crash data can often be gathered for the "before" period after the treatment has been implemented. However, other data, such as traffic volumes, must be collected during both the "before" and the "after" periods if necessary.

The crash estimation is based on the "before" period. The estimated expected average crash frequency based on the "before" period crashes is then adjusted for changes in the various conditions of the "after" period to predict what expected average crash frequency would have been had the treatment not been installed.

Observational Cross-Sectional Studies

The scope of an observational cross-sectional study is the evaluation of a treatment where there are few roadways or facilities where a treatment was implemented, and there are many roadways or facilities that are similar except they do not have the treatment of interest. For example, it is unlikely that an agency has many rural two-lane road segments where horizontal curvature was rebuilt to increase the horizontal curve radius. However, it is likely that an agency has many rural two-lane road segments with horizontal curvature in a certain range, such as 1,500- to 2,000-foot range, and another group of segments with curvature in another range, such as 3,000 to 5,000 feet. These two groups of rural two-lane road segments could be used in a cross-sectional study. Data are collected for a specific time period for both groups. The crash estimation based on the accident frequencies for one group is compared with the crash estimation of the other group. It is, however, very difficult to adjust for differences in the various relevant conditions between the two groups.

3.8. CONCLUSIONS

Chapter 3 summarizes the key concepts, definitions, and methods presented in the HSM. The HSM focuses on crashes as an indicator of safety, and in particular is focused on methods to estimate the crash frequency and severity of a given site type for given conditions during a specific period of time.

Crashes are rare and randomly occurring events which result in injury or property damage. These events are influenced by a number of interdependent contributing factors which affect the events before, during and after a crash.

Crash estimation methods are reliant on accurate and consistent collection of observed crash data. The limitations and potential for inaccuracy inherent in the collection of data apply to all crash estimation methods and need consideration.

As crashes are rare and random events, the observed crash frequency will fluctuate year to year due to both natural random variation and changes in site conditions which affect the number of crashes. The assumption that the observed crash frequency over a short period represents a reliable estimate of the long-term average crash frequency fails to account for the non-linear relationships between crashes and exposure. The assumption also does not account for regression-to-the-mean (RTM) bias (also known as selection bias), resulting in ineffective expenditure of limited safety funds and over (or under) estimation of the effectiveness of a particular treatment type.

In order to account for the effects of RTM bias, and the limitations of other crash estimations methods (discussed in Section 3.4), the HSM provides a predictive method for the estimation of the expected average crash frequency of a site, for given geometric and geographic conditions, in a specific period for a particular AADT.

Expected average crash frequency is the crash frequency expected to occur if the long-term average crash frequency of a site could be determined for a particular type of roadway segment or intersection with no change in the sites conditions. The predictive method (presented in *Part C*) uses statistical models, known as SPFs, and accident modification factors, AMFs, to estimate predicted average crash frequency. These models must be calibrated to local conditions to account for differing crash frequencies between different states and jurisdictions. When appropriate, the statistical estimate is combined with the observed crash frequency of a specific site using the EB Method, to improve the reliability of the estimation. The predictive method also allows for estimation using only SPFs, or only observed data in cases where either a model or observed data in not available.

Effectiveness evaluations are conducted using observational before/after and cross-sectional studies. The evaluation of a treatment's effectiveness involves comparing the expected average crash frequency of a roadway or site with the implemented treatment to the expected average crash frequency of the roadway element or site had the treatment not been installed.

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PART B—ROADWAY SAFETY MANAGEMENT PROCESS

Introduction and Applications Guidance

B.1. Purpose of Part B....B-1
B.2. Part B and the Project Development Process....B-2
B.3. Applying Part B....B-3
B.4. Relationship to Parts A, C, and D of the Highway Safety Manual....B-4
B.5. Summary....B-5

LIST OF EXHIBITS

Exhibit B-1: The Project Development Process B-3

PART B INTRODUCTION AND APPLICATIONS GUIDANCE

B.1. PURPOSE OF PART B

Part B presents procedures and information useful in monitoring and reducing crash frequency on existing roadway networks. Collectively, the chapters in *Part B* are the roadway safety management process.

A roadway safety management process is a quantitative, systematic, process for studying roadway safety on existing transportation systems, and identifying potential safety improvements.

The six steps of the roadway safety management process are:

- *Chapter 4 – Network Screening*: Reviewing a transportation network to identify and rank sites based on the potential for reducing average crash frequency.
- *Chapter 5 – Diagnosis*: Evaluating crash data, historic site data, and field conditions to identify crash patterns.
- *Chapter 6 – Select Countermeasures*: Identifying factors that may contribute to crashes at a site and selecting possible countermeasures to reduce the average crash frequency.
- *Chapter 7 – Economic Appraisal*: Evaluating the benefits and costs of the possible countermeasures and identifying individual projects that are cost-effective or economically justified.
- *Chapter 8 – Prioritize Projects*: Evaluating economically justified improvements at specific sites, and across multiple sites, to identify a set of improvement projects to meet objectives such as cost, mobility, or environmental impacts.
- *Chapter 9 – Safety Effectiveness Evaluation*: Evaluating effectiveness of a countermeasure at one site or multiple sites in reducing crash frequency or severity.

Part B chapters can be used sequentially as a process; or they can be selected and applied individually to respond to the specific problem or project under investigation.

The benefits of implementing a roadway safety management process include:

- Systematic and repeatable process for identifying opportunities to reduce crashes and identifying potential countermeasures resulting in a prioritized list of cost-effective safety countermeasures.
- A quantitative and systematic process that addresses a broad range of roadway safety conditions and tradeoffs.
- The opportunity to leverage funding and coordinate improvements with other planned infrastructure improvement programs.
- Comprehensive methods that consider traffic volume, collision data, traffic operations, roadway geometry, and user expectations.
- The opportunity to use a proactive process to increase the effectiveness of countermeasures intended to reduce crash frequency.

There is no such thing as absolute safety. There is risk in all highway transportation. A universal objective is to reduce the number and severity of crashes within the limits of available resources, science, technology and legislatively-mandated priorities. The material in Part B is one resource for information and methodologies that are used in efforts to reduce crashes on existing roadway networks. Applying these methods does not guarantee that crashes will decrease across all sites; the methods are a set of tools available for use in conjunction with sound engineering judgment.

B.2. PART B AND THE PROJECT DEVELOPMENT PROCESS

Exhibit B-1 illustrates how the various chapters in Part B align with the traditional elements of the project development process introduced in *Chapter 1*. The chapters in *Part B* of the HSM are applicable to the entire process; in several cases individual chapters can be used in multiple stages of the project development process. For example:

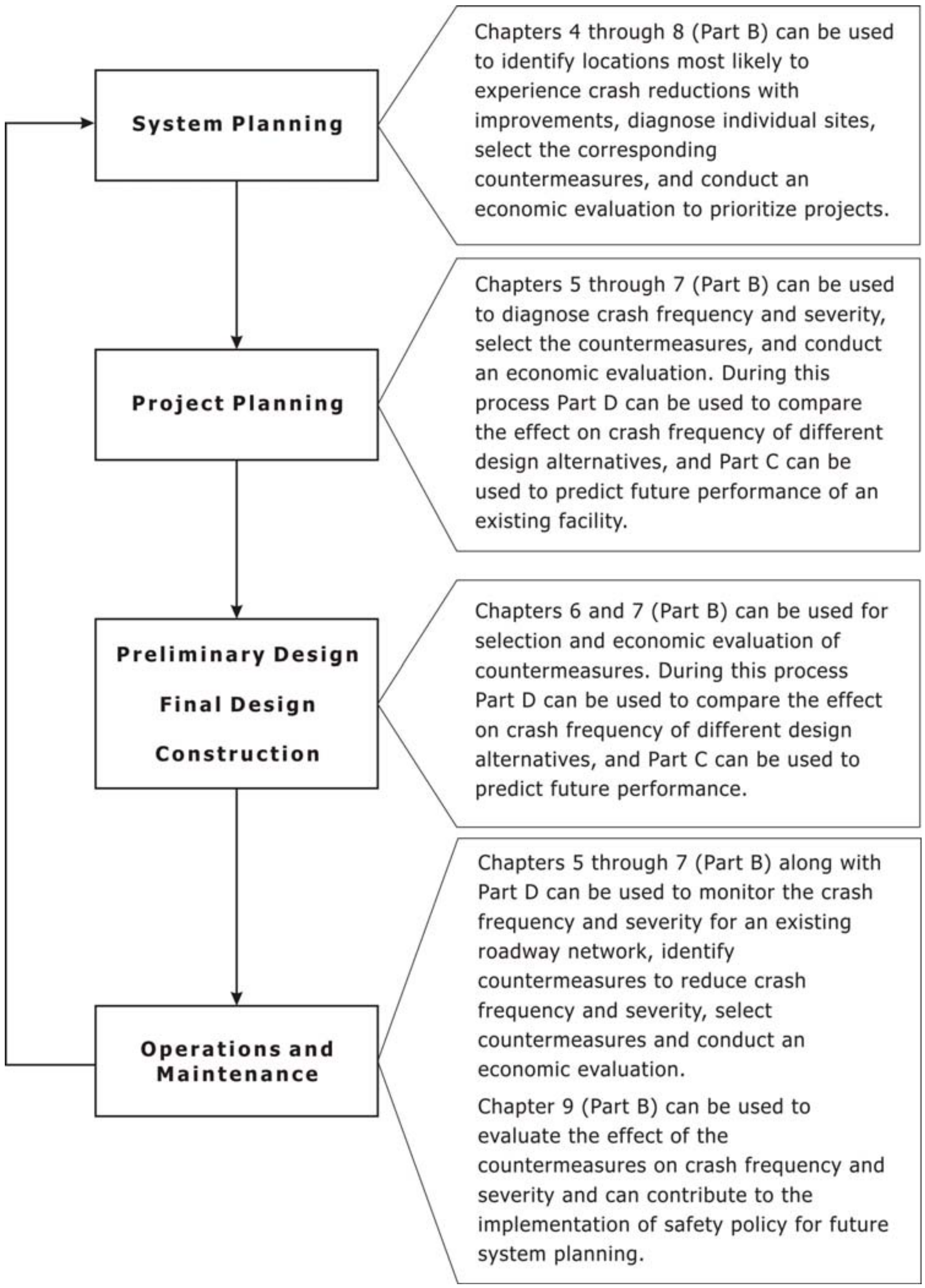
- System Planning: *Chapters 4, 7,* and *8* present methods to identify locations within a network with potential for a change in crash frequency. Projects can then be programmed based on economic benefits of crash reduction. These improvements can be integrated into long-range transportation plans and roadway capital improvement programs.

- Project Planning: As jurisdictions are considering alternative improvements and specifying project solutions, the diagnosis (*Chapter 5*), countermeasure selection (*Chapter 6*), and economic appraisal (*Chapter 7*) methods presented in *Part B* provide performance measures to support integrating crash analysis into a project alternatives analysis.

- Preliminary Design, Final Design and Construction: Countermeasure selection (*Chapter 6*) and Economic Appraisal (*Chapter 7*) procedures can also support the design process. These chapters provide information that could be used to compare various aspects of a design to identify the alternative with the lowest expected crash frequency and cost.

- Operations and Maintenance: Safety Effectiveness Evaluation (*Chapter 9*) procedures can be integrated into a community's operations and maintenance procedures to continually evaluate the effectiveness of investments. In addition, Diagnosis (*Chapter 5*), Selecting Countermeasures (*Chapter 6*), and Economic Appraisal (*Chapter 7*) procedures can be evaluated as part of ongoing overall highway safety system management.

Exhibit B-1: The Project Development Process



B.3. APPLYING PART B

Chapter 4 presents a variety of crash performance measures and screening methods for assessing historic crash data on a roadway system and identifying sites which may respond to a countermeasure. As described in *Chapter 4*, there are strengths and weaknesses to each of the performance measures and screening methods that may influence which sites are identified. Therefore, in practice it may be useful to use multiple performance measures and/or multiple screening methods to identify possible sites for further evaluation.

Chapters 5 and *6* present information to assist with reviewing crash history and site conditions to identify a crash pattern at a particular site and identify potential countermeasures. While the HSM presents these as distinct activities, in practice they may be iterative. For example, evaluating and identifying possible crash contributing factors (*Chapter 6*) may reveal the need for additional site investigation in order to confirm an original assessment (*Chapter 5*).

The final activity in *Chapter 6* is selecting a countermeasure. *Part D* of the HSM presents countermeasures and, when available, their corresponding Accident Modification Factors (AMF). The AMFs presented in *Part D* have satisfied the screening criteria developed for the HSM, which is described in the *Part D Introduction and Applications Guidance*. There are three types of information related to the effects of treatments:

1) a quantitative value representing the change in expected crashes (i.e., an AMF);

2) an explanation of a trend (i.e., change in crash frequency or severity) due to the treatment, but no quantitative information; and,

3) an explanation that information is not currently available.

Chapters 7 and *8* present information necessary for economically evaluating and prioritizing potential countermeasures at any one site or at multiple sites. In *Chapter 7*, the expected reduction in average crash frequency is calculated and converted to a monetary value or cost-effectiveness ratio. *Chapter 8* presents prioritization methods to select financially optimal sets of projects. Because of the complexity of the methods, most projects require application of software to optimize a series of potential treatments.

Chapter 9 presents information on how to evaluate the effectiveness of treatments. This chapter will provide procedures for:

- Evaluating a single project to document the change in crash frequency resulting from that project;
- Evaluating a group of similar projects to document the change in crash frequency resulting from those projects;
- Evaluating a group of similar projects for the specific purpose of quantifying a countermeasure AMF; and,
- Assessing the overall change in crash frequency resulting from specific types of projects or countermeasures in comparison to their costs.

Knowing the effectiveness of the program or project will provide information suitable to evaluate success of a program or project, and subsequently support policy and programming decisions related to improving roadway safety.

B.4. RELATIONSHIP TO PARTS A, C, AND D OF THE HIGHWAY SAFETY MANUAL

Part A: Introduction, Human Factors and Fundamentals

Part A provides introductory and fundamental knowledge for application of the HSM. An overview of Human Factors (*Chapter 2*) is presented to support engineering assessments in *Parts B* and *C*. *Chapter 3* presents fundamentals for the methods and procedures in the HSM. Concepts from *Chapter 3* that are applied in *Part B* include: expected average crashes, safety estimation, regression to the mean and regression-to-the-mean bias, and empirical Bayes methods.

Part C: Predictive Methods

Part C of the HSM introduces techniques for estimating crash frequency of facilities being modified through an alternatives analysis or design process. Specifically, *Chapters 10-12* present a predictive method for two-lane rural highways, multilane rural highways, and urban and suburban arterials, respectively. The predictive method in *Part C* is a proactive tool for estimating the expected change in crash frequency on a facility due to different design concepts. The material in *Part C* can be applied to the *Part B* methods as part of the procedures to estimate the crash reduction expected with implementation of potential countermeasures.

Part D: Accident Modification Factors

Finally, as described above, *Part D* consists of accident modification factors that can be applied in *Chapters 4, 6, 7, and 8*. The accident modification factors are used to estimate the potential crash reduction as the result of implementing a countermeasure(s). The crash reduction estimate can be converted into a monetary value and compared to the cost of the improvement and the cost associated with operational or geometric performance measures (e.g., delay, right-of-way).

B.5. SUMMARY

The roadway safety management process provides information for system planning, project planning, and near-term design, operations, and maintenance of a transportation system. The activities within the roadway safety management process provide:

- Awareness of sites that could benefit from treatments to reduce crash frequency or severity (*Chapter 4 Network Screening*);
- Understanding crash patterns and countermeasure(s) most likely to reduce crash frequency (*Chapter 5 Diagnosis*, *Chapter 6 Select Countermeasures*) at a site;
- Estimating the economic benefit associated with a particular treatment (*Chapter 7 Economic Appraisal*);
- Developing an optimized list of projects to improve (*Chapter 8 Prioritize Projects*); and,
- Assessing the effectiveness of a countermeasure to reduce crash frequency (*Chapter 9 Safety Effectiveness Evaluation*).

The activities within the roadway safety management process can be conducted independently or they can be integrated into a cyclical process for monitoring a transportation network.

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PART B—ROADWAY SAFETY MANAGEMENT PROCESS

CHAPTER 4—NETWORK SCREENING

4.1. Introduction 4-1
4.2. Network Screening Process 4-2
4.2.1. STEP 1 - Establish the Focus of Network Screening 4-2
4.2.2. STEP 2 - Identify the Network and Establish Reference Populations 4-4
4.2.3. STEP 3 - Select Network Screening Performance Measures 4-7
4.2.4. STEP 4 - Select Screening Method 4-16
4.2.5. STEP 5 - Screen and Evaluate Results 4-22
4.3. Summary 4-23
4.4. Performance Measure Methods and Sample Applications 4-24
4.4.1. Intersection Performance Measure Sample Data 4-24
4.4.2. Intersection Performance Measure Methods 4-27
4.4.3. Roadway Segments Performance Measure Sample Data 4-85
4.5. References 4-92

EXHIBITS

Exhibit 4-1: Roadway Safety Management Process 4-1

Exhibit 4-2: The Network Screening Process – Step 1 4-3

Exhibit 4-3: The Network Screening Process – Step 2 4-4

Exhibit 4-4: Example Intersection Reference Populations Defined by Functional Classification and Traffic Control 4-5

Exhibit 4-5: Example Reference Populations for Segments 4-6

Exhibit 4-6: Step 3 of the Network Screening Process 4-7

Exhibit 4-7: Summary of Data Needs for Performance Measures 4-8

Exhibit 4-8: Stability of Performance Measures 4-10

Exhibit 4-9: Strengths and Limitations of the Average Crash Frequency Performance Measure 4-11

Exhibit 4-10: Strengths and Limitations of the Crash Rate Performance Measure 4-11

Exhibit 4-11: Strengths and Limitations of the EPDO Average Crash Frequency Performance Measure 4-12

Exhibit 4-12: Strengths and Limitations of the RSI Performance Measure 4-12

Exhibit 4-13: Strengths and Limitations of the Critical Rate Performance Measure 4-13

Exhibit 4-14: Strengths and Limitations of Excess Average Crash Frequency Using Method of Moments Performance Measure 4-13

Exhibit 4-15: Strengths and Limitations of LOSS Performance Measure 4-14

Exhibit 4-16: Strengths and Limitations of the Excess Predicted Average Crash Frequency Using SPFs Performance Measure 4-14

Exhibit 4-17: Strengths and Limitations of the Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure 4-14

Exhibit 4-18: Strengths and Limitations of the Excess Proportions of Specific Crash Types Performance Measure 4-15

Exhibit 4-19: Strengths and Limitations of the Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment Performance Measure 4-15

Exhibit 4-20: Strengths and Limitations of the EPDO Average Crash Frequency with EB Adjustment Performance Measure 4-16

Exhibit 4-21: Strengths and Limitations of the Excess Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment Performance Measure 4-16

Exhibit 4-22: Network Screening Process: Step 4 – Select Screening Method 4-17

Exhibit 4-23: Example Application of Sliding Window Method 4-18

Exhibit 4-24: Example Application of Expected Crash Frequency with Empirical Bayes Adjustment (Iteration #1) 4-20

Exhibit 4-25: Example Application of Expected Average Crash Frequency with Empirical Bayes Adjustment (Iteration #2) 4-20

Exhibit 4-26: Performance Measure Consistency with Screening Methods..........4-22
Exhibit 4-27: Network Screening Process..4-23
Exhibit 4-28: Intersection Traffic Volumes and Crash Data Summary 4-25
Exhibit 4-29: Intersection Detailed Crash Data Summary (3 Years)...................4-26
Exhibit 4-30: Estimated Predicted Average Crash Frequency from an SPF..........4-27
Exhibit 4-31: Strengths and Limitations of the Average Crash Frequency Performance Measure..4-28
Exhibit 4-32: Intersection Rankings with Frequency Method 4-29
Exhibit 4-33: Strengths and Limitations of the Crash Rate Performance Measure 4-30
Exhibit 4-34: Total Entering Vehicles...4-31
Exhibit 4-35: Ranking Based on Crash Rates..4-32
Exhibit 4-36: Strengths and Limitations of the EPDO Average Crash Frequency Performance Measure 4-33
Exhibit 4-37: Societal Crash Cost Assumptions..4-34
Exhibit 4-38: Sample EPDO Weights..4-35
Exhibit 4-39: Sample EPDO Ranking..4-36
Exhibit 4-40: Strengths and Limitations of the RSI Performance Measure 4-37
Exhibit 4-41: Crash Cost Estimates by Crash Type 4-37
Exhibit 4-42: Intersection 7 Relative Severity Index Costs 4-38
Exhibit 4-43: Average RSI Cost for the Unsignalized Intersection Population......4-39
Exhibit 4-44: Ranking Based on Average RSI Cost per Intersection 4-40
Exhibit 4-45: Strengths and Limitations of the Critical Rate Performance Measure 4-41
Exhibit 4-46: Confidence Levels and P Values for Use in Critical Rate Method 4-41
Exhibit 4-47: Network Reference Populations and Average Crash Rate 4-43
Exhibit 4-48: Critical Rate Method Results..4-45
Exhibit 4-49: Strengths and Limitations of Excess Predicted Average Crash Frequency Using Method of Moments Performance Measure.........4-46
Exhibit 4-50: TWSC Reference Population 4-47
Exhibit 4-51: Signalized Reference Population...4-47
Exhibit 4-52: Reference Population Summary 4-49
Exhibit 4-53: Rank According to PI...4-50
Exhibit 4-54: Strengths and Limitations of LOSS Performance Measure 4-51
Exhibit 4-55: Estimated Number of Crashes from an SPF.................................4-52
Exhibit 4-56: Summary of Standard Deviation Calculations 4-53
Exhibit 4-57: LOSS Categories 4-53
Exhibit 4-58: LOSS Limits for Intersection 7..4-54
Exhibit 4-59: Intersection LOSS Ranking 4-54

Exhibit 4-60: Strengths and Limitations of the Excess Predicted Average Crash Frequency Using SPFs performance measure 4-55
Exhibit 4-61: TWSC Reference Population 4-56
Exhibit 4-62: SPF Predicted Average Crash Frequency 4-57
Exhibit 4-63: Excess Predicted Average Crash Frequency for TWSC Population 4-58
Exhibit 4-64: Ranking of TWSC Population Based Excess Predicted Average Crash Frequency from an SPF 4-58
Exhibit 4-65: Strengths and Limitations of the Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure 4-59
Exhibit 4-66: Estimated Threshold Proportion of Angle Collisions 4-60
Exhibit 4-68: Alpha and Beta Calculations 4-62
Exhibit 4-69: Probability Calculations 4-63
Exhibit 4-70: Ranking Based on Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure 4-63
Exhibit 4-71: Strengths and Limitations of the Excess Proportions of Specific Crash Types Performance Measure 4-64
Exhibit 4-72: Ranking Based on Excess Proportion 4-65
Exhibit 4-73: Strengths and Limitations Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment 4-66
Exhibit 4-74: Annual Correction Factors for all TWSC Intersections 4-68
Exhibit 4-75: Weighted Adjustments for TWSC Intersections 4-69
Exhibit 4-76: Year 3 – EB-Adjusted Expected Average Crash Frequency 4-71
Exhibit 4-77: Year 3- Variance of EB-Adjusted Expected Average Crash Frequency 4-72
Exhibit 4-78: EB-Adjusted Average Crash Frequency Ranking 4-72
Exhibit 4-79: Strengths and Limitations of the EPDO Average Crash Frequency with EB Adjustment Performance Measure 4-73
Exhibit 4-80: Societal Crash Cost Assumptions 4-73
Exhibit 4-81: Example EPDO Weights 4-74
Exhibit 4-82: Estimated Predicted Average Crash Frequency from an SPF 4-75
Exhibit 4-83: Annual Correction Factors for all TWSC Intersections 4-76
Exhibit 4-84: Year 1 – EB-Adjusted Number of Total Crashes 4-78
Exhibit 4-85: EB-Adjusted Expected Average Crash Frequency for TWSC Intersections 4-79
Exhibit 4-86: EB-Adjusted EPDO Ranking 4-81
Exhibit 4-87: Strengths and Limitations of the Excess Expected Average Crash Frequency with EB Adjustment Performance Measure 4-82
Exhibit 4-88: Societal Crash Cost Assumptions 4-82
Exhibit 4-89: Summary of Performance Measure Calculations for Steps 1, 4, and 5 4-83
Exhibit 4-90: EB-Adjusted Excess Expected Crash Ranking 4-85
Exhibit 4-91: EB-Adjusted Severity Weighted Excess Crash Ranking 4-85

Exhibit 4-92: Relative Severity Index Crash Costs 4-86
Exhibit 4-93: Roadway Segment Characteristics 4-86
Exhibit 4-94: Roadway Segment Detail Crash Data Summary (3 Years) 4-87
Exhibit 4-95: Segment 1 Sliding Window Parameters 4-87
Exhibit 4-96: Segment 1 Crash Data per Sliding Window Subsegments 4-88
Exhibit 4-97: Crash Type Summary for Segment 1 Window Subsegments 4-89
Exhibit 4-98: Average RSI Crash Cost per Window Subsegment 4-90
Exhibit 4-99: Average RSI Cost for Two-Lane Undivided Rural Highway Population 4-91

APPENDIX A

Appendix A – Crash Cost Estimates 1
Appendix References 5

CHAPTER 4 NETWORK SCREENING

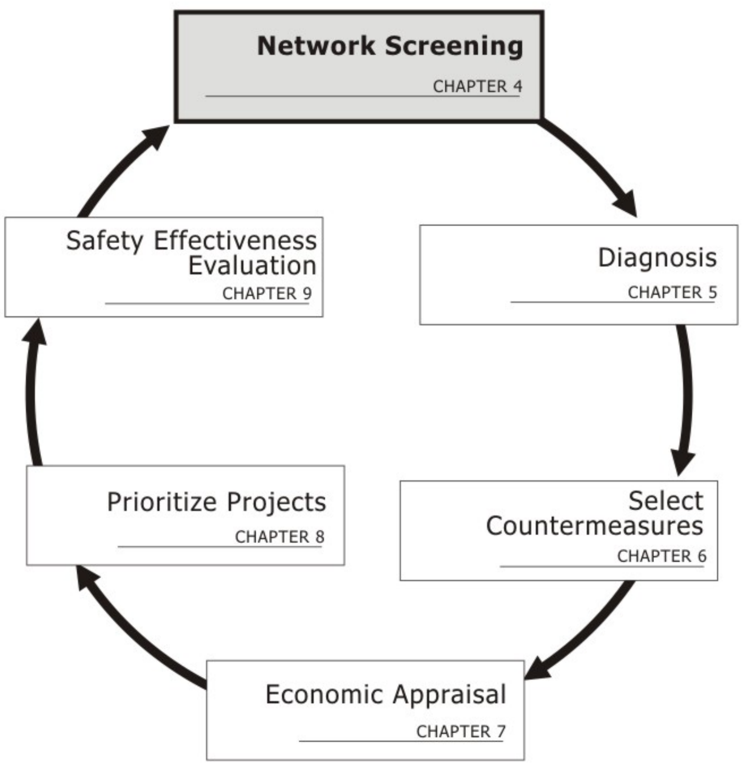
4.1. INTRODUCTION

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

Chapter 4 presents the performance measures and methods for conducting network screening.

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

Exhibit 4-1: Roadway Safety Management Process



4.2. NETWORK SCREENING PROCESS

Section 4.2 describes the steps in the network screening process.

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

1. Establish Focus: Identify the purpose or intended outcome of the network screening analysis. This decision will influence data needs, the selection of performance measures and the screening methods which can be applied.
2. Identify Network and Establish Reference Populations: Specify the type of sites or facilities being screened (i.e., segments, intersections, at-grade rail crossings) and identify groupings of similar sites or facilities.
3. Select Performance Measures: There are a variety of performance measures available to evaluate the potential to reduce crash frequency at a site. In this step the performance measure is selected as a function of the screening focus and the data and analytical tools available.
4. Select Screening Method: There are three principle screening methods described in this chapter (i.e., ranking, sliding window, and peak searching). The advantages and disadvantages of each are described in order to help identify the most appropriate method for a given situation.
5. Screen and Evaluate Results: The final step in the process is to conduct the screening analysis and evaluate results.

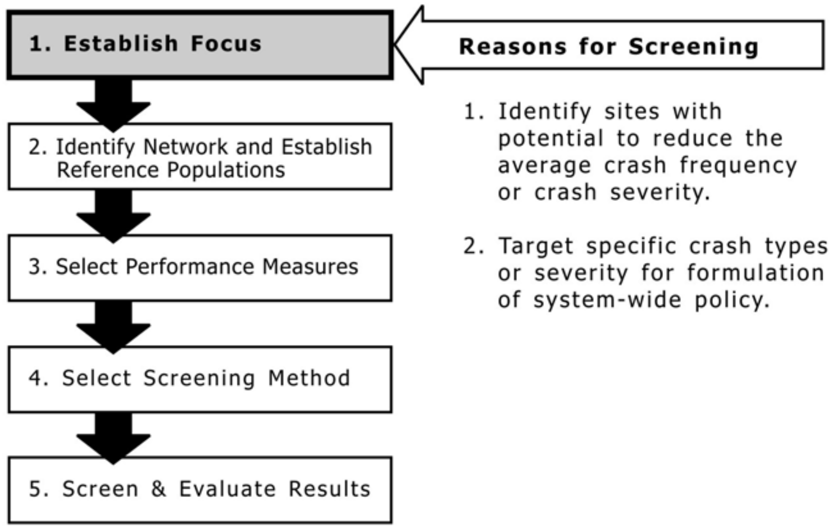
The following sections explain each of the five major steps in more detail.

4.2.1. STEP 1 - Establish the Focus of Network Screening

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

1. Identifying and ranking sites where improvements have potential to reduce the number of crashes; and/or,
2. Evaluating a network to identify sites with a particular crash type or severity in order to formulate and implement a policy (e.g., identify sites with a high number of run-off-the-road crashes to prioritize the replacement of non-standard guardrail statewide).

Exhibit 4-2: The Network Screening Process – Step 1



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

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

Determining the Network Screening Focus

Question

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

Answer

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

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

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

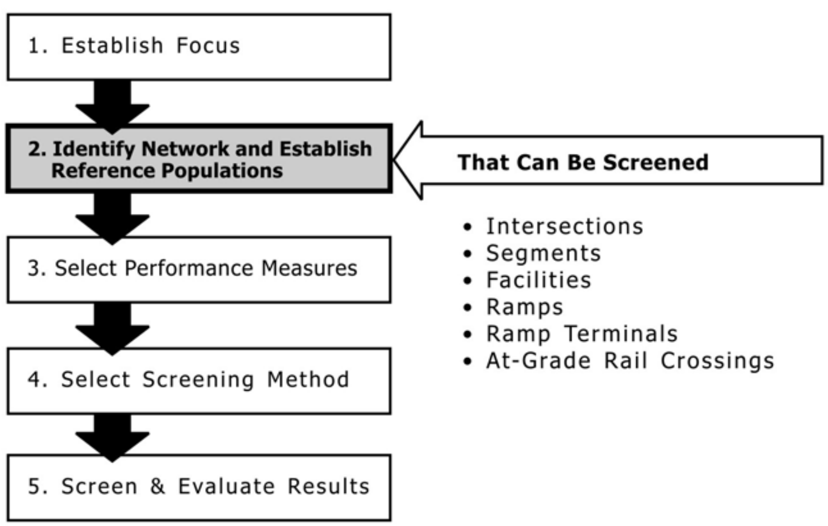
- An agency desires to identify projects for a Capital Improvement Program (CIP) or other established funding sources. In this case all sites would be screened.
- An agency has identified a specific crash type of concern and desires to implement a system-wide program to reduce that type of crash. In this case all sites would be screened to identify those with more of the specific crashes than expected.
- An agency has identified sites within a sub-area or along a corridor that are candidates for further safety analysis. Only the sites on the corridor would be screened.
- An agency has received funding to apply a program or countermeasure(s) system-wide to improve safety (e.g., red-light running cameras). Network screening would be conducted at all signalized intersections; a subset of the whole transportation system.

Roadway network elements that can be screened include intersections, roadway segments, facilities, ramps, ramp terminal intersections, and at-grade rail crossings.

4.2.2. STEP 2 - Identify the Network and Establish Reference Populations

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

Exhibit 4-3: The Network Screening Process – Step 2



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

Intersection Reference Populations

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

- Traffic control (e.g., signalized, two-way or four-way stop control, yield control, roundabout);
- Number of approaches (e.g., three-leg or four-leg intersections);
- Cross-section (e.g., number of through lanes and turning lanes);
- Functional classification (e.g., arterial, collector, local);
- Area type (e.g., urban, suburban, rural);
- Traffic volume ranges (e.g., total entering volume (TEV), peak hour volumes, average annual daily traffic (AADT)); and/or,
- Terrain (e.g., flat, rolling, mountainous).

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

Establishing Reference Populations for Intersection Screening

Exhibit 4-4 provides an example of data for several intersections within a network that have been sorted by functional classification and traffic control. These reference populations may be appropriate for an agency that has received funding to apply red-light running cameras or other countermeasure(s) system-wide to improve safety at signalized intersections. As such the last grouping of sites would not be studied since they are not signalized.

Exhibit 4-4: Example Intersection Reference Populations Defined by Functional Classification and Traffic Control

Reference Population	Segment ID	Street Type 1	Street Type 2	Traffic Control	Fatal	Injury	PDO	Total	Exposure Range (TEV/Average Annual Day)
Arterial-Arterial Signalized Intersections	3	Arterial	Arterial	Signal	0	41	59	100	55,000 to 70,000
	4	Arterial	Arterial	Signal	0	50	90	140	55,000 to 70,000
	10	Arterial	Arterial	Signal	0	28	39	67	55,000 to 70,000
Arterial-Collector Signalized Intersections	33	Arterial	Collector	Signal	0	21	52	73	30,000 to 55,000
	12	Arterial	Collector	Signal	0	40	51	91	30,000 to 55,000
	23	Arterial	Collector	Signal	0	52	73	125	30,000 to 55,000
Collector-Local All-Way Stop Intersections	22	Collector	Local	All-way Stop	1	39	100	140	10,000 to 15,000
	26	Collector	Local	All-way Stop	0	20	47	67	10,000 to 15,000

Segment Reference Populations

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

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

- Number of lanes per direction;
- Access density (e.g., driveway and intersection spacing);
- Traffic volumes ranges (e.g., TEV, peak hour volumes, AADT);
- Median type and/or width;
- Operating speed or posted speed;
- Adjacent land use (e.g., urban, suburban, rural);
- Terrain (e.g., flat, rolling, mountainous); and,
- Functional classification (e.g., arterial, collector, local).

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

Establishing Reference Populations for Segment Screening

Example:

Data is provided in Exhibit 4-5 for several roadway segments within a network. The segments have been sorted by median type and cross-section. These reference populations may be appropriate for an agency that desires to implement a system-wide program to employ access management techniques in order to potentially reduce the number of left-turn crashes along roadway segments.

Exhibit 4-5: Example Reference Populations for Segments

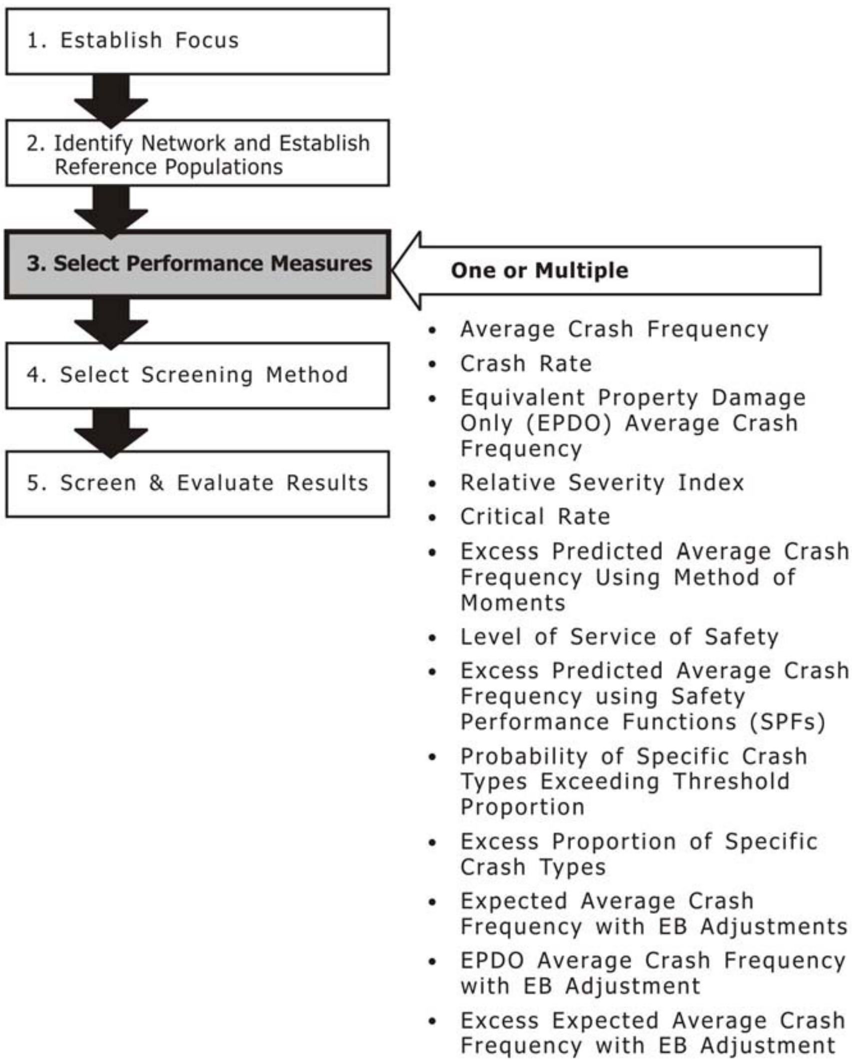
Reference Population	Segment ID	Cross-Section (lanes per direction)	Median Type	Segment Length (miles)
4-Lane Divided Roadways	A	2	Divided	0.60
	B	2	Divided	0.40
	C	2	Divided	0.90
5-Lane Roadway with Two-Way Left-Turn Lane	D	2	TWLTL	0.35
	E	2	TWLTL	0.55
	F	2	TWLTL	0.80

4.2.3. STEP 3 - Select Network Screening Performance Measures

The third step in the network screening process is to select one or several performance measures to be used in evaluating the potential to reduce the number of crashes or crash severity at a site (Exhibit 4-6). Just as intersection traffic operations analysis can be measured as a function of vehicle delay, queue length, or a volume-to-capacity ratio, intersection safety can be quantitatively measured in terms of average crash frequency, expected average crash frequency, a critical crash rate, or several other performance measures. In network screening using multiple performance measures to evaluate each site may improve the level of confidence in the results.

The third step in the network screening process is to select the screening performance measure(s). Multiple performance measures may be used.

Exhibit 4-6: Step 3 of the Network Screening Process



Key Criteria for Selecting Performance Measures

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

Data and Input Availability

Criteria for selecting performance measures are: data input and availability, regression-to-the-mean bias, and performance threshold.

Typical data required for the screening analysis includes the facility information for establishing reference populations, crash data, traffic volume data and in some cases safety performance functions. The amount of data and inputs that are available limits the number of performance measures that can be used. If traffic volume data is not available or cost prohibitive to collect, fewer performance measures are available for ranking sites. If traffic volumes are collected or made available, but calibrated safety performance functions and overdispersion parameters are not, the network could be prioritized using a different set of performance measures. Exhibit 4-7 summarizes the data and inputs needed for each performance measure.

Exhibit 4-7: Summary of Data Needs for Performance Measures

	Data and Inputs				
Performance Measure	**Crash Data**	**Roadway Information for Categorization**	**Traffic Volume[1]**	**Calibrated Safety Performance Function and Overdispersion Parameter**	**Other**
Average Crash Frequency	X	X			
Crash Rate	X	X	X		
Equivalent Property Damage Only (EPDO) Average Crash Frequency	X	X			EPDO Weighting Factors
Relative Severity Index	X	X			Relative Severity Indices
Critical Rate	X	X	X		
Excess Predicted Average Crash Frequency Using Method of Moments[2]	X	X	X		
Level of Service of Safety	X	X	X	X	
Excess Predicted Average Crash Frequency using Safety Performance Functions (SPFs)	X	X	X	X	
Probability of Specific Crash Types Exceeding Threshold Proportion	X	X			
Excess Proportion of Specific Crash Types	X	X			
Expected Average Crash Frequency with EB Adjustment	X	X	X	X	
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment	X	X	X	X	EPDO Weighting Factors
Excess Expected Average Crash Frequency with EB Adjustment	X	X	X	X	

Notes: [1] Traffic volume could be AADT, ADT, or peak hour volumes.

[2] Traffic volume is needed to apply Method of Moments to establish the reference populations based on ranges of traffic volumes as well as site geometric characteristics.

Regression-to-the-Mean Bias

Chapter 3 provides a discussion of regression-to-the-mean and regression-to-the-mean bias.

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

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

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

Performance Threshold

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

The method for determining a threshold performance value is dependent on the performance measure selected. The threshold performance value can be a subjectively assumed value, or calculated as part of the performance measure methodology. For example, threshold values are estimated based on: the average of the observed crash frequency for the reference population; an appropriate safety performance function; or, Empirical Bayes methods. Exhibit 4-8 summarizes whether or not each of the performance measures accounts for regression-to-the-mean bias and/or estimates a performance threshold. The performance measures are presented in relative order of complexity, from least to most complex. Typically, the methods that require more data and address RTM bias produce more reliable performance threshold values.

Exhibit 4-8: Stability of Performance Measures

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 but does not account for RTM bias	Yes
Excess Predicted Average Crash Frequency Using Method of Moments	Considers data variance but does not account for RTM bias	Yes
Level of Service of Safety	Considers data variance but does not account for RTM bias	Expected average crash frequency plus/minus 1.5 standard deviations
Excess Expected Average Crash Frequency Using SPFs	No	Predicted average crash frequency at the site
Probability of Specific Crash Types Exceeding Threshold Proportion	Considers data variance; not effected by RTM Bias	Yes
Excess Proportions of Specific Crash Types	Considers data variance; not effected by RTM Bias	Yes
Expected Average Crash Frequency with EB Adjustments	Yes	Expected average crash frequency at the site
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment	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

Definition of Performance Measures

The following defines the performance measures in the HSM and the strengths and limitations of each measure. The definitions below, in combination with Exhibits Exhibit 4-7 and Exhibit 4-8, provide guidance on selecting performance measures. The procedures to apply each performance measures are presented in detail in Section 4.4.

The strengths and limitation of network screening performance measures are explained in this section.

Average Crash Frequency

The site with the most total crashes or the most crashes of a particular crash severity or type, in a given time period, is given the highest rank. The site with the second highest number of crashes in total or of a particular crash severity or type, in the same time period, is ranked second, and so on. Exhibit 4-9 summarizes the strengths and limitations of the Average Crash Frequency performance measure.

Exhibit 4-9: Strengths and Limitations of the Average Crash Frequency Performance Measure

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

Crash Rate

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

Exhibit 4-10 summarizes the strengths and limitations of the Crash Rate performance measure.

Exhibit 4-10: Strengths and Limitations of the Crash Rate Performance Measure

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

Equivalent Property Damage Only (EPDO) Average Crash Frequency

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

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

Exhibit 4-11 summarizes the strengths and limitations of the EPDO Average Crash Frequency performance measure.

Exhibit 4-11: Strengths and Limitations of the EPDO Average Crash Frequency Performance Measure

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

Relative Severity Index

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

Exhibit 4-12 summarizes the strengths and limitations of the RSI performance measure.

Exhibit 4-12: Strengths and Limitations of the RSI Performance Measure

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

Critical Rate

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

Exhibit 4-13 summarizes the strengths and limitations of the Critical Rate performance measure.

Exhibit 4-13: Strengths and Limitations of the Critical Rate Performance Measure

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

Excess Predicted Average Crash Frequency Using Method of Moments

A site's observed average crash frequency is adjusted based on the variance in the crash data and average crash frequency for the site's reference population.[4] The adjusted observed average crash frequency for the site is compared to the average crash frequency for the reference population. This comparison yields the potential for improvement which can serve as a measure for ranking sites.

Exhibit 4-14 summarizes the strengths and limitations of the Excess Predicted Average Crash Frequency Using Method of Moments performance measure.

Exhibit 4-14: Strengths and Limitations of Excess Average Crash Frequency Using Method of Moments Performance Measure

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

Level of Service of Safety (LOSS)

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

Exhibit 4-15 summarizes the strengths and limitations of the LOSS performance measure.

Exhibit 4-15: Strengths and Limitations of LOSS Performance Measure

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

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

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

Exhibit 4-16 summarizes the strengths and limitations of the Excess Predicted Average Crash Frequency Using SPFs performance measure.

Exhibit 4-16: Strengths and Limitations of the Excess Predicted Average Crash Frequency Using SPFs Performance Measure

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

Probability of Specific Crash Types Exceeding Threshold Proportion

Sites are prioritized based on the *probability* that the true proportion, p_i, of a particular crash type or severity (e.g., long-term predicted proportion) is greater than the threshold proportion, p^*_i.[6] A threshold proportion (p^*_i) is selected for each population, typically based on the proportion of the target crash type or severity in the reference population. This method can also be applied as a diagnostic tool to identify crash patterns at an intersection or on a roadway segment (*Chapter 5*).

Exhibit 4-17 summarizes the strengths and limitations of the Probability of Specific Crash Types Exceeding Threshold Proportion performance measure.

Exhibit 4-17: Strengths and Limitations of the Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure

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

Excess Proportions of Specific Crash Types

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

Exhibit 4-18 summarizes the strengths and limitations of the Excess Proportions of Specific Crash Types performance measure.

Exhibit 4-18: Strengths and Limitations of the Excess Proportions of Specific Crash Types Performance Measure

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

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

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

Details of Empirical Bayes methods, safety performance functions, and calibration techniques are included in Chapter 3 and Part C of the manual.

Exhibit 4-19 summarizes the strengths and limitations of the Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment performance measure.

Exhibit 4-19: Strengths and Limitations of the Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment Performance Measure

Strengths	Limitations
• Accounts for RTM bias	• Requires SPFs calibrated to local conditions

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

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

Exhibit 4-20 summarizes the strengths and limitations of the EPDO Average Crash Frequency with EB Adjustment performance measure.

Exhibit 4-20: Strengths and Limitations of the EPDO Average Crash Frequency with EB Adjustment Performance Measure

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

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

Details of Empirical Bayes methods, safety performance functions, and calibration techniques are included in Chapter 3 and Part C of the manual.

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

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

Exhibit 4-21 summarizes the strengths and limitations of the Excess Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment performance measure.

Exhibit 4-21: Strengths and Limitations of the Excess Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment Performance Measure

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

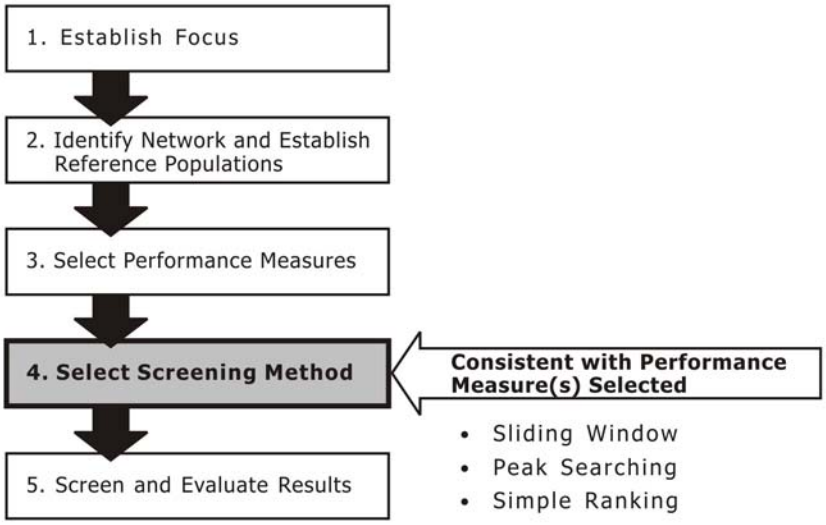
4.2.4. STEP 4 - Select Screening Method

Section 4.2.4 presents the screening methods: simple ranking, sliding window, and peak searching.

The fourth step in the network screening process is to select a network screening method (Exhibit 4-22). In a network screening process, the selected performance measure would be applied to all sites under consideration using a screening method. In the HSM, there are three types of three categories of screening methods:

- Segments (e.g., roadway segment or ramp) are screened using either sliding window or peak searching methods.
- Nodes (e.g., intersections or ramp terminal intersections) are screened using simple ranking method.
- Facilities (combination of nodes and segments) are screened using a combination of segment and node screening methods.

Exhibit 4-22: Network Screening Process: Step 4 – Select Screening Method



Segment Screening Methods

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

Sliding Window Method

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

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

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

Sliding Window Method

Question

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

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

Exhibit 4-23 shows the results for each window. Which sub-segment would define the potential for reduction in crash frequency or severity of the entire segment?

Exhibit 4-23: Example Application of Sliding Window Method

Sub-segment	Window Position	Excess Predicted Average Crash Frequency
A1	0.00 to 0.30 miles	1.20
A2	0.10 to 0.40 miles	0.80
A3	0.20 to 0.50 miles	1.10
A4	0.30 to 0.60 miles	1.90

Answer

As shown above there are four 0.30 sub-segments (i.e., window positions) on Segment A.

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

Peak Searching Method

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

The first step in the peak searching method is to divide a given roadway segment (or ramp) into 0.1 mile windows. The windows do not overlap, with the possible exception that the last window may overlap with the previous. If the segment is less than 0.1 mile in length, then the segment length equals the window length. The performance measure is then calculated for each window, and the results are subjected to precision testing. If the performance measure calculation for at least one sub-segment satisfies the desired precision level, the segment is ranked based upon the maximum performance measure from all of the windows that meet the desired precision level. If none of the performance measures for the initial 0.1 mile windows are found to have the desired precision, the length of each window is incrementally moved forward; growing the windows to a length of 0.2 mile. The calculations are performed again to assess the precision of the performance measures. The methodology continues in this fashion until a maximum performance measure with the desired precision is found or the window length equals the site length.

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

$$\text{Coefficient of Variation (CV)} = \frac{\sqrt{\text{Var(Performance Measure)}}}{\text{PerformanceMeasure}} \qquad (4\text{-}1)$$

A large CV indicates a low level of precision in the estimate, and a small CV indicates a high level of precision in the estimate. The calculated CV is compared to a specified limiting CV. If the calculated CV is less than or equal to the CV limiting value, the performance measure meets the desired precision level, and the performance measure for a given window can potentially be considered for use in ranking the segment. If the calculated CV is greater than the CV limiting value, the window is automatically removed from further consideration in potentially ranking the segment based upon the value of the performance measure.

There is no specific CV value that is appropriate for all network screening applications. However, by adjusting the CV value the user can vary the number of sites identified by network screening as candidates for further investigation. An appropriate initial or default value for the CV is 0.5.

Peak Searching Method

Question

Segment B, in an urban four-lane divided arterial reference population, will be screened using the Excess Expected Average Crash Frequency performance measure. Segment B is 0.47 miles long. The CV limiting value is assumed to be 0.25. If the peak searching method is used to study this segment, how is the methodology applied and how is the segment potentially ranked relative to other sites considered in the screening?

Answer

Iteration #1

Exhibit 4-24 shows the results of the first iteration. In the first iteration, the site is divided into 0.1 mi windows. For each window, the performance measure is calculated along with the CV.

The variance is given as:

$$VAR_B = \frac{(5.2-5.7)^2 + (7.8-5.7)^2 + (1.1-5.7)^2 + (6.5-5.7)^2 + (7.8-5.7)^2}{(5-1)} = 7.7$$

The Coefficient of Variation for Segment B1 is calculated using Equation 4-1 as shown below:

$$CV_{B1} = \frac{\sqrt{7.7}}{5.7} = 0.53$$

Exhibit 4-24: Example Application of Expected Average Crash Frequency with Empirical Bayes Adjustment (Iteration #1)

Sub-segment	Window Position	Excess Expected Average Crash Frequency	Coefficient of Variation (CV)
B1	0.00 to 0.10 miles	5.2	0.53
B2	0.10 to 0.20 miles	7.8	0.36
B3	0.20 to 0.30 miles	1.1	2.53
B4	0.30 to 0.40 miles	6.5	0.43
B5	0.37 to 0.47 miles	7.8	0.36
	Average	5.7	-

Because none of the calculated CVs are less than the CV limiting value, none of the windows meet the screening criterion, so a second iteration of the calculations is required.

Iteration #2

Exhibit 4-25 shows the results of the second iteration. In the second iteration, the site is analyzed using 0.2 mi windows. For each window, the performance measure is calculated along with the CV.

Exhibit 4-25: Example Application of Expected Average Crash Frequency with Empirical Bayes Adjustment (Iteration #2)

Sub-segment	Window Position	Excess Expected Average Crash Frequency	Coefficient of Variation (CV)
B1	0.00 to 0.20 miles	6.50	0.25
B2	0.10 miles to 0.30 miles	4.45	0.36
B3	0.20 miles to 0.40 miles	3.80	0.42
B4	0.27 miles to 0.47 miles	7.15	0.22
	Average	5.5	

In this second iteration, the CVs for sub-segments B1 and B4 are less than or equal to the CV limiting value of 0.25. Segment B would be ranked based upon the maximum value of the performance measures calculated for sub-segments B1 and B4. In this instance Segment B would be ranked and compared to other segments according to the 7.15 Excess Expected Crash Frequency calculated for sub-segment B4.

If during Iteration 2, none of the calculated CVs were less than the CV limiting value, a third iteration would have been necessary with 0.3 mile window lengths, and so on, until the final window length considered would be equal to the segment length of 0.47 miles.

Simple Ranking Method

A simple ranking method can be applied to nodes and segments. In this method, the performance measures are calculated for all of the sites under consideration, and the results are ordered from high to low. The simplicity of this method is the greatest strength. However, for segments, the results are not as reliable as the other segment screening methods.

Node-Based Screening

Node-based screening focuses on intersections, ramp terminal intersections, and at-grade rail crossings. A simple ranking method may be applied whereby the performance measures are calculated for each site, and the results are ordered from high to low. The outcome is a list showing each site and the value of the selected performance measure. All of the performance measures can be used with simple ranking for node-based screening.

A variation of the peak searching method can be applied to intersections. In this variation, the precision test is applied to determine which performance measure to rank upon. Only intersection-related crashes are included in the node-based screening analyses.

Facility Screening

A facility is a length of highway composed of connected roadway segments and intersections. When screening facilities, the connected roadway segments are recommended to be approximately 5 to 10 miles in length. This length provides for more stable results.

Exhibit 4-26 summarizes the performance measures that are consistent with the screening methods.

Exhibit 4-26: Performance Measure Consistency with Screening Methods

Performance Measure	Segments			Nodes	Facilities
	Simple Ranking	Sliding Window	Peak Searching	Simple Ranking	Simple Ranking
Average Crash Frequency	Yes	Yes	No	Yes	Yes
Crash Rate	Yes	Yes	No	Yes	Yes
Equivalent Property Damage Only (EPDO) Average Crash Frequency	Yes	Yes	No	Yes	Yes
Relative Severity Index	Yes	Yes	No	Yes	No
Critical Crash Rate	Yes	Yes	No	Yes	Yes
Excess Predicted Average Crash Frequency Using Method of Moments	Yes	Yes	No	Yes	No
Level of Service of Safety	Yes	Yes	No	Yes	No
Excess Predicted Average Crash Frequency using SPFs	Yes	Yes	No	Yes	No
Probability of Specific Crash Types Exceeding Threshold Proportion	Yes	Yes	No	Yes	No
Excess Proportions of Specific Crash Types	Yes	Yes	No	Yes	No
Expected Average Crash Frequency with EB Adjustments	Yes	Yes	Yes	Yes	No
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment	Yes	Yes	Yes	Yes	No
Excess Expected Average Crash Frequency with EB Adjustments	Yes	Yes	Yes	Yes	No

4.2.5. STEP 5 - Screen and Evaluate Results

The final step in the network screening process is to screen the sites/facilities under consideration.

The performance measure and the screening method are applied to the segments, nodes, and/or facilities according to the methods outlined in Steps 3 and 4. Conceptually, for each segment or node under consideration, the selected performance measure is calculated and recorded. Results can be recorded in a table or on maps as appropriate or feasible.

The results of the screening analysis will be a list of sites ordered according to the selected performance measure. Those sites higher on the list are considered most likely to benefit from countermeasures intended to reduce crash frequency. Further study of these sites will indicate what kinds of improvements are likely to be most effective (see *Chapters 5*, *6*, and *7*).

In general it can be useful to apply multiple performance measures to the same data set. In doing so, some sites will repeatedly be at the high or low end of the resulting list. Sites that repeatedly appear at the higher end of the list could become the focus of more detailed site investigations, while those that appear at the low end of the list could be ruled out for needing further investigation. Differences in the rankings produced by the various performance measures will become most evident at sites which are ranked in the middle of the list.

4.3. SUMMARY

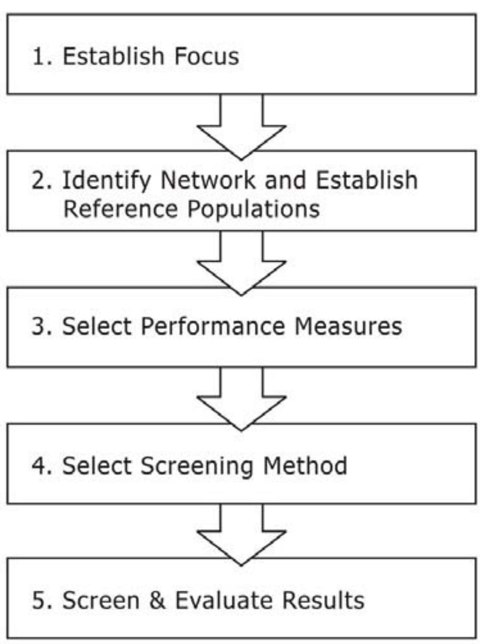
This chapter explains the five steps of the network screening process, illustrated in Exhibit 4-27, that can be applied with one of three screening methods for conducting network screening. The results of the analysis are used to determine the sites that are studied in further detail. The objective of studying these sites in more detail is to identify crash patterns and the appropriate countermeasures to reduce the number of crashes; these activities are discussed in *Chapters 5*, *6*, and *7*.

When selecting a performance measure and screening method there are three key considerations. The first is related to the data that is available or can be collected for the study. It is recognized that this is often the greatest constraint; therefore, methods are outlined in the chapter that do not require a significant amount of data.

The second and third considerations relate to the performance of the methodology results. The most accurate study methodologies provide for the ability to: 1) account for regression-to-the-mean bias, and 2) estimate a threshold level of performance in terms of crash frequency or crash severity. These methods can be trusted with a greater level of confidence than those methods that do not.

Section 4.4 provides a detailed overview of the procedure for calculating each of the performance measures in this chapter. The section also provides step-by-step sample applications for each method applied to intersections. These same steps can be used on ramp terminal intersections and at-grade rail crossings. Section 4.4 also provides step-by-step sample applications demonstrating use of the peak searching and sliding window methods to roadway segments. The same steps can be applied to ramps.

Exhibit 4-27: Network Screening Process



Section 4.4 provides the detailed calculations for each of the performance measures.

4.4. PERFORMANCE MEASURE METHODS AND SAMPLE APPLICATIONS

4.4.1. Intersection Performance Measure Sample Data

The following sections provide sample data to be used to demonstrate application of each performance measure.

Sample Situation

A roadway agency is undertaking an effort to improve safety on their highway network. They are screening twenty intersections to identify sites with potential for reducing the crash frequency.

The Facts

- All of the intersections have four approaches and are in rural areas;
- 13 are signalized intersections and 7 are unsignalized (two-way stop controlled) intersections;
- Major and Minor Street AADT volumes are provided in Exhibit 4-;
- A summary of crash data over the same three years as the traffic volumes is shown in Exhibit 4-28; and,
- Three years of detailed intersection crash data is shown in Exhibit 4-.

Assumptions

- The roadway agency has locally calibrated Safety Performance Functions (SPFs) and associated overdispersion parameters for the study intersections. Predicted average crash frequency from an SPF is provided in Exhibit 4-30 for the sample intersections.
- The roadway agency supports use of FHWA crash costs by severity and type.

Intersection Characteristics and Crash Data

Exhibit 4-28 and Exhibit 4-29 summarize the intersection characteristics and crash data.

Exhibit 4-28: Intersection Traffic Volumes and Crash Data Summary

Intersections	Traffic Control	Number of Approaches	Major AADT	Minor AADT	Crash Data		
					Total Year 1	Total Year 2	Total Year 3
1	Signal	4	30,100	4,800	9	8	5
2	TWSC	4	12,000	1,200	9	11	15
3	TWSC	4	18,000	800	9	8	6
4	Signal	4	11,200	10,900	8	2	3
5	Signal	4	30,700	18,400	3	7	5
6	Signal	4	31,500	3,600	6	1	2
7	TWSC	4	21,000	1,000	11	9	14
8	Signal	4	23,800	22,300	2	4	3
9	Signal	4	47,000	8,500	15	12	10
10	TWSC	4	15,000	1,500	7	6	4
11	Signal	4	42,000	1,950	12	15	11
12	Signal	4	46,000	18,500	10	14	8
13	Signal	4	11,400	11,400	4	1	1
14	Signal	4	24,800	21,200	5	3	2
15	TWSC	4	26,000	500	6	3	8
16	Signal	4	12,400	7,300	7	11	3
17	TWSC	4	14,400	3,200	4	4	5
18	Signal	4	17,600	4,500	2	10	7
19	TWSC	4	15,400	2,500	5	2	4
20	Signal	4	54,500	5,600	4	2	2

Exhibit 4-29: Intersection Detailed Crash Data Summary (3 Years)

Intersections	Total	Crash Severity			Crash Type							
		Fatal	Injury	PDO	Rear End	Sideswipe/ Overtaking	Right Angle	Ped	Bike	Head-On	Fixed Object	Other
1	22	0	6	16	11	4	4	0	0	0	1	2
2	35	2	23	10	4	2	21	0	2	5	0	1
3	23	0	13	10	11	5	2	1	0	0	4	0
4	13	0	5	8	7	2	3	0	0	0	1	0
5	15	0	4	11	9	4	2	0	0	0	0	0
6	9	0	2	7	3	2	3	0	0	0	1	0
7	34	1	17	16	19	7	5	0	0	0	3	0
8	9	0	2	7	4	3	1	0	0	0	0	1
9	37	0	22	15	14	4	17	2	0	0	0	0
10	17	0	7	10	9	4	2	0	0	0	1	1
11	38	1	19	18	6	5	23	0	0	4	0	0
12	32	0	15	17	12	2	14	1	0	2	0	1
13	6	0	2	4	3	1	2	0	0	0	0	0
14	10	0	5	5	5	1	1	1	0	0	1	1
15	17	1	4	12	9	4	1	0	0	0	1	2
16	21	0	11	10	8	4	7	0	0	0	1	1
17	13	1	5	7	6	2	2	0	0	1	0	2
18	19	0	8	11	8	7	3	0	0	0	0	1
19	11	1	5	5	5	4	0	1	0	0	0	1
20	8	0	3	5	2	3	2	0	0	0	1	0

Exhibit 4-30: Estimated Predicted Average Crash Frequency from an SPF

Intersection	Year	AADT Major Street	AADT Minor Street	Predicted Average Crash Frequency from an SPF	Average 3-Year Predicted Crash Frequency from an SPF
2	1	12,000	1,200	1.7	1.7
	2	12,200	1,200	1.7	
	3	12,900	1,300	1.8	
3	1	18,000	800	2.1	2.2
	2	18,900	800	2.2	
	3	19,100	800	2.2	
7	1	21,000	1,000	2.5	2.6
	2	21,400	1,000	2.5	
	3	22,500	1,100	2.7	
10	1	15,000	1,500	2.1	2.2
	2	15,800	1,600	2.2	
	3	15,900	1,600	2.2	
15	1	26,000	500	2.5	2.3
	2	26,500	300	2.2	
	3	27,800	200	2.1	
17	1	14,400	3,200	2.5	2.6
	2	15,100	3,400	2.6	
	3	15,300	3,400	2.6	
19	1	15,400	2,500	2.4	2.5
	2	15,700	2,500	2.5	
	3	16,500	2,600	2.6	

4.4.2. Intersection Performance Measure Methods

The following sections provide step-by-step procedures for applying the performance measures described in Section 4.2.3, which provides guidance for selecting an appropriate performance measure.

4.4.2.1. Average Crash Frequency

Applying the Crash Frequency performance measure produces a simple ranking of sites according to total crashes or crashes by type and/or severity. This method can be used to select an initial group of sites with high crash frequency for further analysis.

Data Needs

- Crash data by location

Strengths and Limitations

Exhibit 4-31 summarizes the strengths and limitations of the Crash Frequency performance measure.

Exhibit 4-31: Strengths and Limitations of the Average Crash Frequency Performance Measure

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

Procedure

STEP 1 – Sum Crashes for Each Location

Count the number of crashes that occurred at each intersection

STEP 2 – Rank Locations

The intersections can be ranked in descending order by the number of total crashes, fatal and injury crashes, and/or PDO crashes.

> Ranking of the 20 sample intersections is shown below in Exhibit 4-32. Column A shows the ranking by total crashes, Column B is the ranking by fatal and injury crashes, and Column C is the ranking by property damage-only crashes.
>
> As shown in Exhibit 4-32, ranking based on crash severity may lead to one intersection achieving a different rank depending on the ranking priority. The rank of Intersection 1 demonstrates this variation.

Exhibit 4-32: Intersection Rankings with Frequency Method

Column A		Column B		Column C	
Intersection	Total Crashes	Intersection	Fatal and Injury	Intersection	PDO Crashes
11	38	2	25	11	18
9	37	9	22	12	17
2	35	11	20	1	16
7	34	7	18	7	16
12	32	12	15	9	15
3	23	3	13	15	12
1	22	16	11	5	11
16	21	18	8	18	11
18	19	10	7	2	10
10	17	1	6	3	10
15	17	17	6	10	10
5	15	19	6	16	10
4	13	4	5	4	8
17	13	14	5	6	7
19	11	15	5	8	7
14	10	5	4	17	7
6	9	20	3	14	5
8	9	6	2	19	5
20	8	8	2	20	5
13	6	13	2	13	4

4.4.2.2. Crash Rate

The crash rate performance measure normalizes the number of crashes relative to exposure (traffic volume) by dividing the total number of crashes by the traffic volume. The traffic volume includes the total number of vehicles entering the intersection, measured as million entering vehicles (MEV).

Data Needs

- Crashes by location
- Traffic Volume

Strengths and Limitations

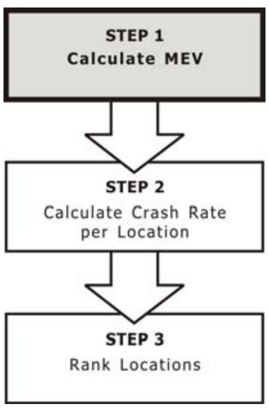
Exhibit 4-33 summarizes the strengths and limitations of the Crash Rate performance measure.

Exhibit 4-33: Strengths and Limitations of the Crash Rate Performance Measure

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

Procedure

The following outlines the assumptions and procedure for ranking sites according to the crash rate method. The calculations for Intersection 7 are used throughout the remaining sample problems to highlight how to apply each method.



STEP 1 – Calculate MEV

Calculate the million entering vehicles for all 3 years. Use Equation 4-2 to calculate the exposure in terms of million entering vehicles (MEV) at an intersection.

$$MEV = \left(\frac{TEV}{1{,}000{,}000} \right) \times (n) \times (365) \qquad (4\text{-}2)$$

Where,

MEV= Million entering vehicles

TEV = Total entering vehicles per day

n = Number of years of crash data

Exhibit 4-34 summarizes the total entering volume (TEV) for all sample intersections. The TEV is a sum of the major and minor street AADT found in Exhibit 4-28.

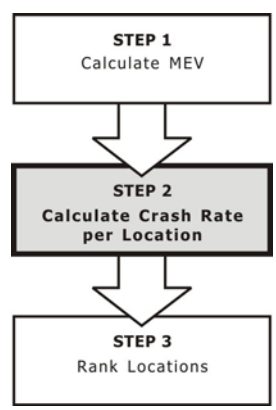
TEV is converted to MEV as shown in the following equation for Intersection 7.

$$MEV = \left(\frac{22{,}000}{1{,}000{,}000}\right) \times (3) \times (365) = 24.1$$

Exhibit 4-34: Total Entering Vehicles

Intersection	TEV/day	MEV
1	34900	38.2
2	13200	14.5
3	18800	20.6
4	22100	24.2
5	49100	53.8
6	35100	38.4
7	22000	24.1
8	46100	50.5
9	55500	60.8
10	16500	18.1
11	43950	48.1
12	64500	70.6
13	22800	25.0
14	46000	50.4
15	26500	29.0
16	19700	21.6
17	17600	19.3
18	22100	24.2
19	17900	19.6
20	60100	65.8

STEP 2 – Calculate the Crash Rate



Calculate the crash rate for each intersection by dividing the total number of crashes by MEV for the 3-year study period as shown in Equation 4-3.

$$R_i = \frac{N_{observed,i(TOTAL)}}{MEV_i} \qquad (4\text{-}3)$$

Where,

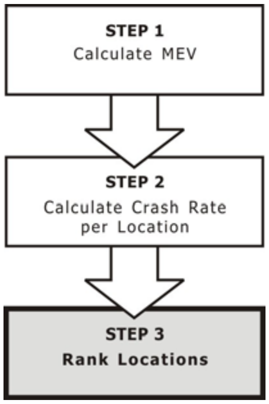
R_i = Observed crash rate at intersection i

$N_{observed,i(TOTAL)}$ = Total observed crashes at intersection i

MEV_i = Million entering vehicles at intersection i

Below is the crash rate calculation for Intersection 7. The total number of crashes for each intersection is summarized in Exhibit 4-28.

$$\text{Crash Rate} = \frac{34}{24.1} = 1.4 \text{ [crashes/MEV]}$$



Step 3 – Rank Intersections

Rank the intersections based on their crash rates.

Exhibit 4-35 summarizes the results from applying the crash rate method.

Exhibit 4-35: Ranking Based on Crash Rates

Intersection	Crash Rate
2	2.4
7	1.4
3	1.1
16	1.0
10	0.9
11	0.8
18	0.8
17	0.7
9	0.6
15	0.6
1	0.6
19	0.6
4	0.5
12	0.5
5	0.3
13	0.2
6	0.2
14	0.2
8	0.2
20	0.1

4.4.2.3. Equivalent Property Damage Only (EPDO) Average Crash Frequency

The Equivalent Property Damage Only (EPDO) Average Crash Frequency performance measure assigns weighting factors to crashes by severity to develop a single combined frequency and severity score per location. The weighting factors are calculated relative to Property Damage Only (PDO) crashes. To screen the network, sites are ranked from the highest to the lowest score. Those sites with the highest scores are evaluated in more detail to identify issues and potential countermeasures.

This method is heavily influenced by the weighting factors for fatal and injury crashes. A large weighting factor for fatal crashes has the potential to rank sites with one fatal crash and a small number of injury and/or PDO crashes above sites with no fatal crashes and a relatively high number of injury and/or PDO crashes. In some applications fatal and injury crashes are combined into one category of Fatal and/or Injury (FI) crashes to avoid over-emphasizing fatal crashes. Fatal crashes are tragic events; however, the fact that they are fatal is often the outcome of factors (or a combination of factors) that is out of the control of the engineer and planner.

Data Needs

- Crash data by severity and location
- Severity weighting factors
- Crash costs by crash severity

Strengths and Limitations

Exhibit 4-36 summarizes the strengths and limitations of the EPDO Average Crash Frequency performance measure.

Exhibit 4-36: Strengths and Limitations of the EPDO Average Crash Frequency Performance Measure

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

Procedure

Societal crash costs are used to calculate the EPDO weights. State and local jurisdictions often have accepted societal crash costs by type and/or severity. When available, locally-developed crash cost data is preferred. If local information is not available, national crash cost data is available from the Federal Highway Administration (FHWA). In order to improve acceptance of study results that use monetary values, it is important that monetary values be reviewed and endorsed by the jurisdiction in which the study is being conducted.

The FHWA report prepared in October 2005, "Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries," documented mean comprehensive societal costs by severity as listed below in Exhibit 4-37 (rounded to the nearest hundred dollars).[(2)] As of December 2008 this was the most recent FHWA crash cost information, although these costs represent 2001 values.

Appendix A includes a summary of crash costs and outlines a process to update monetary values to current year values.

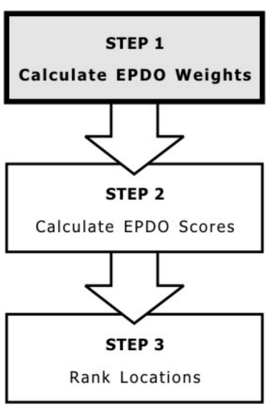
Exhibit 4-37: Societal Crash Cost Assumptions

Severity	Comprehensive Crash Cost (2001 Dollars)
Fatality (K)	$4,008,900
Injury Crashes (A/B/C)	$82,600
PDO (O)	$7,400

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA - HRT - 05-051, October 2005.

A discussion of crash severity coding systems is provided in Chapter 3 of the manual.

The values in Exhibit 4-37 were published in the FHWA study. A combined disabling (A), evident (B), and possible (C) injury crash cost was provided by FHWA to develop an average injury (A/B/C) cost. Injury crashes could also be subdivided into disabling injury, evident injury, and possible injury crashes depending on the amount of detail in the crash data and crash costs available for analysis.



STEP 1 – Calculate EDPO Weights

Calculate the EPDO weights for fatal, injury, and PDO crashes. The fatal and injury weights are calculated using Equation 4-4. The cost of a fatal or injury crash is divided by the cost of a PDO crash, respectively. Weighting factors developed from local crash cost data typically result in the most accurate results. If local information is not available, nationwide crash cost data is available from the Federal Highway Administration (FHWA). Appendix A provides more information on the national data available.

The weighting factors are calculated as follows:

$$f_{y(weight)} = \frac{CC_y}{CC_{PDO}} \qquad (4\text{-}4)$$

Where,

$f_{y(weight)}$= Weighting factor based on crash severity, y

CC_y= Crash cost for crash severity, y

CC_{PDO}= Crash cost for PDO crash severity

Below is a sample calculation for the injury (A/B/C) EPDO weight ($f_{inj(weight)}$):

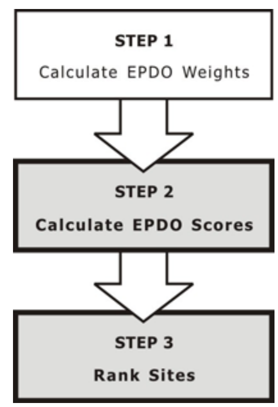
$$f_{inj(weight)} = \$82{,}600 / \$7{,}400 = 11$$

Therefore the weighting factors for all crash severities are shown in Exhibit 4-38.

Exhibit 4-38: Sample EPDO Weights

Severity	Cost	Weight
Fatal (K)	$4,008,900	542
Injury (A/B/C)	$82,600	11
PDO (O)	$7,400	1

STEP 2- Calculate EPDO Scores



For each intersection, multiply the EPDO weights by the corresponding number of fatal, injury, and PDO crashes as shown in Equation 4-5. The frequency of PDO, Injury and Fatal crashes is based on the number of crashes, not the number of injuries per crash.

$$Total\ EPDO\ Score = f_{K(weight)}\left(N_{observed,i(F)}\right) + f_{inj(weight)}\left(N_{observed,i(I)}\right) + f_{PDO(weight)}\left(N_{observed,i(PDO)}\right) \tag{4-5}$$

Where,

$f_{K(weight)}$ = Fatal Crash Weight

$N_{observed,i(F)}$ = Number of Fatal Crashes per intersection, i

$f_{inj(weight)}$ = Injury Crash Weight

$N_{observed,i(I)}$ = Number of Injury Crashes per intersection, i

$f_{PDO(weight)}$ = PDO Crash Weight

$N_{observed,i(PDO)}$ = Number of PDO Crashes per intersection, i

STEP 3 – Rank Locations

The intersections can be ranked in descending order by the EPDO score.

The calculation of EPDO Score for Intersection 7 is shown below. Exhibit 4-29 summarizes the number of fatal, injury, and PDO crashes for each intersection. Exhibit 4-39 summarizes the EPDO score.

$$Total\ EPDO\ Score_7 = (542 \times 1) + (11 \times 17) + (1 \times 16) = 745$$

The calculation is repeated for each intersection.

The ranking for the 20 intersections based on EPDO method is displayed in Exhibit 4-39. The results of calculations for Intersection 7 are highlighted.

Exhibit 4-39: Sample EPDO Ranking

Intersection	EPDO Score
2	1347
11	769
7	745
17	604
19	602
15	598
9	257
12	182
3	153
16	131
18	99
10	87
1	82
4	63
14	60
5	55
20	38
6	29
8	29
13	26

4.4.2.4. Relative Severity Index (RSI)

Jurisdiction-specific societal crash costs are developed and assigned to crashes by crash type and location. These societal crash costs make up a relative severity index. Relative Severity Index (RSI) crash costs are assigned to each crash at each site based on the crash type. An average RSI crash cost is calculated for each site and for each population. Sites are ranked based on their average RSI cost and are also compared to the average RSI cost for their respective population.

Data Needs

- Crashes by type and location
- RSI Crash Costs

Strengths and Limitations

Exhibit 4-40 summarizes the strengths and limitations of the RSI performance measure.

Exhibit 4-40: Strengths and Limitations of the RSI Performance Measure

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

Procedure

The RSI costs listed in Exhibit 4-41 are used to calculate the average RSI cost for each intersection and the average RSI cost for each population. The values shown represent 2001 dollar values and are rounded to the nearest hundred dollars. Appendix A provides a method for updating crash costs to current year values.

Exhibit 4-41: Crash Cost Estimates by Crash Type

Crash Type	Crash Cost (2001 Dollars)
Rear End – Signalized Intersection	$26,700
Rear End – Unsignalized Intersection	$13,200
Sideswipe/Overtaking	$34,000
Angle – Signalized Intersection	$47,300
Angle – Unsignalized Intersection	$61,100
Pedestrian/Bike at an Intersection	$158,900
Head-On – Signalized Intersection	$24,100
Head-On – Unsignalized Intersection	$47,500
Fixed Object	$94,700
Other/Undefined	$55,100

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA - HRT - 05-051, October 2005.

STEP 1 – Calculate RSI Costs per Crash Type

For each intersection, multiply the observed average crash frequency for each crash type by their respective RSI crash cost.

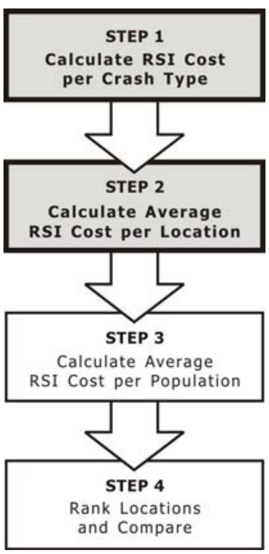
The RSI crash cost per crash type is calculated for each location under consideration. Exhibit 4-42 contains the detailed summary of the crashes by type at each intersection.

Exhibit 4-42 summarizes the number of crashes by crash type at Intersection 7 over the last three years and the corresponding RSI costs for each crash type.

Exhibit 4-42: Intersection 7 Relative Severity Index Costs

Intersection 7	Number of Observed Crashes	Crash Costs	RSI Costs
Rear End - Unsignalized Intersection	19	$13,200	$250,800
Sideswipe Crashes - Unsignalized Intersection	7	$34,000	$238,000
Angle Crashes - Unsignalized Intersection	5	$61,100	$305,500
Fixed Object Crashes - Unsignalized Intersection	3	$94,700	$284,100
Total RSI Cost for Intersection 7			$1,078,400

Note: Crash types that were not reported to have occurred at Intersection 7 were omitted from the table; the RSI value for these crash types is zero.



STEP 2 – Calculate Average RSI Cost for Each Intersection

Sum the RSI crash costs for all crash types and divide by the total number of crashes at the intersection to arrive at an average RSI value for each intersection.

$$\overline{RSI_i} = \frac{\sum_{j=1}^{n} RSI_j}{N_{observed,i}} \qquad (4\text{-}6)$$

Where,

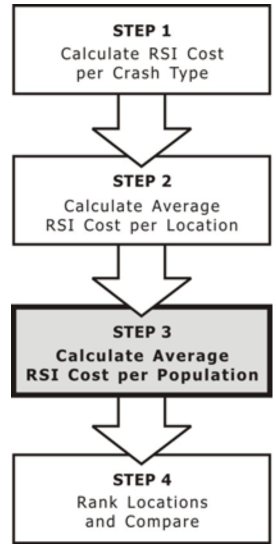
$\overline{RSI_i}$ = Average RSI cost for the intersection, *i*;

RSI_j = RSI cost for each crash type, *j*

$N_{observed,i}$ = Number of observed crashes at the site *i*.

The RSI calculation for intersection 7 is shown below.

$$\overline{RSI_7} = \frac{\$1,078,400}{34} = \$31,700$$



STEP 3 – Calculate the Average RSI Cost for Each Population

Calculate the average RSI cost for the population (the control group) by summing the total RSI costs for each site and dividing by the total number of crashes within the population.

$$\overline{RSI_{av(control)}} = \frac{\sum_{i=1}^{n} RSI_i}{\sum_{i=1}^{n} N_{observed,i}} \tag{4-7}$$

Where,

$\overline{RSI_{av(control)}}$ = Average RSI cost for the reference population (control group);

RSI_i = Total RSI cost at site *i*; and

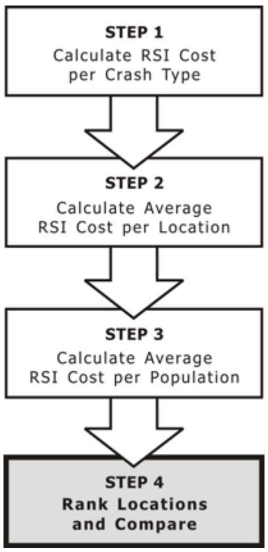
$N_{observed,i}$ = number of observed crashes at site *i*.

In this sample problem, Intersection 7 is in the unsignalized intersection population. Therefore, illustrated below is the calculation for the average RSI cost for the unsignalized intersection population.

The average RSI cost for the population ($\overline{RSI_P}$) is calculated using Exhibit 4-41. Exhibit 4-43 summarizes the information needed to calculate the average RSI cost for the population.

Exhibit 4-43: Average RSI Cost for the Unsignalized Intersection Population

Unsignalized Intersection	Rear End	Sideswipe	Angle	Ped/Bike	Head-On	Fixed Object	Other	Total
Number of Crashes Over Three Years								
2	4	2	21	2	5	0	1	35
3	11	5	2	1	0	4	0	23
7	19	7	5	0	0	3	0	34
10	9	4	2	0	0	1	1	17
15	9	4	1	0	0	1	2	17
17	6	2	2	0	1	0	2	13
19	5	4	0	1	0	0	1	11
Total Crashes in Unsignalized Intersection Population								*150*
RSI Crash Costs per Crash Type								
2	$52,800	$68,000	$1,283,100	$317,800	$237,500	$0	$55,100	$2,014,300
3	$145,200	$170,000	$122,200	$158,900	$0	$378,800	$0	$975,100
7	$250,800	$238,000	$305,500	$0	$0	$284,100	$0	$1,078,400
10	$118,800	$136,000	$122,200	$0	$0	$94,700	$55,100	$526,800
15	$118,800	$136,000	$61,100	$0	$0	$94,700	$110,200	$520,800
17	$79,200	$68,000	$122,200	$0	$47,500	$0	$110,200	$427,100
19	$66,000	$136,000	$0	$158,900	$0	$0	$55,100	$416,000
Sum of Total RSI Costs for Unsignalized Intersections								*$5,958,500*
Average RSI Cost for Unsignalized Intersections ($5,958,500/150)								*$39,700*



STEP 4 – Rank Locations and Compare

The average RSI costs are calculated by dividing the RSI crash cost for each intersection by the number of crashes for the same intersection. The average RSI cost per intersection is also compared to the average RSI cost for its respective population.

Exhibit 4-44 shows the intersection ranking for all 20 intersections based on their average RSI costs. The RSI costs for Intersection 7 would be compared to the average RSI cost for the unsignalized intersection population. In this instance, the average RSI cost for Intersection 7 ($31,700) is less than the average RSI cost for all unsignalized intersections ($39,700 from Exhibit 4-43).

Exhibit 4-44: Ranking Based on Average RSI Cost per Intersection

Intersection	Average RSI Cost[1]	Exceeds RSI$_p$
2	$57,600	X
14	$52,400	X
6	$48,900	X
9	$44,100	X
20	$43,100	X
3	$42,400	X
4	$42,000	X
12	$41,000	X
11	$39,900	X
16	$39,500	
19	$37,800	
1	$37,400	
13	$34,800	
8	$34,600	
18	$34,100	
17	$32,900	
7	$31,700	
5	$31,400	
10	$31,000	
15	$30,600	

Note: [1]Average RSI Costs per Intersection are rounded to the nearest $100.

4.4.2.5. Critical Rate

The observed crash rate at each site is compared to a calculated critical crash rate that is unique to each site. Sites that exceed their respective critical rate are flagged for further review. The critical crash rate depends on the average crash rate at similar sites, traffic volume, and a statistical constant that represents a desired confidence level. Exhibit 4-45 provides a summary of the strengths and limitations of the performance measure.

Data Needs

- Crashes by location
- Traffic Volume

Strengths and Limitations

Exhibit 4-45: Strengths and Limitations of the Critical Rate Performance Measure

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

Procedure

The following outlines the assumptions and procedure for applying the critical rate method. The calculations for Intersection 7 are used throughout the sample problems to highlight how to apply each method.

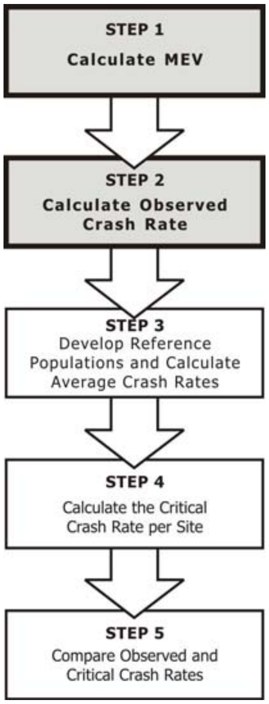
Assumptions

Calculations in the following steps were conducted using a P-value of 1.645 which corresponds to a 95% confidence level. Other possible confidence levels are shown in Exhibit 4-46, based on a Poisson distribution and one-tailed standard normal random variable.

Exhibit 4-46: Confidence Levels and P Values for Use in Critical Rate Method

Confidence Level	P_c – Value
85 Percent	1.036
90 Percent	1.282
95 Percent	1.645
99 Percent	2.326
99.5 Percent	2.576

Source: Road Safety Manual, PIARC Technical Committee on Road Safety, 2003, p. 113



STEP 1 – Calculate MEV for Each Intersection

Calculate the volume in terms of million entering vehicles for all 3 years. Equation 4-8 is used to calculate the million entering vehicles (MEV) at an intersection.

$$MEV = \left({}^{TEV}\!/_{1,000,000} \right) \times (n) \times (365) \tag{4-8}$$

Where,

MEV = Million entering vehicles

TEV = Total entering vehicles per day

n = Number of years of crash data

> Shown below is the calculation for Intersection 7. The TEV is found in Exhibit 4-28.
>
> $$MEV = \left({}^{22,000}\!/_{1,000,000} \right) \times (3) \times (365) = 24.1$$

STEP 2 – Calculate the Crash Rate for Each Intersection

Calculate the crash rate for each intersection by dividing the number of crashes by MEV, as shown in Equation 4-9.

$$R_i = {}^{N_{observed,i(TOTAL)}}\!/_{MEV_i} \tag{4-9}$$

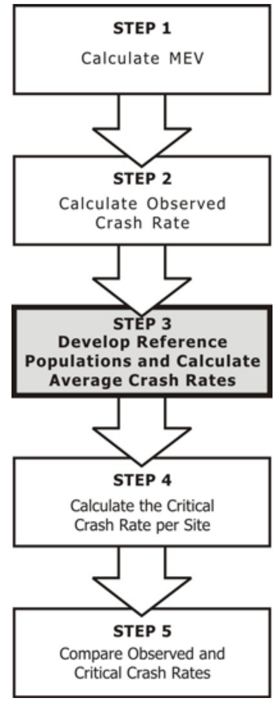
Where,

R_i = Observed crash rate at intersection i

$N_{observed,i(TOTAL)}$ = Total observed crashes at intersection i

MEV_i = Million entering vehicles at intersection i

> Below is the crash rate calculation for Intersection 7. The total number of crashes for each intersection is summarized in Exhibit 4- and the MEV is noted in Step 1.
>
> $$R_i = {}^{34}\!/_{24.1} = 1.41 \text{ [crashes/MEV]}$$



STEP 3 - Calculate Weighted Average Crash Rate per Population

Divide the network into reference populations based on operational or geometric differences and calculate a weighted average crash rate for each population weighted by traffic volume using Equation 4-10.

$$R_a = \frac{\sum_{i=1} (TEV_i \times R_i)}{\sum_{i=1} (TEV_i)} \qquad (4\text{-}10)$$

Where,

R_a = Weighted average crash rate for reference population

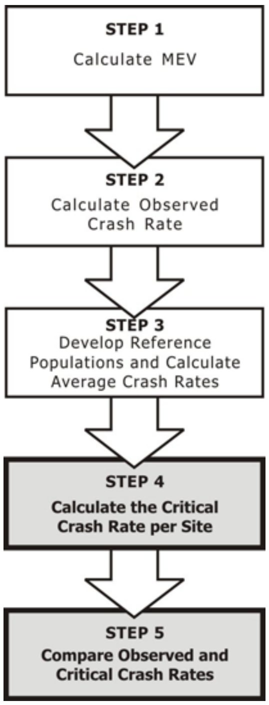
R_i = Observed crash rate at site *i*

TEV_i = Total entering vehicles per day for intersection *i*

For this sample problem the populations are two-way stop-controlled intersections (TWSC) and intersections controlled by traffic signals as summarized in Exhibit 4-47.

Exhibit 4-47: Network Reference Populations and Average Crash Rate

Two-way Stop Controlled	Crash Rate	Weighted Average Crash Rate
2	2.42	1.03
3	1.12	
7	1.41	
10	0.94	
15	0.59	
17	0.67	
19	0.56	
Signalized	Crash Rate	**Weighted Average Crash Rate**
1	0.58	0.42
4	0.54	
5	0.28	
6	0.23	
8	0.18	
9	0.61	
11	0.79	
12	0.45	
13	0.24	
14	0.20	
16	0.97	
18	0.79	
20	0.12	



STEP 4 – Calculate Critical Crash Rate for Each Intersection

Calculate a critical crash rate for each intersection using Equation 4-11.

$$R_{c,i} = R_a + \left[P \times \sqrt{\frac{R_a}{MEV_i}} \right] + \left[\frac{1}{(2 \times (MEV_i))} \right] \tag{4-11}$$

Where,

$R_{c,i}$ = Critical crash rate for intersection *i*

R_a = Weighted average crash rate for reference population

P = P-value for corresponding confidence level

MEV_i = Million entering vehicles for intersection i

> For Intersection 7, the calculation of the critical crash rate is shown below.
>
> $$R_{C,7} = 1.03 + \left[1.645 \times \sqrt{\left(\frac{1.03}{24.1} \right)} \right] + \left[\frac{1}{(2 \times 24.1)} \right] = 1.40 \text{ [crashes/MEV]}$$

STEP 5– Compare Observed Crash Rate with Critical Crash Rate

Observed crash rates are compared with critical crash rates. Any intersection with an observed crash rate greater than the corresponding critical crash rate is flagged for further review.

> The critical crash rate for Intersection 7 is compared to the observed crash rate for Intersection 7 to determine if further review of Intersection 7 is warranted.
>
> - Critical Crash Rate for Intersection 7 = 1.40 [crashes/MEV]
> - Observed Crash Rate for Intersection 7 = 1.41 [crashes/MEV]
>
> Since 1.41>1.40, Intersection 7 is identified for further review.

Exhibit 4-48 summarizes the results for all 20 intersections being screened by the roadway agency.

Exhibit 4-48: Critical Rate Method Results

Intersection	Observed Crash Rate (crashes/MEV)	Critical Crash Rate (crashes/MEV)	Identified for Further Review
1	0.58	0.60	
2	2.42	1.51	X
3	1.12	1.43	
4	0.54	0.66	
5	0.28	0.57	
6	0.23	0.60	
7	1.41	1.40	X
8	0.18	0.58	
9	0.61	0.56	X
10	0.94	1.45	
11	0.79	0.58	X
12	0.45	0.55	
13	0.24	0.65	
14	0.20	0.58	
15	0.59	1.36	
16	0.97	0.67	X
17	0.67	1.44	
18	0.79	0.66	X
19	0.56	1.44	
20	0.12	0.56	

4.4.2.6. Excess Predicted Average Crash Frequency Using Method of Moments

In the method of moments, a site's observed accident frequency is adjusted to partially account for regression to the mean. The adjusted observed average crash frequency is compared to the average crash frequency for the reference population to determine the potential for improvement (PI). The potential for improvement of all reference populations (e.g., signalized four-legged intersections, unsignalized three-legged intersections, urban and rural, etc.) are combined into one ranking list as a basic multiple-facility network screening tool.

Data Needs

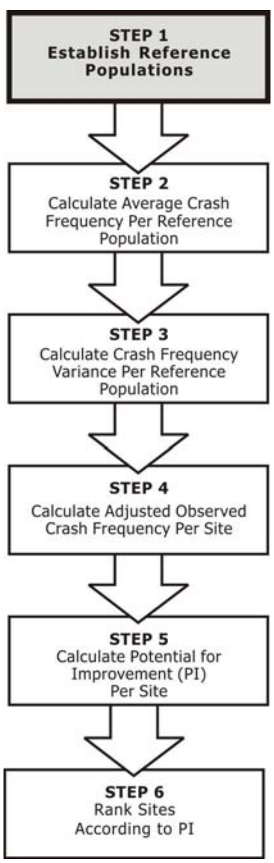
- Crashes by location
- Multiple reference populations

Strengths and Limitations

Exhibit 4-49 provides a summary of the strengths and limitations of the performance measure.

Exhibit 4-49: Strengths and Limitations of Excess Predicted Average Crash Frequency Using Method of Moments Performance Measure

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



Procedure

The following outlines the procedure for ranking intersections using the Method of Moments. The calculations for Intersection 7 are used throughout the sample problems to highlight how to apply each method.

STEP 1 – Establish Reference Populations

Organize historical crash data of the study period based upon factors such as facility type, location, or other defining characteristics.

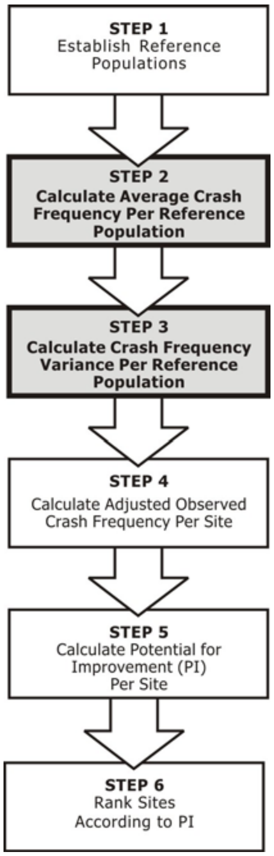
The intersections from Exhibit 4-28 have been organized into two reference populations, as shown in Exhibit 4-50 for two-way stop controlled intersections and Exhibit 4-51 for signalized intersections.

Exhibit 4-50: TWSC Reference Population

Intersection ID	Traffic Control	Number of Approaches	Urban/ Rural	Total Crashes	Average Observed Crash Frequency
2	TWSC	4	U	35	11.7
3	TWSC	4	U	23	7.7
7	TWSC	4	U	34	11.3
10	TWSC	4	U	17	5.7
15	TWSC	4	U	17	5.7
17	TWSC	4	U	13	4.3
19	TWSC	4	U	11	3.7
Sum				*150*	*50.1*

Exhibit 4-51: Signalized Reference Population

Intersection ID	Traffic Control	Number of Approaches	Urban/ Rural	Total Crashes	Average Observed Crash Frequency
1	Signal	4	U	22	7.3
4	Signal	4	U	13	4.3
5	Signal	4	U	15	5.0
6	Signal	4	U	9	3.0
8	Signal	4	U	9	3.0
9	Signal	4	U	37	12.3
11	Signal	4	U	38	12.7
12	Signal	4	U	32	10.7
13	Signal	4	U	6	2.0
14	Signal	4	U	10	3.3
16	Signal	4	U	21	7.0
18	Signal	4	U	19	6.3
20	Signal	4	U	8	2.7
Sum				*239*	*79.6*



STEP 2 – Calculate Average Crash Frequency per Reference Population

Sum the average annual observed crash frequency for each site in the reference population and divide by the number of sites.

$$N_{observed\ rp} = \frac{\sum_{i=1}^{n} N_{observed,i}}{n_{sites}} \qquad (4\text{-}12)$$

Where,

$N_{observed\ rp}$ = Average crash frequency, per reference population

$N_{observed,i}$ = Observed crash frequency at site i

$n_{(sites)}$ = Number of sites per reference population

> Shown below is the calculation for observed average crash frequency in the TWSC reference population.
>
> $$N_{observed,TWSC} = \frac{50}{7} = 7.1 \text{ [crashes per year]}$$

STEP 3 – Calculate Crash Frequency Variance per Reference Population

Use Equation 4-13 to calculate variance. Alternatively, variance can be more easily calculated with common spreadsheet programs.

$$Var(N) = \frac{\sum_{i=1}^{n} \left(N_{observed,\ i} - N_{observed\ \ rp}\right)^{(2)}}{n_{sites} - 1} \qquad (4\text{-}13)$$

Where,

$Var(N)$ = Variance

$N_{observed\ rp}$ = Average crash frequency, per reference population

$N_{observed,i}$ = Observed crash frequency per year at site i

$n_{(sites)}$ = Number of sites per reference population

Shown below is the crash frequency variance calculation for the TWSC reference population. The variance for signal and TWSC reference populations is shown in Exhibit 4-52.

$$s_{TWSC}{}^{2} = \frac{112.8}{6} = 18.8$$

Exhibit 4-52: Reference Population Summary

Reference Population	Crash Frequency	
	Average	Variance
Signal	6.1	10.5
TWSC	7.1	18.8

STEP 4 – Calculate Adjusted Observed Crash Frequency per Site

Using the variance and average crash frequency for a reference population, find the adjusted observed crash frequency for each site using Equation 4-14.

$$N_{observed,i(adj)} = N_{obseved,i} + \frac{N_{observed,rp}}{s^2} \times \left(N_{observed,rp} - N_{observed,i}\right) \quad (4\text{-}14)$$

Where,

$N_{observed,i(adj)}$ = Adjusted observed number of crashes per year, per site

$Var(N)$ = Variance

$N_{observed,\ rp}$ = Average crash frequency, per reference population

$N_{observed,i}$ = Observed average crash frequency per year at site i

Shown below is the adjusted observed average crash frequency calculation for intersection 7.

$$N_{observed,7(adj)} = 11.3 + \frac{7.1}{10.5} \times (7.1 - 11.3) = 8.5 \text{ [crashes per year]}$$

STEP 5 – Calculate Potential for Improvement per Site

Subtract the average crash frequency per reference population from the adjusted observed average crash frequency per site.

$$PI_i = N_{observed,i(adj)} - N_{observed,rp} \quad (4\text{-}15)$$

Where,

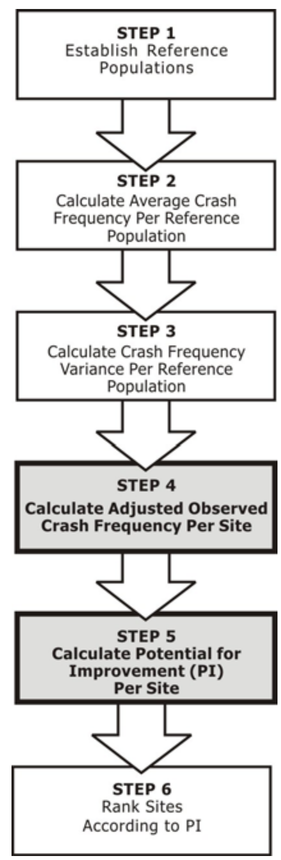
PI_i = Potential for Improvement per site

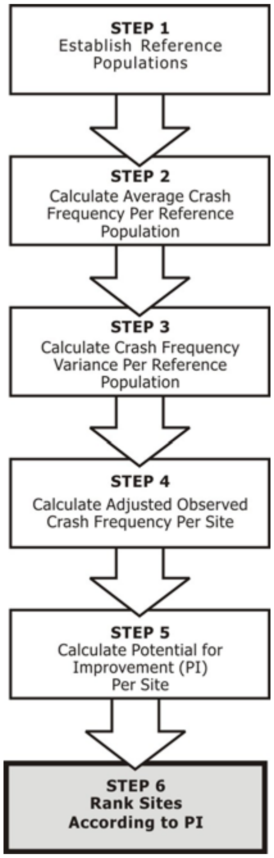
$N_{observed,i(adj)}$ = Adjusted observed average crash frequency per year, per site

$N_{observed\ rp}$ = Average crash frequency, per reference population

Shown below is the potential for improvement calculation for intersection 7.

$$PI_7 = 8.5 - 7.1 = 1.4 \text{ [crashes/year]}$$





STEP 6 – Rank Sites According to PI

Rank all sites from highest to lowest PI value. A negative PI value is not only possible but indicates a low potential for crash reduction.

Exhibit 4-53 summarizes the rankings along with each site's adjusted observed crash frequency.

Exhibit 4-53: Rank According to PI

Intersections	Observed Average Crash Frequency	Adjusted Observed Crash Frequency	PI
11	12.7	9.8	3.6
9	12.3	9.6	3.4
12	10.7	8.6	2.5
2	11.7	8.6	1.4
7	11.3	8.5	1.4
1	7.3	6.8	0.7
16	7.0	6.6	0.5
3	7.7	7.3	0.2
18	6.3	6.2	0.1
10	5.7	6.7	-0.5
15	5.7	6.7	-0.5
5	5.0	5.5	-0.6
17	4.3	6.3	-0.9
4	4.3	5.1	-1.0
19	3.7	6.0	-1.1
14	3.3	4.6	-1.5
6	3.0	4.4	-1.7
8	3.0	4.4	-1.7
20	2.7	4.2	-1.9
13	2.0	3.8	-2.3

4.4.2.7. Level of Service of Safety (LOSS)

Sites are ranked by comparing their observed average crash frequency to the predicted average crash frequency for the entire population under consideration.[1,4,5] The degree of deviation from the predicted average crash frequency is divided into four LOSS classes. Each site is assigned a LOSS based on the difference between the observed average crash frequency and the predicted average crash frequency for the study group. Sites with poor LOSS are flagged for further study.

Data Needs

- Crash data by location (recommended period of 3 to 5 Years)
- Calibrated Safety Performance Function (SPF) and overdispersion parameter
- Traffic volume

Strengths and Limitations

Exhibit 4-54 provides a summary of the strengths and limitations of the performance measure.

Exhibit 4-54: Strengths and Limitations of LOSS Performance Measure

Strengths	Limitations
• Considers variance in crash data	• Effects of RTM bias may still be present in the results
• Accounts for volume	
• Establishes a threshold for measuring crash frequency	

Procedure

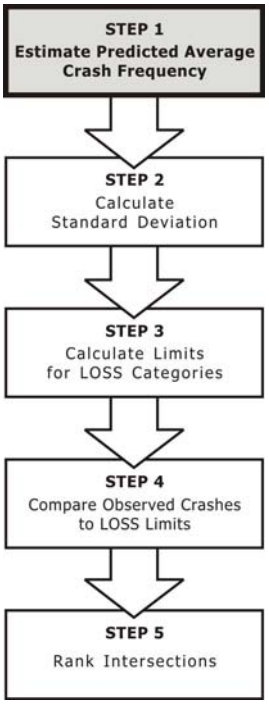
The following sections outline the assumptions and procedure for ranking the intersections using the LOSS performance measure.

Sample Problem Assumptions

The calculations for Intersection 7 are used throughout the sample problem to demonstrate how to apply each method.

The Sample problems provided in this section are intended to demonstrate calculation of the performance measures, not the predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using the predictive method outlined in *Part C* and are provided in Exhibit 4-30 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the base SPF model. It is also assumed that all AMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are to simplify this example and are rarely valid for application of the predictive method to actual field conditions.

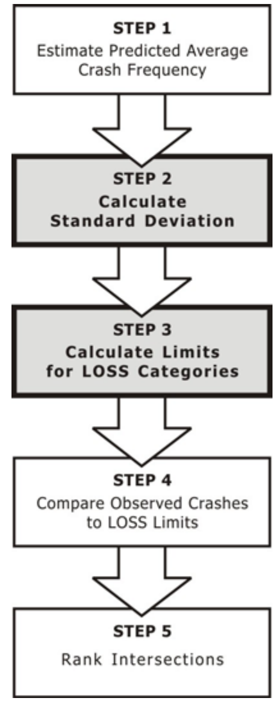


STEP 1 – Estimate Predicted Average Crash Frequency Using an SPF

Use the predictive method and SPFs outlined in *Part C* to estimate the average crash frequency. The predicted average crash frequency is summarized in Exhibit 4-55.

Exhibit 4-55: Estimated Predicted Average Crash Frequency from an SPF

Intersection	Year	AADT Major Street	AADT Minor Street	Predicted Average Crash Frequency from an SPF	Average 3-Year Expected Crash Frequency from an SPF
2	1	12,000	1,200	1.7	1.7
	2	12,200	1,200	1.7	
	3	12,900	1,300	1.8	
3	1	18,000	800	2.1	2.2
	2	18,900	800	2.2	
	3	19,100	800	2.2	
7	1	21,000	1,000	2.5	2.6
	2	21,400	1,000	2.5	
	3	22,500	1,100	2.7	
10	1	15,000	1,500	2.1	2.2
	2	15,800	1,600	2.2	
	3	15,900	1,600	2.2	
15	1	26,000	500	2.5	2.3
	2	26,500	300	2.2	
	3	27,800	200	2.1	
17	1	14,400	3,200	2.5	2.6
	2	15,100	3,400	2.6	
	3	15,300	3,400	2.6	
19	1	15,400	2,500	2.4	2.5
	2	15,700	2,500	2.5	
	3	16,500	2,600	2.6	



STEP 2 – Calculate Standard Deviation

Calculate the standard deviation of the predicted crashes. Equation 4-16 is used to calculate the standard deviation. This estimate of standard deviation is valid since the SPF assumes a negative binomial distribution of crash counts.

$$\sigma = \sqrt{N_{predicted} + k \times N_{predicted}{}^{2}} \qquad (4\text{-}16)$$

Where,

σ = Standard deviation

k = Overdispersion parameter of the SPF

$N_{predicted}$ = Predicted average crash frequency from the SPF

The standard deviation calculations for Intersection 7 are below.

$$\sigma = \sqrt{2.6 + 0.40 \times 2.6^{2}} = 2.3$$

The standard deviation calculation is performed for each intersection. The standard deviation for the TWSC intersections is summarized in Exhibit 4-56.

Exhibit 4-56: Summary of Standard Deviation Calculations

Intersection	Average Observed Crash Frequency	Predicted Average Crash Frequency from an SPF	Standard Deviation
2	11.7	1.7	1.7
3	7.7	2.2	2.0
7	11.3	2.6	2.3
10	5.7	2.2	2.0
15	5.7	2.3	2.1
17	4.3	2.6	2.3
19	3.7	2.5	2.2

STEP 3 – Calculate Limits for LOSS Categories

Calculate the limits for the four LOSS categories for each intersection using the equations summarized in Exhibit 4-57.

Exhibit 4-57: LOSS Categories

LOSS	Condition	Description
I	$0 < K < (N - 1.5 \times (\sigma))$	Indicates a low potential for crash reduction
II	$(N - 1.5 \times (\sigma)) \leq K < N$	Indicates low to moderate potential for crash reduction
III	$N \leq K < (N + 1.5 \times (\sigma))$	Indicates moderate to high potential for crash reduction
IV	$K_i \geq (N + 1.5 \times (\sigma))$	Indicates a high potential for crash reduction

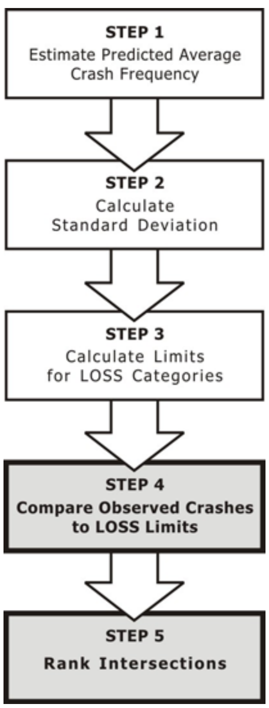
Below is a sample calculation for Intersection 7 that demonstrates the upper limit calculation for LOSS III. The values for this calculation are provided in Exhibit 4-58.

$$N + 1.5 \times (\sigma) = 2.6 + 1.5 \times (2.3) = 6.1$$

A similar pattern is followed for the other LOSS limits.

Exhibit 4-58: LOSS Limits for Intersection 7

Intersection	LOSS I Limits	LOSS II Limits	LOSS III Upper Limit	LOSS IV Limits
7	-	0 to 2.5	2.6 to 6.1	≥ 6.1



STEP 4 – Compare Observed Crashes to LOSS Limits

Compare the total observed crash frequency at each intersection, N_O, to the limits of the four LOSS categories. Assign a LOSS to each intersection based on the category in which the total observed crash frequency falls.

Given that an average of 11.3 crashes were observed per year at intersection 7 and the LOSS IV limits are 6.1 crashes per year, Intersection 7 is categorized as Level IV.

STEP 5 – Rank Intersections

List the intersections based on their LOSS for total crashes.

Exhibit 4-59 summarizes the TWSC reference population intersection ranking based on LOSS.

Exhibit 4-59: Intersection LOSS Ranking

Intersection	LOSS
2	IV
3	IV
7	IV
10	IV
15	IV
17	III
19	III

4.4.2.8. Excess Predicted Average Crash Frequency Using SPFs

Locations are ranked in descending order based on the excess crash frequency or the excess predicted crash frequency of a particular collision type or crash severity.

Data Needs

- Crash data by location

Strengths and Limitations

Exhibit 4-60 provides a summary of the strengths and limitations of the performance measure.

Exhibit 4-60: Strengths and Limitations of the Excess Predicted Average Crash Frequency Using SPFs performance measure

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

Procedure

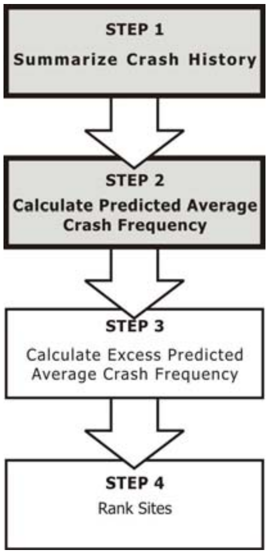
The following sections outline the assumptions and procedure for ranking intersections using the Excess Predicted Crash Frequency using SPFs performance measure.

Sample Problem Assumptions

The Sample problems provided in this section are intended to demonstrate calculation of the performance measures, not predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using predictive method outlined in *Part C* and are provided in Exhibit 4-30 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the SPF. It is also assumed that all AMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the SPF. These assumptions are for theoretical application and are rarely valid for application of Part C predictive method to actual field conditions.

Safety Performance Functions are used to estimate a site's expected crash experience. Chapter 3 Fundamentals explains safety performance functions in more detail.



STEP 1 – Summarize Crash History

Tabulate the number of crashes by type and severity at each site for each reference population being screened.

The reference population for TWSC intersections is shown in Exhibit 4-61 as an example.

Exhibit 4-61: TWSC Reference Population

Intersection	Year	AADT		Observed Number of Crashes	Average Observed Crash Frequency
		Major Street	Minor Street		
2	1	12,000	1,200	9	11.7
	2	12,200	1,200	11	
	3	12,900	1,300	15	
3	1	18,000	800	9	7.7
	2	18,900	800	8	
	3	19,100	800	6	
7	1	21,000	1,000	11	11.3
	2	21,400	1,000	9	
	3	22,500	1,100	14	
10	1	15,000	1,500	7	5.7
	2	15,800	1,600	6	
	3	15,900	1,600	4	
15	1	26,000	500	6	5.7
	2	26,500	300	3	
	3	27,800	200	8	
17	1	14,400	3,200	4	4.3
	2	15,100	3,400	4	
	3	15,300	3,400	5	
19	1	15,400	2,500	5	3.7
	2	15,700	2,500	2	
	3	16,500	2,600	4	

STEP 2 – Calculate Predicted Average Crash Frequency from an SPF

Using the predictive method in *Part C* calculate the predicted average crash frequency, $N_{predicted,n}$, for each year, n, where n = 1,2,…,Y. Refer to *Part C Introduction and Applications Guidance* for a detailed overview of the method to calculate the predicted average crash frequency. The example provided here is simplified to emphasize calculation of the performance measure, not the predictive method.

The predicted average crash frequency from SPFs are summarized for the TWSC intersections for a three-year period in Exhibit 4-62.

Exhibit 4-62: SPF Predicted Average Crash Frequency

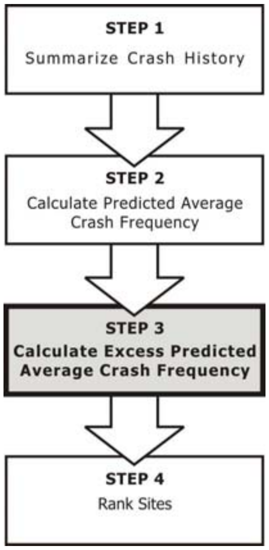
Intersection	Year	Predicted Average Crash Frequency from SPF (Total)	Predicted Average Crash Frequency from an SPF (FI)	Predicted Average Crash Frequency from an SPF (PDO)	Average 3-Year Predicted Crash Frequency from SPF
2	1	1.7	0.6	1.1	1.7
	2	1.7	0.6	1.1	
	3	1.8	0.7	1.1	
3	1	2.1	0.8	1.3	2.2
	2	2.2	0.8	1.4	
	3	2.2	0.9	1.4	
7	1	2.5	1.0	1.6	2.6
	2	2.5	1.0	1.6	
	3	2.7	1.1	1.7	
10	1	2.1	0.8	1.3	2.2
	2	2.2	0.9	1.4	
	3	2.2	0.9	1.4	
15	1	2.5	1.0	1.6	2.3
	2	2.2	0.9	1.4	
	3	2.1	0.8	1.3	
17	1	2.5	1.0	1.5	2.6
	2	2.6	1.0	1.6	
	3	2.6	1.0	1.6	
19	1	2.4	1.0	1.5	2.5
	2	2.5	1.0	1.5	
	3	2.6	1.0	1.6	

STEP 3 – Calculate Excess Predicted Average Crash Frequency

For each intersection the excess predicted average crash frequency is based upon the average of all years of data. The excess is calculated as the difference in the observed average crash frequency and the predicted average crash frequency from an SPF.

$$Excess(N) = \overline{N_{observed,i}} - \overline{N_{predicted,i}} \quad (4\text{-}17)$$

Where,



$\overline{N}_{observed,i}$ = Observed average crash frequency for site *i*

$\overline{N}_{predicted,i}$ = Predicted average crash frequency from SPF for site.

Shown below is the predicted excess crash frequency calculation for Intersection 7.

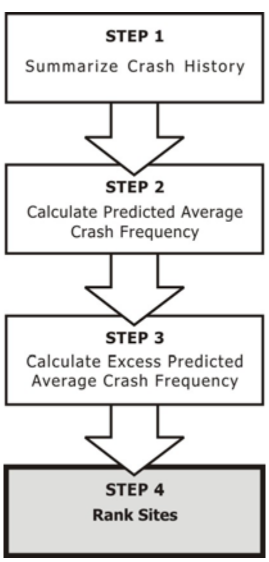
$$Excess_{(TWSC)} = 11.3 - 2.6 = 8.7 \text{ [crashes per year]}$$

Exhibit 4-63 shows the excess expected average crash frequency for the TWSC reference population.

Exhibit 4-63: Excess Predicted Average Crash Frequency for TWSC Population

Intersection	Observed Average Crash Frequency	Predicted Average Crash Frequency from an SPF	Excess Predicted Average Crash Frequency
2	11.7	1.7	10.0
3	7.7	2.2	5.5
7	11.3	2.6	8.7
10	5.7	2.2	3.5
15	5.7	2.3	3.4
17	4.3	2.6	1.7
19	3.7	2.5	1.2

STEP 4 – Rank Sites



Rank all sites in each reference population according to the excess predicted average crash frequency.

The ranking for the TWSC intersections are below in Exhibit 4-64, according to the excess predicted average crash frequency.

Exhibit 4-64: Ranking of TWSC Population Based on Excess Predicted Average Crash Frequency from an SPF

Intersection	Excess Predicted Average Crash Frequency
2	10.0
7	8.7
3	5.5
10	3.5
15	3.4
17	1.7
19	1.2

4.4.2.9. Probability of Specific Crash Types Exceeding Threshold Proportion

Sites are prioritized based on the probability that the true proportion, p_i, of a particular crash type or severity (e.g., long-term predicted proportion) is greater than the threshold proportion, p^*_i.[6] A threshold proportion (p^*_i) is identified for each crash type.

Data Needs

- Crash data by type and location

Strengths and Limitations

Exhibit 4-65 summarizes the strengths and limitations of the Probability of Specific Crash Types Exceeding Threshold Proportion performance measure.

Exhibit 4-65: Strengths and Limitations of the Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure

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

Procedure

Organize sites into reference populations and screen to identify those that have a high proportion of a specified collision type or crash severity.

> The sample intersections are to be screened for a high proportion of angle crashes. Prior to beginning the method, the 20 intersections are organized into two subcategories (i.e., reference populations): TWSC intersections, and signalized intersections.

STEP 1 – Calculate Observed Proportions

A. Determine which collision type or crash severity to target and calculate observed proportion of target collision type or crash severity for each site.

B. Identify the frequency of the collision type or crash severity of interest and the total observed crashes of all types and severity during the study period at each site.

C. Calculate the observed proportion of the collision type or crash severity of interest for each site that has experienced two or more crashes of the target collision type or crash severity using Equation 4-18.

$$p_i = {N_{observed,i}} \Big/ {N_{observed,i(TOTAL)}} \qquad (4\text{-}18)$$

Where,

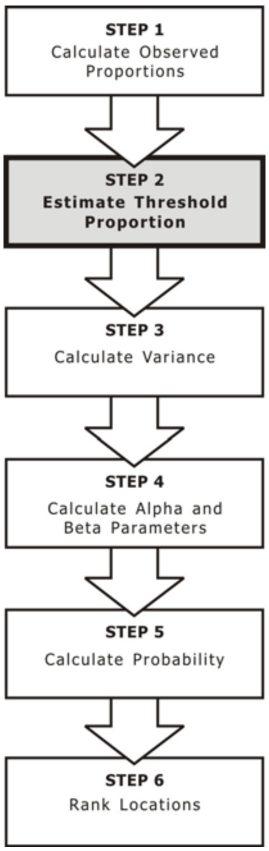
p_i = Observed proportion at site i

$N_{observed,i}$ = Number of observed target crashes at site i

$N_{observed,i(TOTAL)}$ = Total number of crashes at site i

Shown below is the calculation for angle crashes for Intersection 7. The values used in the calculation are found in Exhibit 4-.

$$p_i = 5/34 = 0.15$$



STEP 2 – Estimate a Threshold Proportion

Select the threshold proportion of crashes, p*i, for a specific collision type. A useful default starting point is the proportion of target crashes in the reference population under consideration. For example, if considering rear end crashes, it would be the observed average rear-end crash frequency experienced at all sites in the reference population divided by the total observed average crash frequency at all sites in the reference population. The proportion of a specific crash type in the entire population is calculated using Equation 4-19.

$$p^*_i = \sum N_{observed,i} \Big/ \sum N_{observed,i(TOTAL)} \qquad (4\text{-}19)$$

Where,

p^*_i = Threshold proportion

$\sum N_{observed,i}$ = Sum of observed target crash frequency within the population

$\sum N_{observed,i(TOTAL)}$ = Sum of total observed crash frequency within the population

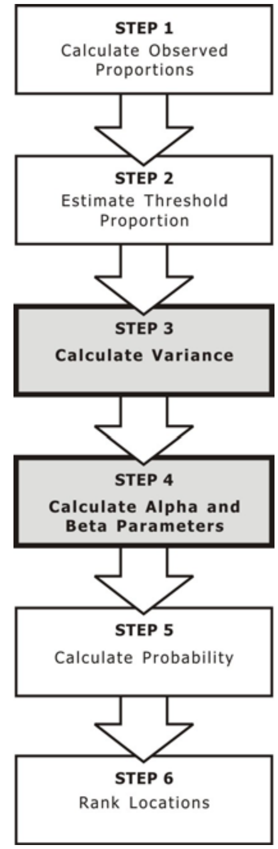
Below is the calculation for threshold proportion of angle collisions for TWSC intersections.

$$p^*_i = 33/150 = 0.22$$

Exhibit 4-66 summarizes the threshold proportions for the reference populations.

Exhibit 4-66: Estimated Threshold Proportion of Angle Collisions

Reference Population	Angle Crashes	Total Crashes	Observed Threshold Proportion (p^*_i)
TWSC	33	150	0.22
Traffic Signals	82	239	0.34



STEP 3 – Calculate Sample Variance

Calculate the sample variance (s^2) for each subcategory. The sample variance is different than population variance. Population variance is commonly used in statistics and many software tools and spreadsheets use the population variance formula as the default variance formula.

For this method, be sure to calculate the sample variance using Equation 4-20:

$$Var(N) = \left(\frac{1}{n_{sites}-1}\right) \times \left[\sum_{i=1}^{n} \left(\frac{N_{observed,\, i}^{\,2} - N_{observed,\, i}}{N_{observed,\, i(TOTAL)}^{\,2} - N_{observed,\, i(TOTAL)}} \right) - \left(\frac{1}{n_{sites}}\right) \times \left(\sum_{i=1}^{n} \frac{N_{observed,\, i}}{N_{observed,\, i(TOTAL)}} \right)^{(2)} \right] \quad \text{(4-20)}$$

for $N_{observed,i(TOTAL)} \geq 2$

Where,

n_{sites} = Number of sites in the subcategory

$N_{observed,i}$ = Observed target crashes for a site *i*

$N_{observed,i(TOTAL)}$ = Total number of crashes for a site *i*

Exhibit 4-67 summarizes the calculations for the two-way stop-controlled subcategory.

Exhibit 4-67: Sample Variance Calculation[1]

TWSC	Angle Crashes ($N_{Observed,i}$)	$(N_{Observed,i})^2$	Total Crashes ($N_{Observed,i(TOTAL)}$)	$(N_{Observed,i(TOTAL)})^2$	n	TWSC Variance
2	21	441	35	1225	7	0.034
7	5	25	34	1156		
3	2	4	23	529		
10	2	4	17	289		
17	2	4	13	169		
15	1	1	17	289		
19	0	0	11	121		

STEP 4 – Calculate Alpha and Beta Parameters

Calculate Alpha (α) and Beta (β) for each subcategory using Equations 4-21 and 4-22.

$$\alpha = \frac{\overline{p^*_i}^{\,2} - \overline{p^*_i}^{\,3} - s^2(\overline{p^*_i})}{s^2} \quad \text{(4-21)}$$

$$\beta = \frac{\alpha}{\overline{p^*_i}} - \alpha \quad \text{(4-22)}$$

Where,

$Var(N)$ = Variance

$\overline{p^*_i}$ = Mean proportion

Below is the calculation for the two-way stop-controlled subcategory. The numerical values shown in the equations below are summarized in Exhibit 4-68.

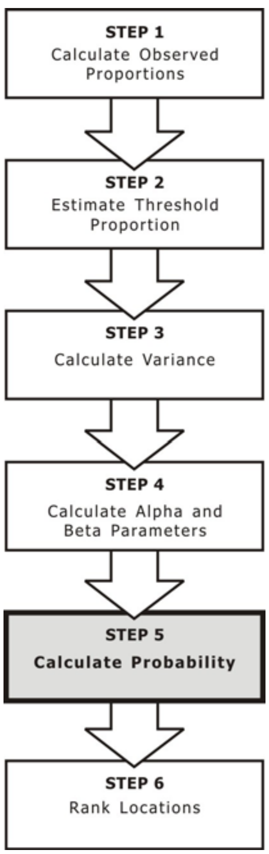
$$\alpha = \frac{0.22^2 - 0.22^3 - 0.034x0.22}{0.034} = 0.91$$

$$\beta = \left(0.91 \middle/ 0.22\right) - 0.91 = 3.2$$

Exhibit 4-68 summarizes the alpha and beta calculations for the TWSC intersections.

Exhibit 4-68: Alpha and Beta Calculations

Subcategories	s^2	$\overline{p^*_i}$	α	β
TWSC	0.034	0.22	0.91	3.2



STEP 5 – Calculate the Probability

Using a "betadist" spreadsheet function, calculate the probability for each intersection as shown in Equation 4-23.

$$P\left(p_i > \overline{p^*_i} \,/\, N_{observedj}, N_{observedj(TOTAL)}\right) = 1 - betadist\left(\overline{p^*_i}, \alpha + N_{observedj}, \beta + N_{observedj(TOTAL)} - N_{observedj}\right) \quad (4\text{-}23)$$

Where:

$\overline{p^*_i}$ = Threshold proportion

p_i = Observed proportion

$N_{observed,i}$ = Observed target crashes for a site i

$N_{observed,i(TOTAL)}$ = Total number of crashes for a site i

Below is the probability calculation for Intersection 7.

$$P\left(p_i > \overline{p*_i} \;/\; N_{Observed,\ i}, N_{Observed,\ i(TOTAL)}\right) = 1 - betadist\ (0.22, 0.78 + 5, 2.8 + 34 - 5)$$

Exhibit 4-69 summarizes the probability calculation for Intersection 7.

Exhibit 4-69: Probability Calculations

TWSC	Angle Crashes ($N_{Observed,i}$)	Total Crashes ($N_{Observed,i}$)	p_i	$\overline{p^*_i}$	α	β	Probability
7	5	34	0.15	0.22	0.91	3.2	0.14

For Intersection 7, the resulting probability is interpreted as "There is a 14% chance that the long-term expected proportion of angle crashes at Intersection 7 is actually greater than the long-term expected proportion for TWSC intersections." Therefore, in this case, with such a small probability there is limited need of additional study of Intersection 7 with regards to angle crashes.

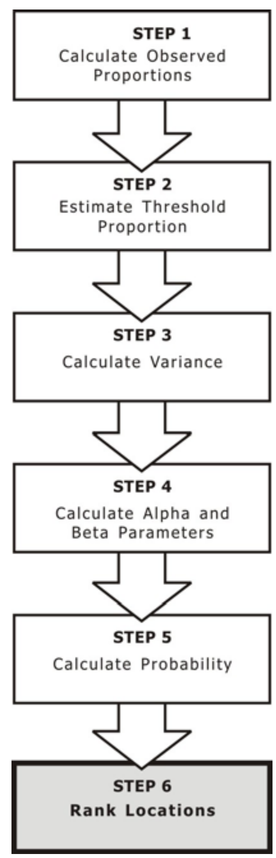
STEP 6 – Rank Locations

Rank the intersections based on the probability of angle crashes occurring at the intersection.

The TWSC intersection population is ranked based on the Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure as shown in Exhibit 4-70.

Exhibit 4-70: Ranking Based on Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure

Intersections	Probability
2	1.00
11	0.97
9	0.72
12	0.63
16	0.32
6	0.32
13	0.32
17	0.26
20	0.26
4	0.21
8	0.15
10	0.14
7	0.14
14	0.13
5	0.11
1	0.10
18	0.09
3	0.05
15	0.04
19	0.02



4.4.2.10. Excess Proportion of Specific Crash Types

Sites are evaluated to quantify the extent to which a specific crash type is overrepresented compared to other crash types at a location. The sites are ranked based on excess proportion, which is the difference between the true proportion, p_i, and the threshold proportion, p^*_i. The excess is calculated for a site if the probability that a site's long-term observed proportion is higher than the threshold proportion, p*i, exceeds a certain limiting probability (e.g., 90 percent).

Data Needs

- Crash data by type and location

Strengths and Limitations

Exhibit 4-71 summarizes the strengths and limitations of the Excess Proportions of Specific Crash Types Proportion performance measure.

Exhibit 4-71: Strengths and Limitations of the Excess Proportions of Specific Crash Types Performance Measure

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

Procedure

Calculation of the excess proportion follows the same procedure outlined in Steps 1 through 6 of the Probability of Specific Crash Types Exceeding Threshold Proportions method. Therefore, the procedure outlined here builds on the previous method and applies results of sample calculations shown in Exhibit 4-70.

> For the sample situation the limiting probability is selected to be 60-percent. The selection of a limiting probability can vary depending on the probabilities of each specific crash types exceeding a threshold proportion. For example, if many sites have high probability, the limiting probability can be correspondingly higher in order to limit the number of sites to a reasonable study size. In this example, a 60-percent limiting probability results in four sites that will be evaluated based on the Excess Proportions performance measure.

STEP 6 – Calculate the Excess Proportion

Calculate the difference between the true observed proportion and the threshold proportion for each site using Equation 4-24:

$$p_{DIFF} = p_i - \overline{p^*_i} \tag{4-24}$$

Where,

$\overline{p^*_i}$ = Threshold proportion

p_i = Observed proportion

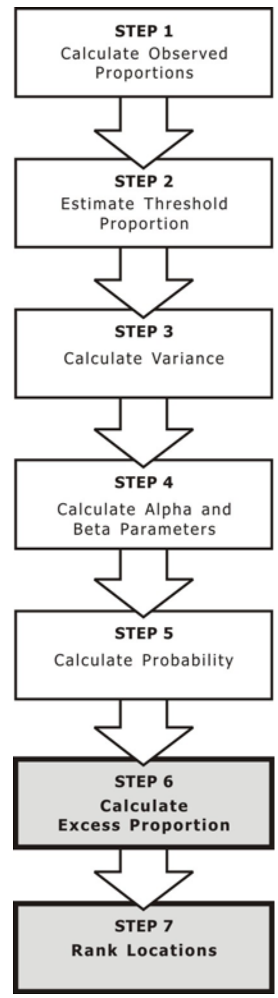
STEP 7 – Rank Locations

Rank locations in descending order by the value of P_{DIFF}. The greater the difference between the observed and threshold proportion, the greater the likelihood that the site will benefit from a countermeasure targeted at the collision type under consideration.

The four intersections that met the limiting probability of 60-percent are ranked in Exhibit 4-72 below.

Exhibit 4-72: Ranking Based on Excess Proportion

Intersections	Probability	Observed Proportion	Threshold Proportion	Excess Proportion
2	1.00	0.60	0.22	0.38
11	0.97	0.61	0.34	0.27
9	0.72	0.46	0.34	0.12
12	0.63	0.44	0.34	0.10



4.4.2.11. Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

The Empirical Bayes (EB) method is applied in the estimation of expected average crash frequency. The EB method, as implemented in this chapter, is implemented in a slightly more sophisticated manner than in the Appendix to *Part C* of the HSM. The version of the EB method implemented here uses yearly correction factors for consistency with network screening applications in the *SafetyAnalyst* software tools.

Data Needs

- Crash data by severity and location
- Traffic volume
- Basic site characteristics (i.e., roadway cross-section, intersection control, etc.)
- Calibrated Safety Performance Functions (SPFs) and overdispersion parameters

Strengths and Limitations

Exhibit 4-73 provides a summary of the strengths and limitations of the Expected Average Crash Frequency with EB Adjustment performance measure.

Exhibit 4-73: Strengths and Limitations Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

Strengths	Limitations
• Accounts for RTM bias	• Requires SPFs calibrated to local conditions

Procedure

The following sample problem outlines the assumptions and procedure for ranking intersections based on the expected average crash frequency with Empirical Bayes adjustments. The calculations for Intersection 7 are used throughout the sample problems to highlight how to apply each method.

Sample Problem Assumptions

The sample problems provided in this section are intended to demonstrate calculation of the performance measures, not predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using predictive method outlined in *Part C* and are provided in Exhibit 4-30 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the SPF. It is also assumed that all AMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are for theoretical application and are rarely valid for application of the *Part C* predictive method to actual field conditions.

STEP 1 – Calculate the Predicted Average Crash Frequency from an SPF

Using the predictive method in *Part C* calculate the predicted average crash frequency, $N_{predicted,n}$, for each year, n, where n = 1,2,…,Y. Refer to *Part C Introduction and Applications Guidance* for a detailed overview of the method to calculate the predicted average crash frequency. The example provided here is simplified to emphasize calculation of the performance measure, not predictive method.

In the following steps this prediction will be adjusted using an annual correction factor and an Empirical Bayes weight. These adjustments will account for annual fluctuations in crash occurrence due to variability in roadway conditions and other similar factors; they will also incorporate the historical crash data specific to the site.

STEP 2 – Calculate Annual Correction Factor

Calculate the annual correction factor (C_n) at each intersection for each year and each severity (i.e., TOTAL and FI).

The annual correction factor is predicted average crash frequency from an SPF for year *n* divided by the predicted average crash frequency from an SPF for year 1. This factor is intended to capture the effect that annual variations in traffic, weather, and vehicle mix have on crash occurrences. [(3)]

$$C_{n(TOT)} = \frac{N_{predicted,n(TOTAL)}}{N_{predicted,1(TOTAL)}} \text{ and } C_{n(FI)} = \frac{N_{predicted,n(FI)}}{N_{predicted,1(FI)}} \qquad (4\text{-}25)$$

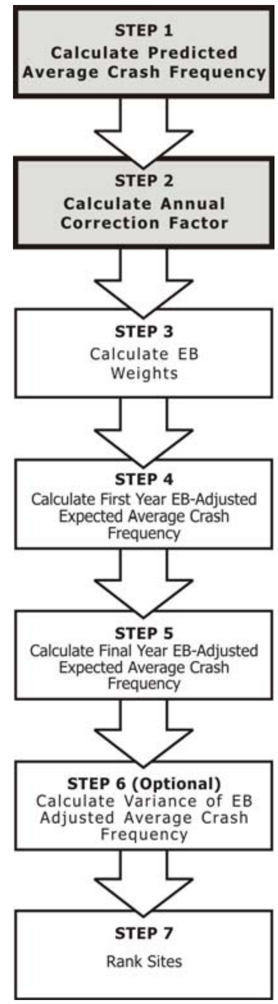
Where,

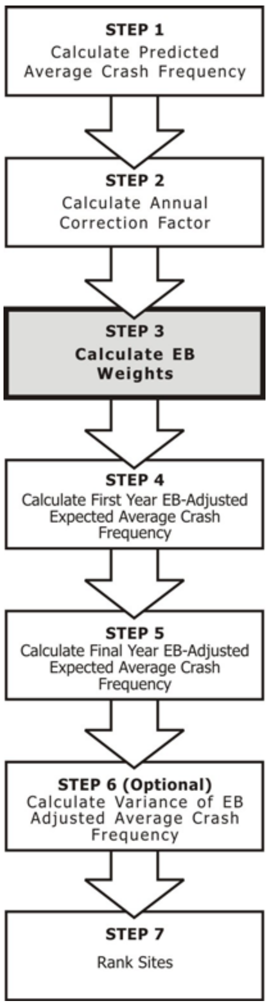
$C_{n(TOTAL)}$ = Annual correction factor for total crashes

$C_{n(F,I)}$ = Annual correction factor for fatal and/or injury crashes

$N_{predicted,n(TOTAL)}$ = Predicted number of total crashes for year n

$N_{predicted,n(FI)}$ = Predicted number of fatal and/or injury crashes for year n





Shown below is the calculation for Intersection 7 based on the annual correction factor for year 3. The predicted crashes shown in the equation are the result of Step 1 and are summarized in Exhibit 4-74.

$$C_{3\,(TOTAL)} = \frac{2.7}{2.5} = 1.1$$

$$C_{3\,(FI)} = \frac{1.1}{1.0} = 1.1$$

This calculation is repeated for each year and each intersection. Exhibit 4-74 summarizes the annual correction factor calculations for the TWSC intersections.

Exhibit 4-74: Annual Correction Factors for all TWSC Intersections

Intersection	Year	Predicted Average Crash Frequency from SPF (TOTAL)	Predicted Average Crash Frequency from SPF (FI)	Correction Factor (TOTAL)	Correction Factor (FI)
2	1	1.7	0.6	1.0	1.0
	2	1.7	0.6	1.0	1.0
	3	1.8	0.7	1.1	1.2
3	1	2.1	0.8	1.0	1.0
	2	2.2	0.8	1.0	1.0
	3	2.2	0.9	1.0	1.1
7	1	2.5	1.0	1.0	1.0
	2	2.5	1.0	1.0	1.0
	3	2.7	1.1	1.1	1.1
10	1	2.1	0.8	1.0	1.0
	2	2.2	0.9	1.0	1.1
	3	2.2	0.9	1.0	1.1
15	1	2.5	1.0	1.0	1.0
	2	2.2	0.9	0.9	0.9
	3	2.1	0.8	0.8	0.8
17	1	2.5	1.0	1.0	1.0
	2	2.6	1.0	1.0	1.0
	3	2.6	1.0	1.0	1.0
19	1	2.4	1.0	1.0	1.0
	2	2.5	1.0	1.0	1.0
	3	2.6	1.0	1.1	1.0

STEP 3 – Calculate Weighted Adjustment

Calculate the weighted adjustment, w, for each intersection and each severity (i.e., TOT and FI). The weighted adjustment accounts for the reliability of the safety performance function that is applied. Crash estimates produced using Safety Performance Functions with overdispersion parameters that are low (which indicates higher reliability) have a larger weighted adjustment. Larger weighting factors place a heavier reliance on the SPF estimate.

$$w_{TOTAL} = \frac{1}{1 + k_{TOT} \times \sum_{n=1}^{N} N_{predicted,\,n(TOTAL)}} \quad \text{and} \quad w_{FI} = \frac{1}{1 + k_{FI} \times \sum_{n=1}^{N} N_{predicted,\,n(FI)}} \tag{4-26}$$

Where,

W = Empirical Bayes weight

k= Overdispersion parameter of the SPF

$N_{predicted,n(TOTAL)}$ = Predicted average total crash frequency from an SPF in year n

$N_{predicted,n(FI)}$ = Predicted average fatal and injury crash frequency from an SPF in year n

Shown below is the weighted adjustment calculation for total and fatal/injury crashes for Intersection 7.

The sum of the predicted crashes shown below (7.7 and 3.1) is the result of summing the annual predicted crashes summarized in Exhibit 4-74 for Intersection 7.

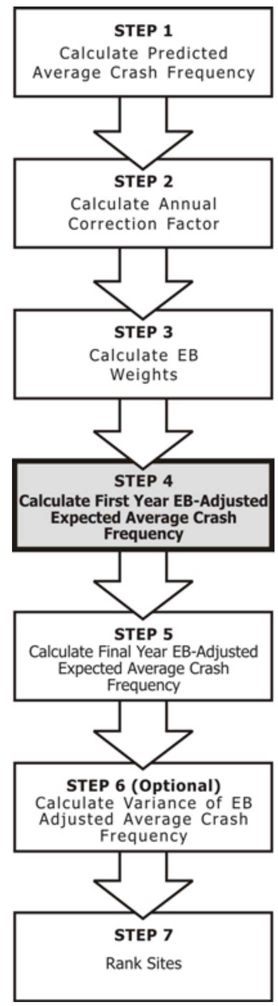
$$w_{TOTAL} = \frac{1}{(1 + (0.49 \times 7.7))} = 0.2$$

$$w_{FI} = \frac{1}{(1 + (0.74 \times 3.1))} = 0.3$$

The calculated weights for the TWSC intersections are summarized in Exhibit 4-75.

Exhibit 4-75: Weighted Adjustments for TWSC Intersections

Intersection	W_{TOTAL}	W_{FI}
2	0.3	0.4
3	0.2	0.4
7	0.2	0.3
10	0.2	0.3
15	0.2	0.3
17	0.2	0.3
19	0.2	0.3



STEP 4 – Calculate First Year EB-adjusted Expected Average Crash Frequency

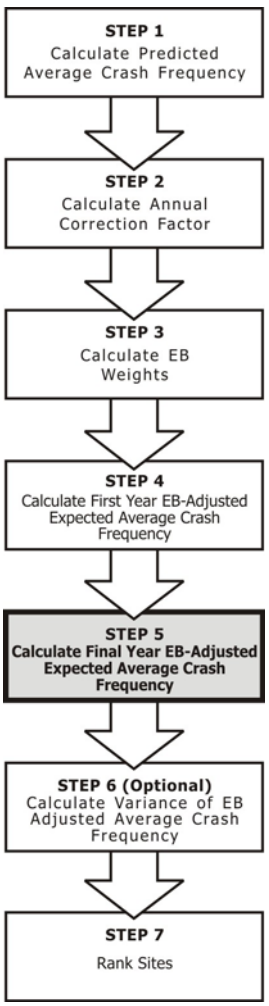
Calculate the base EB-adjusted expected average crash frequency for year 1, $N_{expected,1}$ using Equations 4-26 and 4-27.

This stage of the method integrates the observed crash frequency with the predicted average crash frequency from an SPF. The larger the weighting factor, the greater the reliance on the SPF to estimate the long-term predicted average crash frequency per year at the site. The observed crash frequency on the roadway segments is represented in the equations below as $N_{observed,n}$.

$$N_{expected,1(TOTAL)} = w_{TOTAL} \times N_{predicted,1(TOTAL)} + (1 - w_{TOTAL}) \times \left(\frac{\sum_{n=1}^{N} N_{observed,y(TOTAL)}}{\sum_{n=1}^{N} C_{n(TOTAL)}} \right) \quad (4\text{-}27)$$

and

$$N_{expected,1(FI)} = w_{FI} \times N_{predicted,1(FI)} + (1 - w_{FI}) \times \left(\frac{\sum_{n=1}^{N} N_{observed,y(FI)}}{\sum_{n=1}^{N} C_{n(FI)}} \right) \quad (4\text{-}28)$$



Where,

$N_{expected,1}$ = EB-adjusted estimated average crash frequency for year 1

w = Weight

$N_{predicted,1(TOTAL)}$ = Estimated average crash frequency for year 1 for the intersection

$N_{observed,n}$ = Observed crash frequency at the intersection

C_n = Annual correction factor for the intersection

n = year

Shown below is the total and fatal/injury calculation for Intersection 7.

These calculations are based on information presented in Exhibit 4-74 and Exhibit 4-75.

$$N_{expected,1(TOTAL)} = 0.2 \times (2.5) + (1 - 0.2) \times \frac{34}{3.1} = 9.3$$

$$N_{expected,1(FI)} = 0.3 \times (1.0) + (1 - 0.3) \times \frac{18}{3.1} = 4.4$$

STEP 5 – Calculate Final Year EB-adjusted Expected Average Crash Frequency

Calculate the EB-adjusted expected number of fatal and injury crashes and total crashes for the final year (in this example, the final year is year 3).

$$N_{expected,n(TOTAL)} = N_{expected,1(TOTAL)} \times C_{n(TOTAL)} \quad (4\text{-}29)$$

$$N_{expected,n(FI)} = N_{expected,1(FI)} \times C_{n(FI)} \quad (4\text{-}30)$$

Where,

$N_{expected,n}$ = EB-adjusted expected average crash frequency for final year

$N_{expected,1}$ = EB-adjusted expected average crash frequency for year 1

C_n = Annual correction factor for year, n

Shown below are the calculations for Intersection 7.

$$N_{expected,3(TOTAL)} = 9.3 \times (1.1) = 10.2$$

$$N_{expected,3(FI)} = 4.4 \times (1.1) = 4.8$$

$$N_{expected,3(PDO)} = N_{expected,3(TOTAL)} - N_{expected,3(FI)}$$

Exhibit 4-76 summarizes the calculations for Intersection 7.

Exhibit 4-76: Year 3 – EB-Adjusted Expected Average Crash Frequency[1]

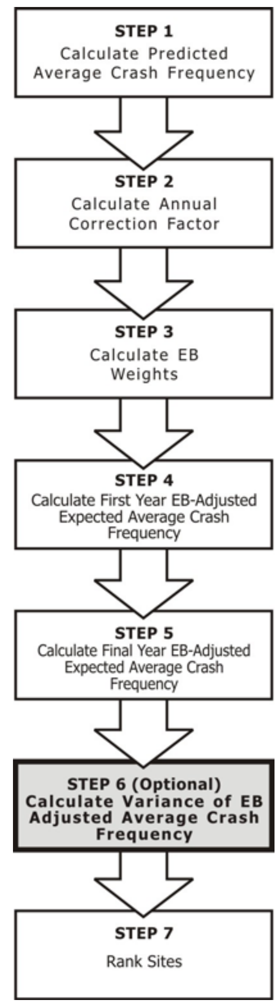
	Fatal and/or Injury Crashes			Total Crashes			PDO Crashes
Intersection	$N_{E,1(FI)}$	$C_{3(FI)}$	$N_{E,3(FI)}$	$N_{E,1(TOTAL)}$	$C_{3(TOTAL)}$	$N_{E,3(TOTAL)}$	$N_{E,3(PDO)}$

STEP 6 – Calculate the Variance of the EB-Adjusted Average Crash Frequency (Optional)

When using the peak searching method (or an equivalent method for intersections), calculate the variance of the EB-adjusted expected number of crashes for year *n*. Equation 4-31 is applicable to roadway segments and ramps, and Equation 4-32 is applicable to intersections.

$$Var(N_{expected,n})_{roadways} = N_{expected,n} \times \left(\frac{(1-w)}{L}\right) \times \frac{C_n}{\sum_{n=1}^{N} C_n} \qquad (4\text{-}31)$$

$$Var(N_{expected,n})_{intersections} = N_{expected,n} \times (1-w) \times \frac{C_n}{\sum_{n=1}^{n} C_n} \qquad (4\text{-}32)$$

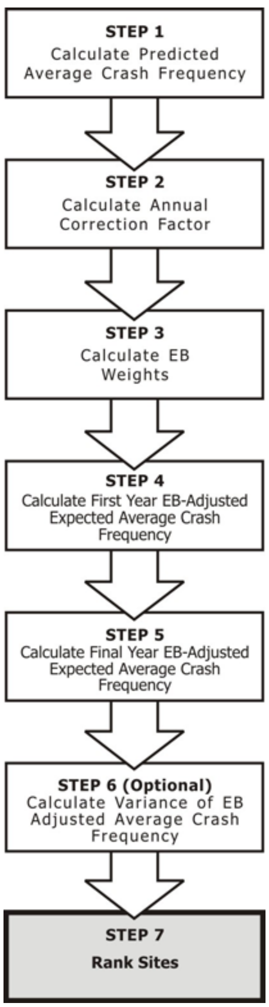


Shown below are the variation calculations for Year 3 at Intersection 7. Exhibit 4-77 summarizes the calculations for Year 3 at Intersection 7.

$$Var(N_{expected,3(TOTAL)})_{intersections} = 10.2 \times (1 - 0.2) \times \frac{1.1}{3.1} = 2.9$$

Exhibit 4-77: Year 3 – Variance of EB-Adjusted Expected Average Crash Frequency

Intersection	Variance
2	2.1
3	1.4
7	2.9
10	1.1
15	1.0
17	1.0
19	1.0



STEP 7 – Rank Sites

Rank the intersections based on the EB-adjusted expected average crash frequency for the final year in the analysis, as calculated in Step 5.

Exhibit 4-78 summarizes the ranking based EB-Adjusted Crash Frequency for the TWSC Intersections.

Exhibit 4-78: EB-Adjusted Expected Average Crash Frequency Ranking

Intersection	EB-Adjusted Average Crash Frequency
7	10.2
2	9.6
3	6.1
10	4.5
15	4.3
17	3.9
19	3.7

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

Equivalent Property Damage Only (EPDO) Method assigns weighting factors to crashes by severity to develop a single combined frequency and severity score per location. The weighting factors are calculated relative to Property Damage Only (PDO) crashes. To screen the network, sites are ranked from the highest to the lowest score. Those sites with the highest scores are evaluated in more detail to identify issues and potential countermeasures.

The frequency of PDO, Injury and Fatal crashes is based on the number of crashes, not the number of injuries per crash.

Data Needs

- Crashes by severity and location
- Severity weighting factors
- Traffic volume on major and minor street approaches
- Basic site characteristics (i.e., roadway cross-section, intersection control, etc.)
- Calibrated safety performance functions (SPFs) and overdispersion parameters

Strengths and Limitations

Exhibit 4-79 provides a summary of the strengths and limitations of the performance measure.

Exhibit 4-79: Strengths and Limitations of the EPDO Average Crash Frequency with EB Adjustment Performance Measure

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

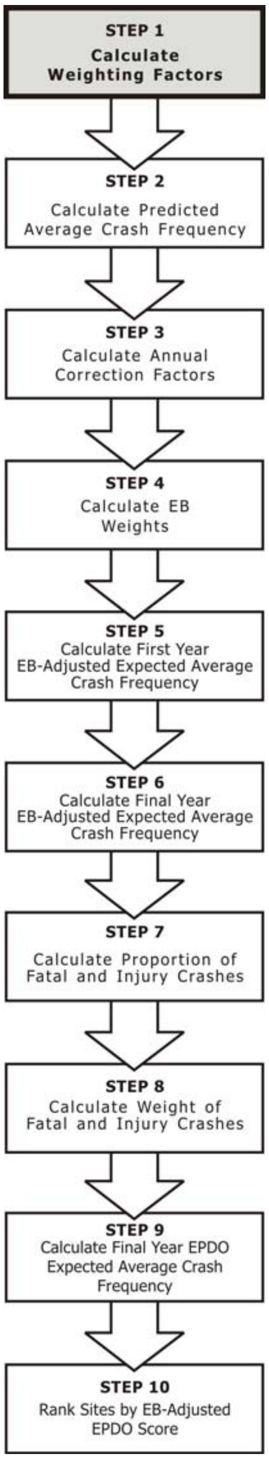
Assumptions

The societal crash costs listed in Exhibit 4-80 are used to calculate the EPDO weights.

Exhibit 4-80: Societal Crash Cost Assumptions

Severity	Cost
Fatality (K)	$4,008,900
Injury Crashes (A/B/C)	$82,600
PDO (O)	$7,400

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA - HRT - 05-051, October 2005.



Sample Problem Assumptions

The Sample problems provided in this section are intended to demonstrate calculation of the performance measures, not predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using predictive method outlined in *Part C* and are provided in Exhibit 4-30 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the base SPF model. It is also assumed that all AMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are for theoretical application and are rarely valid for application of predictive method to actual field conditions.

STEP 1 – Calculate Weighting Factors for Crash Severity

Calculate the EPDO weights for fatal, injury, and PDO crashes. The fatal and injury weights are calculated using Equation 4-33. The cost of a fatal or injury crash is divided by the cost of a PDO crash, respectively. Weighting factors developed from local crash cost data typically result in the most accurate results. If local information is not available, nationwide crash cost data is available from the Federal Highway Administration (FHWA). Appendix A provides information on the national data available and a method for updating crash costs to current dollar values.

The weighting factors are calculated as follows:

$$f_{y(weight)} = \frac{CC_y}{CC_{PDO}} \qquad (4\text{-}33)$$

Where,

$f_{y(weight)}$= EPDO weighting factor based on crash severity, *y*;

CC_y= Crash cost for crash severity, *y*; and,

CC_{PDO}= Crash cost for PDO crash severity.

Incapacitating (A), evident (B), and possible (C) injury crash costs developed by FHWA were combined to develop an average injury (A/B/C) cost. Below is a sample calculation for the injury (A/B/C) EPDO weight (W_I):

$$f_{inj(weight)} = \$82{,}600 / \$7{,}400 = 11$$

Therefore the EPDO weighting factors for all crash severities are shown in Exhibit 4-81.

Exhibit 4-81: Example EPDO Weights

Severity	Cost	Weight
Fatal (K)	$4,008,900	542
Injury (A/B/C)	$82,600	11
PDO (O)	$7,400	1

STEP 2 – Calculate Predicted Average Crash Frequency from an SPF

Using the predictive method in *Part C* calculate the predicted average crash frequency, $N_{predicted,n}$, for each year, n, where n = 1, 2,…, N. Refer to *Part C Introduction and Applications Guidance* for a detailed overview of the method to calculate the predicted average crash frequency. The example provided here is simplified to emphasize calculation of the performance measure, not the predictive method. The predicted average crash frequency from SPFs is summarized for the TWSC intersections for a three-year period in Exhibit 4-82.

Calculations will have to be made for both total and Fatal/Injury crashes, or for Fatal/Injury and Property Damage Only crashes. This example calculates total and Fatal/Injury crashes, from which Property Damage Only crashes are derived.

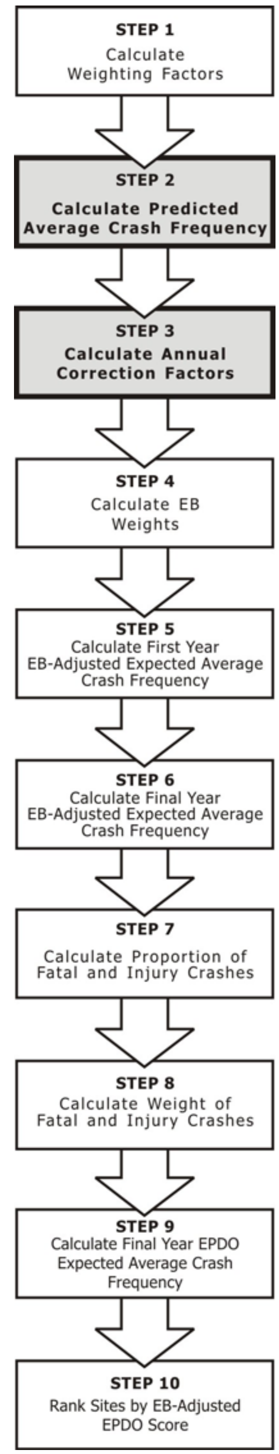
Exhibit 4-82: Estimated Predicted Average Crash Frequency from an SPF

Intersection	Year	AADT		Predicted Average Crash Frequency from an SPF	Average 3-Year Predicted Crash Frequency from an SPF
		Major Street	Minor Street		
2	1	12,000	1,200	1.7	1.7
	2	12,200	1,200	1.7	
	3	12,900	1,300	1.8	
3	1	18,000	800	2.1	2.2
	2	18,900	800	2.2	
	3	19,100	800	2.2	
7	1	21,000	1,000	2.5	2.6
	2	21,400	1,000	2.5	
	3	22,500	1,100	2.7	
10	1	15,000	1,500	2.1	2.2
	2	15,800	1,600	2.2	
	3	15,900	1,600	2.2	
15	1	26,000	500	2.5	2.3
	2	26,500	300	2.2	
	3	27,800	200	2.1	
17	1	14,400	3,200	2.5	2.6
	2	15,100	3,400	2.6	
	3	15,300	3,400	2.6	
19	1	15,400	2,500	2.4	2.5
	2	15,700	2,500	2.5	
	3	16,500	2,600	2.6	

STEP 3 – Calculate Annual Correction Factors

Calculate the annual correction factors (C_n) at each intersection for each year and each severity using Equation 4-34.

The annual correction factor is predicted average crash frequency from an SPF for year *y* divided by the predicted average crash frequency from an SPF for year 1. This factor is intended to capture the effect that annual variations in traffic, weather, and vehicle mix have on crash occurrences.(3)



$$C_{n(TOTAL)} = \frac{N_{predicted,n(TOTAL)}}{N_{predicted,\,n(TOTAL)}} \text{ and } C_{y(FI)} = \frac{N_{predicted,\,n(FI)}}{N_{predicted,1(FI)}} \qquad (4\text{-}34)$$

Where,

$C_{n(TOT)}$ = Annual correction factor for total crashes

$C_{n(F,I)}$ = Annual correction factor for fatal and/or injury crashes

$N_{predicted,n(TOT)}$ = Predicted number of total crashes for year, n

$N_{predicted,1(TOT)}$ = Predicted number of total crashes for year 1

$N_{predicted,n(FI)}$ = Predicted number of fatal and/or injury crashes for year, n

$N_{predicted,1(FI)}$ = Predicted number of fatal and/or injury crashes for year 1

Shown below is the calculation for Intersection 7 based on the yearly correction factor for year 3. The predicted crashes shown in the equation are the result of Step 2.

$$C_{3\,(TOTAL)} = \frac{2.7}{2.5} = 1.1 \qquad C_{3\,(FI)} = \frac{1.1}{1.0} = 1.1$$

The annual correction factors for all TWSC intersections are summarized in Exhibit 4-83.

Exhibit 4-83: Annual Correction Factors for all TWSC Intersections

Intersection	Year	Predicted Average Crash Frequency from an SPF (TOTAL)	Predicted Average Crash Frequency from an SPF (FI)	Correction Factor (TOTAL)	Correction Factor (FI)
2	1	1.7	0.6	1.0	1.0
	2	1.7	0.6	1.0	1.0
	3	1.8	0.7	1.1	1.2
3	1	2.1	0.8	1.0	1.0
	2	2.2	0.8	1.0	1.0
	3	2.2	0.9	1.0	1.1
7	1	2.5	1.0	1.0	1.0
	2	2.5	1.0	1.0	1.0
	3	2.7	1.1	1.1	1.1
10	1	2.1	0.8	1.0	1.0
	2	2.2	0.9	1.0	1.1
	3	2.2	0.9	1.0	1.1
15	1	2.5	1.0	1.0	1.0
	2	2.2	0.9	0.9	0.9
	3	2.1	0.8	0.8	0.8
17	1	2.5	1.0	1.0	1.0
	2	2.6	1.0	1.0	1.0
	3	2.6	1.0	1.0	1.0
19	1	2.4	1.0	1.0	1.0
	2	2.5	1.0	1.0	1.0
	3	2.6	1.0	1.1	1.0

STEP 4 – Calculate Weighted Adjustment

Calculate the weighted adjustment, w, for each intersection and each severity. The weighted adjustment accounts for the reliability of the safety performance function that is applied. Crash estimates produced using safety performance functions with overdispersion parameters that are low (which indicates higher reliability) have a larger weighted adjustment. Larger weighting factors place a heavier reliance on the SPF to predict the long-term predicted average crash frequency per year at a site. The weighted adjustments are calculated using Equation 4-35.

$$w_{TOT} = \frac{1}{1 + k_{TOTAL} \times \sum_{n=1}^{N} N_{predicted,n(TOTAL)}} \quad \text{and} \quad w_{FI} = \frac{1}{1 + k_{FI} \sum_{n=1}^{N} N_{predicted,n(FI)}} \tag{4-35}$$

Where,

W = Empirical Bayes weight

n = years

k = Overdispersion parameter of the SPF

$N_{predicted,n}$ = Predicted average crash frequency from an SPF in year n

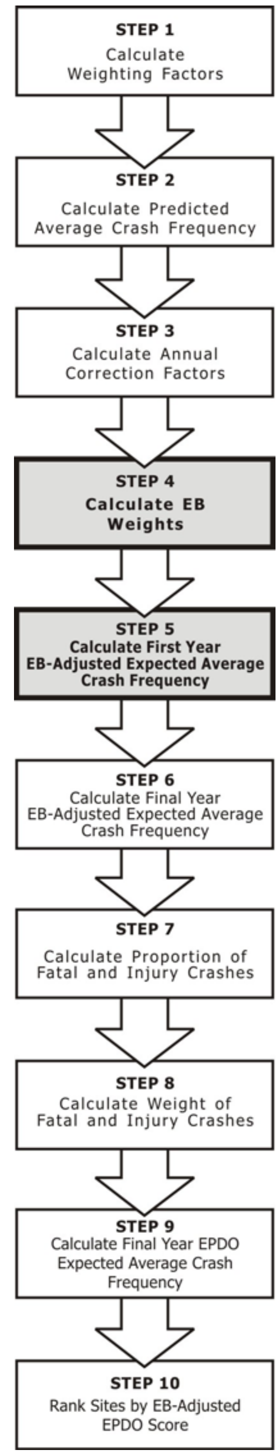
> Shown below is the weighted adjustment calculation for fatal/injury and total crashes for Intersection 7.
>
> The overdispersion parameters shown below are found in *Part C* along with the SPFs. The sum of the predicted crashes shown below (7.7 and 3.1) is the result of summing the annual predicted crashes summarized in Exhibit 4-83 for Intersection 7.
>
> $$w_{TOTAL} = \frac{1}{(1 + (0.49 \times 7.7))} = 0.2$$
>
> $$w_{FI} = \frac{1}{(1 + (0.74 \times 3.1))} = 0.3$$
>
> The TOT and FI weights are summarized for the TWSC intersections in Exhibit 4-84.

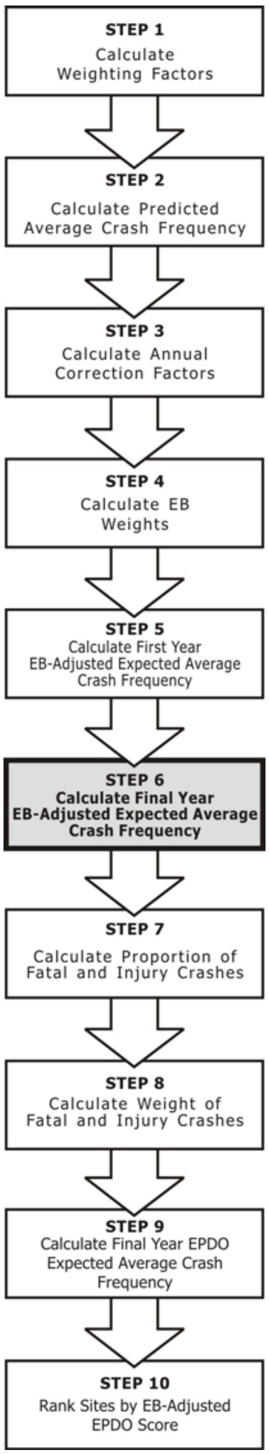
STEP 5 – Calculate First Year EB-adjusted Expected Average Crash Frequency

Calculate the base EB-adjusted expected average crash frequency for year 1, $N_{E,1}$.

This stage of the method integrates the observed crash frequency with the predicted average crash frequency from an SPF. The larger the weighting factor, the greater the reliance on the SPF to estimate the long-term expected average crash frequency per year at the site. The observed crash frequency, $N_{observed,y}$, on the roadway segments is represented in Equations 4-36 and 4-37 below.

$$N_{expected,1(TOTAL)} = w_{TOTAL} \times N_{predicted,1(TOTAL)} + (1 - w_{TOTAL}) \times \left(\frac{\sum_{n=1}^{N} N_{observed,n(TOTAL)}}{\sum_{n=1}^{N} C_{n(TOTAL)}} \right) \tag{4-36}$$





and

$$N_{expected,1(FI)} = w_{FI} \times N_{predicted,1(FI)} + (1 - w_{FI}) \times \left(\frac{\sum_{n=1}^{N} N_{observed,n(FI)}}{\sum_{n=1}^{N} C_{n(FI)}} \right) \quad (4\text{-}37)$$

Where,

$N_{expected,1}$ = EB-adjusted expected average crash frequency for year 1

w = Weight

$N_{predicted,1}$ = Predicted average crash frequency for year 1

$N_{observed,n}$ = Observed average crash frequency at the intersection

C_n = Annual correction factor for the intersection

n = years

Shown below is the total crash calculation for Intersection 7.

$$N_{expected,1(TOT)} = 0.2 \times (2.5) + (1 - 0.2) \times \frac{34}{3.1} = 9.3$$

Exhibit 4-84 summarizes the calculations for total crashes at Intersection 7.

Exhibit 4-84: Year 1 – EB-Adjusted Number of Total Crashes

Intersection	$N_{predicted,1(TOTAL)}$	w_{TOTAL}	$N_{observed,n(TOTAL)}$ (All Years)	Sum of TOT Correction Factors ($C_1 + C_2 + C_3$)	$N_{expected,1(TOTAL)}$
7	2.5	0.2	34	3.1	9.3

The EB-adjusted expected average crash frequency calculations for all TWSC intersections are summarized in 4-87.

STEP 6 – Calculate Final Year EB-adjusted Average Crash Frequency

Calculate the EB-adjusted expected number of fatal and injury crashes and total crashes for the final year. Total and fatal and injury EB-adjusted expected average crash frequency for the final year is calculated using Equations 4-38 and 4-39, respectively.

$$N_{expected,\ n(TOTAL)} = N_{expected,1\ (TOTAL)} \times C_{n(TOTAL)} \quad (4\text{-}38)$$

$$N_{expected,n(FI)} = N_{expected,1(FI)} \times C_{n(FI)} \quad (4\text{-}39)$$

Where,

$N_{expected,n}$ = EB-adjusted expected average crash frequency for final year, n (the final year of analysis in this sample problem is n=3).

$N_{expected,1}$ = EB-adjusted expected average crash frequency for first year, n = 1

C_n = Annual correction factor for year, n

Shown below are the calculations for Intersection 7. The annual correction factors shown below are summarized in Exhibit 4-83 and the EB-adjusted crashes for Year 1 are values from Step 4.

$$N_{expected,3(TOTAL)} = 9.3 \times (1.1) = 10.2$$

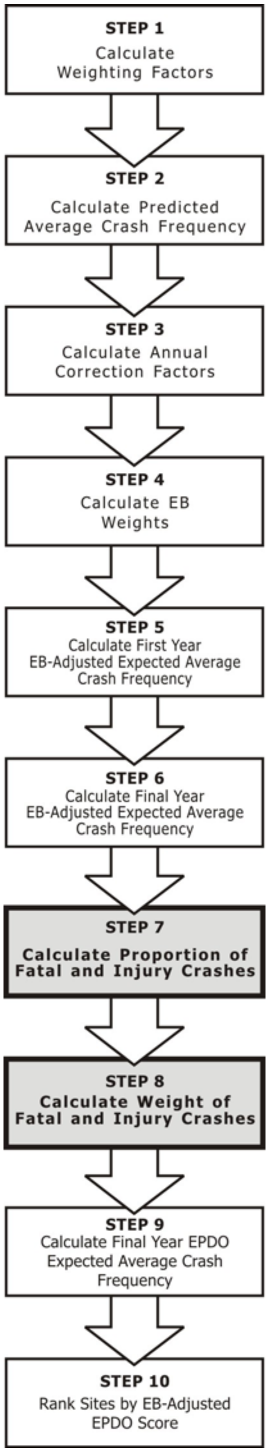
$$N_{expected,3(FI)} = 4.4 \times (1.1) = 4.8$$

$$N_{expected,3(PDO)} = 10.2 - 4.8 = 5.4$$

The calculation of $N_{expected,3(PDO)}$ is based on the difference between the Total and FI expected average crash frequency. Exhibit 4-85 summarizes the results of Steps 4 through 6, including the EB-adjusted expected average crash frequency for all TWSC intersections.

Exhibit 4-85: EB-Adjusted Expected Average Crash Frequency for TWSC Intersections

Intersection	Year	Observed Number of Crashes (TOT)	Predicted Average Crash Frequency from an SPF (TOTAL)	Weight (Total)	Weight (FI)	EB-Adjusted Expected Average Crash Frequency (TOT)	EB-Adjusted Expected Average Crash Frequency (FI)	EB-Adjusted Expected Average Crash Frequency (PDO)
2	1	9.0	1.7	0.3	0.4	8.7	4.9	3.8
	2	11.0	1.7			8.7	4.9	3.8
	3	15.0	1.8			9.6	5.8	3.8
3	1	9.0	2.1	0.2	0.4	6.1	3.0	3.1
	2	8.0	2.2			6.1	3.0	3.1
	3	6.0	2.2			6.1	3.3	2.8
7	1	11.0	2.5	0.2	0.3	9.3	4.3	5.0
	2	9.0	2.5			9.3	4.3	5.0
	3	14.0	2.7			10.2	4.8	5.4
10	1	7.0	2.1	0.2	0.3	4.5	1.7	2.8
	2	6.0	2.2			4.7	1.9	2.8
	3	4.0	2.2			4.5	1.9	2.6
15	1	6.0	2.5	0.2	0.3	5.4	1.6	3.8
	2	3.0	2.2			4.8	1.4	3.4
	3	8.0	2.1			4.3	1.3	3.0
17	1	4.0	2.5	0.2	0.3	3.9	1.7	2.2
	2	4.0	2.6			4.1	1.7	2.4
	3	5.0	2.6			3.9	1.7	2.2
19	1	5.0	2.4	0.2	0.3	3.4	1.7	1.7
	2	2.0	2.5			3.5	1.7	1.8
	3	4.0	2.6			3.7	1.7	2.0



STEP 7 - Calculate the Proportion of Fatal and Injury Crashes

Equations 4-40 and 4-41 are used to identify the proportion of fatal crashes with respect to all non-PDO crashes in the reference population and injury crashes with respect to all non-PDO crashes in the reference population.

$$P_F = \frac{\sum N_{observed,(F)}}{\sum N_{observed,(FI)}} \tag{4-40}$$

$$P_I = \frac{\sum N_{observed,(I)}}{\sum N_{observed,(FI)}} \tag{4-41}$$

Where,

$N_{observed,(F)}$ = Observed number of fatal crashes from the reference population;

$N_{observed,(I)}$ = Observed number of injury crashes from the reference population;

$N_{observed,(FI)}$ = Observed number of fatal-and-injury crashes from the reference population;

P_F = Proportion of observed number of fatal crashes out of FI crashes from the reference population;

P_I = Proportion of observed number of injury crashes out of FI crashes from the reference population.

> Shown below are the calculations for the TWSC intersection reference population.
>
> $$P_F = \frac{6}{80} = 7.5\%$$
>
> $$P_I = \frac{74}{80} = 92.5\%$$

STEP 8 – Calculate the Weight of Fatal and Injury Crashes

Compared to PDO crashes the relative EPDO weight of fatal and injury crashes is calculated using Equation 4-42.

$$w_{EPDO,FI} = P_F \times f_{K(weight)} + P_I \times f_{inj(weight)} \tag{4-42}$$

Where,

$f_{inj(weight)}$ = EPDO injury weighting factor;

$f_{K(weight)}$ = EPDO fatality weighting factor;

P_F = Proportion of observed number of fatal crashes out of FI crashes from the reference population;

> Shown below is the calculation for Intersection 7. The EPDO weights, $f_{K(weight)}$ and W_I are summarized in Exhibit 4-81.
>
> $$w_{EPDO,FI} = (0.075 \times 542) + (0.925 \times 11) = 50.8$$

STEP 9 – Calculate the Final Year EPDO Expected Average Crash Frequency

Equation 4-43 can be used to calculate the EPDO expected average crash frequency for the final year for which data exist for the site.

$$N_{expected,n(EPDO)} = N_{expected,n(PDO)} + w_{EPDO,FI} \times N_{expected,n(FI)} \quad (4\text{-}43)$$

> Shown below is the calculation for Intersection 7.
>
> $$N_{expected,3(EPDO)} = 5.4 + 50.8 \times 4.8 = 249.2$$

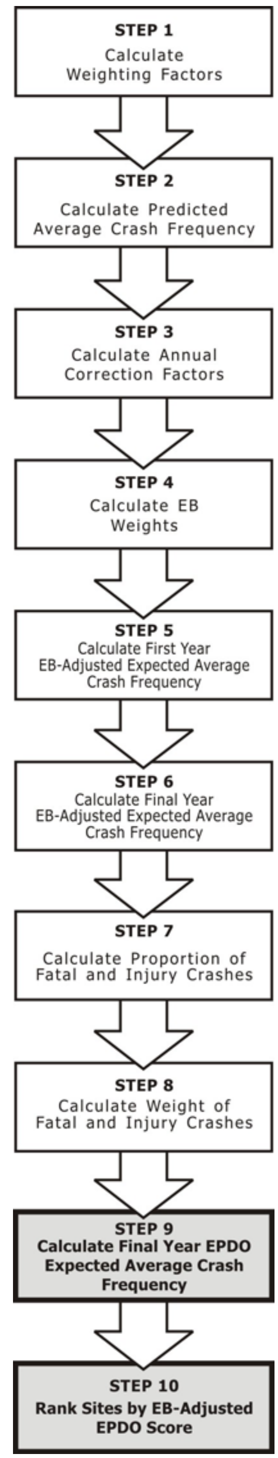
STEP 10 – Rank Sites by EB-adjusted EPDO Score

Order the database from highest to lowest by EB-adjusted EPDO score. The highest EPDO score represents the greatest opportunity to reduce the number of crashes.

> Exhibit 4-86 summarizes the EB-Adjusted EPDO Ranking for the TWSC Intersections.

Exhibit 4-86: EB-Adjusted EPDO Ranking

Intersection	EB-Adjusted EPDO
2	298.4
7	249.2
3	170.4
10	99.1
17	88.6
19	88.4
15	69.0



4.4.2.13. Excess Expected Average Crash Frequency with EB Adjustments

The empirical Bayes Method is applied to estimate expected crash frequency. The *Part C Introduction and Applications Guidance* explains how to apply the EB Method. Intersections are ranked based on the difference between the predicted estimates and EB-adjusted estimates for each intersection, the excess expected average crash frequency per year.

Data Needs

- Crash data by severity and location
- Traffic volume
- Basic site characteristics (i.e., roadway cross-section, intersection control)
- Calibrated Safety Performance Functions (SPFs) and overdispersion parameters

Strengths and Limitations

Exhibit 4-87 provides a summary of the strengths and limitations of the Excess Expected Average Crash Frequency with EB Adjustments performance measure.

Exhibit 4-87: Strengths and Limitations of the Excess Expected Average Crash Frequency with EB Adjustment Performance Measure

Strengths	Limitations
• Accounts for RTM bias	• None
• Identifies a threshold to indicate sites experiencing more crashes than expected for sites with similar characteristics.	

Procedure

The following sample problem outlines the assumptions and procedure for ranking seven TWSC intersections based on the expected crash frequency with empirical Bayes adjustments. The calculations for Intersection 7 are used throughout the sample problems to highlight how to apply each method.

Exhibit 4-88: Societal Crash Cost Assumptions

Crash Severity	Crash Cost
Combined Cost for Crashes with a Fatality and/or Injury (K/A/B/C)	$158,200
PDO (O)	$7,400

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity Within Selected Crash Geometries, FHWA - HRT - 05-051, October 2005.

As shown in Exhibit 4-88, the crash cost that can be used to weigh the expected number of FI crashes is $158,200. The crash cost that can be used to weigh the expected number of PDO crashes is $7,400. More information on crash costs, including updating crash cost values to current year of study values is provided in Appendix A.

Sample Problem Assumptions

The sample problems provided in this section are intended to demonstrate calculation of the performance measures, not predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using predictive method outlined in *Part C* and are provided in Exhibit 4-30 for use in sample problems.

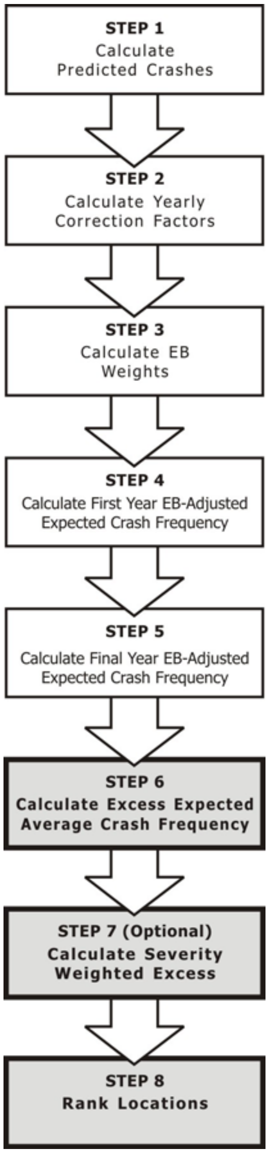
The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the SPF. It is also assumed that all AMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are for theoretical application and are rarely valid for application of the Part C predictive method to actual field conditions.

Calculation of this performance measure follows Steps 1-5 outlined for the Expected Average Crash Frequency with EB Adjustments performance measure.

The results of Steps 1-5 that are used in calculations of the excess expected average crash frequency are summarized in Exhibit 4-89.

Exhibit 4-89: Summary of Performance Measure Calculations for Steps 1, 4, and 5

Intersection	Year	Observed Average Crash Frequency (FI)	Observed Average Crash Frequency (PDO)	SPF Predicted Average Crash Frequency (FI)	SPF Predicted Average Crash Frequency (PDO)	EB-Adjusted Expected Average Crash Frequency (FI)	EB-Adjusted Expected Average Crash Frequency (PDO)
2	1	8	1	0.6	1.1	4.9	3.8
	2	8	3	0.6	1.1	4.9	3.8
	3	9	6	0.7	1.1	5.8	3.8
3	1	8	1	0.8	1.3	3.0	3.1
	2	3	5	0.8	1.4	3.0	3.1
	3	2	4	0.9	1.4	3.3	2.8
7	1	5	6	1.0	1.6	4.3	5.0
	2	5	4	1.0	1.6	4.3	5.0
	3	8	6	1.1	1.7	4.8	5.4
10	1	4	3	0.8	1.3	1.7	2.8
	2	2	4	0.9	1.4	1.9	2.8
	3	1	3	0.9	1.4	1.9	2.6
15	1	1	5	1.0	1.6	1.6	3.8
	2	1	2	0.9	1.4	1.4	3.4
	3	3	5	0.8	1.3	1.3	3.0
17	1	2	2	1.0	1.5	1.7	2.2
	2	2	2	1.0	1.6	1.7	2.4
	3	2	3	1.0	1.6	1.7	2.2
19	1	3	2	1.0	1.5	1.7	1.7
	2	1	1	1.0	1.5	1.7	1.8
	3	2	2	1.0	1.6	1.7	2.0



STEP 6 – Calculate the Excess Expected Average Crash Frequency

The difference between the predicted estimates and EB-adjusted estimates for each intersection is the excess as calculated by Equation 4-44.

$$Excess_y = \left(N_{expected,n(PDO)} - N_{predicted,n(PDO)}\right) + \left(N_{expected,\,n(F,I)} - N_{predicted,n(F,I)}\right) \qquad (4\text{-}44)$$

Where,

$Excess_y$ = Excess expected crashes for year, *n*

$N_{expected,n}$ = EB-adjusted expected average crash frequency for year, *n*

$N_{predicted,n}$ = SPF predicted average crash frequency for year, *n*

> Shown below is the calculation for Intersection 7.
>
> $Excess_3 = 5.4 – 1.7 + 4.8 – 1.1 = 7.4$ [crashes/year]
>
> Exhibit 4-90 summarizes the calculations for all TWSC intersections.

STEP 7 – Calculate Severity Weighted Excess (Optional)

Calculate the severity weighted EB-adjusted excess expected crash value in dollars.

$$Excess_{(SW)} = \left(N_{expected,n(PDO)} - N_{predicted,n(PDO)}\right) \times CC_{(PDO)} + \left(N_{expected,n(FI)} - N_{predicted,n(FI)}\right) \times CC_{(FI)} \qquad (4\text{-}45)$$

Where,

$Excess_{(SW)}$ = Severity weighted EB-adjusted expected excess crash value

$CC(Y)$= Crash cost for crash severity, Y

> Shown below is the calculation for Intersection 7.
>
> $$Excess_{(SW)} = (5.4 - 1.7) \times \$7{,}400 + (4.8 - 1.1) \times \$158{,}200 = \$612{,}720$$
>
> Exhibit 4-91 summarizes the calculations for all TWSC intersections.

STEP 8 – Rank Locations

Rank the intersections based on either EB-adjusted expected excess crashes calculated in Step 6 or based on EB-adjusted severity weighted excess crashes calculated in Step 7. Exhibit 4-90 shows the ranking of TWSC intersections based on the EB-adjusted expected excess crashes calculated in Step 6. The intersection ranking shown in Exhibit 4-91 is based on the EB-adjusted severity weighted excess crashes calculated in Step 7.

Exhibit 4-90: EB-Adjusted Excess Expected Crash Ranking

Intersection	Excess
2	7.8
7	7.4
3	3.8
10	2.2
15	2.2
17	1.3
19	1.1

Exhibit 4-91: EB-Adjusted Severity Weighted Excess Crash Ranking

Intersection	$Excess_{(SW)}$[1]
2	$826,800
7	$612,700
3	$390,000
10	$167,100
17	$115,200
19	$113,700
15	$91,700

Note: [1]All $Excess_{(SW)}$ values rounded to the nearest hundred dollars.

4.4.3. Roadway Segments Performance Measure Sample Data

The Situation

A roadway agency is undertaking an effort to improve safety on their highway network. There are ten roadway segments from which the roadway agency wants to identify sites that will be studied in more detail because they show a potential for reducing the average crash frequency.

After reviewing the guidance in Section 4.2, the agency chooses to apply the sliding window method using the RSI performance measure to analyze each roadway segment. If desired, the agency could apply other performance measures or the peak searching method to compare results and confirm ranking.

The Facts

- The roadway segments are comprised of:
    - 1.2 miles of rural undivided two-lane roadway
    - 2.1 miles are undivided urban/suburban arterial with four lanes
    - 0.6 miles of divided urban/suburban two-lane roadway
- Segment characteristics and a three-year summary of crash data is in Exhibit 4-93.

- Three years of detailed roadway segment crash data is shown in Exhibit 4-94.

Assumptions

- The roadway agency has accepted the FHWA crash costs by severity and type as shown in Exhibit 4-92.

Exhibit 4-92: Relative Severity Index Crash Costs

Crash Type	RSI Crash Costs
Rear End - Non-Intersection	$30,100
Sideswipe/Overtaking	$34,000
Angle - Non-Intersection	$56,100
Pedestrian/Bike Non-Intersection	$287,900
Head-On - Non-Intersection	$375,100
Roll-Over	$239,700
Fixed Object	$94,700
Other/Undefined	$55,100

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA - HRT - 05-051, October 2005.

Roadway Segment Characteristics and Crash Data

Exhibit 4-93 and Exhibit 4-94 summarize the roadway segment characteristics and crash data.

Exhibit 4-93: Roadway Segment Characteristics

Segments	Cross-Section (Number of Lanes)	Segment Length (miles)	AADT	Undivided/ Divided	Crash Data		
					Total Year 1	Total Year 2	Total Year 3
1	2	0.80	9,000	U	16	15	14
2	2	0.40	15,000	U	12	14	10
3	4	0.50	20,000	D	6	9	5
4	4	0.50	19,200	D	7	5	1
5	4	0.35	22,000	D	18	16	15
6	4	0.30	25,000	D	14	12	10
7	4	0.45	26,000	D	12	11	13
8	2	0.20	10,000	U	2	1	3
9	2	0.25	14,000	U	3	2	1
10	2	0.15	15,000	U	1	2	1

Exhibit 4-94: Roadway Segment Detail Crash Data Summary (3 Years)

		Crash Severity			Crash Type							
Segment	Total	Fatal	Injury	PDO	Rear-End	Angle	Head-On	Sideswipe	Pedestrian	Fixed Object	Roll - Over	Other
1	45	3	17	25	0	0	6	5	0	15	19	0
2	36	0	5	31	0	1	3	3	3	14	10	2
3	20	0	9	11	1	0	5	5	0	5	3	1
4	13	0	5	8	3	0	1	2	0	4	0	3
5	49	0	9	40	1	1	21	12	2	5	5	2
6	36	0	5	31	4	0	11	10	0	5	4	2
7	36	0	6	30	2	0	13	11	0	4	3	3
8	6	0	1	5	2	0	0	1	0	1	0	2
9	6	0	1	5	1	0	0	1	0	2	0	2
10	4	0	0	4	2	0	0	0	0	1	0	1

Sliding Window Procedure

The sliding window approach is one analysis method that can be applied when screening roadway segments. It consists of conceptually sliding a window of a specified length along the road segment in increments of a specified size. The method chosen to screen the segment is applied to each position of the window and the results of the analysis are recorded for each window. The window that shows the greatest potential for improvement is used to represent the total performance of the segment. After all segments are ranked according to the respective highest window value, those segments with the greatest potential for reduction in crash frequency or severity are studied in detail to identify potential countermeasures.

The following assumptions are used to apply the sliding window analysis technique in the roadway segment sample problems:

- Segment 1 extends from mile point 1.2 to 2.0
- The length of window in the sliding window analysis is 0.3 miles
- The window slides in increments of 0.1 miles

The name of the window subsegments and the limits of each subsegment are summarized in Exhibit 4-95.

Exhibit 4-95: Segment 1 Sliding Window Parameters

Window Subsegments	Beginning Limit (Mile Point)	Ending Limit (Mile Point)
1a	1.2	1.5
1b	1.3	1.6
1c	1.4	1.7
1d	1.5	1.8
1e	1.6	1.9
1f	1.7	2.0

The windows shown above in Exhibit 4-95 are the windows used to evaluate Segment 1 throughout the roadway segment sample problems. Therefore, whenever window subsegment 1a is referenced it is the portion of Segment 1 that extends from mile point 1.2 to 1.5 and so forth.

Exhibit 4-96 summarizes the crash data for each window subsegment within Segment 1. This data will be used throughout the roadway segment sample problems to illustrate how to apply each screening method.

Exhibit 4-96: Segment 1 Crash Data per Sliding Window Subsegments

Window Subsegments	Total	Crash Severity			Crash Type			
		Fatal	Injury	PDO	Head-On	Sideswipe	Fixed Object	Roll - Over
1a	8	0	3	5	0	0	3	5
1b	8	0	4	4	1	1	3	3
1c	7	0	3	4	3	1	0	3
1d	11	2	3	6	1	2	5	3
1e	4	0	0	4	0	0	1	3
1f	7	1	4	2	1	1	3	2

When the sliding window approach is applied to a method, each segment is ranked based on the highest value found on that segment.

STEP 1 – Calculate RSI Crash Costs per Crash Type

For each window subsegment, multiply the average crash frequency for each crash type by their respective RSI crash type.

Exhibit 4-97 summarizes the observed average crash frequency by crash type for each window subsegment over the last three years and the corresponding RSI crash costs for each crash type.

Exhibit 4-97: Crash Type Summary for Segment 1 Window Subsegments

Window Subsegments	Head-On	Side-swipe	Fixed Object	Roll – Over	Total
Observed Average Crash Frequency					
1a	0	0	3	5	8
1b	1	1	3	3	8
1c	3	1	0	3	7
1d	1	2	5	3	11
1e	0	0	1	3	4
1f	1	1	3	2	7
RSI Crash Costs per Crash Type					
1a	$0	$0	$284,100	$1,198,500	$1,482,600
1b	$375,100	$34,000	$284,100	$719,100	$1,412,300
1c	$1,125,300	$34,000	$0	$719,100	$1,878,400
1d	$375,100	$68,000	$473,500	$719,100	$1,635,700
1e	$0	$0	$94,700	$719,100	$813,800
1f	$375,100	$34,000	$284,100	$479,400	$1,172,600

Table Notes:
1. Crash types that were not reported to have occurred on Roadway Segment 1 were omitted from the table. The RSI costs for these crash types are zero.
2. The values in this table are the result of multiplying the average crash frequency for each crash type by the corresponding RSI cost.

The calculation for Window Subsegment 1d is shown below.

$$\text{Total RSI Cost} = (1\times\$375{,}100)+(2\times\$34{,}000)+(5\times\$94{,}700)+(3\times\$239{,}700)=\$1{,}635{,}700$$

STEP 2 – Calculate Average RSI Cost per Subsegment

Sum the RSI costs for all crash types and divide by the total average crash frequency for the specific window subsegment as shown in Equation 4-46. The result is an Average RSI cost for each window subsegment.

$$\text{Average RSI Cost per Subsegment} = \text{Total RSI Cost} / N_{observed,i(TOTAL)} \tag{4-46}$$

Where,

$N_{observed,i(TOTAL)}$ = Total observed crashes at site, i

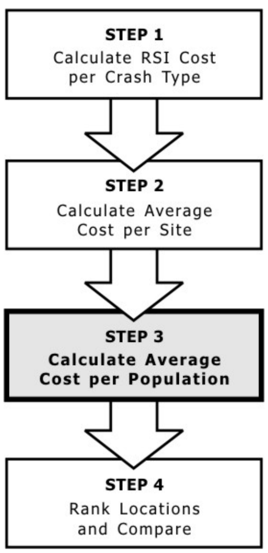
The calculation for Window Subsegment 1d is shown below.

$$\text{Average RSI Cost} = \$1{,}635{,}700 / 11 = \$148{,}700$$

Exhibit 4-98 summarizes the Average RSI Crash Cost calculation for each window subsegment within Segment 1.

Exhibit 4-98: Average RSI Crash Cost per Window Subsegment

Window Subsegment	Total Number of Crashes	Total RSI Value	Average RSI Value
1a	8	$1,482,600	$185,300
1b	8	$1,412,300	$176,500
1c	7	$1,878,400	$268,300
1d	11	$1,635,700	$148,700
1e	4	$813,800	$203,500
1f	7	$1,172,600	$167,500



STEP 3 – Calculate Average RSI Cost for the Population

Calculate the average RSI cost for the entire population by summing the total RSI costs for each site and dividing by the total average crash frequency within the population. In this sample problem, the population consists of Segment 1 and Segment 2. Preferably, there are more than two Segments within a population; however, for the purpose of illustrating the concept and maintaining brevity this set of example problems only has two segments within the population.

The average RSI cost for the population ($\overline{\mathrm{RSI}_{\mathrm{P}}}$) is calculated using Equation 4-47.

$$\overline{RSI_P} = \frac{\sum_{i=1}^{n} RSI_i}{\sum_{i=1}^{n} N_{observed,i}} \qquad (4\text{-}47)$$

Where,

$\overline{\mathrm{RSI}_{\mathrm{P}}}$ = Average RSI cost for the population

RSI_i = RSI cost per site in the population

$N_{observed,i}$ = Number of observed crashes in the population

Exhibit 4-99 summarizes the information needed to calculate the average RSI cost for the population.

Exhibit 4-99: Average RSI Cost for Two-Lane Undivided Rural Highway Population

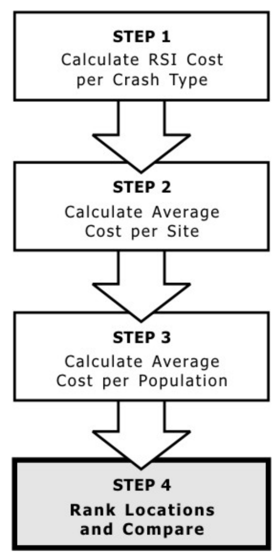
Roadway Segments	Angle	Head-On	Side-swipe	Pedestrian	Fixed Object	Roll-Over	Other	Total
Average Crash Frequency Over Three Years								
1	0	6	5	0	15	19	0	45
2	1	3	3	3	14	10	2	36
RSI Crash Costs per Crash Type								
1	$0	$2,250,600	$170,000	$0	$1,420,500	$4,554,300	$0	$8,395,400
2	$56,100	$1,125,300	$102,000	$863,700	$1,325,800	$2,397,000	$110,000	$5,979,900

Below is the average RSI cost calculation for the Rural Two-Lane Highway population. This can be used as a threshold for comparison of RSI cost of individual sub-segments within a segment.

$$\overline{RSI_P} = \frac{\sum_{i=1}^{n} RSI_i}{\sum_{i=1}^{n} N_{observed,i}} = \frac{\$8{,}395{,}400 + \$5{,}979{,}900}{45 + 36} = \$177{,}500$$

STEP 4 – Rank Locations and Compare

Steps 1 and 2 are repeated for each roadway segment and Step 3 is repeated for each population. The roadway segments are ranked using the highest average RSI cost calculated for each roadway segment. For example, Segment 1 would be ranked using the highest average RSI cost shown in Exhibit 4-98 from Window Subsegment 1c ($268,300). The highest average RSI cost for each roadway segment is also compared to the average RSI cost for the entire population. This comparison indicates whether or not the roadway segment's average RSI cost is above or below the average value for similar locations.



4.5. REFERENCES

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APPENDIX A – CRASH COST ESTIMATES

State and local jurisdictions often have accepted crash costs by crash severity and crash type. When available, these locally-developed crash cost data can be used with procedures in the HSM. If local information is not available, nationwide crash cost data is available from the Federal Highway Administration (FHWA) and the USDOT. This edition of the HSM develops crash costs from the FHWA report "Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries."[3] The costs cited in this 2005 report are presented in 2001 dollars. Exhibits B-1 and B-2 summarize the relevant information for use in the HSM (rounded to the nearest hundred dollars). [3]

The FHWA report presents human capital crash costs and comprehensive crash costs by crash type and severity. Human capital crash cost estimates include the monetary losses associated with medical care, emergency services, property damage, and lost productivity. Comprehensive crash costs include the human capital costs in addition to nonmonetary costs related to the reduction in the quality of life in order to capture a more accurate level of the burden of injury. Comprehensive costs are also generally used in analyses conducted by other federal and state agencies outside of transportation.

Exhibit A-1: Crash Cost Estimates by Crash Severity

Crash Type	Human Capital Crash Costs	Comprehensive Crash Costs
Fatality (K)	$1,245,600	$4,008,900
Disabling Injury (A)	$111,400	$216,000
Evident Injury (B)	$41,900	$79,000
Possible Injury (C)	$28,400	$44,900
PDO (O)	$6,400	$7,400

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA - HRT - 05-051, October 2005.

Exhibit A-2: Crash Cost Estimates by Crash Type

Crash Type	Human Capital Crash Costs	Comprehensive Crash Costs
Rear End – Signalized Intersection	$16,700	$26,700
Rear End – Unsignalized Intersection	$10,900	$13,200
Sideswipe/Overtaking	$17,600	$34,000
Angle – Signalized Intersection	$24,300	$47,300
Angle – Unsignalized Intersection	$29,700	$61,100
Pedestrian/Bike at an Intersection	$72,800	$158,900
Pedestrian/Bike Non-Intersection	$107,800	$287,900
Head-On – Signalized Intersection	$15,600	$24,100
Head-On – Unsignalized Intersection	$24,100	$47,500
Fixed Object	$39,600	$94,700
Other/Undefined	$24,400	$55,100

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA - HRT - 05-051, October 2005.

Crash cost data presented in Exhibits B-1 and B-2 is applied in the HSM to calculate performance measures used in network screening (Chapter 4) and to convert safety benefits to a monetary value (*Chapter 7*). These values can be updated to current year values using the method presented in the following section.

Annual Adjustments

National crash cost studies are not typically updated annually; however, current crash cost dollar values are needed to effectively apply the methods in the HSM. A two-step process based on data from the US Bureau of Labor Statistics (USBLS) can be used to adjust annual crash costs to current dollar values. As noted in the FHWA report, this procedure is expected to provide adequate cost estimates until the next national update of unit crash cost data and methods.[3]

In general, the annual adjustment of crash costs utilizes federal economic indexes to account for the economic changes between the documented past year and the year of interest. Adjustment of the 2001 crash costs (Exhibits B-1 and B-2) to current year values involves multiplying the known crash cost dollar value for a past year by an adjustment ratio. The adjustment ratio is developed from a Consumer Price Index (CPI), published monthly, and an Employment Cost Index (ECI), published quarterly, by the USBLS. The recommended CPI can be found in the “all items” category of expenditures in the Average Annual Indexes tables of the USBLS Consumer Price Index Detailed Report published online.[1] The recommended ECI value for use includes total compensation for private industry workers and is not seasonally adjusted. The ECI values for use can be found in the ECI Current-Dollar Historical Listings published and regularly updated online.[2]

Crash costs estimates can be developed and adjusted based on human capital costs only or comprehensive societal costs. When human capital costs only are used a ratio based on the Consumer Price Index (CPI) is applied. When comprehensive crash costs are used, a ratio based on the Consumer Price Index (CPI) is applied to the human capital portion and a ratio based on the Employment Cost Index (ECI) is applied to the difference between the Comprehensive Societal costs and the Human Capital Costs. Adding the results together yields the adjusted crash cost. A short example of the recommended process for adjusting annual comprehensive crash costs to the year of interest is provided in the shaded box below.

Crash Cost Annual Adjustment

An agency wants to apply the EPDO Crash Frequency performance measure in order to prioritize high-crash locations within a city. Given human capital and comprehensive societal cost data from FHWA in 2001 dollars[1], what is the 2007 dollar value of crashes of various severity?

STEP 1: Adjust Human Capital Costs Using CPI

Multiply human capital costs by a ratio of the CPI for the year of interest divided by the CPI for 2001. Based on US Bureau of Labor Statistics data the CPI for year 2001 was 177.1 and in 2007 was 207.3.[2]

$$\text{CPI Ratio}_{(2001\text{-}2007)} = \frac{207.3}{177.1} = 1.2$$

The 2007 CPI-adjusted human capital costs can be estimated by multiplying the CPI ratio by 2001 human capital costs. For fatal crashes the CPI-Adjusted Human Capital Costs are calculated as:

2007 Human Capital Cost of Fatal Crash = $1,245,600 × 1.2 = $1,494,700 [per fatal crash]

The 2007 human capital costs for all crash severity levels are summarized in Exhibit B-3.

Exhibit A-3: 2007 CPI-Adjusted Human Capital Crash Costs

Crash Severity	2001 Human Capital Costs	2001 Comprehensive Societal Costs	2007 CPI-Adjusted Human Capital Costs
Fatal (K)	$1,245,600	$4,008,900	$1,494,700
Disabling Injury (A)	$111,400	$216,000	$133,700
Evident Injury (B)	$41,900	$79,000	$50,300
Possible Injury (C)	$28,400	$44,900	$34,100
PDO (O)	$6,400	$7,400	$7,700

STEP 2: Adjust Comprehensive Costs using ECI

Recall that comprehensive costs include the human capital costs. Therefore, in order to adjust the portion of the comprehensive costs that are not human capital costs, the difference between the comprehensive cost and the human capital cost is identified. For example, the unit crash cost difference in 2001 dollars for fatal (K) crashes is calculated as:

$4,008,900 - $1,245,600 = $2,763,300 [per fatal crash]

The differences for each crash severity level are shown in Exhibit B-4.

STEP 3: Adjust the Difference Calculated in Step 2 Using the ECI

The comprehensive crash cost portion that does not include human capital costs is adjusted using a ratio of the ECI for the year of interest divided by the ECI for 2001. Based on US Bureau of Labor Statistics data the Employment Cost Index for year 2001 was 85.8 and in 2007 was 104.9.[3] The ECI ratio can then be calculated as:

$$\text{ECI Ratio(2001-2007)} = \frac{104.9}{85.8} = 1.2$$

This ratio is then multiplied by the calculated difference between the 2001 human capital and 2001 comprehensive cost for each severity level. For example, the 2007 ECI-adjusted difference for the fatal crash cost is:

1.2 × $2,763,300=$3,316,000 [per fatal crash]

Exhibit A-4: 2007 ECI-Adjusted Crash Costs

Crash Severity	2001 Human Capital Costs	2001 Comprehensive Societal Costs	Cost Difference	2007 ECI-Adjusted Cost Difference
Fatal (K)	$1,245,600	$4,008,900	$2,763,300	$3,316,000
Disabling Injury (A)	$111,400	$216,000	$104,600	$125,500
Evident Injury (B)	$41,900	$79,000	$37,100	$44,500
Possible Injury (C)	$28,400	$44,900	$16,500	$19,800
PDO (O)	$6,400	$7,400	$1,000	$1,200

STEP 4: Calculate the 2007 Comprehensive Costs

The 2007 CPI-adjusted costs (Exhibit B-3) and the 2007 ECI-adjusted cost differences (Exhibit B-4) are summed, as shown in Exhibit B-5, to determine the 2007 Comprehensive Costs.

For example, the 2007 Comprehensive Cost for a fatal crash is calculated as:

2007 Comprehensive Fatal Crash Cost = $1,494,700+$3,316,000=$4,810,700 [per fatal crash]

Exhibit A-5: Adjusted 2007 Comprehensive Crash Costs

Crash Severity	2007 CPI-Adjusted Human Capital Costs	2007 ECI-Adjusted Cost Difference	2007 Comprehensive Costs
Fatal (K)	$1,494,700	$3,316,000	$4,810,700
Disabling Injury (A)	$133,700	$125,500	$259,200
Evident Injury (B)	$50,300	$44,500	$94,800
Possible Injury (C)	$34,100	$19,800	$53,900
PDO (O)	$7,700	$1,200	$8,900

Appendix References

1. Bureau of Labor Statistics. Washington, DC, 20212. Available from http://www.bls.gov/cpi/cpi_dr.htm#2001.

2. Bureau of Labor Statistics. *Employment Cost Index Historical Listing Current-dollar March 2001 – June 2008 (December 2005=100)*. Bureau of Labor Statistics, Office of Compensation Levels and Trends. Washington, DC, 20212-0001. Available from http://www.bls.gov/ncs/ect/#tables.

3. Council, F.M., E. Zaloshnja, T. Miller, and B. Persaud. *Crash Cost Estimates by Maximum Police Reported Injury Severity within Selected Crash Geometries.* Publication No. FHWA-HRT-05-051, Federal Highway Administration, U.S. Department of Transportation, October, 2005.

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PART B—ROADWAY SAFETY MANAGEMENT PROCESS

CHAPTER 5—DIAGNOSIS

5.1. Introduction 5-1
5.2. Step 1: Safety Data Review 5-2
 5.2.1. Descriptive Crash Statistics 5-2
 5.2.2. Summarizing Crashes by Location 5-4
5.3. Step 2: Assess Supporting Documentation..5-9
5.4. Step 3: Assess Field Conditions...5-10
5.5. Identify Concerns...5-12
5.6. Conclusions 5-12
5.7. Sample Problems 5-14
 5.7.1. Intersection 2 Assessment...5-16
 5.7.2. Intersection 9 Assessment...5-18
 5.7.3. Segment 1 Assessment 5-20
 5.7.4. Segment 5 Assessment 5-22
5.8. References 5-25

EXHIBITS

Exhibit 5–1: Roadway Safety Management Process Overview 5-1
Exhibit 5–2: Example Graphical Summary 5-3
Exhibit 5–3: Example Tabular Summary 5-4
Exhibit 5–4: Example of an Intersection Collision Diagram...5-5
Exhibit 5–5: Example Collision Diagram Symbols 5-6
Exhibit 5–6: Example Condition Diagram 5-8
Exhibit 5-7: Sites Selected For Further Review 5-14
Exhibit 5-8: Intersection Crash Data Summary...5-15
Exhibit 5-9: Roadway Segment Crash Data Summary 5-15
Exhibit 5-10: Crash Summary Statistics for Intersection 2..5-16
Exhibit 5-11: Collision Diagram for Intersection 2..5-17
Exhibit 5-12: Condition Diagram for Intersection 2..5-17
Exhibit 5-13: Crash Summary Statistics for Intersection 9..5-18
Exhibit 5-14: Collision Diagram for Intersection 9..5-19
Exhibit 5-15: Condition Diagram of Intersection 9 5-19
Exhibit 5-16: Crash Summary Statistics for Segment 1 5-21
Exhibit 5-17: Collision Diagram for Segment 1 5-21
Exhibit 5-18: Condition Diagram for Segment 1 5-22
Exhibit 5-19: Crash Summary Statistics for Segment 5 5-23
Exhibit 5-20: Collision Diagram for Segment 5 5-23
Exhibit 5-21: Condition Diagram for Segment 5 5-24
Exhibit A-1: Police Traffic Crash Form..5-26
Exhibit A-2: Police Traffic Crash Form (page 2) 5-27

APPENDICES

Appendix A – Example of Police Crash Report 5-26
Appendix B – Site Characteristic Considerations 5-28
Appendix C – Preparation for Conducting an Assessment of Field Conditions 5-31
Appendix D – Field Review Checklist 5-32
Appendices References 5-35

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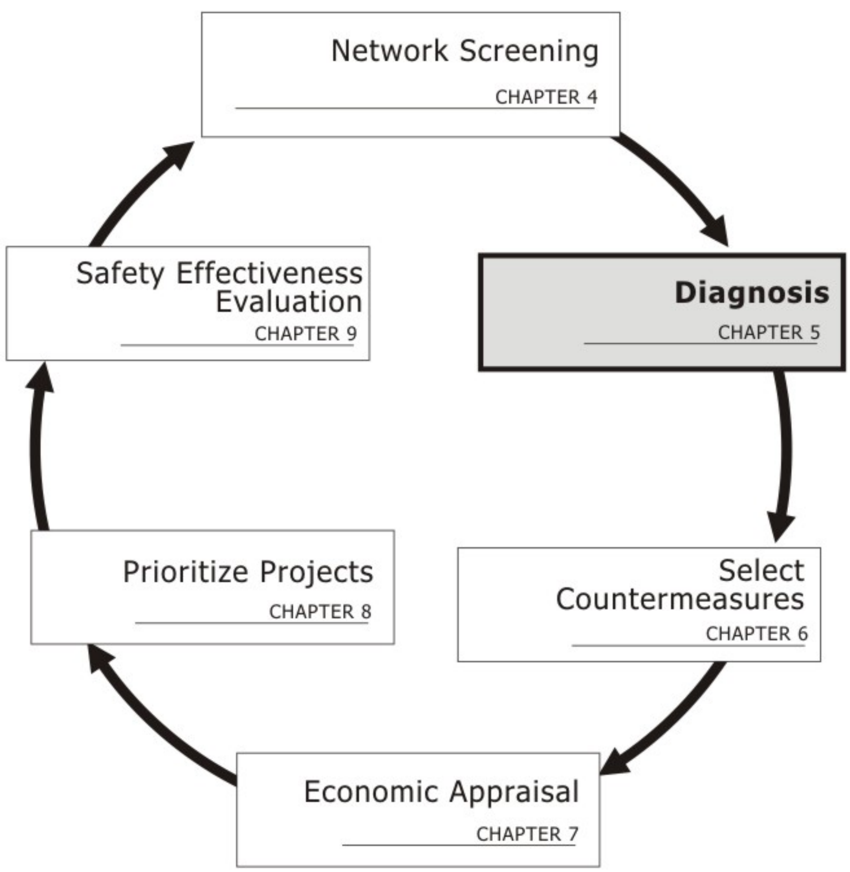
CHAPTER 5 DIAGNOSIS

5.1. INTRODUCTION

Diagnosis is the second step in the roadway safety management process (*Part B*), as shown in Exhibit 5-1. *Chapter 4* described the network screening process from which several sites are identified as the most likely to benefit from safety improvements. The activities included in the diagnosis step provide an understanding of crash patterns, past studies, and physical characteristics before potential countermeasures are selected. The intended outcome of a diagnosis is the identification of the causes of the collisions and potential safety concerns or crash patterns that can be evaluated further, as described in *Chapter 6*.

The purpose of site/crash diagnosis is to develop an understanding of factors that may lead to crashes.

Exhibit 5–1: Roadway Safety Management Process Overview



The assessment of a site begins with a review of crash data that may identify any patterns in the types of crashes and/or severity of crashes that have occurred.

The diagnosis procedure presented in this chapter represents the best available knowledge and is suitable for projects of various complexities. The procedure outlined in this chapter involves the following three steps; some steps may not apply to all projects:

- Step 1: Safety Data Review
  - o Review crash types, severities, and environmental conditions to develop summary descriptive statistics for pattern identification and,

  - Review crash locations.
- Step 2: Assess Supporting Documentation
  - Review past studies and plans covering the site vicinity to identify known issues, opportunities, and constraints.
- Step 3: Assess Field Conditions
  - Visit the site to review and observe multi-modal transportation facilities and services in the area, particularly how users of different modes travel through the site.

5.2. STEP 1: SAFETY DATA REVIEW

A site diagnosis begins with a review of safety data that may identify patterns in crash type, crash severity, or roadway environmental conditions (e.g., pavement, weather, and/or lighting conditions). The review may identify patterns related to time of day, direction of travel prior to crashes, weather conditions, or driver behaviors. Compiling and reviewing three to five years of safety data is suggested to improve the reliability of the diagnosis. The safety data review considers:

- Descriptive statistics of crash conditions (e.g., counts of crashes by type, severity, and/or roadway or environmental conditions); and
- Crash locations (i.e., collision diagrams, condition diagrams, and crash mapping using GIS tools).

5.2.1. Descriptive Crash Statistics

Crash data review may reveal patterns in crashes at a site.

Crash databases generally summarize crash data into three categories: information about the crash, the vehicle in the crash, and the people in the crash. In this step, crash data are reviewed and summarized to identify potential patterns. Descriptive crash statistics include summaries of:

- Crash Identifiers: date, day of week, time of day;
- Crash Type: defined by a police officer at the scene or, if self-reporting is used, according to the victims involved. Typical crash types are:
  - Rear-end
  - Sideswipe
  - Angle
  - Turning
  - Head-on
  - Run-off the road
  - Fixed object
  - Animal
  - Out of control
  - Work zone

Crash severity is often divided into categories according to the KABCO scale, which is defined in *Chapter 3, Section 3.2.2*

- Crash Severity: typically summarized according to the KABCO scale for defining crash severity (described in *Chapter 3*);

Descriptive crash statistics provide information about the crash, the vehicle, and people in the crash.

- Sequence of Events:
  - Direction of Travel;
  - Location of Parties Involved: northbound, southbound, eastbound, westbound; specific approach at a specific intersection or specific roadway milepost;
- Contributing Circumstances:
  - Parties Involved: vehicle only, pedestrian and vehicle, bicycle and vehicle;
  - Road Condition at the Time of the Crash: dry, wet, snow, ice;
  - Lighting Condition at the Time of the Crash: dawn, daylight, dusk, darkness without lights, darkness with lights;
  - Weather Conditions at the Time of the Crash: clear, cloudy, fog, rain, snow, ice; and
  - Impairments of Parties Involved: alcohol, drugs, fatigue.

These data are compiled from police reports. An example of a police report from Oregon is shown in Appendix A.

Bar charts, pie charts, or tabular summaries are useful for displaying the descriptive crash statistics. The purpose of the graphical summaries is to make patterns visible. Exhibits 5-2 and 5-3 provide examples of graphical and tabular summaries of crash data.

Exhibit 5–2: Example Graphical Summary

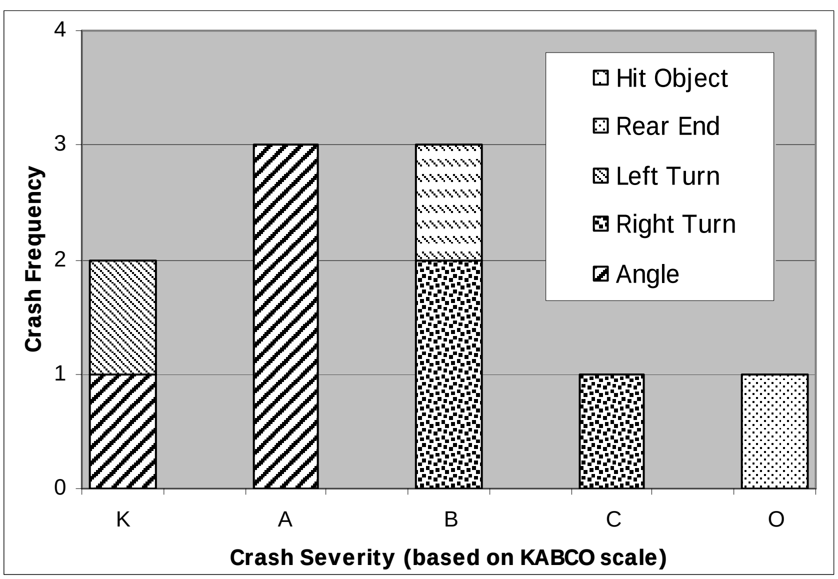


Exhibit 5–3: Example Tabular Summary

Accident Number	1	2	3	4	5	6	7	8	9	10
Date	1/3/92	2/5/92	8/11/92	7/21/93	1/9/93	2/1/93	9/4/94	12/5/08	4/7/94	2/9/94
Day of Week	SU	SA	SU	TU	WE	TH	SA	TH	MO	SU
Time of Day	2115	2010	1925	750	1310	950	1115	1500	1710	2220
Severity	A	A	O	B	K	K	B	C	A	B
Accident Type	Angle	Angle	Rear End	Right Turn	Angle	Left Turn	Right Turn	Right Turn	Angle	Hit Object
Road Condition	Wet	Dry	Dry	Dry	Wet	Dry	Dry	Dry	Wet	Wet
Light Condition	Dark	Dark	Dark	Dusk	Light	Light	Light	Light	Dusk	Dark
Direction	N	N	SW	W	S	W	N	S	N	N
Alcohol (BAC)	0.05	0.08	0.00	0.05	0.00	0.00	0.07	0.00	0.00	0.15

Adapted from Ogden(5)

Chapter 4 outlines the Probability of Specific Crash Types Exceeding Threshold Proportion performance measure which can also be used as a crash diagnosis tool.

Specific Crash Types Exceeding Threshold Proportion

If crash patterns are not obvious from a review of the descriptive statistics, mathematical procedures can sometimes be used as a diagnostic tool to identify whether a particular crash type is overrepresented at the site. The Probability of Specific Crash Types Exceeding Threshold Proportion performance measure described in *Chapter 4* is one example of a mathematical procedure that can be used in this manner.

The Probability of Specific Crash Types Exceeding Threshold Proportion performance measure can be applied to identify whether one crash type has occurred in higher proportions at one site than the observed proportion of the same crash type at other sites. Those crash types that exceed a determined crash frequency threshold can be studied in further detail to identify possible countermeasures. Sites with similar characteristics are suggested to be analyzed together because crash patterns will naturally differ depending on the geometry, traffic control devices, adjacent land uses, and traffic volumes at a given site. *Chapter 4* provides a detailed outline of this performance measure and sample problems demonstrating its use.

5.2.2. Summarizing Crashes by Location

Crash location can be summarized using three tools: collision diagrams, condition diagrams, and crash mapping. Each is a visual tool that may show a pattern related to crash location that may not be identifiable in another format.

Collision Diagram

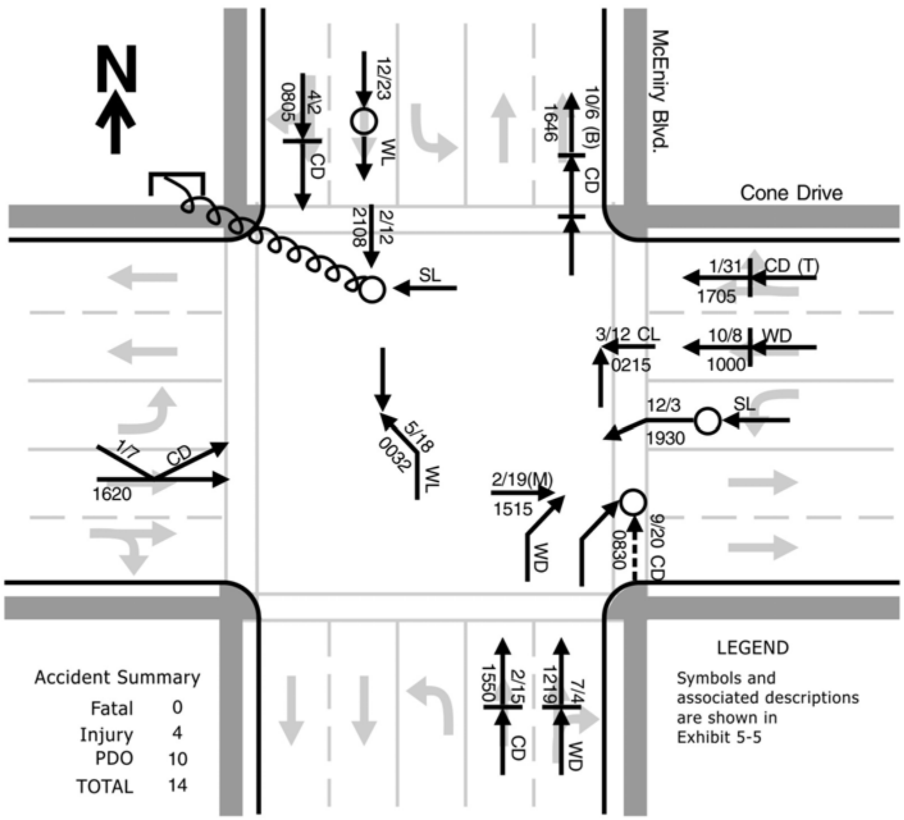
A collision diagram is a two-dimensional plan view representation of the crashes that have occurred at a site within a given time period. A collision diagram simplifies the visualization of crash patterns. Crash clusters or particular patterns of crashes by collision type (e.g., rear-end collisions on a particular intersection approach) may become evident on the crash diagram that were otherwise overlooked.

Visual trends identified in a collision diagram may not reflect a quantitative or statistically reliable assessment of site trends; however, they do provide an indication of whether or not patterns exist. If multiple sites are under consideration, it can be more efficient to develop the collision diagrams with software, if available.

Exhibit 5-4 provides an example of a collision diagram. Crashes are represented on a collision diagram by arrows that indicate the type of crash and the direction of travel. Additional information associated with each crash is also provided next to each symbol. The additional information can be any of the above crash statistics, but often includes some combination (or all) of severity, date, time of day, pavement condition, and light condition. A legend indicates the meaning of the symbols, the site location, and occasionally other site summary information.

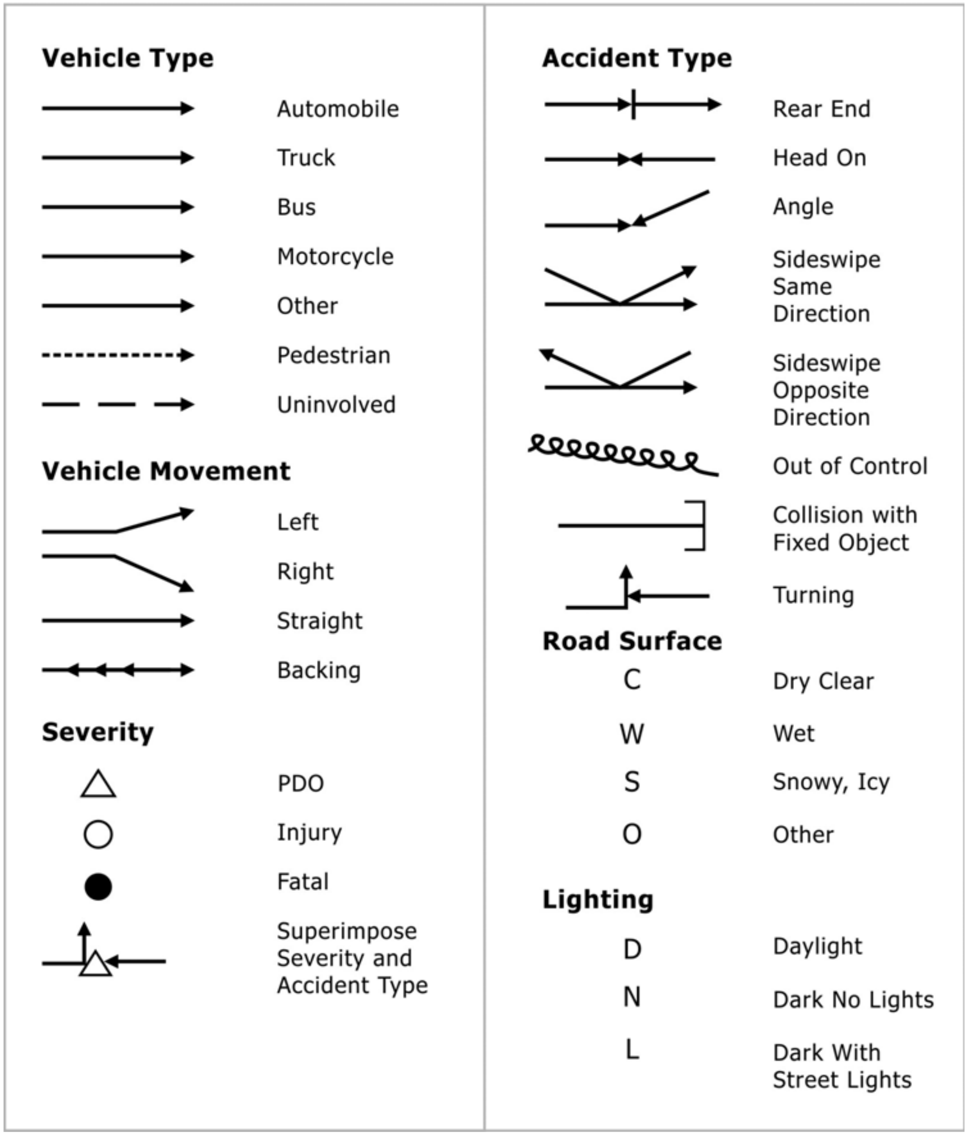
The collision diagram can be drawn by hand or developed using software. It does not need to be drawn to scale. It is beneficial to use a standard set of symbols for different crash types to simplify review and assessment. Example arrow symbols for different crash types are shown in Exhibit 5-5. These can be found in many safety textbooks and state transportation agency procedures.

Exhibit 5–4: Example of an Intersection Collision Diagram



Adapted from ITE Manual of Transportation Engineering Studies.[4]

Exhibit 5–5: Example Collision Diagram Symbols



Adapted from ITE Manual of Transportation Engineering Studies.[4]

Condition Diagram

A condition diagram is a plan view drawing of site characteristics including: roadway geometry, adjacent land use, & pavement conditions.

A condition diagram is a plan view drawing of as many site characteristics as possible.[2] Characteristics that can be included in the condition diagram are:

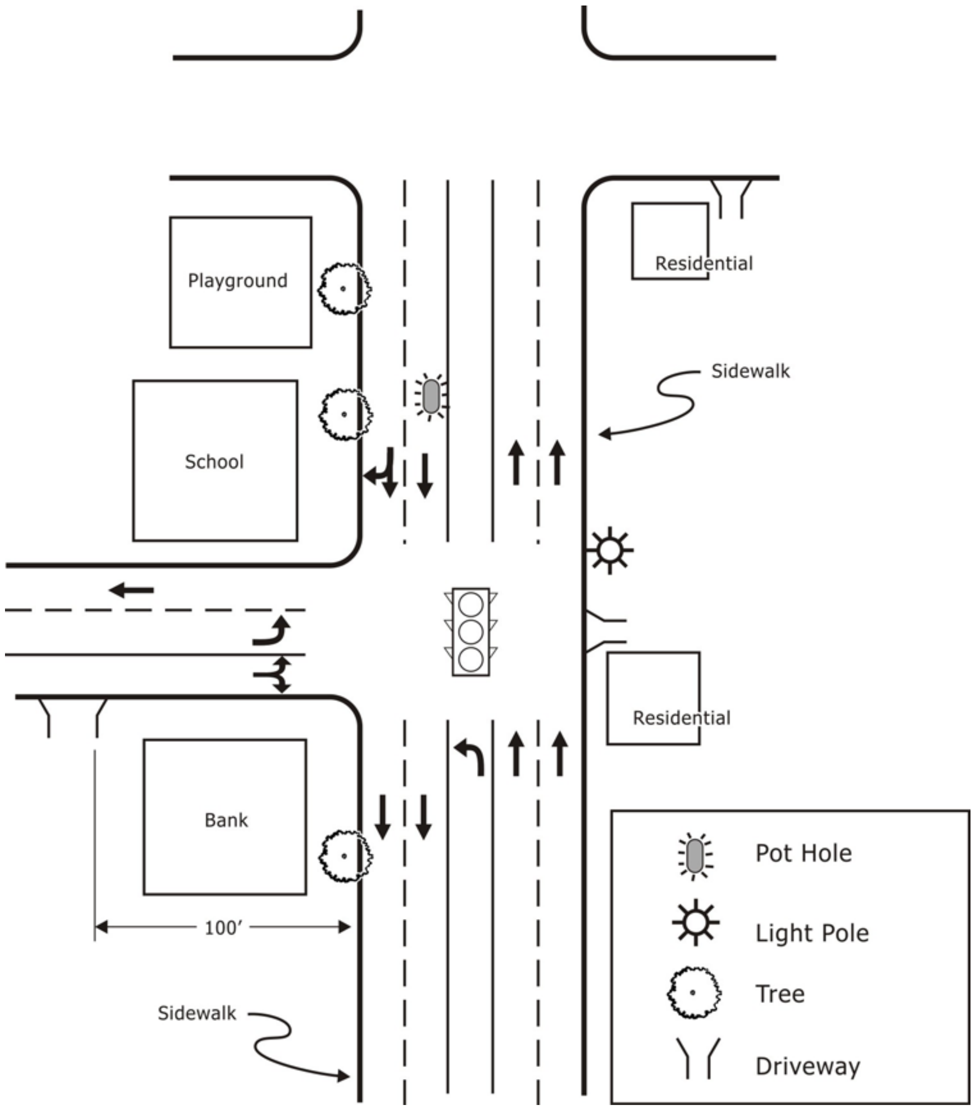
- Roadway
    - Lane configurations and traffic control;
    - Pedestrian, bicycle, and transit facilities in the vicinity of the site;
    - Presence of roadway medians;
    - Landscaping;
    - Shoulder or type of curb and gutter; and,

  - Locations of utilities (e.g., fire hydrants, light poles, telephone poles).
- Land Uses
  - Type of adjacent land uses (e.g., school, retail, commercial, residential) and;
  - Driveway access points serving these land uses.
- Pavement Conditions
  - Locations of potholes, ponding, or ruts.

The purpose of the condition diagram is to develop a visual site overview that can be related to the collision diagram's findings. Conceptually, the two diagrams could be overlaid to further relate crashes to the roadway conditions. Exhibit 5-6 provides an example of a condition diagram; the content displayed will change for each site depending on the site characteristics that may contribute to crash occurrence. The condition diagram is developed by hand during the field investigation and can be transcribed into an electronic diagram if needed. The diagram does not have to be drawn to scale.

A condition diagram can be related to a collision diagram to further understand potential patterns.

Exhibit 5-6: Example Condition Diagram



Crash Mapping

Jurisdictions that have electronic databases of their roadway network and geocoded crash data can integrate the two into a Geographic Information Systems (GIS) database.(3) GIS allows data to be displayed and analyzed based on spatial characteristics. Evaluating crash locations and trends with GIS is called crash mapping. The following describes some of the crash analysis techniques and advantages of using GIS to analyze a crash location (not an exhaustive list):

- Scanned police reports and video/photo logs for each crash location can be related to the GIS database to make the original data and background information readily available to the analyst.

- Data analyses can integrate crash data (e.g., location, time of day, day of week, age of participants, sobriety) with other database information, such as the presence of schools, posted speed limit signs, rail crossings, etc.

- The crash database can be queried to report crash clusters; that is, crashes within a specific distance of each other, or within a specific distance of a particular land use. This can lead to regional crash assessments and analyses of the relationship of crashes to land uses.

- Crash frequency or crash density can be evaluated along a corridor to provide indications of patterns in an area.

- Data entry quality control checks can be conducted easily and, if necessary, corrections can be made directly in the database.

The accuracy of crash location data is the key to achieving the full benefits of GIS crash analysis. The crash locating system that police use is most valuable when it is consistent with, or readily converted to, the locational system used for the GIS database. When that occurs, global positioning system (GPS) tools are used to identify crash locations. However, database procedures related to crash location can influence analysis results. For example, if all crashes within 200 feet of an intersection are entered into the database at the intersection centerline, the crash map may misrepresent actual crash locations and possibly lead to misinterpretation of site issues. These issues can be mitigated by advanced planning of the data set and familiarity with the process for coding crashes.

5.3. STEP 2: ASSESS SUPPORTING DOCUMENTATION

Assessing supporting documentation is the second step in the overall diagnosis of a site. The goal of this assessment is to obtain and review documented information or personal testimony of local transportation professionals that provides additional perspective to the crash data review described in Section 5.2. The supporting documentation may identify new safety concerns or verify the concerns identified from the crash data review.

Reviewing past site documentation provides historical context about the study site. Observed patterns in the crash data may be explained by understanding operational and geometric changes documented in studies conducted in the vicinity of a study site. For example, a review of crash data may reveal that the frequency of left-turning crashes at a signalized intersection increased significantly three years ago and have remained at that level. Associated project area documentation may show a corridor roadway widening project had been completed at that time, which may have led to the increased observed crash frequency due to increased travel speeds and/or the increase in the number of lanes opposing a permitted left turn.

Supporting documentation such as as-built plans, past studies, and past traffic counts further inform of conditions at a site.

Identifying the site characteristics through supporting documentation also helps define the roadway environment type (e.g., high-speed suburban commercial environment, or low-speed urban residential environment). This provides the context in which an assessment can be made as to whether certain characteristics have potentially contributed to the observed crash pattern. For example, in a high-speed rural environment a short horizontal curve with a small radius may increase the risk of a crash, whereas in a low-speed residential environment the same horizontal curve length and radius may be appropriate to help facilitate slower speeds.

The following types of information may be useful as supporting documentation to a site safety assessment:[6]

- Current traffic volumes for all travel modes;

- As-built construction plans;

- Relevant design criteria and pertinent guidelines;
- Inventory of field conditions (e.g., traffic signs, traffic control devices, number of travel lanes, posted speed limits, etc.);
- Relevant photo or video logs;
- Maintenance logs;
- Recent traffic operations and/or transportation studies conducted in the vicinity of the site;
- Land use mapping and traffic access control characteristics;
- Historic patterns of adverse weather;
- Known land use plans for the area;
- Records of public comments on transportation issues;
- Roadway improvement plans in the site vicinity; and,
- Anecdotal information about travel through the site.

A thorough list of questions and data to consider when reviewing past site documentation is provided in Appendix B.

5.4. STEP 3: ASSESS FIELD CONDITIONS

The diagnosis can be supported by a field investigation. Field observations can serve to validate safety concerns identified by a review of crash data or supporting documentation. During a field investigation, firsthand site information is gathered to help understand motorized and non-motorized travel to and through the site. Careful preparation, including participant selection and coordination, helps get the most value from field time. Appendix C includes guidance on how to prepare for assessing field conditions.

A comprehensive field assessment involves travel through the site from all possible directions and modes. If there are bike lanes, a site assessment could include traveling through the site by bicycle. If U-turns are legal, the assessment could include making U-turns through the signalized intersections. The goal is to notice, characterize, and record the "typical" experience of a person traveling to and through the site. Visiting the site during different times of the day and under different lighting or weather conditions will provide additional insights into the site's characteristics.

The following list provides several examples (not an exhaustive list) of useful considerations during a site review:[1]

- Roadway and roadside characteristics:
  - Signing and striping
  - Posted speeds
  - Overhead lighting
  - Pavement condition
  - Landscape condition

A field visit to experience site conditions may provide additional information about crashes.

  - Sight distances
  - Shoulder widths
  - Roadside furniture
  - Geometric design (e.g., horizontal alignment, vertical alignment, cross-section)
- Traffic conditions:
  - Types of facility users
  - Travel condition (e.g., free-flow, congested)
  - Adequate queue storage
  - Excessive vehicular speeds
  - Traffic control
  - Adequate traffic signal clearance time
- Traveler behavior:
  - Drivers—aggressive driving, speeding, ignoring traffic control, making maneuvers through insufficient gaps in traffic;
  - Bicyclists—riding on the sidewalk instead of the bike lane, riding excessively close to the curb or travel lane within the bicycle lane; ignoring traffic control, not wearing helmets; and,
  - Pedestrians—ignoring traffic control to cross intersections or roadways, insufficient pedestrian crossing space and signal time, roadway design that encourages pedestrians to improperly use facilities.
- Roadway consistency: Roadway cross-section is consistent with the desired functionality for all modes, and visual cues are consistent with the desired behavior;
- Land uses: Adjacent land use type is consistent with road travel conditions, degree of driveway access to and from adjacent land uses, and types of users associated with the land use (e.g., school-age children, elderly, commuters);
- Weather conditions: Although it will most likely not be possible to see the site in all weather conditions, consideration of adverse weather conditions and how they might affect the roadway conditions may prove valuable; and,
- Evidence of problems, for example:
  - Broken glass
  - Skid marks
  - Damaged signs
  - Damaged guard rail
  - Damaged road furniture
  - Damaged landscape treatments

Prompt lists are useful at this stage to help maintain a comprehensive assessment. These tools serve as a reminder of various considerations and assessments that can be made in the field. Prompt lists can be acquired from a variety of sources, including road safety audit guidebooks and safety textbooks. Alternately, jurisdictions can develop their own. Example prompt lists for different types of roadway environments are provided in Appendix D.

An assessment of field conditions is different from a road safety audit (RSA). A RSA is a formal examination that could be conducted on an existing or future facility and is completed by an independent and interdisciplinary audit team of experts. RSAs include an assessment of field conditions, as described in this section, but also include a detailed analysis of human factors and other additional considerations. The sites selected for a RSA are also selected differently than those selected through the network screening process described in *Chapter 4*. A RSA will often be conducted as a proactive means of reducing crashes and the site may or may not exhibit a known crash pattern or safety concern in order to warrant study.

5.5. IDENTIFY CONCERNS

Once the field assessment, crash data review, and supporting documentation assessment is completed the information can be compiled to identify any specific crash patterns that could be addressed by a countermeasure. Comparing observations from the field assessment, crash data review, and supporting documentation assessment may lead observations that would not have otherwise been identified. For example, if the crash data review showed a higher average crash frequency at one particular approach to an intersection, and the field investigation showed potential sight-distance constraints at this location, these two pieces of information may be related and warrant further consideration. Alternatively, the background site document assessment may reveal that the intersection's signal timing had recently been modified in response to capacity concerns. In the latter case, conditions may be monitored at the site to confirm that the change in signal timing is achieving the desired effect.

In some cases the data review, documentation review, and field investigation may not identify any potential patterns or concerns at a site. If the site was selected for evaluation through the network screening process, it may be that there are multiple minor factors contributing to crashes. Most countermeasures are effective in addressing a single contributing factor, and therefore it may require multiple countermeasures to realize a reduction in the average crash frequency.

5.6. CONCLUSIONS

This chapter described steps for diagnosing crash conditions at a site. The expected outcome of a diagnosis is an understanding of site conditions and the identification of any crash patterns or concerns, and recognizing the site conditions may relate to the patterns.

This chapter outlined three steps for diagnosing sites:

A site diagnosis is completed with a crash data review, review of supporting documentation, and a field visit.

- Step 1: Crash Data Review – The review considers descriptive statistics of crash conditions and locations that may help identify data trends. Collision diagrams, condition diagrams, and crash mapping are illustrative tools that can help summarize crash data in such a way that patterns become evident.
- Step 2: Assess Supporting Documentation – The assessment provides information about site conditions, including: infrastructure improvements, traffic operations, geometry, traffic control, travel modes in use, and relevant public comments. Appendix B provides a list of questions to consider when assessing supporting documentation.

- Step 3: Field Conditions Assessment – First-hand site information is gathered and compared to the findings of Steps 1 and 2. The on-site information gathered includes roadway and roadside characteristics, live traffic conditions, traveler behavior, land uses, roadway consistency, weather conditions, and any unusual characteristics not identified previously. The effectiveness of a field investigation is increased when conducted from a multi-modal, multi-disciplinary perspective. Appendices C and D provide additional guidance for preparing and conducting a field conditions assessment.

At this point in the roadway safety management process, sites have been screened from a larger network and a comprehensive diagnosis has been completed. Site characteristics are known and specific crash patterns have been identified. *Chapter 6* provides guidance on identifying the factors contributing to the safety concerns or crash patterns and identifying countermeasures to address them.

5.7. SAMPLE PROBLEMS

The Situation

Using the network screening methods outlined in *Chapter 4*, the roadway agency has screened the transportation network and identified five intersections and five roadway segments with the highest potential for safety improvement. The locations are shown in Exhibit 5-7.

Exhibit 5-7: Sites Selected For Further Review

Intersection #	Traffic Control	Number of Approaches	Major AADT	Minor AADT	Urban / Rural	Crash Totals		
						Year 1	Year 2	Year 3
2	Two-way stop	4	22,100	1,650	U	9	11	15
7	Two-way stop	4	40,500	1,200	U	11	9	14
9	Signal	4	47,000	8,500	U	15	12	10
11	Signal	4	42,000	1,950	U	12	15	11
12	Signal	4	46,000	18,500	U	10	14	8

Segment #	Cross-section (lanes)	Length (miles)	AADT	Undivided / Divided	Crash Totals		
					Year 1	Year 2	Year 3
1	2	0.60	9,000	U	16	15	14
2	2	0.4	15,000	U	12	14	10
5	4	0.35	22,000	U	18	16	15
6	4	0.3	25,000	U	14	12	10
7	4	0.45	26,000	U	12	11	13

Intersections 2 and 9 and Segments 1 and 5 will be studied in detail in this example. In a true application, all five intersections and segments would be studied in detail.

The Question

What are the crash summary statistics, collision diagrams, and condition diagrams for Intersections 2 and 9 and Segments 1 and 5?

The Facts

Intersections

- Three years of intersection crash data are shown in Exhibit 5-8.
- All study intersections have four approaches and are located in urban environments.
- The minor road is stop controlled.

Roadway Segments

- Three years of roadway segment crash data are shown in Exhibits 5-7.
- The roadway cross-section and length is shown in Exhibit 5-7.

Assumptions

- The roadway agency has generated crash summary characteristics, collision diagrams, and condition diagrams.
- The roadway agency has qualified staff available to conduct a field assessment of each site.

Exhibit 5-8: Intersection Crash Data Summary

		Crash Severity			Crash Type							
Intersection #	Total	Fatal	Injury	PDO	Rear End	Side-swipe/ Over taking	Right Angle	Ped	Bike	Head-On	Fixed Object	Other
2	**35**	**2**	**25**	**7**	**4**	**2**	**21**	**0**	**2**	**5**	**0**	**1**
7	34	1	17	16	19	7	5	0	0	0	3	0
9	**37**	**0**	**22**	**15**	**14**	**4**	**17**	**2**	**0**	**0**	**0**	**0**
11	38	1	19	18	6	5	23	0	0	4	0	0
12	32	0	15	17	12	2	14	1	0	2	0	1

Exhibit 5-9: Roadway Segment Crash Data Summary

		Crash Severity			Crash Type							
Segment #	Total	Fatal	Injury	PDO	Rear End	Angle	Head-On	Side-swipe	Ped	Fixed Object	Roll-Over	Other
1	**47**	**3**	**15**	**29**	**0**	**0**	**7**	**6**	**0**	**15**	**19**	**0**
2	36	0	5	31	0	1	3	3	3	14	10	2
5	**42**	**0**	**5**	**37**	**0**	**0**	**22**	**10**	**0**	**5**	**5**	**0**
6	36	0	5	31	4	0	11	10	0	5	4	2
7	36	0	6	30	2	0	13	11	0	4	3	3

Solution

The diagnoses for Intersections 2 and 9 are presented, followed by the diagnoses for Segments 1 and 5.

The following information is presented for each site:

- A set of pie charts summarizing the crash data;
- Collision diagram;
- Condition diagram; and
- A written assessment and summary of the site diagnosis.

The findings are used in the *Chapter 6* examples to select countermeasures for Intersections 2 and 9 and Segments 1 and 5.

5.7.1. Intersection 2 Assessment

Exhibit 5-10 contains crash summary statistics for Intersection 2. Exhibit 5-11 illustrates the collision diagram for Intersection 2. Exhibit 5-12 is the condition diagram for Intersection 2. All three exhibits were generated and analyzed to diagnose Intersection 2.

Exhibit 5-10: Crash Summary Statistics for Intersection 2

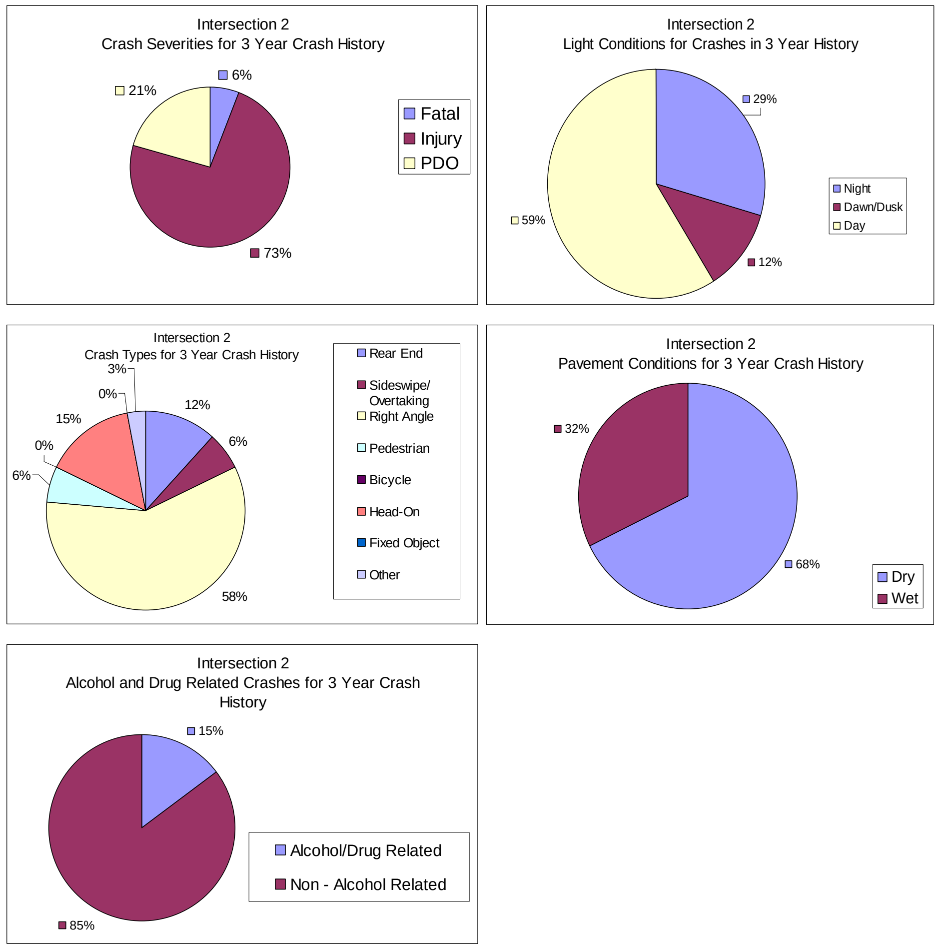


Exhibit 5-11: Collision Diagram for Intersection 2

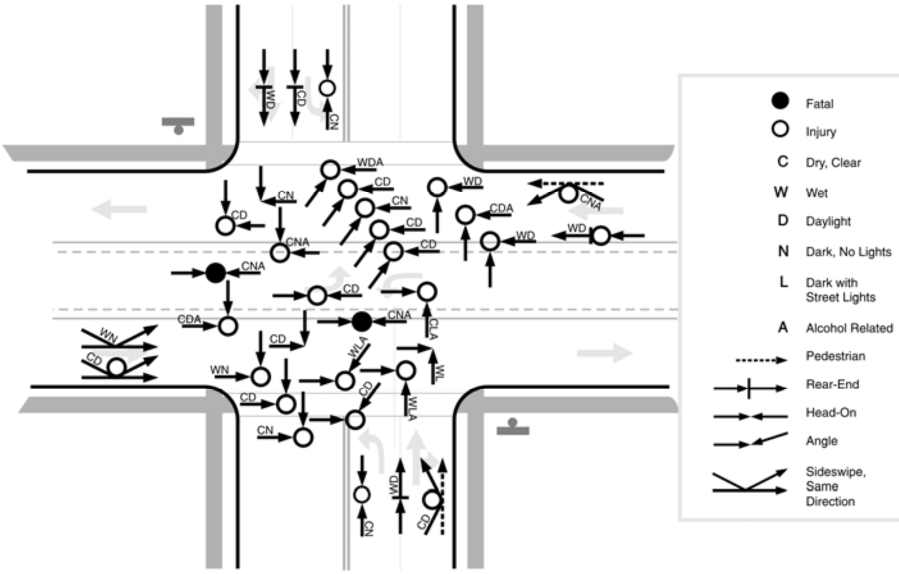
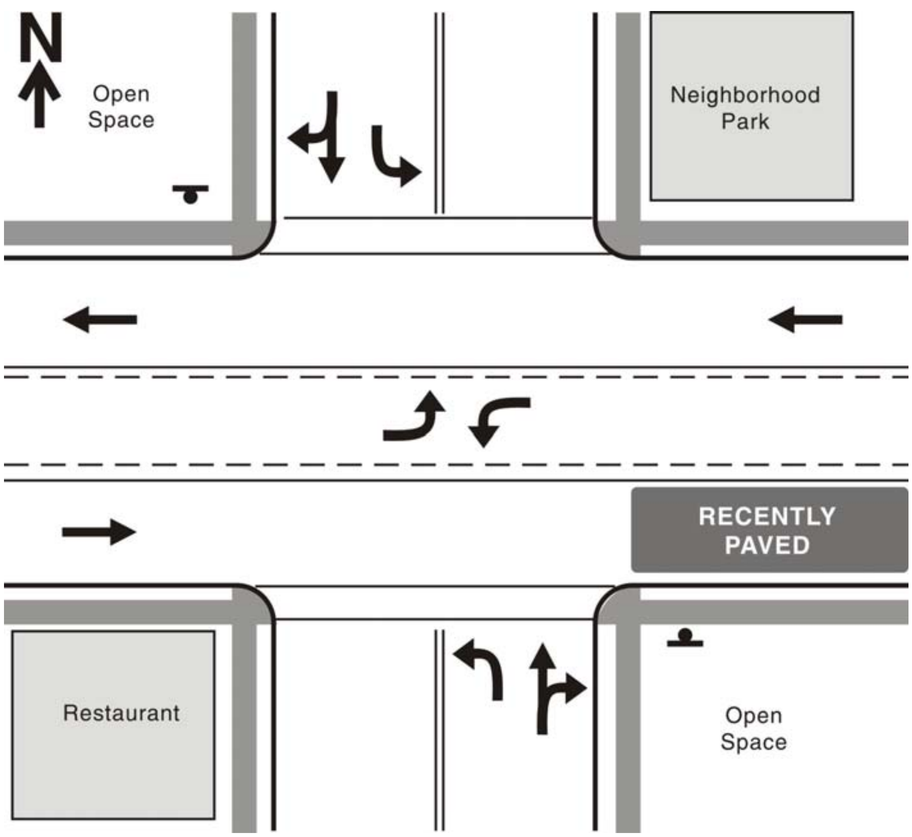


Exhibit 5-12: Condition Diagram for Intersection 2



The crash summary statistics and collision diagram for Intersection 2 indicate angle collisions (including right-angle collisions) comprise a large proportion of crashes. Vehicle direction and movement at the time of the collisions indicate that the angle crashes result from vehicles turning onto and off of the minor road as well as vehicles traveling through the intersection on the minor road across the major road. In the last three years, there have also been five head-on collisions, two of which resulted in a fatality.

An Intersection 2 field assessment confirmed the crash data review. It also revealed that because of the free flow condition on the major street, very few gaps are available for vehicles traveling onto or from the minor street. Sight distances on all four approaches were measured and considered adequate. During the off-peak field assessment, vehicle speeds on the major street were over 10 miles per hour faster than the posted speed limit and inappropriate for the desired character of the roadway.

5.7.2. Intersection 9 Assessment

Exhibit 5-13 contains crash summary characteristics for Intersection 9. Exhibit 5-14 illustrates the collision diagram for Intersection 9. Exhibit 5-15 is the condition diagram for Intersection 9. These exhibits were generated and analyzed to diagnose the safety concern at Intersection 9.

Exhibit 5-13: Crash Summary Statistics for Intersection 9

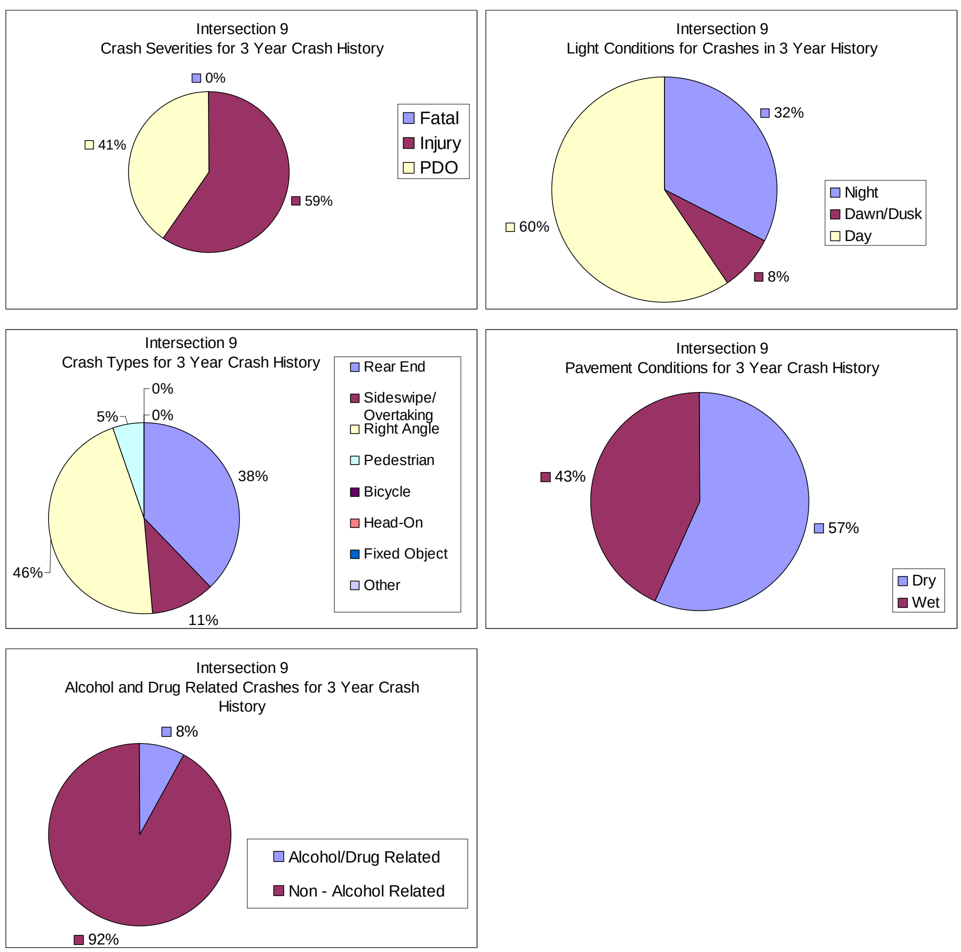


Exhibit 5-14: Collision Diagram for Intersection 9

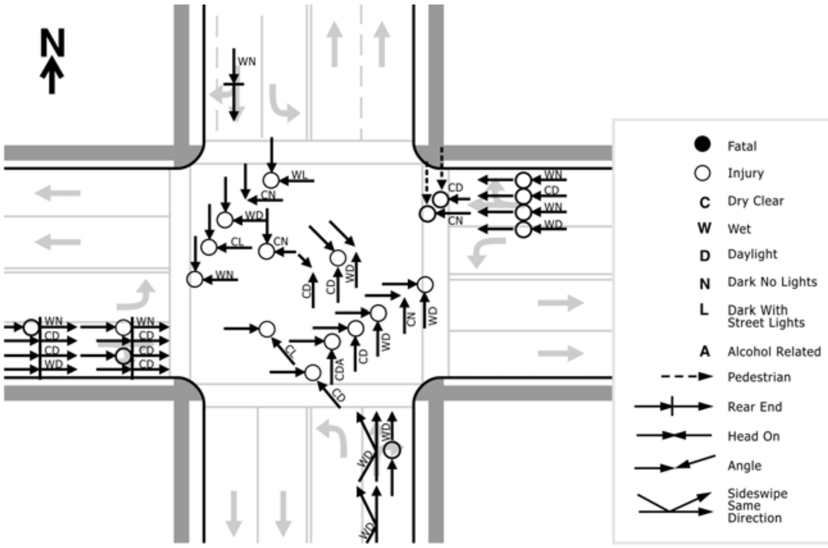
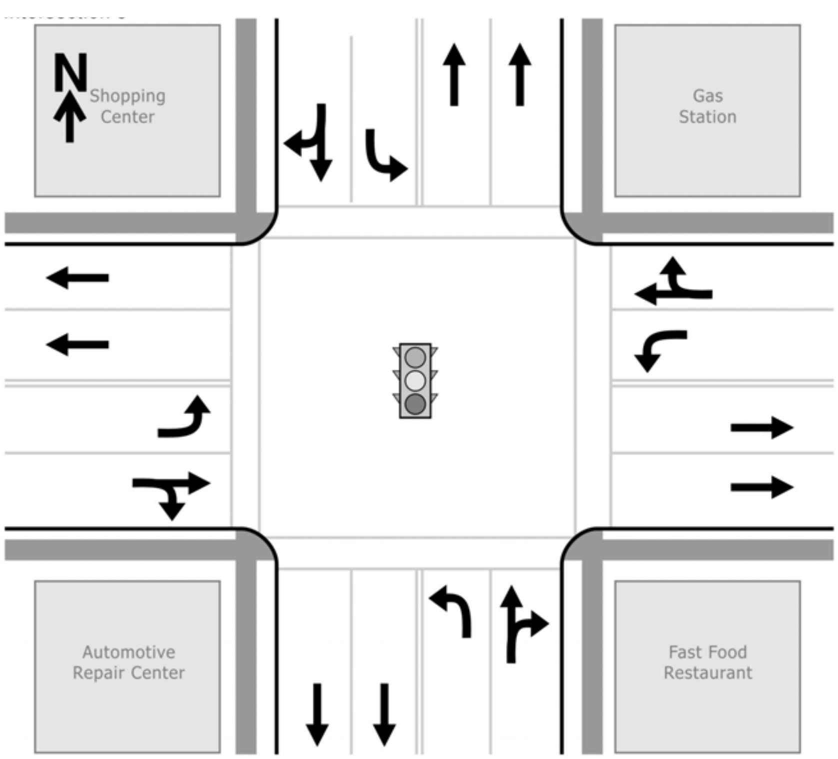


Exhibit 5-15: Condition Diagram of Intersection 9



The crash summary statistics and collision diagram indicate that a majority of the crashes at Intersection 9 are rear-end and angle collisions. In the past three years, the rear-end collisions occurred primarily on the east- and westbound approaches, and the angle collisions occurred in the middle of the intersection. All of the crashes were injury or PDO collisions.

A review of police crash reports indicates that many of the rear-end collisions on the east- and westbound approaches were partially due to the abrupt stop of vehicles traveling east- and westbound. Police crash reports also indicate that many of the angle collisions resulted from vehicles attempting to stop at the last second and continuing into the intersection or vehicles speeding up at the last second in an attempt to make it through the intersection during a yellow light.

Observations of local transportation officials reported that motorists on the east- and westbound approaches are not able to see the signal lenses far enough in advance of the intersection to stop in time for a red light. Local officials confirmed that national criteria for sight distance were met. Horizontal or vertical curves were not found to limit sight distance; however, morning and evening sun glare appears to make it difficult to determine signal color until motorists are essentially at the intersection. The average speed on the roadway also indicates that the existing 8-inch lenses may not be large enough for drivers to see at an appropriate distance to respond to the signal color. Other possible factors are that the length of the yellow interval and the clearance interval can be lengthened considering the limited visibility of the signal lenses. Factors of this sort are suggested to be evaluated further and compared with established criteria.

5.7.3. Segment 1 Assessment

Exhibit 5-16 contains crash summary characteristics for Segment 1. Exhibits 5-17 and 5-18 illustrate the collision diagram and the condition diagram for Segment 1, respectively. All three of these exhibits were generated and analyzed to diagnose the safety concern at Segment 1.

Exhibit 5-16: Crash Summary Statistics for Segment 1

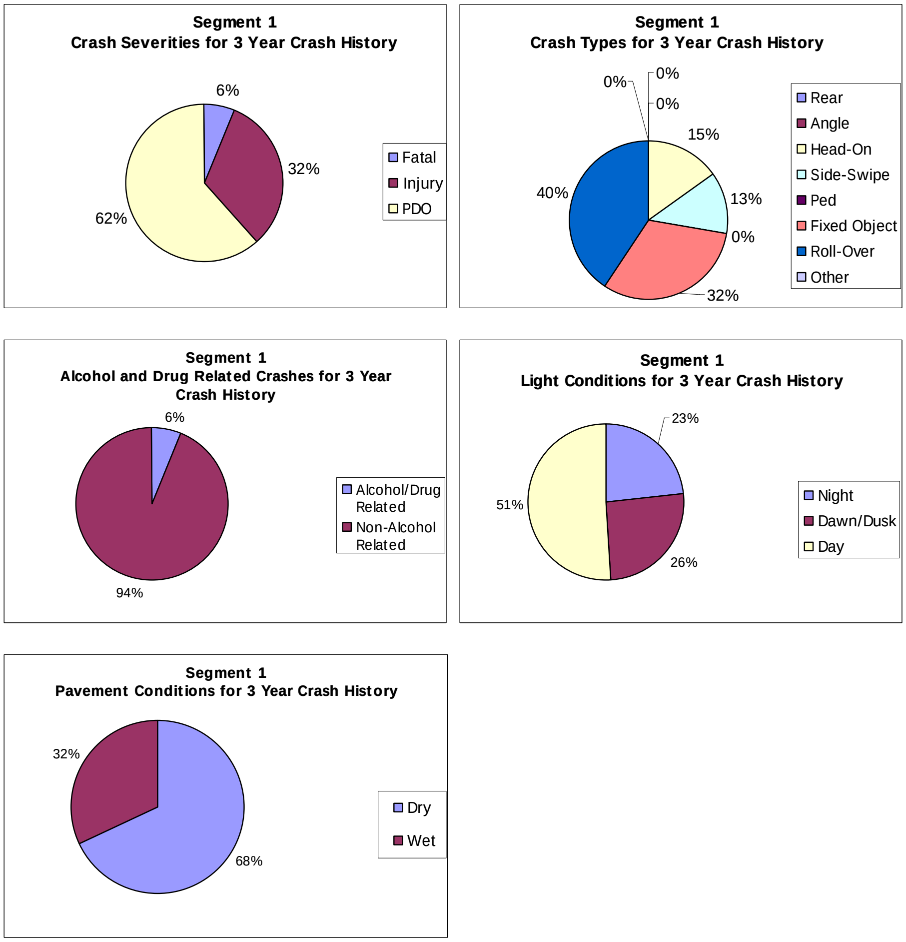


Exhibit 5-17: Collision Diagram for Segment 1

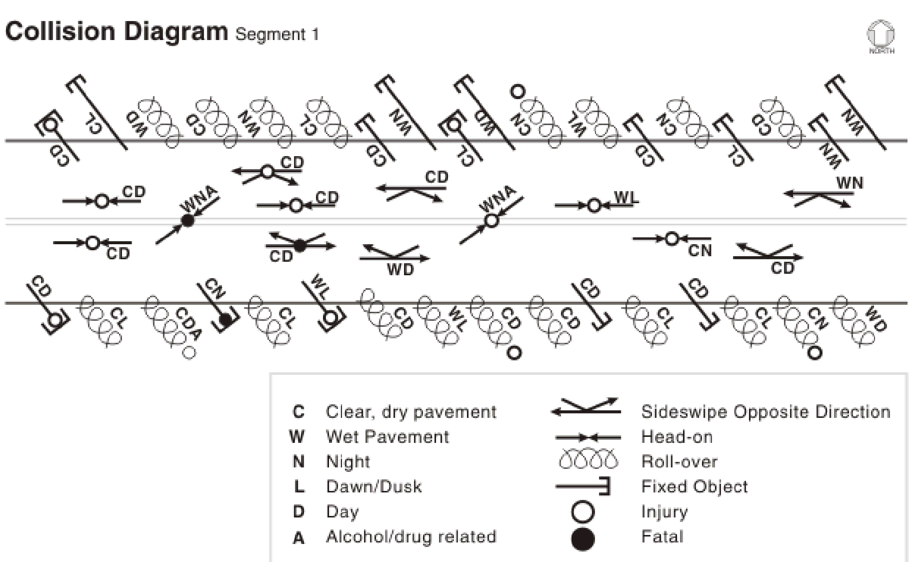
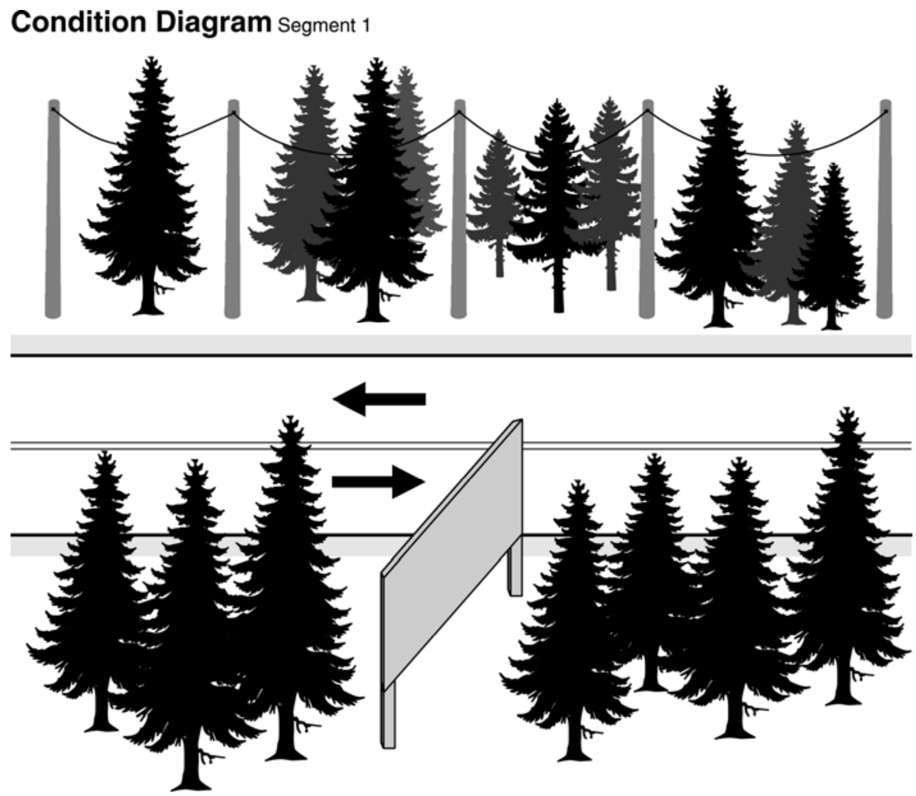


Exhibit 5-18: Condition Diagram for Segment 1



Segment 1 is an undivided two-lane rural highway; the end points of the segment are defined by intersections. The descriptive crash statistics indicate that three-quarters of the crashes on this segment in the last three years involved vehicles running off the road (i.e., roll-over or fixed object). The statistics and crash reports do not show a strong correlation between the run-off-the-road crashes and lighting conditions.

A detailed review of documented site characteristics and a field assessment indicate that the roadway is built to the roadway agency's criteria and is included in the roadway maintenance cycle. Past speed studies and observations made by the roadway agency's engineers indicate that vehicle speeds on the rural two-lane roadway are within 5 to 8 mph of the posted speed limit. Sight distance and delineation were also determined to be appropriate.

5.7.4. Segment 5 Assessment

Exhibit 5-19 contains crash summary characteristics for Segment 5. Exhibit 5-20 illustrates the collision diagram for Segment 5. Exhibit 5-21 is the condition diagram for Segment 5. All three of these exhibits were generated and analyzed to diagnose Segment 5.

Exhibit 5-19: Crash Summary Statistics for Segment 5

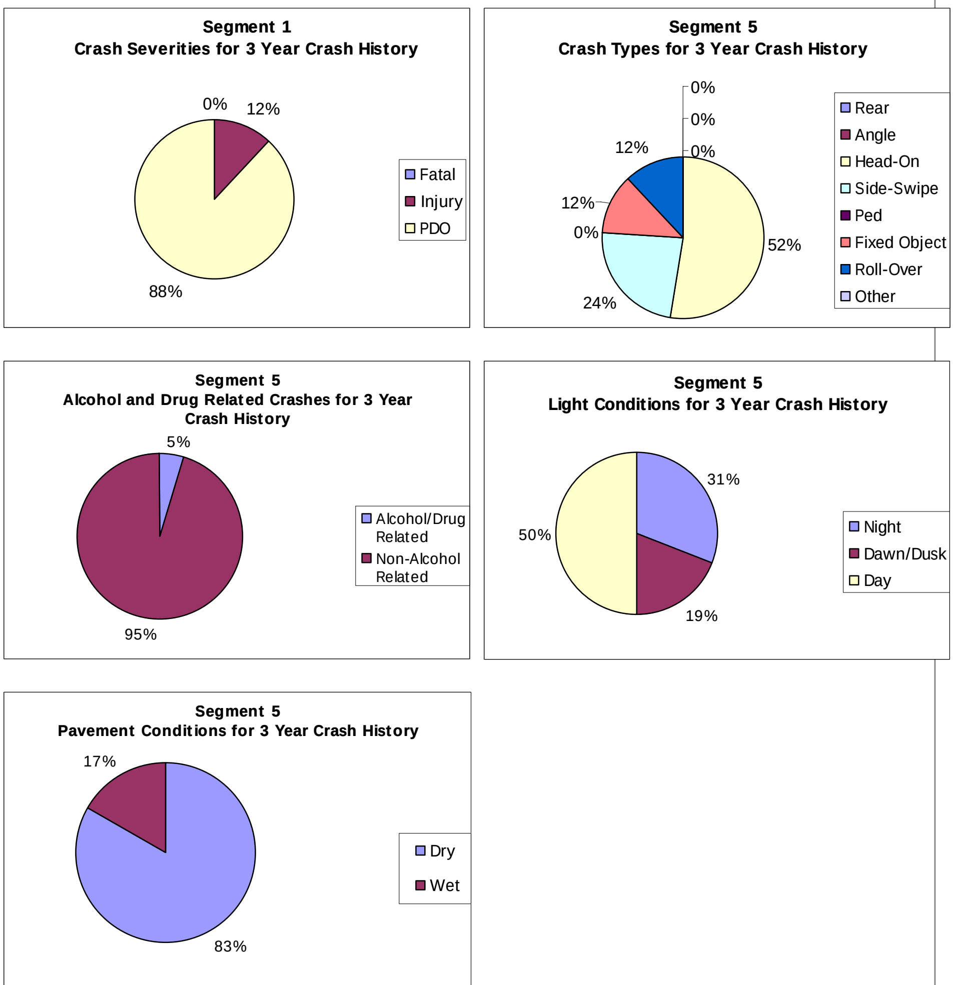


Exhibit 5-20: Collision Diagram for Segment 5

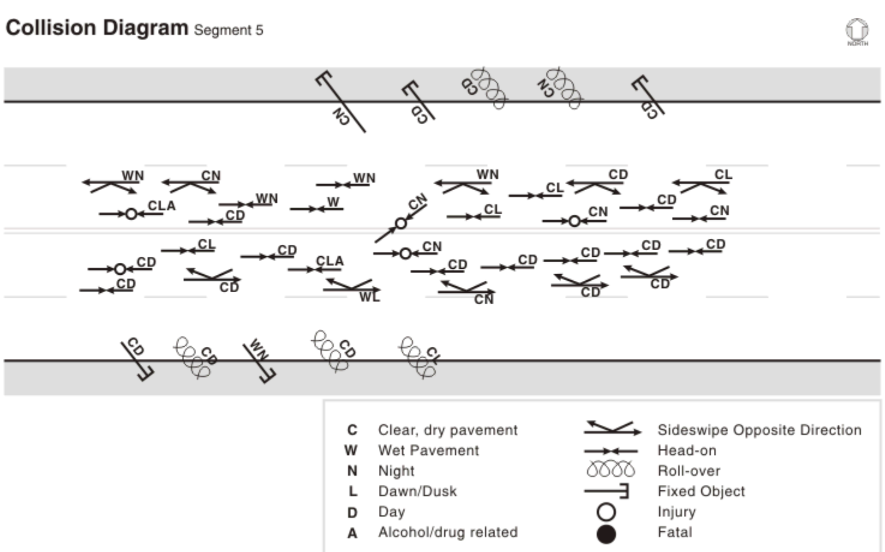
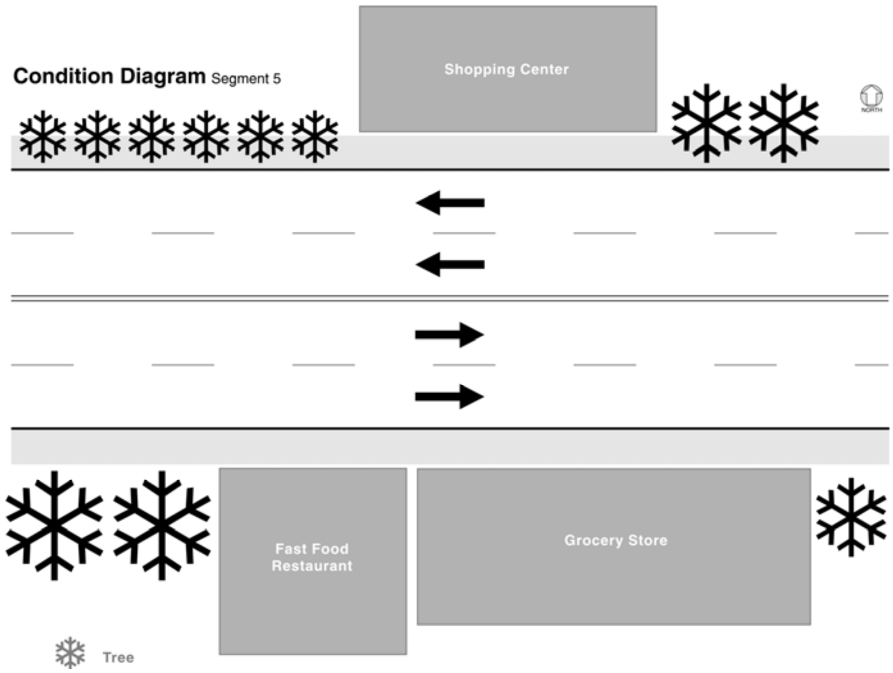


Exhibit 5-21: Condition Diagram for Segment 5



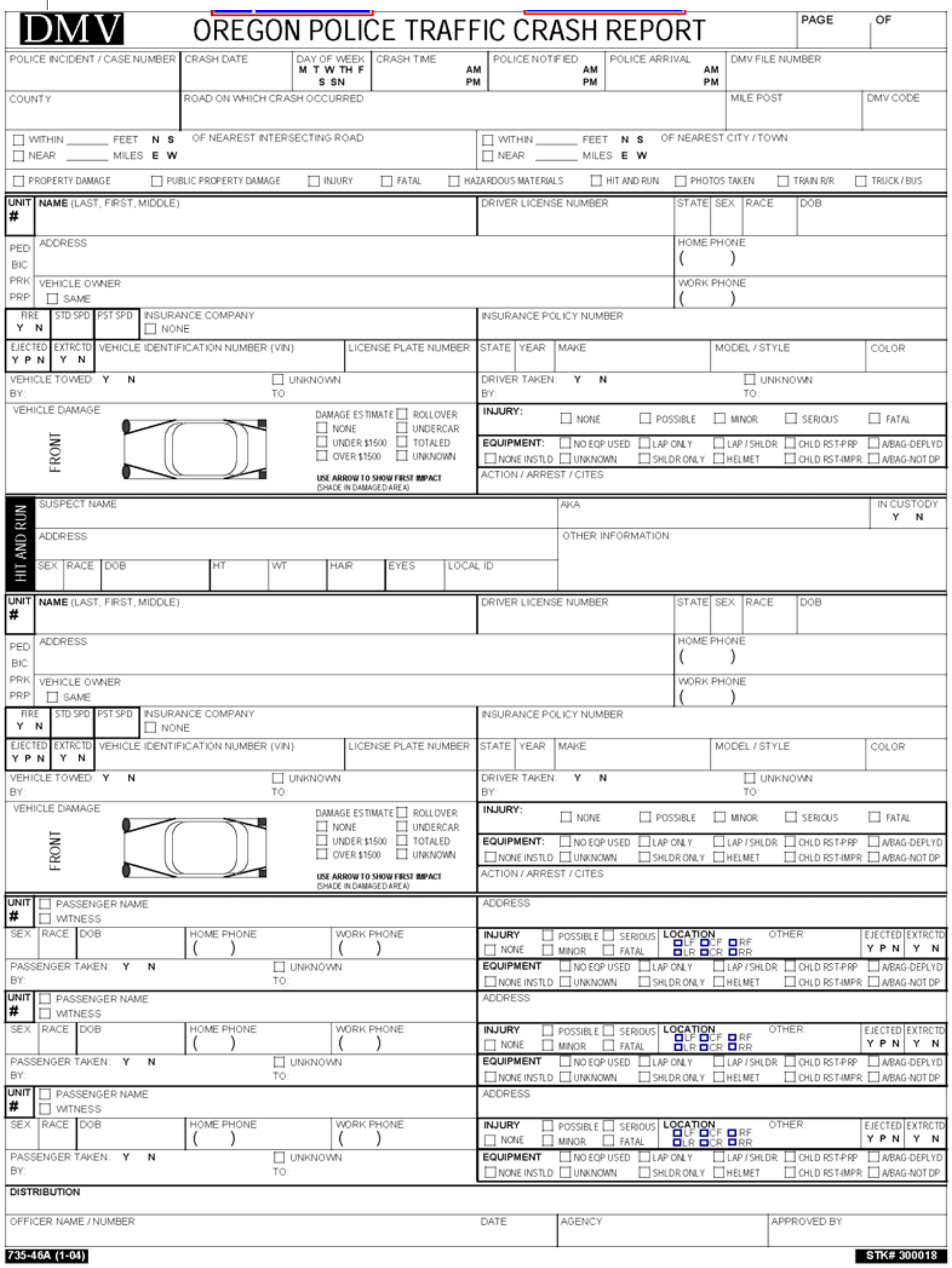
Segment 5 is a four-lane undivided urban arterial. It was originally constructed as a two-lane undivided highway. As a nearby city has grown, suburbs have developed around it, creating the need for the current four-lane roadway. During the past three years, the traffic volumes have increased dramatically, and the crash history over the same three years includes a high percentage (76%) of cross-over crashes (i.e., head-on and opposite direction side-swipe).

5.8. REFERENCES

1. Austroads. *Road Safety Audit.* Austroads 2nd Ed., 2002.
2. FHWA. *Road Safety Fundamentals*. Federal Highway Administration Office of Safety by BMI-SG (draft), 2004.
3. Harkey, D. *GIS-Based Crash Referencing and Analysis System.* Highway Safety Information System Summary Report No. FHWA-RD-99-081, Federal Highway Administration, U.S. Department of Transportation, McLean, VA, February 1999.
4. ITE. *Manual of Transportation Engineering Studies*, Institute of Transportation Engineers, Washington, DC, 1994.
5. Ogden, K.W. *Safer Roads: A Guide to Road Safety Engineering.* Ashgate Publishing Limited, 1996.
6. PIARC Technical Committee on Road Safety (C13). *Road Safety Manual.* World Road Association, 2003.

APPENDIX A – EXAMPLE OF POLICE CRASH REPORT

Exhibit A-1: Police Traffic Crash Form



DMV OREGON POLICE TRAFFIC CRASH REPORT PAGE OF

POLICE INCIDENT / CASE NUMBER | CRASH DATE | DAY OF WEEK M T W TH F S SN | CRASH TIME AM PM | POLICE NOTIFIED AM PM | POLICE ARRIVAL AM PM | DMV FILE NUMBER

COUNTY | ROAD ON WHICH CRASH OCCURRED | MILE POST | DMV CODE

☐ WITHIN ______ FEET N S ☐ NEAR ______ MILES E W OF NEAREST INTERSECTING ROAD | ☐ WITHIN ______ FEET N S ☐ NEAR ______ MILES E W OF NEAREST CITY / TOWN

☐ PROPERTY DAMAGE ☐ PUBLIC PROPERTY DAMAGE ☐ INJURY ☐ FATAL ☐ HAZARDOUS MATERIALS ☐ HIT AND RUN ☐ PHOTOS TAKEN ☐ TRAIN R/R ☐ TRUCK / BUS

UNIT # | NAME (LAST, FIRST, MIDDLE) | DRIVER LICENSE NUMBER | STATE | SEX | RACE | DOB

PED BIC PRK PRP | ADDRESS | HOME PHONE ( )

VEHICLE OWNER ☐ SAME | WORK PHONE ( )

FIRE Y N | STD SPD | PST SPD | INSURANCE COMPANY ☐ NONE | INSURANCE POLICY NUMBER

EJECTED Y P N | EXTRCTD Y N | VEHICLE IDENTIFICATION NUMBER (VIN) | LICENSE PLATE NUMBER | STATE | YEAR | MAKE | MODEL / STYLE | COLOR

VEHICLE TOWED: Y N BY: ☐ UNKNOWN TO: | DRIVER TAKEN: Y N BY: ☐ UNKNOWN TO:

VEHICLE DAMAGE FRONT | DAMAGE ESTIMATE ☐ NONE ☐ UNDER $1500 ☐ OVER $1500 ☐ ROLLOVER ☐ UNDERCAR ☐ TOTALED ☐ UNKNOWN | USE ARROW TO SHOW FIRST IMPACT (SHADE IN DAMAGED AREA)

INJURY: ☐ NONE ☐ POSSIBLE ☐ MINOR ☐ SERIOUS ☐ FATAL

EQUIPMENT: ☐ NO EQP USED ☐ LAP ONLY ☐ LAP / SHLDR ☐ CHLD RST-PRP ☐ A/BAG-DEPLYD ☐ NONE INSTLD ☐ UNKNOWN ☐ SHLDR ONLY ☐ HELMET ☐ CHLD RST-IMPR ☐ A/BAG-NOT DP

ACTION / ARREST / CITES

HIT AND RUN | SUSPECT NAME | AKA | IN CUSTODY Y N

ADDRESS | OTHER INFORMATION:

SEX | RACE | DOB | HT | WT | HAIR | EYES | LOCAL ID

UNIT # | NAME (LAST, FIRST, MIDDLE) | DRIVER LICENSE NUMBER | STATE | SEX | RACE | DOB

PED BIC PRK PRP | ADDRESS | HOME PHONE ( )

VEHICLE OWNER ☐ SAME | WORK PHONE ( )

FIRE Y N | STD SPD | PST SPD | INSURANCE COMPANY ☐ NONE | INSURANCE POLICY NUMBER

EJECTED Y P N | EXTRCTD Y N | VEHICLE IDENTIFICATION NUMBER (VIN) | LICENSE PLATE NUMBER | STATE | YEAR | MAKE | MODEL / STYLE | COLOR

VEHICLE TOWED: Y N BY: ☐ UNKNOWN TO: | DRIVER TAKEN: Y N BY: ☐ UNKNOWN TO:

VEHICLE DAMAGE FRONT | DAMAGE ESTIMATE ☐ NONE ☐ UNDER $1500 ☐ OVER $1500 ☐ ROLLOVER ☐ UNDERCAR ☐ TOTALED ☐ UNKNOWN | USE ARROW TO SHOW FIRST IMPACT (SHADE IN DAMAGED AREA)

INJURY: ☐ NONE ☐ POSSIBLE ☐ MINOR ☐ SERIOUS ☐ FATAL

EQUIPMENT: ☐ NO EQP USED ☐ LAP ONLY ☐ LAP / SHLDR ☐ CHLD RST-PRP ☐ A/BAG-DEPLYD ☐ NONE INSTLD ☐ UNKNOWN ☐ SHLDR ONLY ☐ HELMET ☐ CHLD RST-IMPR ☐ A/BAG-NOT DP

ACTION / ARREST / CITES

UNIT # | ☐ PASSENGER NAME ☐ WITNESS | ADDRESS

SEX | RACE | DOB | HOME PHONE ( ) | WORK PHONE ( ) | INJURY ☐ NONE ☐ POSSIBLE ☐ MINOR ☐ SERIOUS ☐ FATAL | LOCATION ☐LF ☐CF ☐RF ☐LR ☐CR ☐RR | OTHER | EJECTED Y P N | EXTRCTD Y N

PASSENGER TAKEN: Y N BY: ☐ UNKNOWN TO: | EQUIPMENT ☐ NO EQP USED ☐ LAP ONLY ☐ LAP / SHLDR ☐ CHLD RST-PRP ☐ A/BAG-DEPLYD ☐ NONE INSTLD ☐ UNKNOWN ☐ SHLDR ONLY ☐ HELMET ☐ CHLD RST-IMPR ☐ A/BAG-NOT DP

UNIT # | ☐ PASSENGER NAME ☐ WITNESS | ADDRESS

SEX | RACE | DOB | HOME PHONE ( ) | WORK PHONE ( ) | INJURY ☐ NONE ☐ POSSIBLE ☐ MINOR ☐ SERIOUS ☐ FATAL | LOCATION ☐LF ☐CF ☐RF ☐LR ☐CR ☐RR | OTHER | EJECTED Y P N | EXTRCTD Y N

PASSENGER TAKEN: Y N BY: ☐ UNKNOWN TO: | EQUIPMENT ☐ NO EQP USED ☐ LAP ONLY ☐ LAP / SHLDR ☐ CHLD RST-PRP ☐ A/BAG-DEPLYD ☐ NONE INSTLD ☐ UNKNOWN ☐ SHLDR ONLY ☐ HELMET ☐ CHLD RST-IMPR ☐ A/BAG-NOT DP

UNIT # | ☐ PASSENGER NAME ☐ WITNESS | ADDRESS

SEX | RACE | DOB | HOME PHONE ( ) | WORK PHONE ( ) | INJURY ☐ NONE ☐ POSSIBLE ☐ MINOR ☐ SERIOUS ☐ FATAL | LOCATION ☐LF ☐CF ☐RF ☐LR ☐CR ☐RR | OTHER | EJECTED Y P N | EXTRCTD Y N

PASSENGER TAKEN: Y N BY: ☐ UNKNOWN TO: | EQUIPMENT ☐ NO EQP USED ☐ LAP ONLY ☐ LAP / SHLDR ☐ CHLD RST-PRP ☐ A/BAG-DEPLYD ☐ NONE INSTLD ☐ UNKNOWN ☐ SHLDR ONLY ☐ HELMET ☐ CHLD RST-IMPR ☐ A/BAG-NOT DP

DISTRIBUTION

OFFICER NAME / NUMBER | DATE | AGENCY | APPROVED BY

735-46A (1-04) STK# 300018

Source: Oregon Department of Motor Vehicles

Exhibit A-2: Police Traffic Crash Form (page 2)

POLICE INCIDENT / CASE NUMBER | EMS NOTIFIED AM PM | EMS ARRIVAL AM PM | LOCAL CODES A B C D E | PAGE OF

Check ONE box in all categories. Check ALL boxes that apply in categories with (★).

FIRST HARMFUL EVENT

NON COLLISION
☐ OVERTURN
☐ FIRE / EXPLOSION
☐ IMMERSION
☐ GAS INHALATION
☐ OTHER NON COLLISION
☐ MEDICAL (Explain)

COLLISION WITH
☐ PEDESTRIAN
☐ PARKED MOTOR VEHICLE
☐ RAILWAY TRAIN
☐ BICYCLIST

CRASH TYPE
☐ HEAD ON
☐ REAR END
☐ ANGLE
☐ SIDESWIPE
☐ MANNER UNKNOWN

FIXED OBJECT
☐ BARRICADE
☐ BOULDER / ROCK
☐ BRIDGE O/PASS or RAILING
☐ BUILDING
☐ CULVERT HEADWALL
☐ CURBING
☐ DITCH
☐ DIVIDER - CNCRT or STEEL
☐ FENCE - NOT MEDIAN
☐ FIRE HYDRANT
☐ HIGHWAY GUARDRAIL
☐ HIGHWAY SIGN
☐ IMPACT ABSORBER
☐ LIGHT STANDARD
☐ MAILBOX
☐ OVERHEAD SIGN POST
☐ OVERHEAD STRUCTURE
☐ PIER or COLUMN
☐ RETAINING WALL
☐ SIDESLOPE EARTH
☐ SIDESLOPE ROCK or STONE
☐ TRAFFIC SIGNAL POST
☐ TREE
☐ UNDERPASS TUNNEL
☐ UTILITY POLE
☐ OTHER FIXED (Explain)

OTHER OBJECT (NOT FIXED)
☐ ANIMAL
☐ THROWN / FALLING OBJECT
☐ UNKNOWN
☐ OTHER OBJECT (Explain)

EVENT LOCATION

ON ROADWAY
☐ NON-INTERSECTION
☐ INTERSECTION
☐ INTERSECTION RELATED
☐ DRIVEWAY ACCESS
☐ INTERCHANGE AREA
☐ RAILROAD CROSSING
☐ BRIDGE
☐ TUNNEL
☐ OTHER ON-ROAD AREA

OFF ROADWAY
☐ SHOULDER
☐ TURNOUT
☐ ROADSIDE
☐ BEYOND RIGHT OF WAY
☐ MEDIAN
☐ DRIVEWAY
☐ PRIVATE DRIVE
☐ RAILROAD CROSSING
☐ OTHER OFF ROAD
☐ PARKING LOT
☐ UNKNOWN

SPECIAL ZONE
☐ NONE
☐ CONSTRUCTION
☐ MAINTENANCE
☐ UTILITY
☐ SNOW
☐ SCHOOL
☐ UNKNOWN WORK
☐ OTHER

WEATHER
☐ CLEAR
☐ CLOUDY (OVERCAST)
☐ RAIN
☐ SNOW
☐ SLEET / HAIL / ETC
☐ FOG / SMOG
☐ SMOKE
☐ BLOWING SAND / DIRT
☐ SEVERE CROSSWIND
☐ OTHER / UNKNOWN

SURFACE CONDITION
#1 #2
☐ ☐ DRY
☐ ☐ WET
☐ ☐ SNOW / SLUSH
☐ ☐ ICY
☐ ☐ MUDDY
☐ ☐ DEBRIS
☐ ☐ RUTS / HOLES / BUMPS
☐ ☐ WORN / POLISHED
☐ ☐ LOW / SOFT SHOULDER
☐ ☐ OTHER / UNKNOWN

SURFACE TYPE
#1 #2
☐ ☐ CONCRETE
☐ ☐ BLACKTOP / ASPHALT
☐ ☐ GRAVEL
☐ ☐ DIRT
☐ ☐ OTHER

LIGHT
☐ FULL DAYLIGHT
☐ DAWN
☐ DUSK
☐ DARK - LIGHTED WAY
☐ DARK - NOT LIGHTED
☐ UNKNOWN

TRAFFIC CONTROL TYPE
#1 #2
☐ ☐ NONE
☐ ☐ SCHOOL BUS LIGHTS
☐ ☐ OFFICER / CROSSING GUARD or FLAGGER
☐ ☐ TRAFFIC SIGNAL w/ PEDESTRIAN CONTROL
☐ ☐ TRAFFIC SIGNAL
☐ ☐ FLASHING BEACON
☐ ☐ STOP SIGN
☐ ☐ YIELD SIGN
☐ ☐ RR CROSSING GATES
☐ ☐ RR CROSSING BUCKS
☐ ☐ RR FLASHING SIGNAL
☐ ☐ RR CROSSING w/ PAVEMENT MARKINGS
☐ ☐ LANE CONTRLS / LINES / STRIPES / DEVICES
☐ ☐ SCHOOL SIGNAL
☐ ☐ OTHER REG SIGN
☐ ☐ TURN LANES
☐ ☐ UNKNOWN

TRAFFIC CONTROL DEVICE CONDITION
#1 #2
☐ ☐ NO MALFUNCTION
☐ ☐ DOWN / MISSING
☐ ☐ TURNED FROM PROPER POSITION
☐ ☐ OBSCURED BY OTHER SIGNS
☐ ☐ OBSCURED BY PARKED VEHICLE
☐ ☐ OBSCURED BY VEGETATION
☐ ☐ LIGHTS MALFUNCTION
☐ ☐ LIGHTS STUCK
☐ ☐ GATES INOPERATIVE
☐ ☐ GATE ARM MISSING
☐ ☐ OTHER RR MALFUNCTN
☐ ☐ OTHER IMPAIRMENT
☐ ☐ UNKNOWN

ROAD CHARACTER
#1 #2
☐ ☐ STRAIGHT and LEVEL
☐ ☐ STRAIGHT w/ GRADE
☐ ☐ CURVED and LEVEL
☐ ☐ CURVED w/ GRADE

VEH #1 ___ NUMBER OF LANES

VEH #2 ___ NUMBER OF LANES

___ TOTAL NUMBER OF LANES

ROAD FLOW
#1 #2
☐ ☐ ONE WAY TRAFFIC
☐ ☐ NOT PHYSLY DIVIDED

MEDIAN TYPE
☐ ☐ UNPAVED
☐ ☐ BARRIER
☐ ☐ PAVED
☐ ☐ CONT LEFT TURN

DRIVER LICENSE VIOLATION
DRIVER
#1 #2
☐ ☐ NONE
☐ ☐ INSTRUCTION PERMIT
☐ ☐ LICENSE RESTRICTION
☐ ☐ EXPIRED LICENSE
☐ ☐ OUT OF CLASS
☐ ☐ SUSPNDED / REVOKED
☐ ☐ UNLICENSED

★ DRIVER FACTORS
DRIVER
#1 #2
☐ ☐ NONE
☐ ☐ CELL PHONE USE
☐ ☐ OBSTRUCTED VIEW
☐ ☐ FAILED TO YIELD ROW
☐ ☐ DISRGRD TRAF SIGN
☐ ☐ TOO FAST FOR COND
☐ ☐ MADE IMPROPER TURN
☐ ☐ WRONG SIDE/WAY
☐ ☐ FOLLOW TOO CLOSELY
☐ ☐ IMPROPER LANE CHNG
☐ ☐ IMPROPER BACKING
☐ ☐ IMPROPER PASSING
☐ ☐ IMPROPER SIGNAL
☐ ☐ IMPROPER PARKING
☐ ☐ FATIGUE / DROWSY
☐ ☐ ILL / BLACKOUT
☐ ☐ UNKNOWN
☐ ☐ OTHER

★ IMPAIRMENT
DRIVER
#1 #2
☐ ☐ NONE
☐ ☐ UNDER INFL - DRUGS
☐ ☐ UNDER INFL - ALCOHOL
☐ ☐ UNDER INFL - MEDS
☐ ☐ UNKNOWN

DETERMINED BY:
☐ ☐ INTOXILYZER TEST
☐ ☐ BLOOD OR URINE TEST
☐ ☐ FIELD SOB. TEST
☐ ☐ OBSERVED (SPEECH, ODOR, ETC.)
☐ ☐ DRE EVALUATION
☐ ☐ STATEMENTS
☐ ☐ UNKNOWN
☐ ☐ OTHER

RESULTS OF TEST:
D1 ____% D2 ____%
☐ ☐ NO TEST GIVEN
☐ ☐ TEST REFUSED
☐ ☐ TESTED FOR DRUGS
☐ ☐ RESLTS NOT AVAILABLE

★VEH RELATED FACTORS
#1 #2
☐ ☐ NONE
☐ ☐ BRAKES
☐ ☐ STEERING
☐ ☐ POWER PLANT
☐ ☐ SUSPENSION
☐ ☐ TIRES
☐ ☐ EXHAUST
☐ ☐ LIGHTS
☐ ☐ SIGNALS
☐ ☐ WINDOWS / WINDSHLD
☐ ☐ RESTRAINT SYSTEM
☐ ☐ WHEELS
☐ ☐ COUPLING
☐ ☐ CARGO
☐ ☐ OTHER

VEHICLE MOVEMENT
#1 #2
☐ ☐ BACKING
☐ ☐ STOPPED
☐ ☐ STRAIGHT AHEAD
☐ ☐ TURNING RIGHT
☐ ☐ TURNING LEFT
☐ ☐ MAKING U-TURN
☐ ☐ ENTER TRAFFIC LANE
☐ ☐ LEAVE TRAFFIC LANE
☐ ☐ OVERTAKING
☐ ☐ CHANGING LANES
☐ ☐ AVOIDING MANEUVER
☐ ☐ MERGING
☐ ☐ PARKING
☐ ☐ NEGOTIATING A CURVE
☐ ☐ OTHER

TRAILER TYPE
#1 #2
☐ ☐ LOG BUNK
☐ ☐ SEMITRAILER
☐ ☐ POLE TRAILER
☐ ☐ FULL TRAILER
☐ ☐ MOBILE HOME
☐ ☐ UTILITY TRAILER
☐ ☐ TRAVEL TRAILER
☐ ☐ BOAT TRAILER
☐ ☐ FARM EQUIPMENT
☐ ☐ HORSE TRAILER
☐ ☐ VEHICLE IN TOW
☐ ☐ OTHER / UNKNOWN

TRUCK CONFIGURATION
#1 #2
☐ ☐ TRUCK (2 or 3 AXLE)
☐ ☐ TRUCK / TRACTOR-SEMI
☐ ☐ TRUCK and TRAILER
☐ ☐ DOUBLE TRAILERS
☐ ☐ TRIPLE TRAILERS
☐ ☐ DROMEDARY and SEMI
☐ ☐ HEAVY HAUL CONFIG
☐ ☐ BUS
☐ ☐ OTHER

★ PASSENGER FACTORS

PASS #1 #2 **UNIT #1**
☐ ☐ NONE
☐ ☐ INTERFERED w/DRIVER
☐ ☐ UNDER INFL - DRUGS
☐ ☐ UNDER INFL - ALCOHOL
☐ ☐ UNKNOWN
☐ ☐ OTHER (Explain)

PASS #1 #2 **UNIT #2**
☐ ☐ NONE
☐ ☐ INTERFERED w/DRIVER
☐ ☐ UNDER INFL - DRUGS
☐ ☐ UNDER INFL - ALCOHOL
☐ ☐ UNKNOWN
☐ ☐ OTHER (Explain)

PEDESTRIAN LOCATION

IN ROAD
☐ IN X-WALK
☐ NOT IN X-WALK
☐ NO X-WALK AVAILABLE

INTERSECTION
☐ IN X-WALK
☐ NOT IN X-WALK
☐ NO X-WALK AVAILABLE

OTHER
☐ NOT IN ROADWAY
☐ SHOULDER
☐ MEDIAN
☐ BIKE LANE
☐ UNKNOWN

PEDESTRIAN TYPE
☐ NONE
☐ PEDESTRIAN
☐ BICYCLIST
☐ CONVEYANCE
☐ WHEELCHAIR
☐ ANIMAL RIDER
☐ RIDER of ANIM DRAWN VEH
☐ UNKNOWN
☐ OTHER (Explain)

★ PEDESTRIAN ACTION
☐ ENTER / CROSS ROAD
☐ WALK / RIDE w/TRAFF
☐ WALK / RIDE AGAINST
☐ STEP ON / OFF VEHICLE
☐ STEP ON / OFF SCH BUS
☐ APPRCH / LEAVE SC BUS
☐ APPROACH / LEAVE VEH
☐ WORK / PUSHING VEHICLE
☐ OTHER WORKING
☐ PLAYING
☐ STANDING
☐ LYING DOWN
☐ UNKNOWN

PED / BIKE VISIBILITY

CLOTHING
☐ NO CONTRAST w/BKGRND
☐ CONTRASTED w/BKGRND
☐ REFLECTIVE

OTHER
☐ OTHER LIGHT SOURCE
☐ UNKNOWN

★ PED / BIKE FACTORS
☐ NONE
☐ FAILED TO YIELD ROW
☐ DISREGARD TRAFFIC SIGN
☐ ILLEGALLY IN ROAD
☐ EQUIPMENT VIOLATION
☐ CLOTHING NOT VISIBLE
☐ UNDER INFL - DRUGS
☐ UNDER INFL - ALCOHOL
☐ UNKNOWN
☐ OTHER (Explain)

North

SKETCH & NARRATIVE

(NOT TO SCALE)

UNIT 1 2

SKID MARKS TO (FEET) ____________ ____________

DISTANCE AFTER (FEET) ____________ ____________

APPENDIX B – SITE CHARACTERISTIC CONSIDERATIONS

The following provides a list of questions and data to consider when reviewing past site documentation.(3) This list is intended to serve as an example and is not exhaustive.

Traffic Operations

- Do past studies indicate excessive speeds at or through the site?
- If the site is a signalized intersection, is there queuing on the intersection approaches?
- If the site is a signalized intersection, what signal warrant does the intersection satisfy? Does the intersection currently satisfy the signal warrants?
- Is there adequate capacity at or through the site?
- What is the proportion of heavy vehicles traveling through the site?
- Does mainline access to adjacent land negatively influence traffic operations?

Geometric Conditions

- Is the roadway geometry in the vicinity of the site consistent with the adopted functional classification?
- What are the available stopping sight distances and corner sight distances at each driveway or intersection?
- Have there been recent roadway geometry changes that may have influenced crash conditions?
- How does the site design compare to jurisdictional design criteria and other related guidelines? Non-compliance and/or compliance does not directly relate to safe or unsafe conditions, though it can inform the diagnostic process.

Physical Conditions

- Do the following physical conditions indicate possible safety concerns:
  - pavement conditions;
  - drainage;
  - lighting;
  - landscaping;
  - signing or striping; and,
  - driveway access.
- Are there specific topographic concerns or constraints that could be influencing conditions?

Planned Conditions

- Are improvements planned at the site or in the vicinity that may influence safety conditions?
- How will the planned conditions affect the function and character of the site? What is the objective of the planned changes (i.e. increase capacity, etc.)? How could these changes influence safety?
- Are there planning or policy statements relating to the site such as:
    - functional classification;
    - driveway access management;
    - pedestrian, bicycle, transit, or freight policies; and,
    - future connections for motorized traffic, pedestrians, or cyclists.

Transit, Pedestrian, and Bicycle Activity

- What transportation modes do people use to travel through the site?
- Is there potential to introduce other travel modes at the site (i.e. new bus stops, sidewalks, bike lanes, or multi-use path)?
- Are bus stops located in the vicinity of the site?
- Is there a continuous bicycle or pedestrian network in the area?
- What visual clues exist to alert motorists to pedestrians and bicyclists (e.g. striped bike lanes, curb extensions at intersections for pedestrians)?
- Is there any historical information relating to multimodal concerns such as:
    - roadway shoulders and edge treatments;
    - transit stop locations;
    - exclusive or shared transit lanes;
    - bicycle lanes;
    - sidewalks; and,
    - adjacent parking.

Heavy Vehicle Activity

- Are there concerns related to heavy vehicles. Such concerns could include:
    - sight distance or signal operations;
    - emergency vehicle access and mobility;
    - freight truck maneuvers in the site vicinity; and,
    - presence of road maintenance or farm vehicles.

Land Use Characteristics

- Do the adjacent land uses lead to a high level of driveway turning movements onto and off of the roadway?

- Do the land uses attract vulnerable user groups (e.g., small children going to school, library or day-care; elderly people walking to and from a retirement center or retirement living facility; a playground or ball field where children may not be focused on the roadway)?
- Are adjacent land uses likely to attract a particular type of transportation mode, such as large trucks or bicycles?
- Do the adjacent land uses lead to a mix of users familiar with the area and others who may not be familiar with the area, such as tourists?

Public Comments

- What is the public perception of site conditions?
- Have comments been received about any specific safety concerns?

APPENDIX C – PREPARATION FOR CONDUCTING AN ASSESSMENT OF FIELD CONDITIONS

Select Participants

The field investigation is most successful when conducted from a multi-modal, multi-disciplinary perspective.(1) It is ideal to include experts in pedestrian, bicycle, transit, and motorized vehicle transportation, as well as law enforcement and emergency service representatives. A multi-modal, multi-disciplinary perspective may produce ideas and observations about the site that enhance the engineering observations and development of countermeasures. However, field investigations can also take place on a smaller scale where two or three people from a roadway agency are involved. In these instances, the individuals conducting the investigation can make an effort to keep multi-modal and multi-disciplinary perspectives in mind while evaluating and conducting the field investigation.

Advanced Coordination

The following activities are suggested to occur in advance of the field investigation in an effort to increase the effectiveness of the investigation:

- Team members review summaries of the crash analyses and site characteristics;
- The team members review a schedule and description of expected roles and outcomes from the investigation.
- A schedule is developed that identifies the number of field reviews and the time of day for each review. If possible, two field trips are useful: one during the day and another at night.

While in the field, the following tools may be useful:

- Still and/or video camera
- Stopwatch
- Safety vest and hardhat
- Measuring device
- Traffic counting board
- Spray paint
- Clipboards and notepads
- Weather protection
- Checklist for site investigation
- As-built design plans
- Summary notes of the site characteristics assessment
- Summary notes of the crash data analysis

APPENDIX D – FIELD REVIEW CHECKLIST

Roadway Segment

A roadway segment may include a portion of two-lane undivided, multi-lane undivided, or multi-lane divided highways in a rural, urban, or suburban area. Access may either be controlled (using grade-separated interchanges) or uncontrolled (via driveways or other access locations). Consideration of horizontal and vertical alignment and cross-sectional elements can help to determine possible accident contributory factors. The presence and location of auxiliary lanes, driveways, interchange ramps, signs, pavement marking delineation, roadway lighting, and roadside hardware is also valuable information. The prompt list below contains several prompts (not intended to be exhaustive) that could be used when performing field investigations on roadway segments: (2)

- Are there clear sight lines between the mainline road and side streets or driveways, or are there obstructions that may hinder visibility of conflicting flows of traffic?
- Does the available stopping sight distance meet local or national stopping sight distance criteria for the speed of traffic using the roadway segment? (See AASHTO's "A Policy on Geometric Design of Highways and Streets" or other guidance documents). Non-compliance and/or compliance does not directly relate to safe or unsafe conditions, though it can inform the diagnostic process.
- Is the horizontal and vertical alignment appropriate given the operating speeds on the roadway segment?
- Are passing opportunities adequate on the roadway segment?
- Are all through travel lanes and shoulders adequate based on the composition of traffic using the roadway segment?
- Does the roadway cross-slope adequately drain rainfall and snow runoff?
- Are auxiliary lanes properly located and designed?
- Are interchange entrance and exit ramps appropriately located and designed?
- Are median and roadside barriers properly installed?
- Is the median and roadside (right of traveled way) free from fixed objects and steep embankment slopes?
- Are bridge widths appropriate?
- Are drainage features within the clear zone traversable?
- Are sign and luminaire supports in the clear zone breakaway?
- Is roadway lighting appropriately installed and operating?
- Are traffic signs appropriately located and clearly visible to the driver?
- Is pavement marking delineation appropriate and effective?

- Is the pavement surface free of defects and does it have adequate skid resistance?
- Are parking provisions satisfactory?

Signalized Intersections

Examples of geometric and other signalized intersection characteristics that may prove valuable in determining a possible crash contributory factor at a signalized intersection include: the number of approach legs and their configuration, horizontal and vertical alignment design, cross-section elements, median type (if any), traffic signal phasing, parking locations, driveway access points, and any turn prohibitions. The signalized intersection safety prompt list provided below contains several examples of questions worthy of consideration when performing field investigations:

- Is appropriate sight distance available to all users on each intersection approach?
- Is the horizontal and vertical alignment appropriate on each approach leg?
- Are pavement markings and intersection control signing appropriate?
- Are all approach lanes adequately designed based on the composition of traffic using the intersection?
- Is the roadway cross-slope adequately draining rainfall and snow runoff?
- Is the median, curbs, and channelization layout appropriate?
- Are turning radii and tapers adequately designed based on the traffic composition using the intersection?
- Is roadway lighting appropriately installed and operating?
- Are traffic signs appropriately located and clearly visible to the driver on each approach leg?
- Is the pavement free of defects and is there adequate skid resistance?
- Are parking provisions satisfactory?
- Is traffic signal phasing appropriate for turning traffic on each approach?
- Are driveways and other access points appropriately located on each intersection approach leg?

Unsignalized Intersections

Unsignalized intersections may be stop or yield controlled or may not contain any control. Unsignalized intersections may contain three or more approach legs and different lane configurations on each leg. Data that may prove valuable in determining a possible crash contributory factor at an unsignalized intersection includes: the number of approach legs and their configuration, type of traffic control (none, yield, or stop), horizontal and vertical alignment design, cross-section elements, median type (if any), parking locations, driveway access points, and any turn prohibitions. The prompt list[2] provided below includes questions to consider when performing field investigations at unsignalized intersections:

- Is appropriate sight distance available to all users on each intersection approach?
- Is the horizontal and vertical alignment appropriate on each approach leg?
- Are pavement markings and intersection control signing appropriate?
- Are all approach lanes adequately designed based on the composition of traffic using the intersection?
- Is the roadway cross-slope adequately draining rainfall and snow runoff?
- Is the layout of the curbs and channelization appropriate?
- Are turning radius and tapers adequately designed based on the traffic composition using the intersection?
- Is roadway lighting appropriately installed and operating?
- Are traffic signs appropriately located and clearly visible to the driver on each approach leg?
- Is the pavement free of defects, and is there adequate skid resistance?
- Are parking provisions satisfactory?
- Are driveways and other access points appropriately located on each intersection approach leg?

Highway-Railroad Grade Crossings

Data that is valuable prior to determining a possible crash contributory factor at a highway-rail grade crossing includes:

- Sight distance on each approach and at the crossing itself;
- Existing pavement marking location and condition; and,
- Traffic control devices (i.e., advance warning signs, signals).

APPENDICES REFERENCES

1. Austroads. *Road Safety Audit*. Austroads 2nd Ed, 2002.
2. Kuhn, B. T., M. T. Pietrucha, and P. M. Garvey. *Development of a Safety Audit Process for Pennsylvania,* Report No. PTI 9702, Pennsylvania Transportation Institute, University Park, PA, August 1996.
3. PIARC Technical Committee on Road Safety (C13). *Road Safety Manual.* World Road Association, 2003.

PART B—ROADWAY SAFETY MANAGEMENT PROCESS

CHAPTER 6—SELECT COUNTERMEASURES

6.1. Introduction 6-1

6.2. Identifying Contributing Factors 6-2

6.2.1. Perspectives to Consider when Evaluating Contributing Factors 6-2

6.2.2. Contributing Factors for Consideration 6-3

6.3. Select Potential Countermeasures 6-9

6.4. Summary of Countermeasure Selection 6-10

6.5. Sample Problems 6-11

6.6. References 6-14

EXHIBITS

Exhibit 6–1: Roadway Safety Management Process Overview 6-1
Exhibit 6–2: Example Haddon Matrix for Rear-End Crash...6-2
Exhibit 6–3: Possible Crash Contributing Factors along Roadway Segments................6-5
Exhibit 6–4: Possible Crash Contributing Factors at Signalized Intersections 6-6
Exhibit 6–5: Possible Crash Contributing Factors at Unsignalized Intersections 6-7
Exhibit 6–6: Possible Crash Contributing Factors along Highway-Rail Grade Crossings...6-8
Exhibit 6–7: Possible Crash Contributing Factors Involving Pedestrians 6-8
Exhibit 6–8: Possible Crash Contributing Factors Involving Bicyclists 6-9
Exhibit 6-9: Assessment Summary 6-11

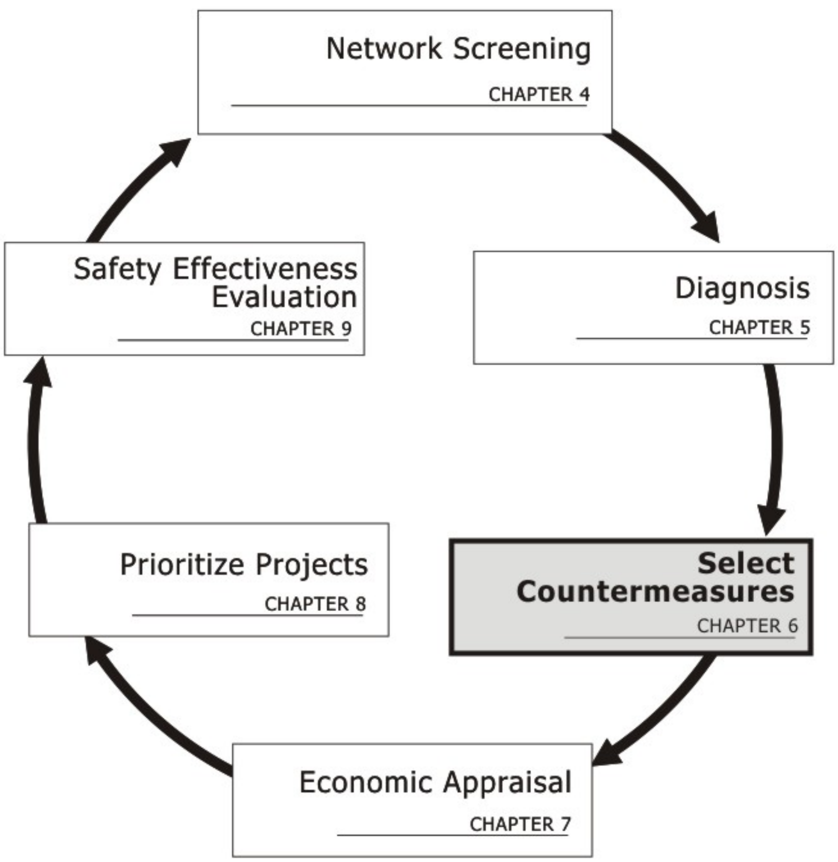
CHAPTER 6 SELECT COUNTERMEASURES

6.1. INTRODUCTION

This chapter outlines the third step in the roadway safety management process: selecting countermeasures to reduce crash frequency or severity at specific sites. The entire roadway safety management process is shown in Exhibit 6-1. In the context of this chapter, a countermeasure is a roadway strategy intended to decrease crash frequency or severity, or both at a site. Prior to selecting countermeasures, crash data and site supporting documentation are analyzed and a field review is conducted, as described in *Chapter 5*, to diagnose the characteristics of each site and identify crash patterns. In this chapter the sites are further evaluated to identify factors that may be contributing to observed crash patterns or concerns and countermeasures are selected to address the respective contributing factors. The selected countermeasures are subsequently evaluated from an economic perspective as described in *Chapter 7*.

Chapter 6 provides information about identifying contributing factors and selecting countermeasures.

Exhibit 6–1: Roadway Safety Management Process Overview



Vehicle- or driver-based countermeasures are not covered explicitly in this edition of the HSM. Examples of vehicle-based countermeasures include occupant restraint systems and in-vehicle technologies. Examples of driver-based countermeasures include educational programs, targeted enforcement, and graduated driver licensing. The following documents provide information about driver- and vehicle-based countermeasures:

- The National Cooperative Highway Research Program (NCHRP) *Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan;*(7) and,
- The National Highway Traffic Safety Administration's (NHTSA) report *Countermeasures that Work: A Highway Safety Countermeasure Guide for State Highway Safety Offices.*(3)

6.2. IDENTIFYING CONTRIBUTING FACTORS

For each identified crash pattern there may be multiple contributing factors. The following sections provide information to assist with development of a comprehensive list of possible crash contributing factors. The intent is to assist in identification of a broad range of possible contributing factors in order to minimize the probability that a major contributing factor will be overlooked.

Once a broad range of contributing factors have been considered, engineering judgment is applied to identify those factors that are expected to be the greatest contributors to each particular crash type or concern. The information obtained as part of the diagnosis process (*Chapter 5*) will be the primary basis for such decisions.

Contributing factors can be divided into three categories: human, vehicle, and roadway.

6.2.1. Perspectives to Consider when Evaluating Contributing Factors

A useful framework for identifying crash contributing factors is the Haddon Matrix.(2) In the Haddon Matrix, the crash contributing factors are divided into three categories: human, vehicle, and roadway. The possible crash conditions before, during, and after a crash are related to each crash contributing factor category to identify possible reasons for the crash. An example of a Haddon Matrix prepared for a rear-end crash is shown in Exhibit 6-2. Additional details on the Haddon Matrix are provided in *Chapter 3*.

Exhibit 6–2: Example Haddon Matrix for Rear-End Crash

Period	Human Factors	Vehicle Factors	Roadway Factors
Before the Crash (Causes of the hazardous situation)	distraction, fatigue, inattention, bad judgment, age, cell phone use, impaired cognitive skills, deficient driving habits	bald tires, worn brakes	wet pavement, polished aggregate, steep downgrade, poor signal coordination, limited stopping sight distance, lack of warning signs
During the Crash (Causes of crash severity)	vulnerability to injury, age, failure to wear a seat belt	bumper heights and energy absorption, headrest design, airbag operations	pavement friction and grade
After the Crash (Factors of crash outcome)	age, gender	ease of removal of injured passengers	the time and quality of the emergency response, subsequent medical treatment

The engineering perspective considers items like crash data, supporting documentation, and field conditions in the context of identifying potential engineering solutions to reduce crash frequency or severity. Evaluation of contributing factors from an engineering perspective may include comparing field conditions to various national and local jurisdictional design guidelines related to signing, striping, geometric design, traffic control devices, roadway classifications, work zones, etc. In reviewing these guidelines, if a design anomaly is identified it may provide a clue to the crash contributing factors. However, it is important to emphasize that consistency with design guidelines does not correlate directly to a safe roadway system; vehicles are driven by humans who are dynamic beings with varied capacity to perform the driving task.

When considering human factors in the context of contributing factors, the goal is to understand the human contributions to the cause of the crash in order to propose solutions that might break the chain of events that led to the crash. The consideration of human factors involves developing fundamental knowledge and principles about how people interact with a roadway system so that roadway system design matches human strengths and weaknesses. The study of human factors is a separate technical field. An overview discussion of human factors is provided in *Chapter 2* of the manual. Several fundamental principles essential to understanding the human factors aspects of the roadway safety management process include:

- Attention and information processing: Drivers can only process limited information and often rely on past experience to manage the amount of new information they must process while driving. Drivers can process information best when it is presented in accordance with expectations; sequentially to maintain a consistent level of demand; and, in a way that helps drivers prioritize the most essential information.

- Vision: Approximately 90% of the information a driver uses is obtained visually.[4] Given that driver visual abilities vary considerably, it is important that the information be presented in a way that users can see, comprehend, and respond to appropriately. Examples of actions that help account for driver vision capabilities include: designing and locating signs and markings appropriately; ensuring that traffic control devices are conspicuous and redundant (e.g., stops signs with red backing and words that signify the desired message); providing advanced warning of roadway hazards; and removing obstructions for adequate sight distance.

- Perception-reaction time: The time and distance needed by a driver to respond to a stimulus (e.g., hazard in road, traffic control device, or guide sign) depends on human elements, including information processing, driver alertness, driver expectations, and vision.

- Speed choice: Each driver uses perceptual and road message cues to determine a travel speed. Information taken in through peripheral vision may lead drivers to speed up or slow down depending on the distance from the vehicle to the roadside objects. Other roadway elements that impact speed choice include roadway geometry and terrain.

Chapter 2 provides an overview of Human Factors.

6.2.2. Contributing Factors for Consideration

Examples of contributing factors associated with a variety of crash types are provided in the following sections. The examples may serve as a checklist to verify that a key contributing factor is not forgotten or overlooked. Many of the specific types of highway crashes or contributing factors are discussed in detail in *NCHRP Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan*, a series of concise documents that were developed to assist state and local agencies in reducing injuries and fatalities in targeted emphasis areas. [1,5,6,8-15]

Section 6.2.2 provides a summary of different crash types and potential contributing factors.

The possible crash contributing factors listed in the following sections are not and can never be a comprehensive list. Each site and crash history are unique and identification of crash contributing factors is can be completed by careful consideration of all the facts gathered during a diagnosis process similar to that described in *Chapter 5*.

Crashes on Roadway Segments

Exhibit 6-3 outlines common crash types and multiple potential contributing factors for crashes on roadway segments. It is important to note that some of the possible contributing factors shown for various crash types in Exhibit 6-3 may overlap, and that there are additional contributing factors that could be identified through the diagnosis process. For example, fixed object crashes may be the result of multiple contributing factors such as excessive speeds on sharp horizontal curves with inadequate signing.

113 **Exhibit 6–3: Possible Crash Contributing Factors along Roadway Segments**

Crash Type	Possible Contributing Factor(s)
Vehicle rollover	Roadside design (e.g., non-traversable side slopes, pavement edge drop off)
	Inadequate shoulder width
	Excessive speed
	Pavement design
Fixed object	Obstruction in or near roadway
	Inadequate lighting
	Inadequate pavement markings
	Inadequate signs, delineators, guardrail
	Slippery pavement
	Roadside design (e.g., inadequate clear distance)
	Inadequate roadway geometry
	Excessive speed
Nighttime	Poor nighttime visibility or lighting
	Poor sign visibility
	Inadequate channelization or delineation
	Excessive speed
	Inadequate sight distance
Wet Pavement	Pavement design (e.g., drainage, permeability)
	Inadequate pavement markings
	Inadequate maintenance
	Excessive speed
Opposite-direction Sideswipe or Head-on	Inadequate roadway geometry
	Inadequate shoulders
	Excessive speed
	Inadequate pavement markings
	Inadequate signing
Run-off-road	Inadequate lane width
	Slippery pavement
	Inadequate median width
	Inadequate maintenance
	Inadequate roadway shoulders
	Poor delineation
	Poor visibility
	Excessive speed
Bridges	Alignment
	Narrow roadway
	Visibility
	Vertical clearance
	Slippery pavement
	Rough surface
	Inadequate barrier system

Crashes at Signalized Intersections

Exhibit 6-4 shows common crash types that occur at signalized intersections and contributing factors for each type. The crash types considered include: right-angle; rear-end or sideswipe; left- or right-turn; nighttime; and wet pavement crashes. The possible contributing factors shown may overlap with various crash types. This is not intended to be a comprehensive list of all crash types and contributing factors.

Exhibit 6–4: Possible Crash Contributing Factors at Signalized Intersections

Crash Type	Possible Contributing Factor(s)
Right-angle	Poor visibility of signals
	Inadequate signal timing
	Excessive speed
	Slippery pavement
	Inadequate sight distance
	Drivers running red light
Rear-end or Sideswipe	Inappropriate approach speeds
	Poor visibility of signals
	Unexpected lane changes on approach
	Narrow lanes
	Unexpected stops on approach
	Slippery pavement
	Excessive speed
Left- or right-turn movement	Misjudge speed of on-coming traffic
	Pedestrian or bicycle conflicts
	Inadequate signal timing
	Inadequate sight distance
	Conflict with right-turn-on-red vehicles
Nighttime	Poor nighttime visibility or lighting
	Poor sign visibility
	Inadequate channelization or delineation
	Inadequate maintenance
	Excessive speed
	Inadequate sight distance
Wet Pavement	Slippery pavement
	Inadequate pavement markings
	Inadequate maintenance
	Excessive speed

Crashes at Unsignalized Intersections

Exhibit 6-5 shows common crash types that occur at unsignalized intersections along with possible contributing factor(s) for each type. The crash types include: angle; rear-end; collision at driveways; head-on or sideswipe; left- or right-turn; nighttime; and wet pavement crashes. This is not intended to be a comprehensive list of all crash types and contributing factors.

Exhibit 6–5: Possible Crash Contributing Factors at Unsignalized Intersections

Crash Type	Possible Contributing Factor(s)
Angle	Restricted sight distance
	High traffic volume
	High approach speed
	Unexpected crossing traffic
	Drivers running "stop" sign
	Slippery pavement
Rear-end	Pedestrian crossing
	Driver inattention
	Slippery pavement
	Large number of turning vehicles
	Unexpected lane change
	Narrow lanes
	Restricted sight distance
	Inadequate gaps in traffic
	Excessive speed
Collisions at driveways	Left-turning vehicles
	Improperly located driveway
	Right-turning vehicles
	Large volume of through traffic
	Large volume of driveway traffic
	Restricted sight distance
	Excessive speed
Head-on or sideswipe	Inadequate pavement markings
	Narrow lanes
Left- or right-turn	Inadequate gaps in traffic
	Restricted sight distance
Nighttime	Poor nighttime visibility or lighting
	Poor sign visibility
	Inadequate channelization or delineation
	Excessive speed
	Inadequate sight distance
Wet pavement	Slippery pavement
	Inadequate pavement markings
	Inadequate maintenance
	Excessive speed

Crashes at Highway-Rail Grade Crossings

Exhibit 6-6 lists common crash types that occur at highway-rail grade crossings and possible contributing factors associated with each type. This is not intended to be a comprehensive list of all crash types and contributing factors.

Exhibit 6–6: Possible Crash Contributing Factors along Highway-Rail Grade Crossings

Crash Type	Possible Contributing Factor(s)
Collision at highway-rail grade crossings	Restricted sight distance
	Poor visibility of traffic control devices
	Inadequate pavement markings
	Rough or wet crossing surface
	Sharp crossing angle
	Improper pre-emption timing
	Excessive speed
	Drivers performing impatient maneuvers

Crashes Involving Bicyclists and Pedestrians

Common crash types and possible contributing factors in pedestrian crashes are shown in Exhibit 6-7, while possible contributing factors in bicycle crashes are shown in Exhibit 6-8. These are not intended to be comprehensive lists of all crash types and contributing factors.

Exhibit 6–7: Possible Crash Contributing Factors Involving Pedestrians

Crash Type	Possible Contributing Factor(s)
Motor vehicle-pedestrian	Limited sight distance
	Inadequate barrier between pedestrian and vehicle facilities
	Inadequate signals/signs
	Inadequate signal phasing
	Inadequate pavement markings
	Inadequate lighting
	Driver has inadequate warning of mid-block crossings
	Lack of crossing opportunity
	Excessive speed
	Pedestrians on roadway
	Long distance to nearest crosswalk
	Sidewalk too close to travel way
	School crossing area

Exhibit 6–8: Possible Crash Contributing Factors Involving Bicyclists

Crash Type	Possible Contributing Factor(s)
Motor vehicle-bicyclist	Limited sight distance
	Inadequate signs
	Inadequate pavement markings
	Inadequate lighting
	Excessive speed
	Bicycles on roadway
	Bicycle path too close to roadway
	Narrow lanes for bicyclists

6.3. SELECT POTENTIAL COUNTERMEASURES

There are three main steps to selecting a countermeasure(s) for a site:

1. Identify factors contributing to the cause of crashes at the subject site;
2. Identify countermeasures which may address the contributing factors; and,
3. Conduct cost-benefit analysis, if possible, to select preferred treatment(s) (*Chapter 7*).

The material in Section 6.2 and *Chapter 3* provide an overview of a framework for identifying potential contributing factors at a site. Countermeasures (also known as treatments) to address the contributing factors are developed by reviewing the field information, crash data, supporting documentation, and potential contributing factors to develop theories about the potential engineering, education, or enforcement treatments that may address the contributing factor under consideration.

Part D of the HSM presents information regarding the effects of various countermeasures that can be used to estimate the effectiveness of a countermeasure in reducing crashes at a specific location.

Comparing contributing factors to potential countermeasures requires engineering judgment and local knowledge. Consideration is given to issues like 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 example, if at a signalized intersection it is expected that limited sight-distance is the contributing factor to the rear-end crashes, then the possible reasons for the limited sight distance conditions are identified. Examples of possible causes of limited sight distance might include: constrained horizontal or vertical curvature, landscaping hanging low on the street, or illumination conditions.

A variety of countermeasures could be considered to resolve each of these potential reasons for limited sight distance. The roadway could be re-graded or re-aligned to eliminate the sight distance constraint or landscaping could be modified. These various actions are identified as the potential treatments.

Part D of the HSM is a resource for treatments with quantitative accident modification factors (AMFs). The AMFs represent the estimated change in crash frequency with implementation of the treatment under consideration. An AMF value of less than 1.0 indicates that the predicted average crash frequency will be lower with implementation of the countermeasure. For example, changing the traffic control of an urban intersection from a two-way, stop-controlled intersection to a modern roundabout has an AMF of 0.61 for all collision types and crash severities. This indicates that the expected average crash frequency will decrease by 39 percent after converting the intersection control. Application of an AMF will provide an estimate of the change in crashes due to a treatment. There will be variance in results at any particular location. Some countermeasures may have different effects on different crash types or severities. For example, installing a traffic signal in a rural environment at a previously unsignalized two-way stop-controlled intersection has an AMF of 1.58 for rear-end crashes and an AMF of 0.40 for left-turn crashes. The AMFs suggest that an increase in rear-end crashes may occur while a reduction in left-turn crashes may occur.

If an AMF is not available, *Part D* of the HSM also provides information about the trends in crash frequency related to implementation of such treatments. Although not quantitative and therefore not sufficient for a cost-benefit or cost-effectiveness analysis (*Chapter 7*), information about a trend in the change in crashes at a minimum provides guidance about the resulting crash frequency. Finally, accident modification factors for treatments can be derived locally using procedures outline in *Chapter 9* of the HSM.

In some cases a specific contributing factor and/or associated treatment may not be easily identifiable, even when there is a prominent crash pattern or concern at the site. In these cases, conditions upstream or downstream of the site can also be evaluated to determine if there is any influence at the site under consideration. Also, the site is evaluated for conditions which are not consistent with the typical driving environment in the community. Systematic improvements such as: guide signage, traffic signals with mast-arms instead of span-wire, or changes in signal phasing may influence the overall driving environment. Human factors issues may also be influencing driving patterns. Finally, the site can be monitored in the event that conditions may change and potential solutions become evident.

6.4. SUMMARY OF COUNTERMEASURE SELECTION

Chapter 6 provides examples of crash types and possible contributing factors as well as a framework for selecting counter measures.

This chapter outlined the process for selecting countermeasures based on conclusions of a diagnosis of each site (*Chapter 5*). The site diagnosis is intended to identify any patterns or trends in the data and provide comprehensive knowledge of the sites, which can prove valuable in selecting countermeasures.

Several lists of contributing factors are provided in Section 6.2. Connecting the contributing factor to potential countermeasures requires engineering judgment and local knowledge. Consideration is given to 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 are identified that are expected to address the crash pattern or collision type. *Part D* information provides estimates of the change in expected average crash frequency for various countermeasures. If an AMF is not available, in some cases *Part D* of the HSM also provides information about the trends in crash frequency or user behavior related to implementation of some treatments.

When a countermeasure or combination of countermeasures is selected for a specific location, an economic appraisal of all sites under consideration is performed to help prioritize network improvements. *Chapters 7* and *Chapter 8* provide guidance on conducting economic evaluations and prioritizing system improvements.

6.5. SAMPLE PROBLEMS

The Situation

Upon conducting network screening (*Chapter 4*) and diagnostic procedures (*Chapter 5*), a roadway agency has completed a detailed investigation at Intersection 2 and Segment 1. A solid understanding of site characteristics, history, and layout has been acquired so that possible contributing factors can be identified. A summary of the basic findings of the diagnosis is shown in Exhibit 6-9.

Exhibit 6-9: Assessment Summary

Data	Intersection 2	Segment 1
Major / Minor AADT	22,100 / 1,650	9,000
Traffic Control / Facility Type	Two-way stop	Undivided Roadway
Predominant Crash Types	Angle, Head-On	Roll-Over, Fixed Object
Crashes by Severity		
Fatal	6%	6%
Injury	73%	32%
PDO	21%	62%

The Question

What factors are likely contributing to the target crash types identified for each site? What are appropriate countermeasures that have potential to reduce the target crash types?

The Facts

Intersections

- Three years of intersection crash data as shown in *Chapter 5,* Exhibit 5-7.
- All study intersections have four approaches and are located in urban environments.

Roadway Segments

- Three years of roadway segment crash data as shown in *Chapter 5,* Exhibit 5-7.
- The roadway cross-section and length as shown in *Chapter 5*, Exhibit 5-7.

Solution

The countermeasure selection for Intersection 2 is presented, followed by the countermeasure selection for Segment 1. The countermeasures selected will be economically evaluated using economic appraisal methods outlined in *Chapter 7*.

Intersection 2

Exhibit 6-5 identifies possible crash contributing factors at unsignalized intersections by accident type. As shown in the exhibit, possible contributing factors for angle collisions include: restricted sight distance, high traffic volume, high approach speed, unexpected crossing traffic, drivers ignoring traffic control on stop-controlled approaches, and wet pavement surface. Possible contributing factors for head-on collisions include: inadequate pavement markings and narrow lanes.

A review of documented site characteristics indicates that over the past several years the traffic volumes on both the minor and major roadways have increased. An existing conditions traffic operations analysis during the weekday p.m. peak hour indicates an average delay of 115 seconds for vehicles on the minor street and 92 seconds for left-turning vehicles turning from the major street onto the minor street. In addition to the long delay experienced on the minor street, the operations analysis calculated queue lengths as long as 11 vehicles on the minor street.

A field assessment of Intersection 2 confirmed the operations analysis results. It also revealed that because of the traffic flow condition on the major street, very few gaps are available for vehicles traveling to or from the minor street. Sight distances on all four approaches were measured and met local and national guidelines. During the off-peak field assessment, the vehicle speed on the major street was observed to be substantially higher than the posted speed limit and inappropriate for the desired character of the roadway.

The primary contributing factors for the angle collisions were identified as increasing traffic volumes during the peak periods, providing few adequate gaps for vehicles traveling to and from the minor street. As a result, motorists have become increasingly willing to accept smaller gaps, resulting in conflicts and contributing to collisions. Vehicles travel at high speeds on the major street during off-peak periods when traffic volumes are lower; the higher speeds result in a larger speed differential between vehicles turning onto the major street from the minor street. The larger speed differential creates conflicts and contributes to collisions.

Chapter 14 of *Part D* includes information on the crash reduction effects of various countermeasures. Reviewing the many countermeasures provided in *Chapter 14* and considering other known options for modifying intersections, the following countermeasures were identified as having potential for reducing the angle crashes at Intersection 2:

- Convert stop-controlled intersection to modern roundabout
- Convert two-way stop-controlled intersection to all-way stop control
- Provide exclusive left-turn lane on one or more approaches

The following countermeasures were identified as having potential for reducing the head-on crashes at Intersection 2:

- Increasing intersection median width
- Convert stop-controlled intersection to modern roundabout
- Increase lane width for through travel lanes

The potential countermeasures were evaluated based on the supporting information known about the sites and the AMFs provided in *Part D*. Of the three potential countermeasures identified as the most likely to reduce target crashes, the only one that was determined to be able to serve the forecast traffic demand was the modern roundabout option. Additionally the AMFs provided in *Part D* provide support that the roundabout option can be expected to reduce the average crash frequency. Constructing exclusive left-turn lanes on the major approaches would likely reduce the number of conflicts between through traffic and turning traffic, but was not expected to mitigate the need for adequate gaps in major street traffic. Therefore, the roadway agency selected a roundabout as the most appropriate countermeasure to implement at Intersection 2. Further analysis, as outlined in *Chapters 7, 8,* and *9*, is suggested to determine the priority of implementing this countermeasure at this site.

Segment 1

Segment 1 is an undivided two-lane rural highway; the segment end points are defined by intersections. The crash summary statistics in *Chapter 5* indicate that approximately three-quarters of the crashes on the road segment in the last three years involved vehicles running off of the road resulting in either a fixed object crash or roll-over crash. The statistics and crash reports do not show a strong correlation between the run-off-the-road crashes and lighting conditions.

Exhibit 6-3 summarizes possible contributing factors for roll-over and run-off-road crashes. Possible contributing factors include low-friction pavement, inadequate roadway geometric design, inadequate maintenance, inadequate roadway shoulders, inadequate roadside design, poor delineation, and poor visibility.

A detailed review of documented site characteristics and a field assessment indicated that the roadway is built to the agency's standards and is included in its maintenance cycle. Past speed studies and observations made by the roadway agency's engineers indicate that vehicle speeds on the rural two-lane roadway often exceed the posted speed limit by 5 to 15 mph. Given the location of the segment, local agency staff expects that the majority of the trips that use this segment have a total trip length of less than 10 miles. Sight distance and delineation also were assessed to be within reason.

Potential countermeasures that the agency could implement were identified to include: increasing the lane and/or shoulder width, removing or relocating any fixed objects within the clear zone, flattening the sideslope, adding delineation or replacing existing lane striping with retro-reflective material, and adding shoulder rumble strips.

The potential countermeasures were evaluated based on the supporting information known about the site and the AMFs provided in *Part D*. Given that the roadway segment is located between two intersections and they know that most users of the facility are making trips of a total length of less than 10 miles, it is not expected that drivers are becoming drowsy or not paying attention. Therefore adding rumble strips or delineation to alert drivers of the roadway boundaries is not expected to be effective.

The agency believes that increasing the forgiveness of the shoulder and clear zone will be the most effective countermeasure for reducing fixed-object or roll-over crashes. Specifically they suggest flattening the sideslope in order to improve the ability of errant drivers to correct without causing a roll-over crash. The agency will also consider protecting or removing objects within a specified distance from the edge of roadway. The agency will consider the economic feasibility of these improvements on this segment and prioritize among other projects in their jurisdiction using methods in *Chapters 7* and *Chapter 8*.

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PART B — ROADWAY SAFETY MANAGEMENT PROCESS

CHAPTER 7—ECONOMIC APPRAISAL

7.1. Introduction 7-1
7.2. Overview of Project Benefits and Costs 7-2
7.3. Data Needs....7-3
7.4. Assess Expected Project Benefits 7-3
7.4.1. Estimating Change in Crashes for a Proposed Project 7-4
7.4.2. Estimating a Change in Crashes When No Safety Prediction Methodology or AMF is Available 7-4
7.4.3. Converting Benefits to a Monetary Value 7-5
7.5. Estimate Project Costs....7-7
7.6. Economic Evaluation Methods for Individual Sites 7-8
7.6.1. Procedures for Benefit-Cost Analysis 7-9
7.6.2. Procedures for Cost-Effectiveness Analysis 7-11
7.7. Non-Monetary Considerations 7-12
7.8. Conclusions 7-13
7.9. Sample Problem....7-14
7.9.1. Economic Appraisal 7-14
7.10. References 7-23

EXHIBITS

Exhibit 7-1: Roadway Safety Management Process Overview 7-1
Exhibit 7-2: Economic Appraisal Process 7-2
Exhibit 7-3: Data Needs for Calculating Project Benefits 7-3
Exhibit 7-4: Crash Cost Estimates by Crash Severity 7-5
Exhibit 7-5: Strengths and Limitations of NPV Analysis 7-10
Exhibit 7-6: Strengths and Limitations of BCR Analysis 7-11
Exhibit 7-7: Strengths and Limitations of Cost-Effectiveness Analysis 7-12
Exhibit 7-8: Summary of Crash Conditions, Contributory Factors, and Selected Countermeasures 7-14
Exhibit 7-9: Expected Average Crash Frequency at Intersection 2 WITHOUT Installing the Roundabout 7-15
Exhibit 7-10: Societal Crash Costs by Severity 7-16
Exhibit 7-11: Remaining Assumptions 7-16
Exhibit 7-12: Economic Appraisal for Intersection 2 7-17
Exhibit 7-13: Expected Average FI Crash Frequency at Intersection 2 WITH the Roundabout 7-18
Exhibit 7-14: Expected Average Total Crash Frequency at Intersection 2 WITH the Roundabout 7-19
Exhibit 7-15: Change in Expected Average in Crash Frequency at Intersection 2 WITH the Roundabout 7-20
Exhibit 7-16: Annual Monetary Value of Change in Crashes 7-21
Exhibit 7-17: Converting Annual Values to Present Values 7-22

APPENDIX A

A.1 Data Needs to Calculate Change in Crashes 7-24

A.2 Service Life of the Improvement Specific to the Countermeasure 7-25

A.3 Discount Rate 7-25

A.4 Data Needs to Calculate Project Costs 7-26

A.5 Appendix References 7-27

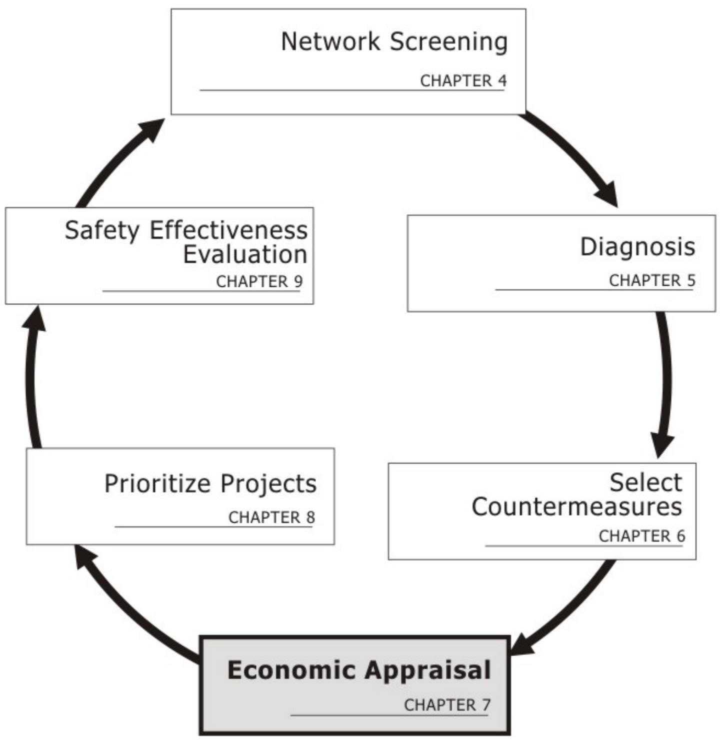
CHAPTER 7 ECONOMIC APPRAISAL

7.1. INTRODUCTION

Economic appraisals are performed to compare the benefits of potential crash countermeasure to its project costs. Site economic appraisals are conducted after the highway network is screened (*Chapter 4*), the selected sites are diagnosed (*Chapter 5*), and potential countermeasures for reducing crash frequency or crash severity are selected (*Chapter 6*). Exhibit 7-1 shows this step in the context of the overall roadway safety management process.

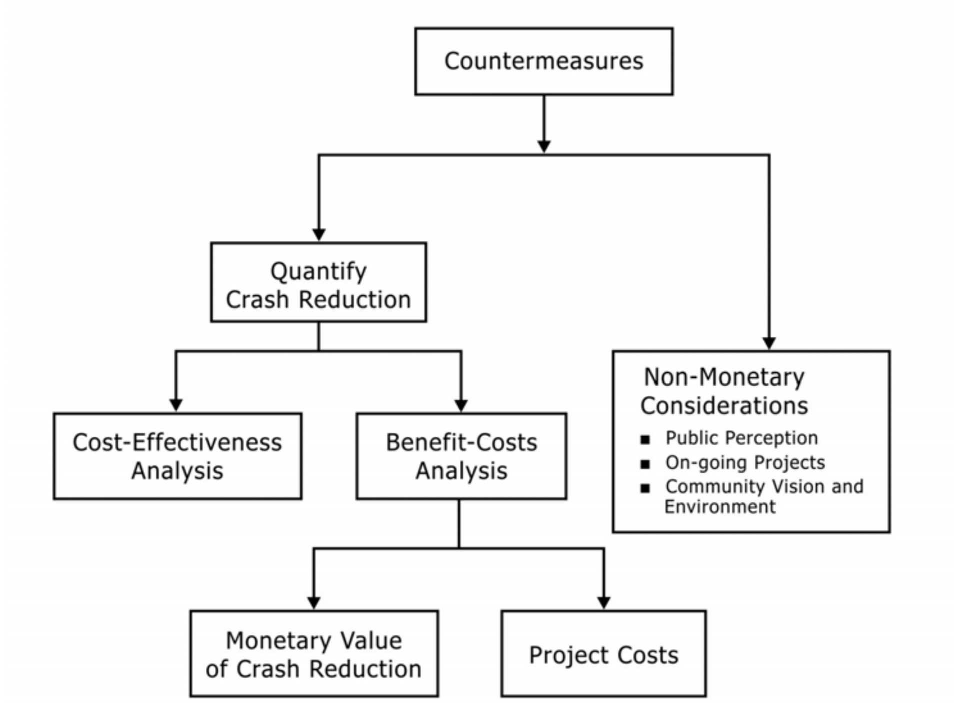
Economic appraisals are used to estimate the monetary benefit of safety improvements.

Exhibit 7-1: Roadway Safety Management Process Overview



In an economic appraisal, project costs are addressed in monetary terms. Two types of economic appraisal – benefit-cost analysis and cost-effectiveness analysis – address project benefits in different ways. Both types begin quantifying the benefits of a proposed project, expressed as the estimated change in crash frequency or severity of crashes, as a result of implementing a countermeasure. In benefit-cost analysis, the expected change in average crash frequency or severity is converted to monetary values, summed, and compared to the cost of implementing the countermeasure. In cost-effectiveness analysis, the change in crash frequency is compared directly to the cost of implementing the countermeasure. This chapter also presents methods for estimating benefits if the expected change in crashes is unknown. Exhibit 7-2 provides a schematic of the economic appraisal process.

Exhibit 7-2: Economic Appraisal Process



As an outcome of the economic appraisal process, the countermeasures for a given site can be organized in descending or ascending order by the following characteristics:

- Project costs
- Monetary value of project benefits
- Number of total crashes reduced
- Number of fatal and incapacitating injury crashes reduced
- Number of fatal and injury crashes reduced
- Net Present Value (NPV)
- Benefit-Cost Ratio (BCR)
- Cost-Effectiveness Index

Ranking alternatives for a given site by these characteristics can assist highway agencies in selecting the most appropriate alternative for implementation.

7.2. OVERVIEW OF PROJECT BENEFITS AND COSTS

In addition to project benefits associated with a change in crash frequency, project benefits such as travel time, environmental impacts, and congestion relief are also considerations in project evaluation. However, the project benefits discussed in *Chapter 7* relate only to changes in crash frequency. Guidance for considering other project benefits, such as travel-time savings and reduced fuel consumption, are found in the American Association of State Highway and Transportation Officials (AASHTO) publication entitled *A Manual of User Benefit Analysis for Highways* (also known as the AASHTO Redbook).[(1)]

The HSM predictive method presented in *Part C* provides a reliable method for estimating the change in expected average crash frequency due to a countermeasure. After applying the *Part C* predictive method to determine expected average crash frequency for existing conditions and proposed alternatives, the expected change in average fatal and injury crash frequency is converted to a monetary value using the societal cost of crashes. Similarly, the expected change in property damage only (PDO) crashes (change in total crashes minus the change in fatal and injury crashes) is converted to a monetary value using the societal cost of a PDO collision. Additional methods for estimating a change in crash frequency are also described in this chapter, although it is important to recognize the results of those methods are not expected to be as accurate as the *Part C* predictive method.

Part C presents methods to estimate a change in the average crash frequency at a site.

7.3. DATA NEEDS

The data needed to calculate the change in crash frequency and countermeasure implementation costs are summarized in Exhibit 7-3. Appendix A includes a detailed explanation of the data needs.

Exhibit 7-3: Data Needs for Calculating Project Benefits

Activity	Data Need
Calculate Monetary Benefit	
Estimate change in crashes by severity	Crash history by severity
	Current and future Average Annual Daily Traffic (AADT) volumes
	Implementation year for expected countermeasure
	SPF for current and future site conditions (if necessary)
	AMFs for all countermeasures under consideration
Convert change in crash frequency to annual monetary value	Monetary value of crashes by severity
	Change in crash frequency estimates
Convert annual monetary value to a present value	Service life of the countermeasure
	Discount rate (minimum rate of return)
Calculate Costs	
Calculate construction and other implementation costs	Subject to standards for the jurisdiction
Convert costs to present value	Service life of the countermeasure(s)
	Project phasing schedule

7.4. ASSESS EXPECTED PROJECT BENEFITS

This section outlines the methods for estimating the benefits of a proposed project based on the estimated change in average crash frequency. The method used will depend on the facility type and countermeasures, and the amount of research that has been conducted on such facilities and countermeasures. The HSM's suggested method for determining project benefits is to apply the predictive method presented in *Part C.*

Section 7.4.1 reviews the applicable methods for estimating a change in average crash frequency for a proposed project. The discussion in Section 7.4.1 is consistent with the guidance provided in the *Part C Introduction and Applications Guidance.* Section 7.4.2 describes how to estimate the change in expected average crash frequency when none of the methods outlined in Section 7.4.1 can be applied. Section 7.4.3 describes how to convert the expected change in average crash frequency into a monetary value.

7.4.1. Estimating Change in Crashes for a Proposed Project

The Part C Introduction and Applications Guidance provides detailed information about the HSM predictive method, SPFs, and AMFs.

The *Part C Predictive Method* provides procedures to estimate the expected average crash frequency when geometric design and traffic control features are specified. This section provides four methods, in order of reliability for estimating the change in expected average crash frequency of a proposed project or project design alterative. These are:

- Method 1 – Apply the *Part C* predictive method to estimate the expected average crash frequency of both the existing and proposed conditions.
- Method 2 – Apply the *Part C* predictive method to estimate the expected average crash frequency of the existing condition and apply an appropriate project AMF from *Part D* to estimate the safety performance of the proposed condition.
- Method 3 – If the *Part C* predictive method is not available, but a Safety Performance Function (SPF) applicable to the existing roadway condition is available (i.e., a SPF developed for a facility type that is not included in *Part C*), use that SPF to estimate the expected average crash frequency of the existing condition and apply an appropriate project AMF from *Part D* to estimate the expected average crash frequency of the proposed condition. A locally-derived project AMF can also be used in Method 3.
- Method 4 – Use observed crash frequency to estimate the expected average crash frequency of the existing condition, and apply an appropriate project AMF from *Part D* to the estimated expected average crash frequency of the existing condition to obtain the estimated expected average crash frequency for the proposed condition. This method is applied to facility types with existing conditions not addressed by the *Part C* predictive method.

When an AMF from *Part D* is used in one of the four methods, the associated standard error of the AMF can be applied to develop a confidence interval around the expected average crash frequency estimate. The range will help to see what type of variation could be expected when implementing a countermeasure.

7.4.2. Estimating a Change in Crashes When No Safety Prediction Methodology or AMF is Available

Section 7.4.1 explains that estimating the expected change in crashes for a countermeasure can be accomplished with the *Part C* predictive method, the *Part D* AMFs, or with locally developed AMFs. When there is no applicable *Part C* predictive method, no applicable SPF, and no applicable AMF, the HSM procedures cannot provide an estimate of the expected project effectiveness.

In order to evaluate countermeasures when no valid AMF is available, an estimate of the applicable AMF may be chosen using engineering judgment. The results of such analysis are considered uncertain and a sensitivity analysis based on a range of AMF estimates could support decision making.

7.4.3. Converting Benefits to a Monetary Value

Converting the estimated change in crash frequency to a monetary value is relatively simple as long as established societal crash costs by severity are available. First the estimated change in crash frequency is converted to an annual monetary value. This annual monetary value may or may not be uniform over the service life of the project. Therefore, in order to obtain a consistent unit for comparison between sites, the annual value is converted to a present value.

7.4.3.1. Calculate Annual Monetary Value

The following data is needed to calculate annual monetary value:

- Accepted monetary value of crashes by severity
- Change in crash estimates for:
  - Total Crashes
  - Fatal/Injury Crashes
  - PDO Crashes

In order to develop an annual monetary value the societal cost associated with each crash severity is multiplied by the corresponding annual estimate of the change in crash frequency. State and local jurisdictions often have accepted crash costs by crash severity and collision type. When available, these locally-developed crash cost data are used with procedures in the HSM. If local information is not available, nationwide crash cost data is available from the Federal Highway Administration (FHWA). This edition of the HSM applies crash costs from the FHWA report *Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries*.(2) The costs cited in this 2005 report are presented in 2001 dollars. The *Chapter 4* appendix includes a summary of a procedure for updating annual monetary values to current year values. Exhibit 7-4 summarizes the relevant information for use in the HSM (rounded to the nearest hundred dollars).

The Chapter 4, *Appendix A* includes a summary of the recommended procedure for updating annual monetary values to current year values.

Exhibit 7-4: Crash Cost Estimates by Crash Severity

Collision Type	Comprehensive Crash Costs
Fatality (K)	$4,008,900
Disabling Injury (A)	$216,000
Evident Injury (B)	$79,000
Fatal/Injury (K/A/B)	$158,200
Possible Injury (C)	$44,900
PDO (O)	$7,400

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA - HRT - 05-051, October 2005.

Because SPFs and AMFs do not always differentiate between a fatal and injury crashes when estimating average crash frequencies, many jurisdictions have established a societal cost that is representative of a combined fatal/injury crash. The value determined by FHWA is shown in *Exhibit 7-4* as $158,200.

A countermeasure is estimated to reduce the expected average crash frequency of fatal/injury crashes by five crashes per year and the number of PDO crashes by 11 per year over the service year of the project. What is the annual monetary benefit associated with the crash reduction? Fatal/Injury Crashes: 5 x $158,200 = $791,000/year PDO crashes: 11 x $7,400 = $81,400/year Total Annual Monetary Benefit: $791,000+$81,400 = $872,400/year

7.4.3.2. Convert Annual Monetary Value to Present Value

This section describes the method to calculate present value of monetary benefits.

There are two methods that can be used to convert annual monetary benefits to present value. The first is used when the annual benefits are uniform over the service life of the project. The second is used when the annual benefits vary over the service life of the project.

The following data is needed to convert annual monetary value to present value:

- Annual monetary benefit associated with the change in crash frequency (as calculated above);
- Service life of the countermeasure(s); and,
- Discount rate (minimum rate of return).

7.4.3.3. Method One: Convert Uniform Annual Benefits to a Present Value

When the annual benefits are uniform over the service life of the project Equations 7-1 and 7-2 can be used to calculate present value of project benefits.

$$PV_{benefits} = TotalAnnualMonetaryBenefits \times (P/A,\ i,\ y) \tag{7-1}$$

Where,

$PV_{benefits}$ = Present value of the project benefits for a specific site, v

$(P/A, i, y)$ = Conversion factor for a series of uniform annual amounts to present value

$$(P/A, i, y) = \frac{(1.0 + i)^{(y)} - 1.0}{i \times (1.0 + i)^{(y)}} \tag{7-2}$$

i = Minimum attractive rate of return or discount rate (i.e., if the discount rate is 4%, the i = 0.04)

y = Year in the service life of the countermeasure(s)

From the previous example, the total annual monetary benefit of a countermeasure is $872,400. What is the present value of the project?

Applying Equation 7-2:

Assume,

i = 0.04

y = 5 years

Then,

$$(P/A,i,y) = \frac{(1.0+0.04)^{(5)} - 1.0}{0.04 \times (1.0+0.04)^{(5)}} = 4.45$$

Applying Equation 7-1:

$PV_{benefits}$ = $872,400 x (4.45)

= $3,882,180

7.4.3.4. Method Two: Convert Non-Uniform Annual Benefits to Present Value

Some countermeasures yield larger changes in expected average crash frequency in the first years after implementation than in subsequent years. In order to account for this occurrence over the service life of the countermeasure, non-uniform annual monetary values can be calculated as shown in Step 1 below for each year of service. The following process is used to convert the project benefits of all non-uniform annual monetary values to a single present value:

1. Convert each annual monetary value to its individual present value. Each future annual value is treated as a single future value; therefore, a different present worth factor is applied to each year.
   a) Substitute the (P/F, i, y) factor calculated for each year in the service life for the (P/A, i, y) factor presented in Equation 7-2.
      i) (P/F, i, y) = a factor that converts a single future value to its present value
      ii) (P/F, i, y) = $(1+i)^{(-y)}$

      Where,

      i = discount rate (i.e., the discount rate is 4%, i = 0.04)

      y = year in the service life of the countermeasure(s)
2. Sum the individual present values to arrive at a single present value that represents the project benefits of the project.

The sample problems at the end of this chapter illustrate how to convert non-uniform annual values to a single present value.

7.5. ESTIMATE PROJECT COSTS

Estimating the cost associated with implementing a countermeasure follows the same procedure as performing cost estimates for other construction or program implementation projects. Similar to other roadway improvement projects, expected project costs are unique to each site and to each proposed countermeasure(s). The cost of implementing a countermeasure or set of countermeasures could include a variety of factors. These may include right-of-way acquisition, construction material costs, grading and earthwork, utility relocation, environmental impacts, maintenance, and other costs including any planning and engineering design work conducted prior to construction.

The *AASHTO Redbook* states "Project costs should include the present value of any obligation to incur costs (or commit to incur costs in the future) that burden the [highway] authority's funds."[(1)] Therefore, under this definition the present value of construction, operating, and maintenance costs over the service life of the project are included in the assessment of expected project costs. *Chapter 6* of the *AASHTO Redbook* provides additional guidance regarding the categories of costs and their proper treatment in a benefit-cost or economic appraisal. Categories discussed in the Redbook include:

- Construction and other development costs
- Adjusting development and operating cost estimates for inflation
- The cost of right-of-way
- Measuring the current and future value of undeveloped land
- Measuring current and future value of developed land
- Valuing already-owned right-of-way
- Maintenance and operating costs
- Creating operating cost estimates

Project costs are expressed as present values for use in economic evaluation. Project construction or implementation costs are typically already present values, but any annual or future costs need to be converted to present values using the same relationships presented for project benefits in Section 7.4.3.

7.6. ECONOMIC EVALUATION METHODS FOR INDIVIDUAL SITES

There are two main objectives for the economic evaluation of a countermeasure or combination of countermeasures:

The two main objectives for economic evaluation are to determine:
1) if a project is economically justified, and
2) which project is most cost-effective.

1. Determine if a project is economically justified (i.e., the benefits are greater than the costs), and
2. Determine which project or alternative is most cost-effective.

Two methods are presented in Section 7.6.1 that can be used to conduct cost-benefit analysis in order to satisfy the first objective. A separate method is described in Section 7.6.2 that can be used to satisfy the second objective. A step-by-step process for using each of these methods is provided, along with an outline of the strengths and limitations of each.

In situations where an economic evaluation is used to compare multiple alternative countermeasures or projects at a single site, the methods presented in *Chapter 8* for evaluation of multiple sites can be applied.

7.6.1. Procedures for Benefit-Cost Analysis

Net present value and benefit-cost ratio are presented in this section. These methods are commonly used to evaluate the economic effectiveness and feasibility of individual roadway projects. They are presented in this section as a means to evaluate countermeasure implementation projects intended to reduce the expected average crash frequency or crash severity. The methods utilize the benefits calculated in Section 7.4 and costs calculated in Section 7.5. The FHWA SafetyAnalyst software provides an economic-appraisal tool that can apply each of the methods described below.(3)

Section 7.6.1 provides a description of the methods to calculate net present value (NPV) and benefit-cost ratio (BCR).

7.6.1.1. Net Present Value (NPV)

The net present value (NPV) method is also referred to as the net present worth (NPW) method. This method is used to express the difference between discounted costs and discounted benefits of an individual improvement project in a single amount. The term "discount" indicates that the monetary costs and benefits are converted to a present value using a discount rate.

Applications

The NPV method is used for the two basic functions listed below:

- Determine which countermeasure or set of countermeasures provides the most cost-efficient means to reduce crashes. Countermeasure(s) are ordered from the highest to lowest NPV.
- Evaluate if an individual project is economically justified. A project with a NPV greater than zero indicates a project with benefits that are sufficient enough to justify implementation of the countermeasure.

Method

1. Estimate the number of crashes reduced due to the safety improvement project (see Section 7.4 and the *Part C Introduction and Applications Guidance*).
2. Convert the change in estimated average crash frequency to an annual monetary value to representative of the benefits (see Section 7.5).
3. Convert the annual monetary value of the benefits to a present value (see Section 7.5).
4. Calculate the present value of the costs associated with implementing the project (see Section 7.5).
5. Calculate the NPV using Equation 7-3:

$$NPV = PV_{benefits} - PV_{costs} \tag{7-3}$$

Where,

$PV_{benefits}$ = Present value of project benefits

PV_{costs} = Present value of project costs

6. If the NPV > 0, then the individual project is economically justified.

Exhibit 7-5 presents the strengths and limitations of NPV Analysis.

Exhibit 7-5: Strengths and Limitations of NPV Analysis

Strengths	Weaknesses
• This method evaluates the economic justification of a project.	• The magnitude cannot be as easily interpreted as a benefit-cost ratio.
• NPV are ordered from highest to lowest value.	
• It ranks projects with the same rankings as produced by the incremental-benefit-to-cost-ratio method discussed in Chapter 8.	

7.6.1.2. Benefit-Cost Ratio (BCR)

A benefit-cost ratio is the ratio of the present-value benefits of a project to the implementation costs of the project (BCR = Benefits/Costs). If the ratio is greater than 1.0, then the project is considered economically justified. Countermeasures are ranked from highest to lowest BCR. An incremental benefit-cost analysis (*Chapter 8*) is needed to use the BCR as a tool for comparing project alternatives.

Applications

This method is used to determine the most valuable countermeasure(s) for a specific site and is used to evaluate economic justification of individual projects. The benefit-cost ratio method is not valid for prioritizing multiple projects or multiple alternatives for a single project; the methods discussed in *Chapter 8* are valid processes to prioritize multiple projects or multiple alternatives.

Method

1. Calculate the present value of the estimated change in average crash frequency (see *Section 7.5*).
2. Calculate the present value of the costs associated with the safety improvement project (see *Section 7.5*).
3. Calculate the benefit-cost ratio by dividing the estimated project benefits by the estimated project costs.

$$BCR = \frac{PV_{benefits}}{PV_{costs}} \tag{7-4}$$

Where,

BCR = Benefit cost ratio

$PV_{benefits}$ = Present value of project benefits

PV_{costs} = Present value of project costs

4. If the BCR is greater than 1.0, then the project is economically justified.

Exhibit 7-6 presents the strengths and limitations of BCR Analysis.

Exhibit 7-6: Strengths and Limitations of BCR Analysis

Strengths	Weaknesses
• The magnitude of the benefit-cost ratio makes the relative desirability of a proposed project immediately evident to decision makers.	• Benefit-cost ratio cannot be directly used in decision making between project alternatives or to compare projects at multiple sites. An incremental benefit-cost analysis would need to be conducted for this purpose (see *Chapter 8*).
• This method can be used by highway agencies in evaluations for the Federal Highway Administration (FHWA) to justify improvements funded through the Highway Safety Improvement Program (HSIP). Projects identified as economically justified (BCR > 1.0) are eligible for federal funding; however, there are instances where implementing a project with a BCR < 1.0 is warranted based on the potential for crashes without the project.	• This method considers projects individually and does not provide guidance for identifying the most cost-effective mix of projects given a specific budget.

7.6.2. Procedures for Cost-Effectiveness Analysis

In cost-effectiveness analysis the predicted change in average crash frequency are not quantified as monetary values, but are compared directly to project costs.

Cost effectiveness is the annual cost per crash reduced. The lower the cost per crash reduced, the more effective the treatment.

The cost-effectiveness of a countermeasure implementation project is expressed as the annual cost per crash reduced. Both the project cost and the estimated average crash frequency reduced must apply to the same time period, either on an annual basis or over the entire life of the project. This method requires an estimate of the change in crashes and cost estimate associated with implementing the countermeasure. However, the change in estimated crash frequency is not converted to a monetary value.

Applications

This method is used to gain a quantifiable understanding of the value of implementing an individual countermeasure or multiple countermeasures at an individual site when an agency does not support the monetary crash cost values used to convert a project's change in estimated average crash frequency reduction to a monetary value.

Method

1. Estimate the change in expected average crash frequency due to the safety improvement project (see Section 7.4 and the *Part C Introduction and Applications Guidance,* Section C.7).
2. Calculate the costs associated with implementing the project (see Section 7.5).
3. Calculate the cost-effectiveness of the safety improvement project at the site by dividing the present value of the costs by the estimated change in average crash frequency over the life of the countermeasure:

$$Cost\text{ - }Effectiveness\ Index = \frac{PV_{costs}}{N_{predicted}\text{ - }N_{observed}} \qquad (7\text{-}5)$$

Where,

PV_{costs} = Present Value of Project Cost

$N_{predicted}$ = Predicted crash frequency for year y

$N_{observed}$ = Observed crash frequency for year y

Exhibit 7-7 presents the strengths and limitations of NPV Analysis.

Exhibit 7-7: Strengths and Limitations of Cost-Effectiveness Analysis

Strengths	Weaknesses
• This method results in a simple and quick calculation that provides a general sense of an individual project's value.	• It does not differentiate between the value of reducing a fatal crash, an injury crash and a PDO crash.
• It produces a numeric value that can be compared to other safety improvement projects evaluated with the same method.	• It does not indicate whether an improvement project is economically justified because the benefits are not expressed in monetary terms.
• There is no need to convert the change in expected average crash frequency by severity or type to a monetary value.	

Section 7.7 describes that non-monetary factors can also be a consideration in project decisions.

7.7. NON-MONETARY CONSIDERATIONS

In most cases, the primary benefits of countermeasure implementation projects can be estimated in terms of the change average crash frequency and injuries avoided and/or monetary values. However, many factors not directly related to changes in crash frequency enter into decisions about countermeasure implementation projects and many cannot be quantified in monetary terms. Non-monetary considerations include:

- Public demand;
- Public perception and acceptance of safety improvement projects;
- Meeting established and community-endorsed policies to improve mobility or accessibility along a corridor;
- Air quality, noise, and other environmental considerations;
- Road user needs; and,
- Providing a context sensitive solution that is consistent with a community's vision and environment.

For example, a roundabout typically provides both quantifiable and non-quantifiable benefits for a community. Quantifiable benefits often include reducing the average delay experienced by motorists, reducing vehicle fuel consumption, and reducing severe angle and head-on injury crashes at the intersection. Each could be converted into a monetary value in order to calculate costs and benefits.

Examples of potential benefits associated with implementation of a roundabout that cannot be quantified or given a monetary value could include:

- Improving aesthetics compared to other intersection traffic control devices;
- Establishing a physical character change that denotes entry to a community (a gateway treatment) or change in roadway functional classification;
- Facilitating economic redevelopment of an area;
- Serving as an access management tool where the splitter islands remove the turbulence of full access driveways by replacing them with right-in/right-out driveways to land uses; and,
- Accommodating U-turns more easily at roundabouts.

For projects intended primarily to reduce crash frequency or severity, a benefit-cost analysis in monetary terms may serve as the primary decision making tool, with secondary consideration of qualitative factors. The decision-making process on larger-scale projects that do not only focus on change in crash frequency may be primarily qualitative, or may be quantitative by applying weighting factors to specific decision criteria such as safety, traffic operations, air quality, noise, etc. *Chapter 8* discusses the application of multi-objective resource allocation tools as one method to make such decisions as quantitative as possible.

7.8. CONCLUSIONS

Chapter 7 provides an overview of methods to estimate the benefits of a countermeasure in terms of a reduction in crash frequency. It also provides methods for comparing the benefits to the costs.

The information presented in this chapter can be used to objectively evaluate countermeasure implementation projects by quantifying the monetary value of each project. The process begins with quantifying the benefits of a proposed project in terms of the change in expected average crash frequency.

Section 7.4.1 provides guidance on how to use the *Part C* safety prediction methodology, the *Part D* AMFs, or locally developed AMFs, to estimate the change in expected average crash frequency for a proposed project. Section 7.4.2 provides guidance for how to estimate the change in expected average crash frequency when there is no applicable *Part C* methodology, no applicable SPF, and no applicable AMF.

Two types of methods are outlined in the chapter for estimating change in average crash frequency in terms of a monetary value. In benefit-cost analysis, the expected reduction in crash frequency by severity level is converted to monetary values, summed, and compared to the cost of implementing the countermeasure. In cost-effectiveness analysis, the expected change in average crash frequency is compared directly to the cost of implementing the countermeasure.

Depending on the objective of the evaluation, the economic appraisal methods described in this chapter can be used by highway agencies to:

1. Identify economically justifiable projects where the benefits are greater than the costs, and
2. Rank countermeasure alternatives for a given site.

Estimating the cost associated with implementing a countermeasure follows the same procedure as performing cost estimates for other construction or program implementation projects. *Chapter 6* of the *AASHTO Redbook* provides guidance regarding the categories of costs and their proper treatment in a benefit-cost or economic appraisal.[(1)]

The ultimate decision of which countermeasure implementation projects are constructed involves numerous considerations beyond those presented in *Chapter 7*. These considerations assess the overall influence of the projects, as well as the current political, social, and physical environment surrounding their implementation.

Chapter 8 presents methods that are intended to identify the most cost-efficient mix of improvement projects over multiple sites, but can also be applied to compare alternative improvements for an individual site.

7.9. SAMPLE PROBLEM

The sample problem presented here illustrates the process for calculating the benefits and costs of projects and subsequent ranking of project alternatives by three of the key ranking criteria illustrated in Section 7.6: cost-effectiveness analysis, benefit-cost analysis, and net present value analysis.

7.9.1. Economic Appraisal

Background/Information

The roadway agency has identified countermeasures for application at Intersection 2. Exhibit 7-8 provides a summary of the crash conditions, contributory factors, and selected countermeasures.

Exhibit 7-8: Summary of Crash Conditions, Contributory Factors, and Selected Countermeasures

Data	Intersection 2
Major/Minor AADT	22,100 / 1,650
Predominate Collision Types	Angle Head-On
Crashes by Severity	
Fatal	6%
Injury	65%
PDO	29%
Contributory Factors	Increase in traffic volumes
	Inadequate capacity during peak hour
	High travel speeds during off-peak
Selected Countermeasure	Install a Roundabout

The Question

What are the benefits and costs associated with the countermeasures selected for Intersection 2?

The Facts

Intersections:

- AMFs for installing a single-lane roundabout in place of a two-way stop controlled intersection (see *Chapter 14*);

- o Total crashes = 0.56; and,
- o Fatal and injury crashes = 0.18.

Assumptions

The roadway agency has the following information:

- Calibrated SPF and dispersion parameters for the intersection being evaluated;
- Societal crash costs associated with crash severities;
- Cost estimates for implementing the countermeasure;
- Discount rate (minimum rate of return);
- Estimate of the service life of the countermeasure; and,
- The roadway agency has calculated the EB-adjusted expected average crash frequency for each year of historical crash data.

The sample problems provided in this section are intended to demonstrate application of the economic appraisal process, not predictive methods. Therefore, simplified crash estimates for the existing conditions at Intersection 2 were developed using predictive methods outlined in *Part C* and are provided in Exhibit 7-9.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the base SPF model. AMFs that are associated with the countermeasures implemented are provided. All other AMFs are assumed to be 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are for theoretical application and are rarely valid for application of predictive methods to actual field conditions.

Exhibit 7-9: Expected Average Crash Frequency at Intersection 2 WITHOUT Installing the Roundabout

Year in service life (y)	Major AADT	Minor AADT	$N_{expected(TOT)}$	$N_{expected(FI)}$
1	23,553	1,758	10.4	5.2
2	23,906	1,785	10.5	5.3
3	24,265	1,812	10.5	5.3
4	24,629	1,839	10.6	5.4
5	24,998	1,866	10.7	5.4
6	25,373	1,894	10.7	5.4
7	25,754	1,923	10.8	5.5
8	26,140	1,952	10.9	5.5
9	26,532	1,981	11.0	5.5
10	26,930	2,011	11.0	5.6
Total			107.1	54.1

The roadway agency finds the societal crash costs shown in Exhibit 7-10 acceptable. The agency decided to conservatively estimate the economic benefits of the countermeasures. Therefore, they are using the average injury crash cost (i.e., the average value of a fatal (K), disabling (A), evident (B), and possible injury crash (C) as the crash cost value representative of the predicted fatal and injury crashes.

Exhibit 7-10: Societal Crash Costs by Severity

Injury Severity	Estimated Cost
Fatality (K)	$4,008,900
Cost for crashes with a fatal and/or injury (K/A/B/C)	$158,200
Disabling Injury (A)	$216,000
Evident Injury (B)	$79,000
Possible Injury (C)	$44,900
PDO (O)	$7,400

Source: Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries, FHWA - HRT - 05-051, October 2005.

Exhibit 7-11 summarizes the assumptions regarding the service life for the roundabout, the annual traffic growth at the site during the service life, the discount rate and the cost of implementing the roundabout.

Exhibit 7-11: Remaining Assumptions

	Intersection 2
Countermeasure	Roundabout
Service Life	10 years
Annual Traffic Growth	2%
Discount Rate (i)	4.0%
Cost Estimate	$695,000

Method

The following steps are required to solve the problem.

- STEP 1 - Calculate the expected average crash frequency at Intersection 2 *without* the roundabout.
- STEP 2 - Calculate the expected average crash frequency at Intersection 2 *with* the roundabout.
- STEP 3 - Calculate the change in expected average crash frequency for total, fatal and injury, and PDO crashes.
- STEP 4 - Convert the change in crashes to a monetary value for each year of the service life.
- STEP 5 - Convert the annual monetary values to a single present value representative of the total monetary benefits expected from installing the countermeasure at Intersection 2.

A summary of inputs, equations, and results of economic appraisal conducted for Intersection 2 is shown in Exhibit 7-12. The methods for conducting the appraisal are outlined in detail in the following sections.

Exhibit 7-12: Economic Appraisal for Intersection 2

ROADWAY SEGMENT ACCIDENT PREDICTION WORKSHEET	
General Information	**Site Information**
Analyst: Mary Smith	Highway: US71
Agency or Company: State DOT	Roadway Section: ________
Date Performed: 02/03/02	Jurisdiction: ________
Analysis Time Period: _____ ________	Analysis Year: 2002
Input Data	
Major/Minor AADT (veh/day)	12,000 / 1,200
Countermeasure	Roundabout
Service Life ($Years_{SL}$)	10 years
Annual Traffic Volume Growth Rate	1.5%
Discount Rate (i)	4.0%
Cost Estimate	$2,000,000
Societal Crash Costs by Severity	
Fatal and Injury	$158,200
Property Damage Only	$7,400
Base Model	
Four-Legged Two-Way Stop Controlled Intersection Multiple Vehicle Collisions (See Chapter 12)	$N_{br} = N_{spf\ rs} \times (AMF_{1r} \times AMF_{2r} \times \ldots \times AMF_{nr})$
EB-Adjusted Expected Average Crash Frequency	
Expected Crashes without Roundabout	See Exhibit 7-9
Expected Crashes with Roundabout Equations 7-6, 7-7	See Exhibit 7-13 and Exhibit 7-14
Expected Change in Crashes Equations 7-8, 7-9, 7-10	See Exhibit 7-15
Yearly Monetary Value of Change in Crashes Equations 7-11, 7-12, 7-13	See Exhibit 7-16
Present Value of Change in Crashes Equations 7-14, 7-15	See Exhibit 7-17
Benefit of installing a roundabout at Intersection 2	$36,860,430

STEP 1 - Calculate the expected average crash frequency at Intersection 2 WITHOUT the roundabout.

The *Part C* prediction method can be used to develop the estimates. Exhibit 7-9 summarizes the EB-adjusted expected crash frequency by severity for each year of the expected service life of the project.

STEP 2 - Calculate the expected average crash frequency at Intersection 2 WITH the roundabout.

Calculate EB-adjusted total (TOT) and fatal and injury (FI) crashes for each year of the service life (y) assuming the roundabout is installed.

Multiply the AMF for converting a stop-controlled intersection to a roundabout found in *Chapter 14* (restated below in Exhibit 7-13) by the expected average crash frequency calculated above in *Exhibit 7-6* using Equations 7-6 and 7-7.

$$N_{expected\ roundabout\ (TOTAL)} = N_{expected(TOTAL)} \times AMF_{(TOTAL)} \tag{7-6}$$

$$N_{expected\ roundabout\ (FI)} = N_{expected(FI)} \times AMF_{(FI)} \tag{7-7}$$

Where,

$N_{expected\ roundabout\ (TOTAL)}$ = EB-adjusted expected average crash frequency in year *y* WITH the roundabout installed;

$N_{expected\ roundabout\ i(FI)}$ = EB-adjusted expected average fatal and injury crash frequency in year *y* WITH the roundabout installed;

$N_{expected\ (TOTAL)}$ = EB-adjusted expected average total crash frequency in year *y* WITHOUT the roundabout installed;

$N_{expected\ (FI)}$ = EB-adjusted expected average fatal and injury crash frequency in year *y* WITHOUT the roundabout installed;

$AMF_{(TOTAL)}$ = Accident Modification Factor for total crashes; and,

$AMF_{(FI)}$ = Accident Modification Factor for fatal and injury crashes.

Exhibit 7-13 summarizes the EB-adjusted average fatal and injury crash frequency for each year of the service life assuming the roundabout is installed.

Exhibit 7-13: Expected Average FI Crash Frequency at Intersection 2 WITH the Roundabout

Year in Service Life (y)	$N_{expected(FI)}$	$AMF_{(FI)}$	$N_{expected\ roundabout(FI)}$
1	5.2	0.18	0.9
2	5.3	0.18	1.0
3	5.3	0.18	1.0
4	5.4	0.18	1.0
5	5.4	0.18	1.0
6	5.4	0.18	1.0
7	5.5	0.18	1.0
8	5.5	0.18	1.0
9	5.5	0.18	1.0
10	5.6	0.18	1.0
Total			9.9

Exhibit 7-14 summarizes the EB-adjusted average total crash frequency for each year of the service life assuming the roundabout is installed.

Exhibit 7-14: Expected Average Total Crash Frequency at Intersection 2 WITH the Roundabout

Year in service life (y)	$N_{expected(TOTAL)}$	$AMF_{(TOTAL)}$	$N_{expected\ roundabout(TOTAL)}$
1	10.4	0.56	5.8
2	10.5	0.56	5.9
3	10.5	0.56	5.9
4	10.6	0.56	5.9
5	10.7	0.56	6.0
6	10.8	0.56	6.0
7	10.8	0.56	6.0
8	10.9	0.56	6.1
9	11.0	0.56	6.2
10	11.0	0.56	6.2
Total			60.0

STEP 3 - Calculate the expected change in crash frequency for total, fatal and injury, and PDO crashes.

The difference between the expected average crash frequency with and without the countermeasure is the expected change in average crash frequency. Equations 7-8, 7-9, and 7-10 are used to estimate this change for total, fatal and injury, and PDO crashes.

$$\Delta N_{expected\ (FI)} = N_{expected(FI)} - N_{expected\ roundabout(FI)} \qquad (7\text{-}8)$$

$$\Delta N_{expected(TOTAL)} = N_{expected(TOTAL)} - N_{expected\ roundabout(TOTAL)} \qquad (7\text{-}9)$$

$$\Delta N_{expected(PDO)} = N_{expected(TOTAL)} - N_{expected(FI)} \qquad (7\text{-}10)$$

Where,

$\Delta N_{expected(TOTAL)}$ = Expected change in average crash frequency due to implementing countermeasure;

$\Delta N_{expected(FI)}$ = Expected change in average fatal and injury crash frequency due to implementing countermeasure; and,

$\Delta N_{expected(PDO)}$ = Expected change in average PDO crash frequency due to implementing countermeasure.

Exhibit 7-15 summarizes the expected change in average crash frequency due to installing the roundabout.

Exhibit 7-15: Change in Expected Average in Crash Frequency at Intersection 2 WITH the Roundabout

Year in service life, *y*	**$\Delta N_{expected(TOTAL)}$**	**$\Delta N_{expected(FI)}$**	**$\Delta N_{expected(PDO)}$**
1	4.6	4.3	0.3
2	4.6	4.3	0.3
3	4.6	4.3	0.3
4	4.7	4.4	0.3
5	4.7	4.4	0.3
6	4.7	4.4	0.3
7	4.8	4.5	0.3
8	4.8	4.5	0.3
9	4.8	4.5	0.3
10	4.8	4.6	0.2
Total	47.1	44.2	2.9

STEP 4 - Convert Change in Crashes to a Monetary Value

The estimated reduction in average crash frequency can be converted to a monetary value for each year of the service life using Equations 7-11 through 7-13.

$$AM_{(PDO)} = \Delta N_{expected(PDO)} \times CC_{(FI)} \qquad (7\text{-}11)$$

$$AM_{(FI)} = \Delta N_{expected(FI)} \times CC_{(FI)} \qquad (7\text{-}12)$$

$$AM_{(TOTAL)} = AM_{(PDO)} \times AM_{(FI)} \qquad (7\text{-}13)$$

Where,

$AM_{(PDO)}$ = Monetary value of the estimated change in average PDO crash frequency for year, *y*;

$CC_{(PDO)}$ = Crash cost for PDO crash severity;

$CC_{(FI)}$ = Crash cost for FI crash severity;

$AM_{(FI)}$ = Monetary value of the estimated change in fatal and injury average crash frequency for year y; and,

$AM_{(TOTAL)}$ = Monetary value of the total estimated change in average crash frequency for year y.

Exhibit 7-16 summarizes the monetary value calculations for each year of the service life.

Exhibit 7-16: Annual Monetary Value of Change in Crashes

Year in service life (y)	ΔN (FI)	FI Crash Cost	AM(FI)	ΔN(PDO)	PDO Crash Cost	AM(PDO)	AM(TOTAL)
1	4.3	$158,200	$680,260	0.3	$7,400	$2,220	$682,480
2	4.3	$158,200	$680,260	0.3	$7,400	$2,220	$682,480
3	4.3	$158,200	$680,260	0.3	$7,400	$2,220	$682,480
4	4.4	$158,200	$696,080	0.3	$7,400	$2,220	$698,300
5	4.4	$158,200	$696,080	0.3	$7,400	$2,220	$698,300
6	4.4	$158,200	$696,080	0.3	$7,400	$2,220	$698,300
7	4.5	$158,200	$711,900	0.3	$7,400	$2,220	$714,120
8	4.5	$158,200	$711,900	0.3	$7,400	$2,220	$714,120
9	4.5	$158,200	$711,900	0.3	$7,400	$2,220	$714,120
10	4.6	$158,200	$727,720	0.2	$7,400	$1,480	$729,200

STEP 5 – Convert Annual Monetary Values to a Present Value

The total monetary benefits expected from installing a roundabout at Intersection 2 are calculated as a present value using Equations 7-14 and 7-15.

Note: A 4% discount rate is assumed for the conversion of the annual values to a present value.

Convert the annual monetary value to a present value for each year of the service life.

$$PV_{benefits} = TotalAnnualMonetaryBenefits \times (P/F, i, y) \qquad (7\text{-}14)$$

Where,

$PV_{benefits}$ = Present value of the project benefits per site in year y;

$(P/F, i, y)$ = Factor that converts a single future value to its present value, calculated as (1+i)-y;

i = Discount rate (i.e., the discount rate is 4%, i = 0.04); and,

y= Year in the service life of the countermeasure.

If the annual project benefits are uniform, then the following factor is used to convert a uniform series to a single present worth:

$$(P/A, i, y) = \frac{(1.0 + i)^{(y)} - 1.0}{i \times (1.0 + i)^{(y)}} \qquad (7\text{-}15)$$

Where,

$(P/A, i, y)$ = a factor that converts a series of uniform future values to a single present value.

Exhibit 7-17 summarizes the results of converting the annual values to present values.

Exhibit 7-17: Converting Annual Values to Present Values

Year in service life (y)	(P/A, i, y)	AM $_{(TOT)}$	Present Value
1	1.0	$682,480	$682,480
2	1.9	$682,480	$1,296,710
3	2.8	$682,480	$1,910,940
4	3.6	$698,300	$2,513,880
5	4.5	$698,300	$3,142,350
6	5.2	$698,300	$3,631,160
7	6.0	$714,120	$4,284,720
8	6.7	$714,120	$4,784,600
9	7.4	$714,120	$5,284,490
10	8.1	$729,200	$5,906,520
Total			$33,437,850

The total present value of the benefits of installing a roundabout at Intersection 2 is the sum of the present value for each year of the service life. The sum is shown above in Exhibit 7-17.

Results

The estimated present value monetary benefit of installing a roundabout at Intersection 2 is $33,437,850.

The roadway agency estimates the cost of installing the roundabout at Intersection 2 is $2,000,000.

If this analysis were intended to determine whether the project is cost effective, the magnitude of the monetary benefit provides support for the project. If the monetary benefit of change in crashes at this site were to be compared to other sites the BCR could be calculated and used to compare to other projects to identify the most economically-efficient project.

7.10. REFERENCES

1. AASHTO. *A Manual of User Benefit Analysis for Highways, 2nd Edition.* American Association of State Highway and Transportation Officials, Washington, DC, 2003.
2. Council, F.M., E. Zaloshnja, T. Miller, and B. Persaud. *Crash Cost Estimates by Maximum Police Reported Injury Severity within Selected Crash Geometries*. Publication No. FHWA-HRT-05-051, Federal Highway Administration, U.S. Department of Transportation, October 2005.
3. Harwood, D.W. et al. *Safety Analyst: Software Tools for Safety Management of Specific Highway Sites Task M Functional Specification for Module 3* – Economic Appraisal and Priority Ranking GSA Contract No. GS-23F-0379K Task No. DTFH61-01-F-00096. November 2003. More information available from http://www.safetyanalyst.org.

APPENDIX A – DATA NEEDS AND DEFINITIONS FOR CHAPTER 7

A.1 Data Needs to Calculate Change in Crashes

Calculating the benefits of a countermeasure or set of countermeasures is a two step process. The first step is to calculate the change in crash frequency and the second is to calculate the monetary value of the change in crashes. The data needed for both of these steps are described below.

1. Calculate Change in Crashes

   The data needed to estimate change in crashes by severity are defined below.

- **Crash history** at the site by severity;
- **Current Average Annual Daily Traffic** (AADT) volumes for the site;
- **Expected implementation year** for the countermeasure(s); and,
- **Future AADT** for the site that correspond with the year in which the countermeasure is implemented.
- **Safety Performance Function (SPF)** for current site conditions (e.g., urban, four-legged, signalized intersection) and for total crashes (TOT) and for fatal and injury crashes (FI). SPFs may be locally developed or calibrated to local conditions.
- If necessary, an **SPF** for site conditions with the countermeasure implemented (e.g. urban, four-legged, roundabout controlled intersection) and for total crashes (TOT) and for fatal and injury crashes (FI). SPFs may be locally developed or calibrated to local conditions.
- **Accident Modification Factors (AMFs)** for the countermeasures under consideration. AMFs are a decimal that when multiplied by the expected average crash frequency without the countermeasure produces the expected average crash frequency with the countermeasure.

2. Convert Change in Crashes to a Monetary Value

The data needed to convert the change in crashes to a monetary value are described below.

- Accepted **monetary value of crashes** by collision type and/or crash severity

State and local jurisdictions often have accepted dollar value of crashes by collision type and/or crash severity that are used to convert the estimated change in crash reduction to a monetary value. The most recent societal costs by severity documented in the October 2005 Federal Highway Administration (FHWA) report "Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries" are listed below (values shown below are rounded to the nearest hundred dollars).[2]

- Fatality (K) = $4,008,900/fatal crash;

- Crashes that include a fatal and/or injury (K/A/B/C) = $158,200/ fatal and/or injury crash;
- Injury (A/B/C) = $82, 600/ injury crash;
- Disabling Injury (A) = $216,000/disabling injury crash;
- Evident Injury (B) = $79,000/evident injury crash;
- Possible Injury (C) = $44,900/possible injury crash; and,
- PDO (O) = $7,400/PDO crash.

The most recent mean comprehensive crash costs by type (i.e., single-vehicle rollover crash, multiple vehicle rear-end crash, and others) are also documented in the October 2005 FHWA report.

The monetary values used to represent the change in crashes are those accepted and endorsed by the jurisdiction in which the safety improvement project will be implemented.

A.2 Service Life of the Improvement Specific to the Countermeasure

All improvement projects have a service life. In terms of a countermeasure, the service life corresponds to the number of years in which the countermeasure is expected to have a noticeable and quantifiable effect on the crash occurrence at the site. Some countermeasures, such as pavement markings, deteriorate as time passes, and need to be renewed. For other countermeasures, other roadway design modifications and changes in the surrounding land uses that occur as time passes may influence the crash occurrence at the site, reducing the effectiveness of the countermeasure. The service life of a countermeasure reflects a reasonable time period in which roadway characteristics and traffic patterns are expected to remain relatively stable.

A.3 Discount Rate

The discount rate is an interest rate that is chosen to reflect the time value of money. The discount rate represents the minimum rate of return that would be considered by an agency to provide an attractive investment. Thus, the minimum attractive rate of return is judged in comparison with other opportunities to invest public funds wisely to obtain improvements that benefit the public. Two basic factors to consider when selecting a discount rate:

1. The discount rate corresponds to the treatment of inflation (i.e., real dollars versus nominal dollars) in the analysis being conducted. If benefits and costs are estimated in real (uninflated) dollars, then a real discount rate is used. If benefits and costs are estimated in nominal (inflated) dollars, then a nominal discount rate is used.
2. The discount rate reflects the private cost of capital instead of the public-sector borrowing rate. Reflecting the private cost of capital implicitly accounts for the element of risk in the investment. Risk in the investment corresponds to the potential that the benefits and costs associated with the project are not realized within the given service life of the project.

Discount rates are used for the calculation of benefits and costs for all improvement projects. Therefore, it is reasonable that jurisdictions are familiar with the discount rates commonly used and accepted for roadway improvements. Further guidance is found in the American Associate of State Highway and Transportation Officials (AASHTO) publication entitled *A Manual of User Benefit Analysis for Highways* (also known as the AASHTO Redbook).[1]

A.4 Data Needs to Calculate Project Costs

Highway agencies and local jurisdictions have sufficient experience with and established procedures for estimating the costs of roadway improvements. Locally derived costs based on specific site and countermeasure characteristics are the most statistically reliable costs to use in the economic appraisal of a project. It is anticipated that costs of implementing the countermeasures will include considerations such as right-of-way acquisition, environmental impacts, and operational costs.

A.5 Appendix References

1. AASHTO. *A Manual of User Benefit Analysis for Highways, 2nd Edition.* American Association of State Highway and Transportation Officials, Washington, DC, 2003.

2. Council, F.M., E. Zaloshnja, T. Miller, and B. Persaud. *Crash Cost Estimates by Maximum Police Reported Injury Severity within Selected Crash Geometries*. Publication No. FHWA-HRT-05-051, Federal Highway Administration, Washington, DC, October 2005.

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PART B — ROADWAY SAFETY MANAGEMENT PROCESS

CHAPTER 8—PRIORITIZE PROJECTS

8.1. Introduction 8-1
8.2. Project Prioritization Methods..8-2
8.2.1. Ranking Procedures 8-3
8.2.2. Optimization Methods..8-4
8.2.3. Summary of Prioritization Methods..8-5
8.3. Understanding Prioritization Results 8-7
8.4. Sample Problems 8-7
8.4.1. The Situation..8-7
8.4.2. The Question..8-9
8.4.3. The Facts 8-9
8.4.4. Solution..8-9
8.4.4.1. Simple Ranking - Cost-Effectiveness 8-9
8.4.4.2. Simple Ranking - Net Present Value (NPV) 8-10
8.5. References 8-14

EXHIBITS

Exhibit 8-1: Roadway Safety Management Process Overview 8-1
Exhibit 8-2: Project Prioritization Process 8-2
Exhibit 8-3: Summary of Project Prioritization Methods...8-6

APPENDIX A

A.1 Linear Programming (LP)........8-15
A.2 Integer Programming (IP)........8-16
A.3 Dynamic Programming (DP)........8-17
A.4 Appendix References........8-18

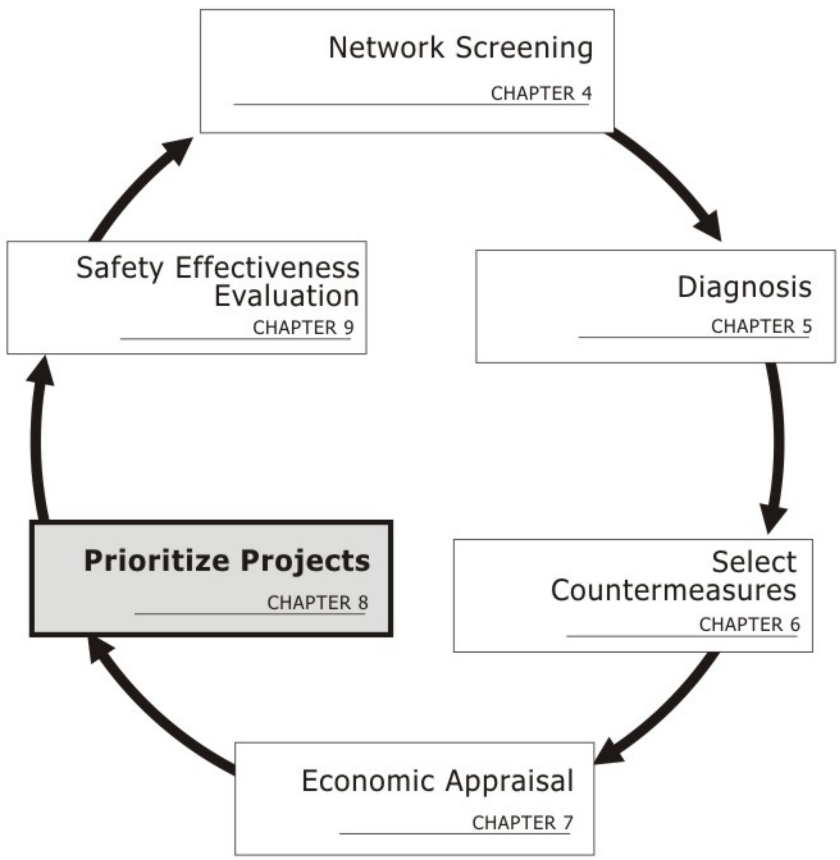
CHAPTER 8 PRIORITIZE PROJECTS

8.1. INTRODUCTION

Chapter 8 presents methods for prioritizing countermeasure implementation projects. Prior to conducting prioritization, one or more candidate countermeasures have been identified for possible implementation at each of several sites, and an economic appraisal has been conducted for each countermeasure. Each countermeasure that is determined to be economically justified by procedures presented in *Chapter 7* is included in the project prioritization process described in this chapter. Exhibit 8-1 provides an overview of the complete Roadway Safety Management process presented in *Part B* of the manual.

Chapter 8 presents prioritization methods to select financially optimal sets of projects.

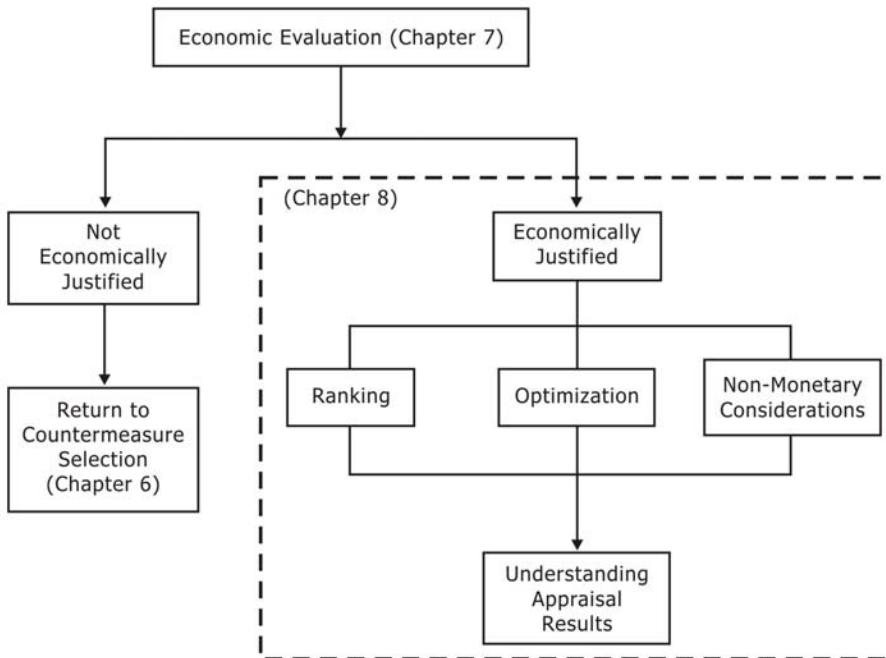
Exhibit 8-1: Roadway Safety Management Process Overview



In the HSM, the term "prioritization" refers to a review of possible projects or project alternatives for construction and developing an ordered list of recommended projects based on the results of ranking and optimization processes. "Ranking" refers to an ordered list of projects or project alternatives based on specific factors or project benefits and costs. "Optimization" is used to describe the process by which a set of projects or project alternatives are selected by maximizing benefits according to budget and other constraints.

This chapter includes overviews of simple ranking and optimization techniques for prioritizing projects. The project prioritization methods presented in this chapter are primarily applicable to developing optimal improvement programs across multiple sites or for an entire roadway system, but they can also be applied to compare improvement alternatives for a single site. This application has been discussed in *Chapter 7*. Exhibit 8-2 provides an overview of the project prioritization process.

Exhibit 8-2: Project Prioritization Process



8.2. PROJECT PRIORITIZATION METHODS

Chapter 8 provides an overview of six methods for prioritizing a list of potential improvements.

The three prioritization methods presented in this chapter are:

- Ranking by economic effectiveness measures
- Incremental benefit-cost analysis ranking
- Optimization methods

Ranking by economic effectiveness measures or by the incremental benefit-cost analysis method provides a prioritized list of projects based on a chosen criterion. Optimization methods, such as linear programming, integer programming, and dynamic programming, provide project prioritization consistent with incremental benefit-cost analysis, but consider the impact of budget constraints in creating an optimized project set. Multiobjective resource allocation can consider the effect of non-monetary elements, including decision factors other than those centered on crash reduction, and can optimize based on several factors.

Incremental benefit-cost analysis is closely related to the benefit-cost ratio (BCR) method presented in *Chapter 7*. Linear programming, integer programming, and dynamic programming are closely related to the net present value (NPV) method presented in *Chapter 7*. There is no generalized multiple-site method equivalent to the cost-effectiveness method presented in *Chapter 7*.

A conceptual overview of each prioritization method is presented in the following sections. Computer software programs are needed to efficiently and effectively use many of these methods, due to their complexity. For this reason, this chapter does not include a step-by-step procedure for these methods. References to additional documentation regarding these methods are provided.

8.2.1. Ranking Procedures

Ranking by Economic Effectiveness Measures

The simplest method for establishing project priorities involves ranking projects or project alternatives by the following measures (identified in *Chapter 7*), including:

- Project costs,
- Monetary value of project benefits,
- Number of total crashes reduced,
- Number of fatal and incapacitating injury crashes reduced,
- Number of fatal and injury crashes reduced,
- Cost-effectiveness index, and,
- Net present value (NPV).

The ranking process develops a list of sites based on particular factors. Examples of these factors are shown in 8.2.1.

As an outcome of a ranking procedure, the project list is ranked high to low on any one of the above measures. Many simple improvement decisions, especially those involving only a few sites and a limited number of project alternatives for each site, can be made by reviewing rankings based on two or more of these criteria.

However, because these methods do not account for competing priorities, budget constraints, or other project impacts, they are too simple for situations with multiple, competing, priorities. Optimization methods are more complicated but will provide information accounting for competing priorities, and will yield a project set that provides the most crash reduction benefits within financial constraints. If ranking sites by benefit-cost ratio, an incremental benefit-cost analysis is performed, as described below.

Incremental Benefit-Cost Analysis

Incremental benefit-cost analysis is an extension of the benefit-cost ratio (BCR) method presented in *Chapter 7*. The following steps describe the method in its simplest form:

1. Perform a BCR evaluation for each individual improvement project as described in *Chapter 7*.
2. Arrange projects with a BCR greater than 1.0 in increasing order based on their estimated cost. The project with the smallest cost is listed first.
3. Beginning at the top of the list, calculate the difference between the first and second project's benefits. Similarly calculate the difference between the costs of the first and second projects. The differences between the benefits of the two projects and the costs of the two are used to compute the BCR for the incremental investment.

4. If the BCR for the incremental investment is greater than 1.0, the project with the higher cost is compared to the next project in the list. If the BCR for the incremental investment is less than 1.0, the project with the lower cost is compared to the next project in the list.
5. Repeat this process. The project selected in the last pairing is considered the best economic investment.

To produce a ranking of projects, the entire evaluation is repeated without the projects previously determined to be the best economic investment until the ranking of every project is determined.

There may be instances where two projects have the same cost estimates resulting in an incremental difference of zero for the costs. An incremental difference of zero for the costs leads to a zero in the denominator for the BCR. If such an instance arises, the project with the greater benefit is selected. Additional complexity is added, where appropriate, to choose one and only one project alternative for a given site. Incremental benefit-cost analysis does not explicitly impose a budget constraint.

It is possible to perform this process manually for a simple application; however, the use of a spreadsheet or special purpose software to automate the calculations is the most efficient and effective application of this method. An example of incremental benefit-cost analysis software used for highway safety analysis is the Roadside Safety Analysis Program (RSAP), which is widely used to establish the economic justification for roadside barriers and other roadside improvements.[3]

8.2.2. Optimization Methods

At a highway network level, a jurisdiction may have a list of improvement projects that are already determined to be economically justified, but there remains a need to determine the most cost-effective set of improvement projects that fit a given budget. Optimization methods are used to identify a project set that will maximize benefits within a fixed budget and other constraints. Thus, optimization methods can be used to establish project priorities for the entire highway system or any subset of the highway system.

It is assumed that all projects or project alternatives to be prioritized using these optimization methods have first been evaluated and found to be economically justified (i.e., project benefits are greater than project costs). The method chosen for application will depend on:

- The need to consider budget and/or other constraints within the prioritization, and
- The type of software accessible, which could be as simple as a spreadsheet or as complex as specialized software designed for the method.

Basic Optimization Methods

There are three specific optimization methods that can potentially be used for prioritization of safety projects. These are:

- Linear programming (LP) optimization
- Integer programming (IP) optimization

- Dynamic programming (DP) optimization

Each of these optimization methods uses a mathematical technique for identifying an optimal combination of projects or project alternatives within user-specified constraints (such as an available budget for safety improvement). *Appendix A* provides a more detailed description of these three optimization methods.

In recent years, integer programming is the most widely used of these three optimization methods for highway safety applications. Optimization problems formulated as integer programs can be solved with Microsoft Excel or with other commercially available software packages. A general purpose optimization tool based on integer programming is available in the FHWA *Safety Analyst* software tools for identifying an optimal set of safety improvement projects to maximize benefits within a budget constraint (www.safetyanalyst.org). A special-purpose optimization tool known as the Resurfacing Safety Resource Allocation Program (RSRAP) is available for identifying an optimal set of safety improvements for implementation in conjunction with pavement resurfacing projects.[(2)]

Multiobjective Resource Allocation

The optimization and ranking methods discussed above are all directly applicable to project prioritization where reducing crashes is the only objective being considered. However, in many decisions concerning highway improvement projects, reducing crashes is just one of many factors that influence project selection and prioritization. Many highway investment decisions that are influenced by multiple factors are based on judgments by decision makers once all of the factors have been listed and, to the extent feasible, quantified.

A class of decision-making algorithms known as multiobjective resource allocation can be used to address such decisions quantitatively. Multiobjective resource allocation can optimize multiple objective functions, including objectives that may be expressed in different units. For example, these algorithms can consider safety objectives in terms of crashes reduced; traffic operational objectives in terms of vehicle-hours of delay reduced; air quality benefits in terms of pollutant concentrations reduced; and noise benefits in terms of noise levels reduced. Thus, multiobjective resource allocation provides a method to consider non-monetary factors, like those discussed in *Chapter 7*, in decision making.

All multiobjective resource allocation methods require the user to assign weights to each objective under consideration. These weights are considered during the optimization to balance the multiple objectives under consideration. As with the basic optimization methods, in the multiobjective resource allocation method an optimal project set is reached by using an algorithm to minimize or maximize the weighted objectives subject to constraints, such as a budget limit.

Examples of multiobjective resource allocation methods for highway engineering applications include Interactive Multiobjective Resource Allocation (IMRA) and Multicriteria Cost-Benefit Analysis (MCCBA).[(1,4)]

8.2.3. Summary of Prioritization Methods

Exhibit 8-3 provides a summary of the prioritization methods described in Section 8.2.

Exhibit 8-3: Summary of Project Prioritization Methods

Method	Input Needs	Outcomes	Considerations
Ranking by Safety-Related Measures	Various; inputs are readily available and/or derived using the methods presented in Chapter 7.	A ranked list or lists of projects based on various cost and/or benefit factors.	• The prioritization can be improved by using a number of ranking criteria. • Not effective for prioritizing many project alternatives or projects across many sites. • The list is not necessarily optimized for a given budget.
Incremental Benefit-Cost Analysis	Present value of monetary benefits and costs for economically justified projects. Spreadsheet and/or a software program.	A ranked list of projects based on the benefits they provide and their cost.	• Multiple benefit cost ratio calculations. • Spreadsheet or software is useful to automate and track the calculations. • The list is not necessarily optimized for a given budget.
Linear Programming (LP)	Present value of monetary benefits and costs for economically justified projects. Spreadsheet and/or a software program.	An optimized list of projects that provide: 1) Maximum benefits for a given budget, or 2) Minimum cost for a predetermined benefit.	• Generally most applicable to roadway projects without defined limits. • Microsoft Excel can be used to solve LP problems for a limited set of values. • Other computer software packages are available to solve LP problems that have many variables. • There are no generally available LP packages specifically customized for highway safety applications.
Integer Programming (IP)	Present value of monetary benefits and costs for economically justified projects. Spreadsheet and/or software program.	An optimized list of projects that provide: 1) Maximum benefits for a given budget, or 2) Minimum cost for a predetermined benefit.	• Generally most applicable to projects with fixed bounds. • Microsoft Excel can be used to solve IP problems for a limited set of values. • Other computer software packages are available to efficiently solve IP problems. • SafetyAnalyst and RSRAP provide IP packages developed specifically for highway safety applications.
Dynamic Programming (DP)	Present value of monetary benefits and costs for economically justified projects. Software program to solve the DP problem.	An optimized list of projects that provide: 1) Maximum benefits for a given budget, or 2) Minimum cost for a predetermined benefit.	• Computer software is needed to efficiently solve DP problems.
Multiobjective Resource Allocation	Present value of monetary benefits and costs for economically justified projects. Software program to solve the multiobjective problem.	A set of projects that optimizes multiple project objectives, including safety and other decision criteria, simultaneously in accordance with user-specified weights for each project objectives.	• Computer software is needed to efficiently solve multiobjective problems. • User must specify weights for each project objective, including crash reduction measures and other decision criteria.

The methods presented in this chapter vary in complexity. Depending on the purpose of the study and access to specialized software for analysis, one method may be more appropriate than another. Each method is expected to provide valuable input into the roadway safety management process.

8.3. UNDERSTANDING PRIORITIZATION RESULTS

The results produced by these prioritization methods can be incorporated into the decision-making process as one key, but not necessarily definitive, piece of information. The results of these prioritization methods are influenced by a variety of factors including:

Prioritization methods are used to select among a variety of projects. This chapter provides an overview of ranking and optimization methods.

- How benefits and costs are assigned and calculated;
- The extent to which the evaluation of costs and benefits are quantified;
- The service lives of the projects being considered;
- The discount rate (i.e., the minimum rate of return); and,
- The confidence intervals associated with the predicted change in crashes.

There are also non-monetary factors to be considered, as discussed in *Chapter 7*. These factors may influence the final allocation of funds through influence on the judgments of key decision makers or through a formal multi-objective resource allocation. As with many engineering analyses, if the prioritization process does not reveal a clear decision, it may be useful to conduct sensitivity analyses to determine incremental benefits of different choices.

8.4. SAMPLE PROBLEMS

The sample problems presented here illustrate the ranking of project alternatives across multiple sites. The linear programming, integer programming, dynamic programming, and multi-objective resource allocation optimization methods described in *Chapter 8* require the use of software and, therefore, no examples are presented here. These methods are useful to generate a prioritized list of countermeasure improvement projects at multiple sites that will optimize the number of crashes reduced within a given budget.

8.4.1. The Situation

The highway agency has identified safety countermeasures, benefits, and costs for the intersections and segments shown in Exhibit 8-4.

Exhibit 8-4: Intersections and Roadway Segments Selected for Further Review

Intersections	Traffic Control	Number of Approaches	Major AADT	Minor AADT	Urban/ Rural	Crash Data		
						Total Year 1	Total Year 2	Total Year 3
2	TWSC	4	22,100	1,650	U	9	11	15
7	TWSC	4	40,500	1,200	U	11	9	14
11	Signal	4	42,000	1,950	U	12	15	11
12	Signal	4	46,000	18,500	U	10	14	8
Segments	**Cross-Section (Number of Lanes)**	**Segment Length (miles)**	**AADT**		**Undivided/ Divided**	**Crash Data (Total)**		
						Year 1	**Year 2**	**Year 3**
1	2	0.60	9,000		U	16	15	14
2	2	0.40	15,000		U	12	14	10
5	4	0.35	22,000		U	18	16	15
6	4	0.30	25,000		U	14	12	10
7	4	0.45	26,000		U	12	11	13

Exhibit 8-5 summarizes the countermeasure, benefits, and costs for each of the sites selected for further review. The present value of crash reduction was calculated for Intersection 2 in *Chapter 7*. Other crash costs represent theoretical values developed to illustrate the sample application of the ranking process.

Exhibit 8-5: Summary of Countermeasure, Crash Reduction, and Cost Estimates for Selected Intersections and Roadway Segments

Intersection	Countermeasure	Present Value of Crash Reduction	Cost Estimate
2	Single-Lane Roundabout	$33,437,850	$695,000
7	Add Right Turn Lane	$1,200,000	$200,000
11	Add Protected Left Turn	$1,400,000	$230,000
12	Install Red Light Cameras	$1,800,000	$100,000
Segment	**Countermeasure**	**Present Value of Safety Benefits**	**Cost Estimate**
1	Shoulder Rumble Strips	$3,517,400	$250,000
2	Shoulder Rumble Strips	$2,936,700	$225,000
5	Convert to Divided	$7,829,600	$3,500,000
6	Convert to Divided	$6,500,000	$2,750,000
7	Convert to Divided	$7,000,000	$3,100,000

8.4.2. The Question

Which safety improvement projects would be selected based on ranking the projects by Cost-Effectiveness, Net Present Value (NPV), and Benefit-Cost Ratio (BCR) measures?

8.4.3. The Facts

Exhibit 8-6 summarizes the crash reduction, monetary benefits and costs for the safety improvement projects being considered.

Exhibit 8-6: Project Facts

Location	Estimated Average Reduction in Crash Frequency	Present Value of Crash Reduction	Cost Estimate
Intersection 2	47	$33,437,850	$695,000
Intersection 7	6	$1,200,000	$200,000
Intersection 11	7	$1,400,000	$230,000
Intersection 12	9	$1,800,000	$100,000
Segment 1	18	$3,517,400	$250,000
Segment 2	16	$2,936,700	$225,000
Segment 5	458	$7,829,600	$3,500,000
Segment 6	110	$6,500,000	$2,750,000
Segment 7	120	$7,000,000	$3,100,000

8.4.4. Solution

The evaluation and prioritization of the intersection and roadway-segment projects are both presented in this set of examples. An additional application of the methods could be to rank multiple countermeasures at a single intersection or segment; however, this application is not demonstrated in the sample problems as it is an equivalent process.

Simple Ranking - Cost-Effectiveness

STEP 1 – Estimate Crash Reduction

Divide the cost of the project by the total estimated crash reduction as shown in Equation 8-1.

Cost-Effectiveness = Cost of the project/Total crashes reduced (8-1)

Exhibit 8-7 summarizes the results of this method.

Exhibit 8-7: Cost-Effectiveness Evaluation

Project	Total	Cost	Cost Effectiveness (Cost/Crash Reduced)
Intersection 2	47	$695,000	$14,800
Intersection 7	6	$200,000	$33,300
Intersection 11	7	$230,000	$32,900
Intersection 12	9	$100,000	$11,100
Segment 1	18	$250,000	$14,000
Segment 2	16	$225,000	$14,100
Segment 5	458	$3,500,000	$7,600
Segment 6	110	$2,750,000	$25,000
Segment 7	120	$3,100,000	$25,800

STEP 2 – Rank Projects by Cost-Effectiveness

The improvement project with the lowest cost-effective value is the most cost-effective at reducing crashes. Exhibit 8-8 shows the countermeasure implementation projects listed based on simple cost-effectiveness ranking.

Exhibit 8-8: Cost-Effectiveness Ranking

Project	Cost-Effectiveness
Segment 5	$7,600
Intersection 12	$11,100
Segment 1	$14,000
Segment 2	$14,100
Intersection 2	$14,800
Segment 6	$25,000
Segment 7	$25,800
Intersection 11	$32,900
Intersection 7	$33,300

Simple Ranking - Net Present Value (NPV)

The net present value (NPV) method is also referred to as the net present worth (NPW) method. This method is used to express the difference between discounted costs and discounted benefits of an individual improvement project in a single amount.

STEP 1 - Calculate the NPV

Subtract the cost of the project from the benefits as shown in Equation 8-2.

$$NPV = \text{Present Monetary Value of the Benefits} - \text{Cost of the project} \quad (8\text{-}2)$$

STEP 2 - Rank Sites Based on NPV

Rank sites based on the NPV as shown in Exhibit 8-9.

Exhibit 8-9: Net Present Value Results

Project	Present Value of Benefits ($)	Cost of Improvement Project ($)	Net Present Value
Intersection 2	$33,437,850	$695,000	$32,742,850
Segment 5	$7,829,600	$3,500,000	$4,329,600
Segment 7	$7,000,000	$3,100,000	$3,900,000
Segment 6	$6,500,000	$2,750,000	$3,750,000
Segment 1	$3,517,400	$250,000	$3,267,400
Segment 2	$2,936,700	$225,000	$2,711,700
Intersection 12	$1,800,000	$100,000	$1,700,000
Intersection 11	$1,400,000	$230,000	$1,170,000
Intersection 7	$1,200,000	$200,000	$1,000,000

As shown in Exhibit 8-9, Intersection 2 has the highest net present value out of the intersection and roadway segment projects being considered.

All of the improvement projects have net present values greater than zero, indicating they are economically feasible projects because the monetary benefit is greater than the cost. It is possible to have projects with net present values less than zero, indicating that the calculated monetary benefits do not outweigh the cost of the project. The highway agency may consider additional benefits (both monetary and non-monetary) that may be brought about by the projects before implementing them.

Incremental Benefit-Cost Analysis

Incremental benefit-cost analysis is an extension of the benefit-cost ratio (BCR) method presented in *Chapter 7*.

STEP 1 – Calculate the BCR

Chapter 7, Section 7.6.1.2 illustrates the process for calculating the BCR for each project.

STEP 2 – Organize Projects by Project Cost

The incremental analysis is applied to pairs of projects ordered by project cost, as shown in Exhibit 8-10.

Exhibit 8-10: Cost of Improvement Ranking

Project	Cost of Improvement
Intersection 12	$100,000
Intersection 7	$200,000
Segment 2	$225,000
Intersection 11	$230,000
Segment 1	$250,000
Intersection 2	$695,000
Segment 6	$2,750,000
Segment 7	$3,100,000
Segment 5	$3,500,000

STEP 3 – Calculate Incremental BCR

Equation 8-3 is applied to a series of project pairs ordered by cost. If the incremental BCR is greater than 1.0, the higher-cost project is preferred to the lower-cost project. If the incremental BCR is a positive value less than 1.0, or is zero or negative, the lower-cost project is preferred to the higher-cost project. The computations then proceed comparing the preferred project from the first comparison to the project with the next highest cost. The preferred alternative from the final comparison is assigned the highest priority. The project with the second-highest priority is then determined by applying the same computational procedure but omitting the highest priority project.

$$\text{Incremental BCR} = (PV_{benefits\ 2} - PV_{benefits\ 1}) / (PV_{costs\ 2} - PV_{costs\ 1}) \tag{8-3}$$

Where,

$PV_{benefits\ 1}$ = Present value of benefits for lower-cost project

$PV_{benefits\ 2}$ = Present value of benefits for higher-cost project

$PV_{costs\ 1}$ = Present value of cost for lower-cost project

$PV_{costs\ 2}$ = Present value of cost for higher-cost project

Exhibit 8-11 illustrates the sequence of incremental benefit-cost comparisons needed to assign priority to the projects.

Exhibit 8-11: Incremental BCR Analysis

Comparison	Project	$PV_{benefits}$	PV_{costs}	Incremental BCR	Preferred Project
1	Intersection 12	$1,800,000	$100,000	-6	Intersection 12
	Intersection 7	$1,200,000	$200,000		
2	Intersection 12	$1,800,000	$100,000	9	Segment 2
	Segment 2	$2,936,700	$225,000		
3	Segment 2	$2,936,700	$225,000	-307	Segment 2
	Intersection 11	$1,400,000	$230,000		
4	Segment 2	$2,936,700	$225,000	23	Segment 1
	Segment 1	$3,517,400	$250,000		
5	Segment 1	$3,517,400	$250,000	67	Intersection 2
	Intersection 2	$33,437,850	$695,000		
6	Intersection 2	$33,437,850	$695,000	-13	Intersection 2
	Segment 6	$6,500,000	$2,750,000		
7	Intersection 2	$33,437,850	$695,000	-11	Intersection 2
	Segment 7	$7,000,000	$3,100,000		
8	Intersection 2	$33,437,850	$695,000	-9	Intersection 2
	Segment 5	$7,829,600	$3,500,000		

As shown by the comparisons in Exhibit 8-11, the improvement project for Intersection 2 receives the highest priority. In order to assign priorities to the remaining projects, another series of incremental calculations is performed, each time omitting the projects previously prioritized. Based on multiple iterations of this method, the projects were ranked as shown in Exhibit 8-12.

Exhibit 8-12: Ranking Results of Incremental BCR Analysis

Rank	Project
1	Intersection 2
2	Intersection 5
3	Intersection 7
4	Segment 6
5	Segment 1
6	Intersection 2
7	Segment 12
8	Segment 1

Comments

The ranking of the projects by incremental benefit-cost analysis differs from the project rankings obtained with cost-effectiveness and net present value computations. Incremental benefit-cost analysis provides greater insight into whether the expenditure represented by each increment of additional cost is economically justified. Incremental benefit-cost analysis provides insight into the priority ranking of alternative projects, but does not lend itself to incorporating a formal budget constraint.

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APPENDIX A – BASIC OPTIMIZATION METHODS DISCUSSED IN CHAPTER 8

A.1 Linear Programming (LP)

Linear programming is a method commonly used to allocate limited resources to competing activities in an optimal manner. With respect to evaluating improvement projects, the limited resource is funds, the competing activities are different improvement projects, and an optimal solution is one in which benefits are maximized.

A linear program typically consists of a linear function to be optimized (known as the objective function), a set of decision variables that specify possible alternatives, and constraints that define the range of acceptable solutions. The user specifies the objective function and the constraints and an efficient mathematical algorithm is applied to determine the values of the decision variables that optimize the objective function without violating any of the constraints. In an application for highway safety, the objective function represents the relationship between benefits and crash reductions resulting from implementation.

Typical optimization methods are: linear programming, integer programming, dynamic programming, and multi-objective resource allocation.

The constraints put limits on the solutions to be considered. For example, constraints might be specified so that incompatible project alternatives would not be considered at the same site. Another constraint for most highway safety applications is that it is often infeasible to have negative values for the decision variables (e.g., the number of miles of a particular safety improvement type that will be implemented can be zero or positive, but cannot be negative). The key constraint in most highway safety applications is that the total cost of the alternatives selected must not exceed the available budget. Thus, an optimal solution for a typical highway safety application would be decision-variable values that represent the improvements which provide the maximum benefits within the available budget.

An optimized linear programming objective function contains continuous (i.e., non-discrete) values of the decision variables, so is most applicable to resource allocation problems for roadway segments without predefined project limits. A linear program could be used to determine an optimum solution that indicates, for example, how many miles of lane widening or shoulder widening and paving would provide maximum benefits within a budget constraint.

While there are methods to manually find an optimized solution, computer software programs are typically employed. Microsoft Excel can solve LP problems for a limited set of variables, which is sufficient for simple applications. Other commercial packages with a wide range of capabilities for solving linear programs are also available.

Linear programming has been applied to highway safety resource allocation. Kar and Datta used linear programming to determine the optimal allocation of funding to cities and townships in Michigan based on their crash experience and anticipated crash reductions from safety programs.[4] However, there are no widely available software tools that apply linear programming specifically to decisions related to highway safety. Also, there are no known applications of linear programming in use for prioritizing individual safety improvement projects because integer programming, as described below, is more suited for this purpose.

A.2 Integer Programming (IP)

Integer programming is a variation of linear programming. The primary difference is that decision variables are restricted to integer values. Decision variables often represent quantities that are only meaningful as integer values, such as people, vehicles, or machinery. Integer programming is the term used to represent an instance of linear programming when at least one decision variable is restricted to an integer value.

The two primary applications of integer programming are:

- Problems where it is only practical to have decision variables that are integers; and,
- Problems that involve a number of interrelated "yes or no" decisions such as whether to undertake a specific project or make a particular investment. In these situations there are only two possible answers, "yes" or "no," which are represented numerically as 1 and 0, respectively, and known as binary variables.

Integer programming with binary decision variables is particularly applicable to highway safety resource allocation because a series of "yes" or "no" decisions are typically required (i.e., each project alternative considered either will or will not be implemented). While linear programming may be most appropriate for roadway projects with undetermined length, integer programming may be most appropriate for intersection alternatives or roadway projects with fixed bounds. An integer program could be used to determine the optimum solution that indicates, for example, if and where discrete projects, such as left-turn lanes, intersection lighting, and a fixed length of median barrier, would provide maximum benefits within a budget constraint. Because of the binary nature of project decision making, integer programming has been implemented more widely than linear programming for highway safety applications.

As in the case of linear programming, an integer program would also include a budget limit and a constraint to assure that incompatible project alternatives are not selected for any given site. The objective for an integer program for highway safety resource allocation would be to maximize the benefits of projects within the applicable constraints, including the budget limitation. Integer programming could also be applied to determine the minimum cost of projects that achieve a specified level of benefits, but there are no known applications of this approach.

Integer programs can be solved with Microsoft Excel or with other commercially available software packages. A general purpose optimization tool based on integer programming is available in the FHWA *Safety Analyst* software tools for identifying an optimal set of safety improvement projects to maximize benefits within a budget constraint (www.safetyanalyst.org). A special-purpose optimization tool known as the Resurfacing Safety Resource Allocation Program (RSRAP) is available for identifying an optimal set of safety improvements for implementation in conjunction with pavement resurfacing projects.[3]

A.3 Dynamic Programming (DP)

Dynamic programming is another mathematical technique used to make a sequence of interrelated decisions to produce an optimal condition. Dynamic programming problems have a defined beginning and end. While there are multiple paths and options between the beginning and end, only one optimal set of decisions will move the problem toward the desired solution.

The basic theory of dynamic programming is to solve the problem by solving a small portion of the original problem and finding the optimal solution for that small portion. Once an optimal solution for the first small portion is found, the problem is enlarged and the optimal solution for the current problem is found from the preceding solution. Piece by piece, the problem is enlarged and solved until the entire original problem is solved. Thus, the mathematical principle used to determine the optimal solution for a dynamic program is that subsets of the optimal path through the maze must themselves be optimal.

Most dynamic programming problems are sufficiently complex that computer software is typically used. Dynamic programming was used for resource allocation in Alabama in the past and remains in use for highway safety resource allocation in Kentucky.[1,2]

A.4 Appendix References

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PART B—ROADWAY SAFETY MANAGEMENT PROCESS

CHAPTER 9—SAFETY EFFECTIVENESS EVALUATION

9.1. Chapter Overview 9-4

9.2. Safety Effective Evaluation – Definition and Purpose 9-4

9.3. Study Design and Methods 9-5

- 9.3.1. Observational Before/After Evaluation Studies 9-6
- 9.3.2. Observational Before/After Evaluation Studies Using SPFs – the Empirical Bayes Method 9-7
- 9.3.3. Observational Before/After Evaluation Study Using the Comparison-Group Method 9-8
- 9.3.4. Observational Before/After Evaluation Studies to Evaluate Shifts in Collision Crash Type Proportions 9-9
- 9.3.5. Observational Cross-Sectional Studies 9-9
- 9.3.6. Selection Guide for Observational Before/After Evaluation Study Methods 9-10
- 9.3.7. Experimental Before/After Evaluation Studies 9-11

9.4. Procedures to Implement Safety Evaluation Methods 9-11

- 9.4.1. Implementing the EB Before/After Safety Evaluation Method 9-12
- 9.4.2. Implementing the Before/After Comparison-Group Safety Evaluation Method 9-15
- 9.4.3. Implementing the Safety Evaluation Method for Before/After Shifts in Proportions of Target Collision Types 9-17
- 9.4.4. Implementing the Cross-Sectional Safety Evaluation Method 9-19

9.5. Evaluating a Single Project at a Specific Site to Determine its Safety Effectiveness 9-21

9.6. Evaluating a Group of Similar Projects to Determine their Safety Effectiveness 9-21

9.7. Quantifying AMFs as a Result of a Safety Effectiveness Evaluation 9-22

9.8. Comparison of Safety Benefits and Costs of Implemented Projects 9-22

9.9. Conclusions 9-23

9.10. Sample Problem to Illustrate the EB BEFORE/AFTER Safety Effectiveness Evaluation Method 9-24

- 9.10.1. Basic Input Data 9-24
- 9.10.2. EB Estimation of the Expected Crash Frequency in the Before Period 9-25
- 9.10.3. EB Estimation of the Expected Crash Frequency in the After Period in the Absence of the Treatment 9-27
- 9.10.4. Estimation of the Treatment Effectiveness 9-28
- 9.10.5. Estimation of the Precision of the Treatment Effectiveness 9-30

9.11. Sample Problem to Illustrate the COMPARISON-GROUP Safety Effectiveness Evaluation Method 9-31
9.11.1. Basic Input Data for Treatment Sites 9-31
9.11.2. Basic Input Data for Comparison Group Sites 9-32
9.11.3. Estimation of Mean Treatment Effectiveness 9-32
9.11.4. Estimation of the Overall Treatment Effectiveness and its Precision 9-39
9.12. Sample Problem to Illustrate the SHIFT OF PROPORTIONS Safety Effectiveness Evaluation Method 9-41
9.12.1. Basic Input Data 9-41
9.12.2. Estimate the Average Shift in Proportion of the Target Collision Type 9-42
9.12.3. Assess the Statistical Significance of the Average Shift in Proportion of the Target collision type 9-42
9.13. References 9-45

EXHIBITS

Exhibit 9-1: Roadway Safety Management Overview Process 9-4

Exhibit 9-2: Generic Evaluation Study Design 9-6

Exhibit 9-3: Observational Before/After Evaluation Study Design 9-7

Exhibit 9-4: Observational Cross-Sectional Evaluation Study Design 9-10

Exhibit 9-5: Selection Guide for Observational Before/After Evaluation Methods 9-10

Exhibit 9-6: Experimental Before/After Evaluation Study Design 9-11

Exhibit 9-7: Overview of Data Needs and Inputs for Safety Effectiveness Evaluations 9-12

Exhibit 9-8: Overview of EB Before/After Safety Evaluation.....................................9-13

Exhibit 9-9: Overview of Before/After Comparison-Group Safety Evaluation 9-16

Exhibit 9-10: Overview Safety Evaluation for Before/After Shifts in Proportions 9-18

Exhibit 9-11: Overview of Safety Benefits and Costs Comparison of Implemented Projects 9-23

Exhibit 9-12: Upper Tail Probabilities for the Wilcoxon Signed Rank T+ Statistic (n* = 4 to 10)a (8) A-12

1

APPENDIX A

A.1 Computational Procedure for Implementing the EB BEFORE/AFTER Safety Effectiveness Evaluation Method 1

A.2 Computational Procedure for Implementing the COMPARISON-GROUP Safety Effectiveness Evaluation Method 5

A.3 Computational Procedure for Implementing the SHIFT OF PROPORTIONS Safety Effectiveness Evaluation Method 9

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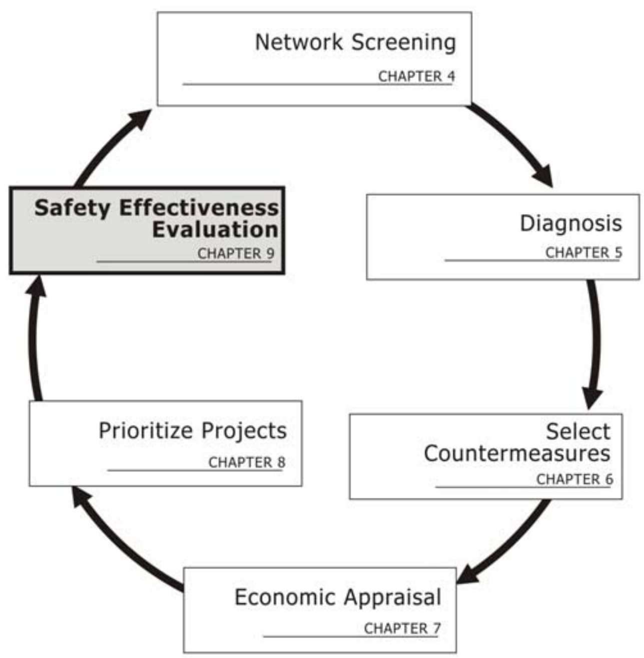
CHAPTER 9 SAFETY EFFECTIVENESS EVALUATION

9.1. CHAPTER OVERVIEW

Evaluating the change in crashes from implemented safety treatments is an important step in the roadway safety evaluation process (see Exhibit 9-1). Safety evaluation leads to an assessment of how crash frequency or severity has changed due to a specific treatment, or a set of treatments or projects. In situations where one treatment is applied at multiple similar sites, safety evaluation can also be used to estimate an accident modification factor (AMF) for the treatment. Finally, safety effectiveness evaluations have an important role in assessing how well funds have been invested in safety improvements. Each of these aspects of safety effectiveness evaluation may influence future decision-making activities related to allocation of funds and revisions to highway agency policies.

Exhibit 9-1: Roadway Safety Management Overview Process



This chapter explains the methods for evaluating the effectiveness of treatment(s) in reducing crash frequency or severity.

The purpose of this chapter is to document and discuss the various methods for evaluating the effectiveness of a treatment, a set of treatments, an individual project, or a group of similar projects after improvements have been implemented to reduce crash frequency or severity. This chapter provides an introduction to the evaluation methods that can be used; highlights which methods are appropriate for assessing safety effectiveness in specific situations; and provides step-by-step procedures for conducting safety effectiveness evaluations.

9.2. SAFETY EFFECTIVE EVALUATION – DEFINITION AND PURPOSE

Safety effectiveness evaluation is the process of developing quantitative estimates of how a treatment, project, or a group of projects has affected crash frequencies or severities. The effectiveness estimate for a project or treatment is a valuable piece of information for future safety decision-making and policy development.

Safety effectiveness evaluation may include:

- Evaluating a single project at a specific site to document the safety effectiveness of that specific project;
- Evaluating a group of similar projects to document the safety effectiveness of those projects;
- Evaluating a group of similar projects for the specific purpose of quantifying an AMF for a countermeasure; and
- Assessing the overall safety effectiveness of specific types of projects or countermeasures in comparison to their costs.

The purpose of safety effectiveness evaluations are summarized here.

If a particular countermeasure has been installed on a system-wide basis, such as the installation of cable median barrier or shoulder rumble strips for the entire freeway system of a jurisdiction, a safety effectiveness evaluation of such a program would be conducted no differently than an evaluation of any other group of similar projects.

Safety effectiveness evaluations may use several different types of performance measures, such as a percentage reduction in crashes, a shift in the proportions of crashes by collision type or severity level, an AMF for a treatment, or a comparison of the safety benefits achieved to the cost of a project or treatment.

The next section presents an overview of available evaluation study designs and their corresponding evaluation methods. Detailed procedures for applying those methods are presented in Section 9.4 and the Appendix to this chapter. Sections 9.5 through 9.8, respectively, describe how the evaluation study designs and methods for each of the evaluation types identified above are implemented.

9.3. STUDY DESIGN AND METHODS

To evaluate the effectiveness of a treatment in reducing crash frequency or severity, the treatment must have been implemented for at least one and, preferably, many sites. Selection of the appropriate study design for a safety effectiveness evaluation depends on the nature of the treatment, the type of sites at which the treatment has been implemented, and the time periods for which data are available for those sites (or will become available in the future). The evaluation is more complex than simply comparing before and after crash data at treatment sites because consideration is also given to what changes in crash frequency would have occurred at the evaluation sites between the time periods before and after the treatment even if the treatment had not been implemented. Many factors that can affect crash frequency may change over time, including changes in traffic volumes, weather, and driver behavior. General trends in crash frequency can also affect both improved and unimproved sites. For this reason, most evaluations use data for both treatment and nontreatment sites. Information can be directly obtained by collecting data on such sites or by making use of safety performance functions for sites with comparable geometrics and traffic patterns.

Exhibit 9-2 presents a generic evaluation study design layout that will be used throughout the following discussion to explain the various study designs that can be used in safety effectiveness evaluation. As the exhibit indicates, study designs usually use data (crash and traffic volume) for both treatment and nontreatment sites and for time periods both before and after the implementation of the treatments. Even though no changes are made intentionally to the nontreatment sites, it is useful to have data for such sites during time periods both before and after improvement of the treatment sites so that general time trends in crash data can be accounted for.

Exhibit 9-2: Generic Evaluation Study Design

Type of Site	Before Treatment	After Treatment
Treatment Sites		
Nontreatment Sites		

This section provides an overview of three basic safety effectiveness evaluation types: observational before/after studies, observational cross-sectional studies, and experimental before/after studies.

There are three basic study designs that are used for safety effectiveness evaluations:

- Observational before/after studies
- Observational cross-sectional studies
- Experimental before/after studies

Both observational and experimental studies are used in safety effectiveness evaluations. In observational studies, inferences are made from data observations for treatments that have been implemented by highway agencies in the normal course of the efforts to improve the road system, not treatments that have been implemented specifically so they can be evaluated. By contrast, experimental studies consider treatments that have been implemented specifically so that their effectiveness can be evaluated. In experimental studies, sites that are potential candidates for improvement are randomly assigned to either a treatment group, at which the treatment of interest is implemented, or a comparison group, at which the treatment of interest is not implemented. Subsequent differences in crash frequency between the treatment and comparison groups are directly attributed to the treatment. Observational studies are much more common in road safety than experimental studies, because highway agencies are generally reluctant to use random selection in assigning treatments. For this reason, the focus of this chapter is on observational studies.

Each of the observational and experimental approaches to evaluation studies are explained below.

9.3.1. Observational Before/After Evaluation Studies

Observational before/after studies are the most common approach used for safety effectiveness evaluation. An example situation that warrants an observational before/after study is when an agency constructs left-turn lanes at specific locations on a two-lane highway where concerns about crash frequency had been identified. Exhibit 9-3 shows the evaluation study design layout for an observational before/after study to identify the effectiveness of the left-turn lanes in reducing crash frequency or severity.

All observational before/after studies use crash and traffic volume data for time periods before and after improvement of the treated sites. The treatment sites do not need to have been selected in a particular way; they are typically sites of projects implemented by highway agencies in the course of their normal efforts to improve the operational and safety performance of the highway system. However, if the sites were selected for improvement because of unusually high crash frequencies, then using these sites as the treatment sites may introduce a selection bias which could result in a high regression-to-the-mean bias since treatment was not randomly assigned to sites. *Chapter 3* of the HSM provides more information about issues associated with regression-to-the-mean bias.

As shown in Exhibit 9-3, the nontreatment sites (i.e. comparison sites) – sites that were not improved between the time periods before and after improvement of the treatment sites – may be represented either by SPFs or by crash and traffic volume data. Evaluation study design using these alternative approaches for consideration of non-treatment sites are not discussed below.

Exhibit 9-3: Observational Before/After Evaluation Study Design

Type of Site	Before Treatment	After Treatment
Treatment Sites	✓	✓
Non-treatment Sites (SPF or comparison group)	✓	✓

Observational before/after studies are the most common approach used for safety effectiveness evaluation.

If an observational before/after evaluation is conducted without any consideration of nontreatment sites (i.e., with no SPFs and no comparison group), this is referred to as a simple or naïve before/after evaluation. Such evaluations do not compensate for regression-to-the-mean bias (see *Chapter 3*) or compensate for general time trends in the crash data.

Naïve before/after evaluations are not recommended because they do not compensate for regression-to-the-mean bias.

9.3.2. Observational Before/After Evaluation Studies Using SPFs – the Empirical Bayes Method

Observational before/after evaluation studies that include non-treatment sites are conducted in one of two ways. The empirical Bayes method is most commonly used. This approach to evaluation studies uses SPFs to estimate what the average crash frequency at the treated sites would have been during the time period after implementation of the treatment, had the treatment not been implemented.

In cases where the treated sites were selected by the highway agency for improvement because of unusually high crash frequencies, this constitutes a selection bias which could result in a high regression-to-the-mean bias in the evaluation. The use of the EB approach, which can compensate for regression-to-the–mean bias, is particularly important in such cases.

EB Method for observational before/after studies is the most common safety effectiveness evaluation study type.

Chapter 3 presents the basic principles of the EB method which is used to estimate a site's expected average crash frequency. The EB method combines a site's observed crash frequency and SPF-based predicted average crash frequency to estimate the expected average crash frequency for that site in the after period had the treatment not been implemented. The comparison of the observed after crash frequency to the expected average after crash frequency estimated with the EB method is the basis of the safety effectiveness evaluation.

A key advantage of the EB method for safety effectiveness evaluation is that existing SPFs can be used. There is no need to collect crash and traffic volume data for nontreatment sites and develop a new SPF each time a new evaluation is performed. However, if a suitable SPF is not available, one can be developed by assembling crash and traffic volume data for a set of comparable nontreatment sites.

The EB method has been explained for application to highway safety effectiveness evaluation by Hauer[5,6] and has been used extensively in safety effectiveness evaluations[2,8,10]. The EB method implemented here is similar to that used in the FHWA *SafetyAnalyst* software tools[3]. Detailed procedures for performing an observational before/after study with SPFs to implement the EB method are presented in Section 9.4.1 and the Appendix to this chapter.

9.3.3. Observational Before/After Evaluation Study Using the Comparison-Group Method

Observational before/after studies may incorporate nontreatment sites into the evaluation as a comparison group. In a before/after comparison-group evaluation method, the purpose of the comparison group is to estimate the change in crash frequency that would have occurred at the treatment sites if the treatment had not been made. The comparison group allows consideration of general trends in crash frequency or severity whose causes may be unknown, but which are assumed to influence crash frequency and severity at the treatment and comparison sites equally. Therefore, the selection of an appropriate comparison group is a key step in the evaluation.

Comparison groups used in before/after evaluations have traditionally consisted of nontreated sites that are comparable in traffic volume, geometrics, and other site characteristics to the treated sites, but without the specific improvement being evaluated. Hauer[5] makes the case that the requirement for matching comparison sites with respect to site characteristics, such as traffic volumes and geometrics, is secondary to matching the treatment and comparison sites based on their crash frequencies over time (multiple years). Matching on the basis of crash frequency over time generally uses crash data for the period before treatment implementation. Once a set of comparison sites that are comparable to the treatment sites has been identified, crash and traffic volume data are needed for the same time periods as are being considered for the treated sites.

Obtaining a valid comparison group is essential when implementing an observational before/after evaluation study using the comparison-group method. It is therefore important that agreement between the treatment group and comparison group data in the yearly time series of crash frequencies during the period before implementation of the treatment be confirmed. During the before period, the rate of change in crashes from year to year should be consistent between a particular comparison group and the associated treatment group. A statistical test using the yearly time series of crash frequencies at the treatment and comparison group sites for the before period is generally used to assess this consistency. Hauer[5] provides a method to assess whether a candidate comparison group is suitable for a specific treatment group.

While the comparison-group method does not use SPF(s) in the same manner as the EB method, SPF(s) are desirable to compute adjustment factors for the nonlinear effects of changes in traffic volumes between the before and after periods.

The before/after comparison-group evaluation method has been explained for application to highway safety effectiveness evaluation by Griffin[1] and by Hauer[5]. A variation of the before/after comparison-group method to handle adjustments to compensate for varying traffic volumes and study period durations between the before and after study periods and between the treatment and comparison sites was formulated by Harwood et al.[2]. Detailed procedures for performing an observational before/after study with the comparison group method are presented in Section 9.4.2 and the Appendix to this chapter.

9.3.4. Observational Before/After Evaluation Studies to Evaluate Shifts in Collision Crash Type Proportions

Observational before/after studies can also be used to test for a change in frequency of a specific collision type.

An observational before/after evaluation study is used to assess whether a treatment has resulted in a shift in the frequency of a specific target collision type as a proportion of total crashes from before to after implementation of the treatment. The target collision types addressed in this type of evaluation may include specific crash severity levels or crash types. The procedures used to assess shifts in proportion are those used in the FHWA *SafetyAnalyst* software tools[3]. The assessment of the statistical significance of shifts in proportions for target collision types is based on the Wilcoxon signed rank test[7]. Detailed procedures for performing an observational before/after evaluation study to assess shifts in crash severity level or crash type proportions are presented in Section 9.4.3 and the Appendix to this chapter.

9.3.5. Observational Cross-Sectional Studies

There are many situations in which a before/after evaluation, while desirable, is simply not feasible, including the following examples:

- When treatment installation dates are not available;
- When crash and traffic volume data for the period prior to treatment implementation are not available; or,
- When the evaluation needs to explicitly account for effects of roadway geometrics or other related features by creating an AMF function, rather than a single value for an AMF.

In such cases, an observational cross-sectional study may be applied. For example, if an agency wants to compare the safety performance of intersections with channelized right-turn lanes to intersections without channelized right-turn lanes and no sites are available that have been converted from one configuration to the other, then an observational cross-sectional study may be conducted comparing sites with these two configurations. Cross-sectional studies use statistical modeling techniques that consider the crash experience of sites with and without a particular treatment of interest (such as roadway lighting or a shoulder rumble strip) or with various levels of a continuous variable that represents a treatment of interest (such as lane width). This type of study is commonly referred to as a “with and without study.” The difference in number of crashes is attributed to the presence of the discrete feature or the different levels of the continuous variable.

Observational cross-sectional studies are used to make inferences about the effectiveness of a treatment when applied to other sites.

As shown in Exhibit 9-4, the data for a cross-sectional study is typically obtained for the same period of time for both the treatment and comparison sites. Since the treatment is obviously in place during the entire study period, a cross-sectional study might be thought of as comparable to a before/after study in which data are only available for the time period after implementation of the treatment.

Exhibit 9-4: Observational Cross-Sectional Evaluation Study Design

Type of Site	Before Treatment	After Treatment
Treatment Sites		✓
Nontreatment Sites		✓

Two cautions related to the observational cross-sectional evaluation study type: there is no good method to compensate for the potential effect of regression-to-the-mean bias, and it is difficult to assess cause and effect.

There are two substantial drawbacks to a cross-sectional study. First, there is no good method to compensate for the potential effect of regression-to-the-mean bias introduced by site selection procedures. Second, it is difficult to assess cause and effect and, therefore, it may be unclear whether the observed differences between the treatment and nontreatment sites are due to the treatment or due to other unexplained factors(4). In addition, the evaluation of the safety effectiveness requires a more involved statistical analysis approach. The recommended approach to performing observational before/after cross-sectional studies is presented in Section 9.4.4.

9.3.6. Selection Guide for Observational Before/After Evaluation Study Methods

Exhibit 9-5 presents a selection guide to the observational before/after evaluation study methods. If, at the start of a safety evaluation, the user has information on both the safety measure to be evaluated and the types of data available, then the exhibit indicates which type(s) of observational before/after evaluation studies are feasible. On the other hand, based on data availability, the information provided in Exhibit 9-5 may also guide the user in assessing additional data needs depending on a desired safety measure (i.e., crash frequency or target collision type as a proportion of total crashes).

Exhibit 9-5: Selection Guide for Observational Before/After Evaluation Methods

Safety measure to be evaluated	Data availability					Appropriate evaluation study method
	Treatment sites		Nontreatment sites			
	Before period data	After period data	Before period data	After period data	SPF	
Crash frequency	✓	✓			✓	Before/after evaluation study using the EB method
	✓	✓	✓	✓		Before/after evaluation study using either the EB method OR the comparison group method
		✓		✓		Cross-sectional study
Target collision type as a proportion of total crashes	✓	✓				Before/after evaluation study for shift in proportions

9.3.7. Experimental Before/After Evaluation Studies

Experimental studies are those in which comparable sites with respect to traffic volumes and geometric features are randomly assigned to a treatment or nontreatment group. The treatment is then applied to the sites in the treatment group, and crash and traffic volume data is obtained for time periods before and after treatment. Optionally, data may also be collected at the nontreatment sites for the same time periods. For example, if an agency wants to evaluate the safety effectiveness of a new and innovative signing treatment, then an experimental study may be conducted. Exhibit 9-6 illustrates the study design for an experimental before/after study.

Experimental study sites are randomly assigned to receive treatments or not. These study types are not feasible because of the random assignments.

Exhibit 9-6: Experimental Before/After Evaluation Study Design

Type of Site	Before Treatment	After Treatment
Treatment Sites ***Required data***	✓	✓
Nontreatment Sites (Comparison Group) ***Optional data***		

The advantage of the experimental over the observational study is that randomly assigning individual sites to the treatment or nontreatment groups minimizes selection bias and, therefore, regression-to-the-mean bias. The disadvantage of experimental studies is that sites are randomly selected for improvement. Experimental before/after evaluations are performed regularly in other fields, such as medicine, but are rarely performed for highway safety improvements because of a reluctance to use random assignment procedures in choosing improvement locations. The layout of the study design for an experimental before/after study is identical to that for an observational before/after evaluation design and the same safety evaluation methods described above and presented in more detail in Section 9.4 can be used.

9.4. PROCEDURES TO IMPLEMENT SAFETY EVALUATION METHODS

This section presents step-by-step procedures for implementing the EB and comparison-group methods for observational before/after safety effectiveness evaluations. The cross-sectional approach to observational before/after evaluation and the applicability of the observational methods to experimental evaluations are also discussed. Exhibit 9-7 provides a tabular overview of the data needs for each of the safety evaluation methods discussed in this chapter.

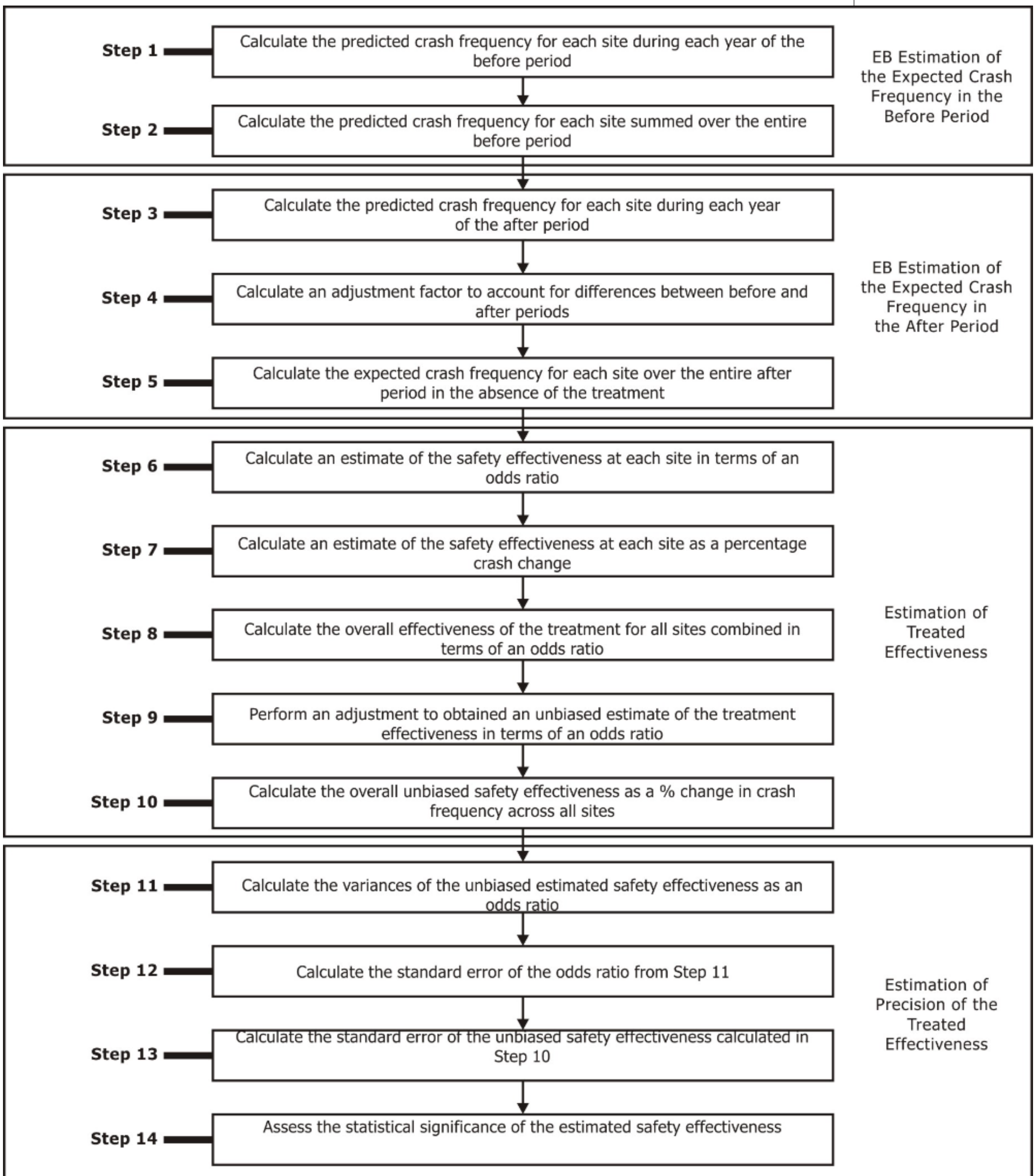
Exhibit 9-7: Overview of Data Needs and Inputs for Safety Effectiveness Evaluations

Data Needs and Inputs	Safety Evaluation Method			
	EB Before/After	**Before/After with Comparison Group**	**Before/After Shift in Proportion**	**Cross-Sectional**
10 to 20 treatment sites	✓	✓	✓	✓
10 to 20 comparable non-treatment sites		✓		✓
A minimum of 650 aggregate crashes in non-treatment sites		✓		
3 to 5 years of crash and volume "before" data	✓	✓	✓	
3 to 5 years of crash and volume "after" data	✓	✓	✓	✓
SPF for treatment site types	✓	✓		
SPF for non-treatment site types		✓		
Target crash type			✓	

9.4.1. Implementing the EB Before/After Safety Evaluation Method

The empirical Bayes (EB) before/after safety evaluation method is used to compare crash frequencies at a group of sites before and after a treatment is implemented. The EB method explicitly addresses the regression-to-the-mean issue by incorporating crash information from other but similar sites into the evaluation. This is done by using an SPF and weighting the observed crash frequency with the SPF-predicted average crash frequency to obtain an expected average crash frequency (see *Chapter 3*). Exhibit 9-8 provides a step-by-step overview of the EB before/after safety effectiveness evaluation method.

Exhibit 9-8: Overview of EB Before/After Safety Evaluation



Data Needs and Inputs

The data needed as input to an EB before/after evaluation include:

- At least 10 to 20 sites at which the treatment of interest has been implemented
- 3 to 5 years of crash and traffic volume data for the period before treatment implementation
- 3 to 5 years of crash and traffic volume for the period after treatment implementation
- SPF for treatment site types

This section summarizes how to implement the EB before/after safety evaluation. The appendix presents computations.

An evaluation study can be performed with fewer sites and/or shorter time periods, but statistically significant results are less likely.

Pre-Evaluation Activities

The key pre-evaluation activities are to:

- Identify the treatment sites to be evaluated
- Select the time periods before and after treatment implementation for each site that will be included in the evaluation.
- Select the measure of effectiveness for the evaluation. Evaluations often use total crash frequency as the measure of effectiveness, but any specific crash severity level and/or crash type can be considered.
- Assemble the required crash and traffic volume data for each site and time period of interest.
- Identify (or develop) an SPF for each type of site being developed. SPFs may be obtained from *SafetyAnalyst* or they may be developed based on the available data as described in *Part C* of the HSM. Typically, separate SPFs are used for specific types of roadway segments or intersections.

The before study period for a site must end before implementation of the treatment began at that site. The after study period for a site normally begins after treatment implementation is complete; a buffer period of several months is usually allowed for traffic to adjust to the presence of the treatment. Evaluation periods that are even multiples of 12 months in length are used so that there is no seasonal bias in the evaluation data. Analysts often choose evaluation periods consisting of complete calendar years because this often makes it easier to assemble the required data. When the evaluation periods consist of entire calendar years, the entire year during which the treatment was installed is normally excluded from the evaluation period.

Computational Procedure

A computational procedure using the EB method to determine the safety effectiveness of the treatment being evaluated, expressed as a percentage change in crashes, θ, and to assess its precision and statistical significance, is presented in the Appendix to this chapter.

9.4.2. Implementing the Before/After Comparison-Group Safety Evaluation Method

The before/after comparison-group safety evaluation method is similar to the EB before/after method except that a comparison group is used, rather than an SPF, to estimate how safety would have changed at the treatment sites had no treatment been implemented. Exhibit 9-9 provides a step-by-step overview of the before/after comparison-group safety effectiveness evaluation method.

Data Needs and Inputs

The data needed as input to a before/after comparison-group evaluation include:

- At least 10 to 20 sites at which the treatment of interest has been implemented
- At least 10 to 20 comparable sites at which the treatment has not been implemented and that have not had other major changes during the evaluation study period
- A minimum of 650 aggregate crashes at the comparable sites at which the treatment has not been implemented
- 3 to 5 years of before crash data is recommended for both treatment and nontreatment sites
- 3 to 5 years of after crash data is recommended for both treatment and nontreatment sites
- SPFs for treatment and nontreament sites

An evaluation study can be performed with fewer sites and/or shorter time periods, but statistically significant results are less likely.

Pre-Evaluation Activities

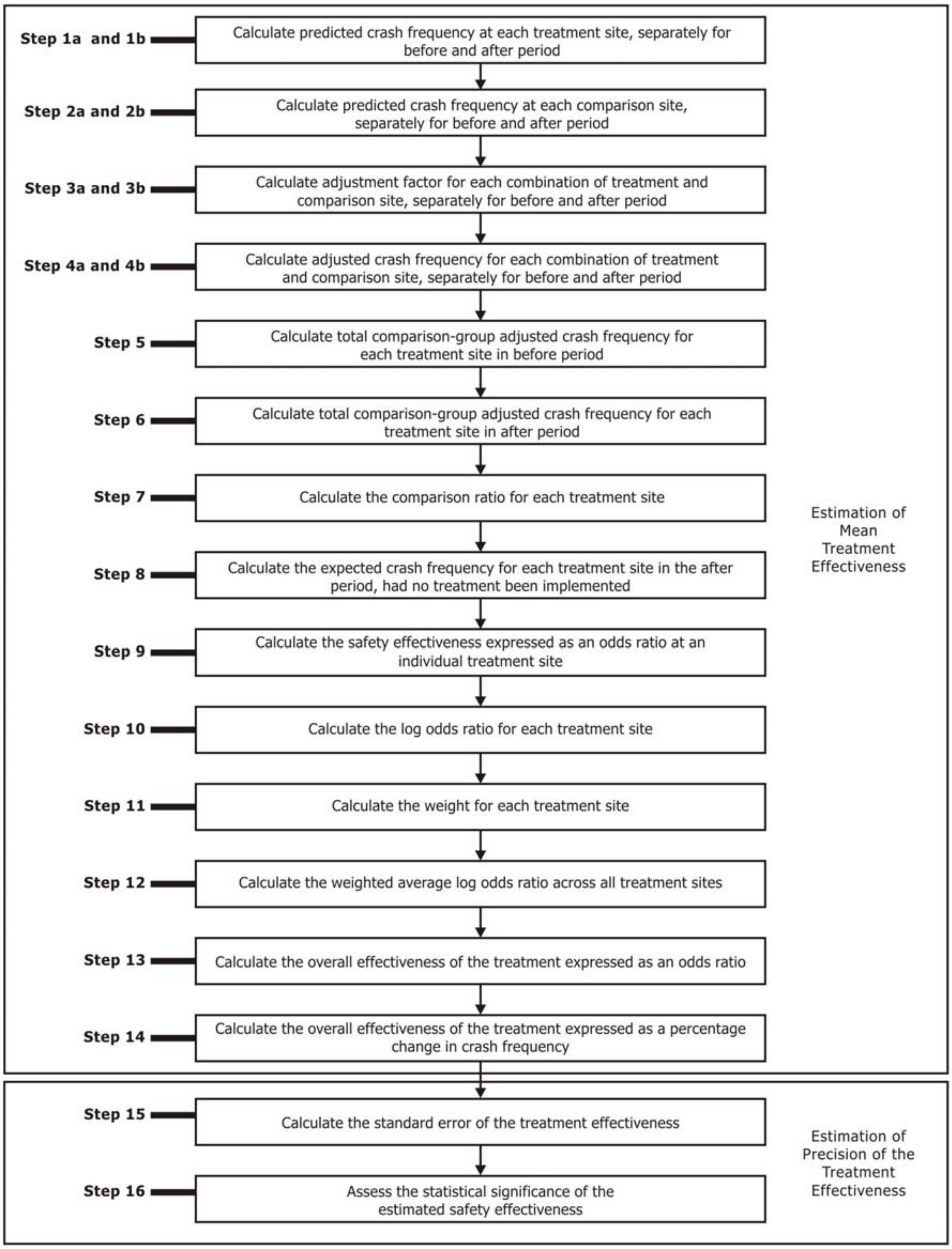
The key pre-evaluation activities are to:

- Identify the treatment sites to be evaluated
- Select the time periods before and after treatment implementation for each site that will be included in the evaluation.
- Select the measure of effectiveness for the evaluation. Evaluations often use total crash frequency as the measure of effectiveness, but any specific crash severity level and/or crash type can be considered.
- Select a set of comparison sites that are comparable to the treatment sites
- Assemble the required crash and traffic volume data for each site and time period of interest, including both treatment and comparison sites.
- Obtain SPF(s) applicable to the treatment and comparison sites. Such SPFs may be developed based on the available data as described in *Part C* of the HSM or from Safety Analysis. In a comparison group evaluation, the SPF(s) are used solely to derive adjustment factors to account for the nonlinear effects of changes in average daily traffic volume. This adjustment for

This section summarizes how to conduct before/after comparison group method effectiveness evaluation. The computational procedures are presented in the appendix.

changes in traffic volume is needed for both the treatment and comparison sites and, therefore, SPFs are needed for all site types included in the treatment and comparison sites. If no SPFs are available and the effects of traffic volume are assumed to be linear, this will make the evaluation results less accurate.

Exhibit 9-9: Overview of Before/After Comparison-Group Safety Evaluation



The before study period for a site must end before implementation of the treatment began at that site. The after study period for a site normally begins after treatment implementation is complete; a buffer period of several months is usually allowed for traffic to adjust to the presence of the treatment. Evaluation periods that are even multiples of 12 months in length are used so that there is no seasonal bias in the evaluation data. Analysts often choose evaluation periods that consist of complete calendar years because this often makes it easier to assemble the required data. When the evaluation periods consist of entire calendar years, the entire year during which the treatment was installed is normally excluded from the evaluation period.

The comparison-group procedures are based on the assumption that the same set of comparison-group sites are used for all treatment sites. A variation of the procedure that is applicable if different comparison group sites are used for each treatment is presented by Harwood et al.[(2)]. Generally, this variation would only be needed for special cases, such as multi-state studies where an in-state comparison group was used for each treatment site.

A weakness of the comparison-group method is that it cannot consider treatment sites at which the observed crash frequency in the period either before or after implementation of the treatment is zero. This may lead to an underestimate of the treatment effectiveness since sites with no crashes in the after treatment may represent locations at which the treatment was most effective.

Computational Procedure

A computational procedure using the comparison-group evaluation study method to determine the effectiveness of the treatment being evaluated, expressed as a percentage change in crashes, θ, and to assess its precision and statistical significance, is presented in the Appendix to this chapter.

9.4.3. Implementing the Safety Evaluation Method for Before/After Shifts in Proportions of Target Collision Types

The safety evaluation method for before/after shifts in proportions is used to quantify and assess the statistical significance of a change in the frequency of a specific target collision type expressed as a proportion of total crashes from before to after implementation of a specific countermeasure or treatment. This method uses data only for treatment sites and does not require data for nontreatment or comparison sites. Target collision types (e.g., run-off road, head-on, rear end) addressed by the method may include all crash severity levels or only specific crash severity levels (fatal-and-serious-injury crashes, fatal-and-injury-crashes, or property-damage-only crashes). Exhibit 9-10 provides a step-by-step overview of the method for conducting a before/after safety effectiveness evaluation for shifts in proportions of target collision types.

Data Needs and Inputs

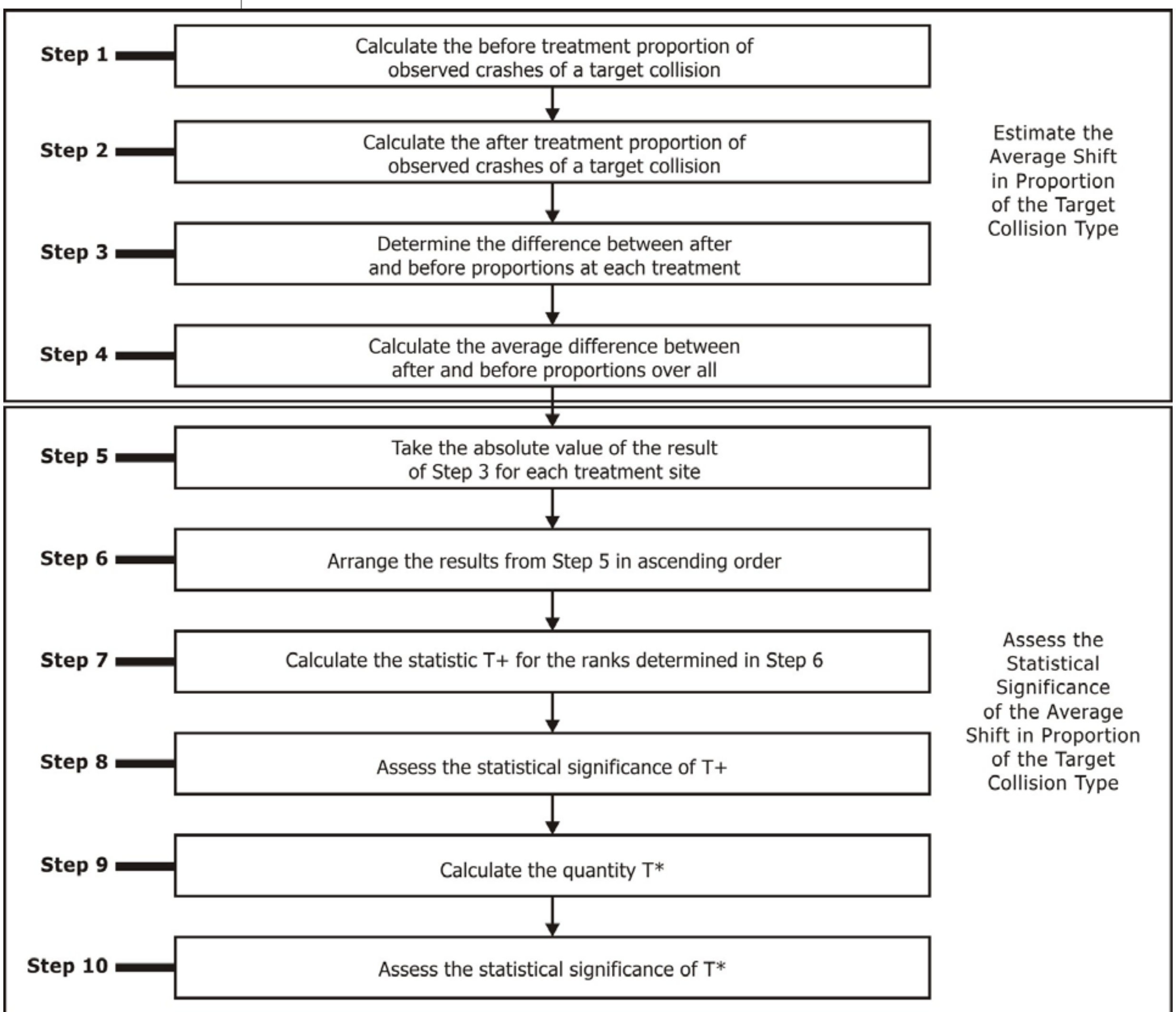
The data needed as input to a before/after evaluation for shifts in proportions of target collision types include:

- At least 10 to 20 sites at which the treatment of interest has been implemented
- 3 to 5 years of before-period crash data is recommended for the treatment sites

- 3 to 5 years of after-period crash data is recommended for the treatment sites

An evaluation study can be performed with fewer sites and/or shorter time periods, but statistically significant results are less likely.

Exhibit 9-10: Overview Safety Evaluation for Before/After Shifts in Proportions



Pre-Evaluation Activities

The key pre-evaluation activities are to:

- Identify the treatment sites to be evaluated
- Select the time periods before and after treatment implementation for each site that will be included in the evaluation
- Select the target collision type for the evaluation
- Assemble the required crash and traffic volume data for each site and time period of interest for the treatment sites

The before study period for a site must end before implementation of the treatment began at that site. The after study period for a site normally begins after treatment implementation is complete; a buffer period of several months is usually allowed for traffic to adjust to the presence of the treatment. Evaluation periods that are even multiples of 12 months in length are used so that there is no seasonal bias in the evaluation data. Analysts often choose evaluation periods consist of complete calendar years because this often makes it easier to assemble the required data. When the evaluation periods consist of entire calendar years, the entire year during which the treatment was installed is normally excluded from the evaluation period.

Computational Method

A computational procedure using the evaluation study method for assessing shifts in proportions of target collision types to determine the safety effectiveness of the treatment being evaluated, $AvgP_{(CT)Diff}$, and to assess its statistical significance, is presented in the Appendix to this chapter.

9.4.4. Implementing the Cross-Sectional Safety Evaluation Method

Definition

In the absence of before data at treatment sites, the cross-sectional safety evaluation method can be used to estimate the safety effectiveness of a treatment through comparison to crash data at comparable nontreatment sites. A cross-sectional safety evaluation generally requires complex statistical modeling and therefore is addressed here in general terms only.

Data Needs and Inputs

- 10 to 20 treatment sites are recommended to evaluate a safety treatment
- 10 to 20 nontreatment sites are recommended for the nontreatment group
- 3 to 5 years of crash data for both treatment and nontreatment sites is recommended

Pre-Evaluation Activities

The key pre-evaluation activities are to:

- Identify the sites both with and without the treatment to be evaluated
- Select the time periods that will be included in the evaluation when the conditions of interest existed at the treatment and nontreatment sites
- Select the safety measure of effectiveness for the evaluation. Evaluations often use total crash frequency as the measure of effectiveness, but any specific crash severity level and/or crash type can be considered.
- Assemble the required crash and traffic volume data for each site and time period of interest.

Method

There is no step-by-step methodology for the cross-sectional safety evaluation method because this method requires model development rather than a sequence of computations that can be presented in equations. In implementing the cross-sectional safety evaluation method, all of the crash, traffic volume, and site characteristics data (including data for both the treatment and nontreatment sites) are analyzed in a single model including either an indicator variable for the presence or absence of the treatment at a site or a continuous variable representing the dimension of the treatment (e.g., lane width or shoulder width). A generalized linear model (GLM) with a negative binomial distribution and a logarithmic link function is a standard approach to model the yearly crash frequencies. Generally, a repeated-measures correlation structure is included to account for the relationship between crashes at a given site across years (temporal correlation). A compound symmetry, autoregressive, or other covariance structure can be used to account for within-site correlation. General estimating equations (GEE) may then be used to determine the final regression parameter estimates, including an estimate of the treatment effectiveness and its precision. An example of application of this statistical modeling approach is presented by Lord and Persaud[8]. This approach may be implemented using any of several commercially available software packages.

The grey box below illustrates a generic application of a cross-sectional safety evaluation analysis.

Overview of a Cross-Sectional Analysis to Evaluate the Safety Effectiveness of a Treatment

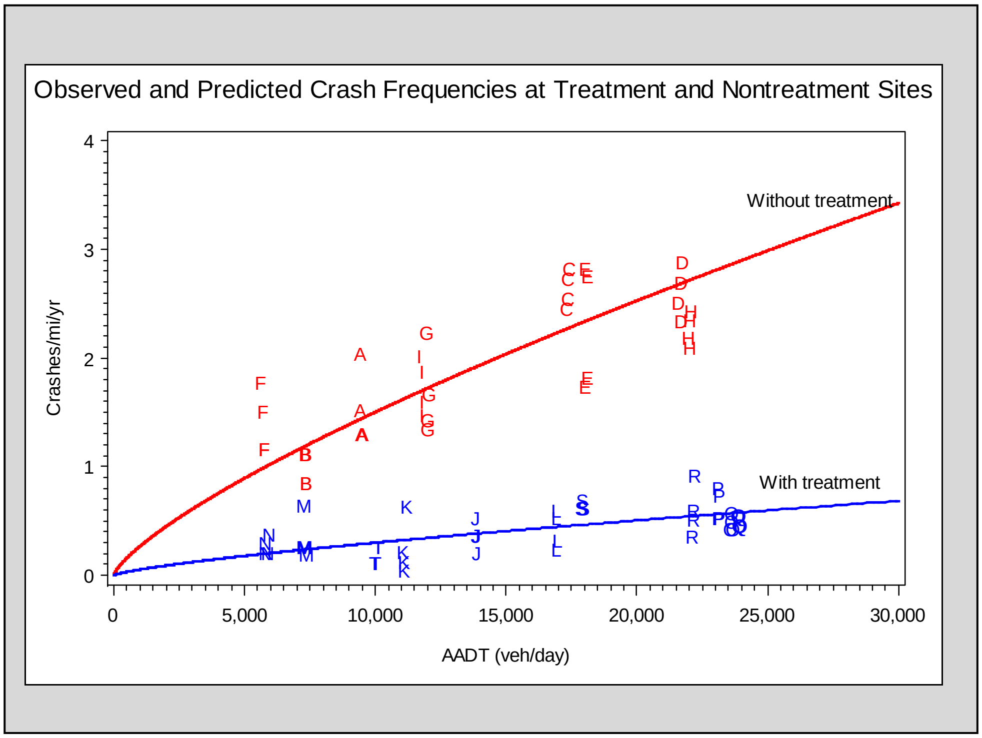
A treatment was installed at 11 sites. Crash data, geometrics, and traffic volume data are available for a 4-year period at each site. Similar data are available for 9 sites without the treatment but with comparable geometrics and traffic volumes. The available data can be summarized as follows:

- 9 nontreatment sites (denoted A through I); 4 years of data at each site
- 11 treatment sites (denoted J through T); 4 years of data at each site

A negative binomial generalized linear model (GLM) was used to estimate the treatment effect based on the entire dataset, accounting for AADT and other geometric parameters (e.g., shoulder width, lane width, number of lanes, roadside hazard rating) as well as the relationship between crashes at a given site over the 4-year period (within-site correlation) using generalized estimating equations (GEE).

The graph illustrates the observed and predicted average crash frequency for the treatment and nontreatment sites. The safety effectiveness of the treatment is assessed by the statistical significance of the treatment effect on crash frequency. This effect is illustrated by the difference in the rate of change in the two curves. In this example, the installation of the treatment significantly reduced crash frequency.

Note that the data shown below are fictional crash and traffic data.



9.5. EVALUATING A SINGLE PROJECT AT A SPECIFIC SITE TO DETERMINE ITS SAFETY EFFECTIVENESS

An observational before/after evaluation can be conducted for a single project at a specific site to determine its effectiveness in reducing crash frequency or severity. The evaluation results provide an estimate of the effect of the project on safety at that particular site. Any of the study designs and evaluation methods presented in Sections 9.3 and 9.4, with the exception of cross-sectional studies which require more than one treatment site, can be applied to such an evaluation. The results of such evaluations, even for a single site, may be of interest to highway agencies in monitoring their improvement programs. However, results from the evaluation of a single site will not be very accurate and, with only one site available, the precision and statistical significance of the evaluation results cannot be assessed.

9.6. EVALUATING A GROUP OF SIMILAR PROJECTS TO DETERMINE THEIR SAFETY EFFECTIVENESS

Observational before/after evaluations can be conducted for groups of similar projects to determine their effectiveness reducing crash frequency or severity. The evaluation results provide an estimate of the overall safety effectiveness of the group of projects as a whole. Any of the study designs and evaluation methods presented in Sections 9.3 and 9.4, with the exception of cross-sectional studies, can be applied to such an evaluation. Cross-sectional studies are intended to make inferences about the effectiveness of a countermeasure or treatment when applied to other sites, not to evaluate the safety effectiveness of projects at particular sites. Therefore cross-sectional studies are not appropriate when the objective of the evaluation is to assess the effectiveness of the projects themselves.

A safety effectiveness evaluation for a group of projects may be of interest to highway agencies in monitoring their improvement programs. Where more than one project is evaluated, the precision of the effectiveness estimate and the statistical significance of the evaluation results can be determined. The guidelines in Section 9.4 indicate that at least 10 to 20 sites generally need to be evaluated to obtain statistically significant results. While this minimum number of sites is presented as a general guideline, the actual number of sites needed to obtain statistically significant results can vary widely as a function of the magnitude of the safety effectiveness for the projects being evaluated and the site-to-site variability of the effect. The most reliable methods for evaluating a group of projects are those that compensate for regression-to-the-mean bias, such as the EB method.

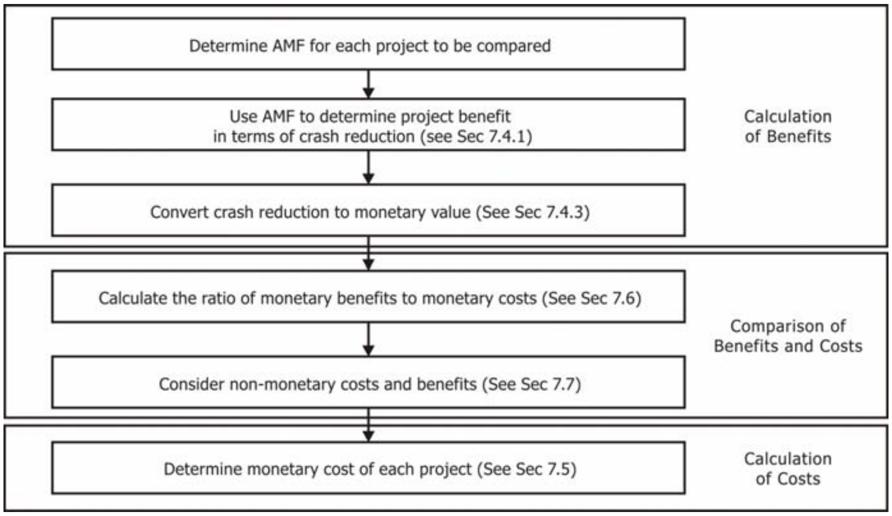
9.7. QUANTIFYING AMFS AS A RESULT OF A SAFETY EFFECTIVENESS EVALUATION

A common application of safety effectiveness evaluation is to quantify the value of an AMF for a countermeasure by evaluating multiple sites where that countermeasure has been evaluated. Any of the study designs and evaluation methods presented in Sections 9.3 and 9.4 can be applied in quantifying an AMF value, although methods that compensate for regression-to-the-mean bias, such as the EB method, are the most reliable. The evaluation methods that can be used to quantify an AMF are the same as those described in Section 9.6 for evaluating a group of projects, except the cross-sectional studies may also be used, though they are less reliable than methods that compensate for regression-to-the- mean bias. As noted above, at least 10 to 20 sites generally need to be evaluated to obtain statistically significant results. While this minimum number of sites is presented as a general guideline, the actual number of sites needed to obtain statistically significant results can vary widely as a function of the magnitude of the safety effectiveness for the projects being evaluated and the site-to-site variability of the effect.

9.8. COMPARISON OF SAFETY BENEFITS AND COSTS OF IMPLEMENTED PROJECTS

Where the objective of an evaluation is to compare the crash reduction benefits and costs of implemented projects, the first step is to determine an AMF for the project, as described above in Section 9.7. The economic analysis procedures presented in *Chapter 7* are then be applied to quantify the safety benefits of the projects in monetary terms, using the AMF, and to compare the safety benefits and costs of the implemented projects. Exhibit 9-11 provides a graphical overview of this comparison.

Exhibit 9-11: Overview of Safety Benefits and Costs Comparison of Implemented Projects



9.9. CONCLUSIONS

Safety effectiveness evaluation is the process of developing quantitative estimates of the reduction in the number of crashes or severity of crashes due to a treatment, project, or a group of projects. Evaluating implemented safety treatments is an important step in the roadway safety evaluation process, and provides important information for future decision-making and policy development.

Safety effectiveness evaluation may include:

- Evaluating a single project at a specific site to document the safety effectiveness of that specific project;
- Evaluating a group of similar projects to document the safety effectiveness of those projects;
- Evaluating a group of similar projects for the specific purpose of quantifying an AMF for a countermeasure; and
- Assessing the overall safety effectiveness of specific types of projects or countermeasures in comparison to their costs.

There are three basic study designs that can be used for safety effectiveness evaluations:

- Observational before/after studies
- Observational cross-sectional studies
- Experimental before/after studies

Both observational and experimental studies may be used in safety effectiveness evaluations, although observational studies are more common among highway agencies.

This chapter documents and discusses the various methods for evaluating the effectiveness of a treatment, a set of treatments, an individual project, or a group of similar projects after safety improvements have been implemented. This chapter provides an introduction to the evaluation methods that can be used; highlights which methods are appropriate for assessing safety effectiveness in specific situations; and provides step-by-step procedures for conducting safety effectiveness evaluations

9.10. SAMPLE PROBLEM TO ILLUSTRATE THE EB BEFORE/AFTER SAFETY EFFECTIVENESS EVALUATION METHOD

This section presents sample problems corresponding to the three observational before/after safety effectiveness evaluation methods presented in Chapter 9, including the EB method, the comparison-group method, and the shift in proportions method. The data used in these sample problems are hypothetical. Appendix A provides a detailed summary of the steps for each of these methods.

Passing lanes have been installed to increase passing opportunities at 13 rural two-lane highway sites. An evaluation is to be conducted to determine the overall effect of the installation of these passing lanes on total crashes at the 13 treatment sites.

Data for total crash frequencies are available for these sites, including five years of data before and two years of data after installation of the passing lanes. Other available data include the site length (L) and the before- and after-period traffic volumes. To simplify the calculations for this sample problem, AADT is assumed to be constant across all years for both the before and after periods. It is also assumed that the roadway characteristics match base conditions and therefore all applicable AMFs as well as the calibration factor (see *Chapter 10*) are equal to 1.0.

Column numbers are shown in the first row of all the tables in this sample problem; the description of the calculations refers to these column numbers for clarity of explanation. For example, the text may indicate that Column 10 is the sum of Columns 5 through 9 or that Column 13 is the sum of Columns 11 and 12. When columns are repeated from table to table, the original column number is kept. Where appropriate, column totals are indicated in the last row of each table.

9.10.1. Basic Input Data

The basic input data for the safety effectiveness evaluation, including the yearly observed before- and after-period crash data for the 13 rural two-lane road segments, are presented below:

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Site No.	**Site length (L) (mi)**	**AADT (veh/day)**		**Observed before total crash frequency by year (crashes/site/year)**					**Observed crash frequency in before period**	**Observed after total crash frequency by year (crashes/site/year)**		**Observed crash frequency in after period**
		Before	**After**	**Y1**	**Y2**	**Y3**	**Y4**	**Y5**		**Y1**	**Y2**	
1	1.114	8,858	8,832	4	4	1	5	2	16	1	1	2
2	0.880	11,190	11,156	2	0	0	2	2	6	0	2	2
3	0.479	11,190	11,156	1	0	2	1	0	4	1	1	2
4	1.000	6,408	6,388	2	5	4	3	2	16	0	1	1
5	0.459	6,402	6,382	0	0	1	0	0	1	0	1	1
6	0.500	6,268	6,250	1	1	0	2	1	5	1	0	1
7	0.987	6,268	6,250	4	3	3	4	3	17	6	3	9
8	0.710	5,503	5,061	4	3	1	1	3	12	0	0	0
9	0.880	5,523	5,024	2	0	6	0	0	8	0	0	0
10	0.720	5,523	5,024	1	0	1	1	0	3	0	0	0
11	0.780	5,523	5,024	1	4	2	1	1	9	3	2	5
12	1.110	5,523	5,024	1	0	2	4	2	9	4	2	6
13	0.920	5,523	5,024	3	2	3	3	5	16	0	1	1
Total				26	22	26	27	21	122	16	14	30

9.10.2. EB Estimation of the Expected Average Crash Frequency in the Before Period

Equation 10-6 of Section 10.6.1 in *Chapter 10* provides the applicable SPF to predict total crashes on rural two-lane roads:

$$N_{spf\ rs} = AADT \times L \times 365 \times 10^{-6} \times e^{(-0.312)} \qquad (10\text{–}6)$$

Where,

$N_{spf\ rs}$ = estimated total crash frequency for roadway segment base conditions;

AADT = average annual daily traffic volume (vehicles per day);

L = length of roadway segment (miles).

The overdispersion parameter is given by Equation 10-7 in *Chapter 10* as:

$$k = \frac{0.236}{L} \qquad (10\text{–}7)$$

Equation 10-1 of Section 10.2 in *Chapter 10* presents the predicted average crash frequency for a specific site type x (roadway, rs, in this example). Note in this example all AMFs and the calibration factor are assumed to equal 1.0.

$$N_{predicted} = N_{spf\ x} \times (AMF_{1x} \times AMF_{2x} \times \ldots \times AMF_{yx}) \times C_x \qquad (10\text{-}1)$$

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*;

AMF_{yx} = Accident Modification Factors specific to site type *x* and specific geometric design and traffic control features *y*;

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

Step 1: Using the above SPF and Columns 2 and 3, Calculate the Predicted Average Crash Frequency for Each Site During Each Year of the Before Period

Using the above SPF and Columns 2 and 3, calculate the predicted average crash frequency for each site during each year of the before period. The results appear in Columns 14 through 18. For use in later calculations, sum these predicted average crash frequencies over the five before years. The results appear in Column 19. Note that because in this example the AADT is assumed constant across years at a given site in the before period, the predicted average crash frequencies do not change from year to year since they are simply a function of segment length and AADT at a given site. This will not be the case in general, when yearly AADT data are available.

(1)	(14)	(15)	(16)	(17)	(18)	(19)
Site No.	Predicted before total crash frequency by year (crashes/year)					Predicted average crash frequency in before period
	Y1	Y2	Y3	Y4	Y5	
1	2.64	2.64	2.64	2.64	2.64	13.18
2	2.63	2.63	2.63	2.63	2.63	13.15
3	1.43	1.43	1.43	1.43	1.43	7.16
4	1.71	1.71	1.71	1.71	1.71	8.56
5	0.79	0.79	0.79	0.79	0.79	3.93
6	0.84	0.84	0.84	0.84	0.84	4.19
7	1.65	1.65	1.65	1.65	1.65	8.26
8	1.04	1.04	1.04	1.04	1.04	5.22
9	1.30	1.30	1.30	1.30	1.30	6.49
10	1.06	1.06	1.06	1.06	1.06	5.31
11	1.15	1.15	1.15	1.15	1.15	5.75
12	1.64	1.64	1.64	1.64	1.64	8.19
13	1.36	1.36	1.36	1.36	1.36	6.79
Total	19.24	19.24	19.24	19.24	19.24	96.19

Step 2: Calculate the Weighted Adjustment, w, for Each Site for the Before Period

Using Equation A-2, the calculated overdispersion parameter (shown in Column 20), and Column 19, calculate the weighted adjustment, w, for each site for the before period. The results appear in Column 21. Using Equation A-1, Columns 21, 19, and

10, calculate the expected average crash frequency for each site, summed over the entire before period. The results appear in Column 22.

(1) Site No.	(20) Overdispersion parameter, k	(21) Weighted adjustment, w	(22) Expected average crash frequency in before period
1	0.212	0.264	15.26
2	0.268	0.221	7.58
3	0.493	0.221	4.70
4	0.236	0.331	13.54
5	0.514	0.331	1.97
6	0.472	0.336	4.73
7	0.239	0.336	14.06
8	0.332	0.366	9.52
9	0.268	0.365	7.45
10	0.328	0.365	3.84
11	0.303	0.365	7.82
12	0.213	0.365	8.70
13	0.257	0.365	12.64
Total			111.81

9.10.3. EB Estimation of the Expected Average Crash Frequency in the After Period in the Absence of the Treatment

Step 3: Calculate the Predicted Average Crash Frequency for Each Site during each year of the After Period

Using the above SPF and Columns 2 and 4, calculate the predicted average crash frequency for each site during each year of the after period. The results appear in Columns 23 and 24. For use in later calculations, sum these predicted average crash frequencies over the two after years. The results appear in Column 25.

(1)	(23)	(24)	(25)	(26)	(27)
Site No.	**Predicted after total crash frequency (crashes/year)**		**Predicted average crash frequency in after period**	**Adjustment factor, r**	**Expected average crash frequency in after period without treatment**
	Y1	**Y2**			
1	2.63	2.63	5.26	0.399	6.08
2	2.62	2.62	5.25	0.399	3.02
3	1.43	1.43	2.86	0.399	1.87
4	1.71	1.71	3.41	0.399	5.40
5	0.78	0.78	1.57	0.399	0.79
6	0.83	0.83	1.67	0.399	1.89
7	1.65	1.65	3.30	0.399	5.61
8	0.96	0.96	1.92	0.368	3.50
9	1.18	1.18	2.36	0.364	2.71
10	0.97	0.97	1.93	0.364	1.40
11	1.05	1.05	2.09	0.364	2.84
12	1.49	1.49	2.98	0.364	3.17
13	1.23	1.23	2.47	0.364	4.60
Total	18.53	18.53	37.06		42.88

Step 4: Calculate the Adjustment Factor, r, to Account for the Differences Between the Before and After Periods in Duration and Traffic Volume at Each Site.

Using Equation A-3 and Columns 25 and 19, calculate the adjustment factor, r, to account for the differences between the before and after periods in duration and traffic volume at each site. The results appear in Column 26 in the table presented in Step 3.

Step 5: Calculate the Expected Average Crash Frequency for each Site over the Entire after Period in the Absence of the Treatment.

Using Equation A-4 and Columns 22 and 26, calculate the expected average crash frequency for each site over the entire after period in the absence of the treatment. The results appear in Column 27 in the table presented in Step 3.

9.10.4. Estimation of the Treatment Effectiveness

Step 6: Calculate an Estimate of the Safety Effectiveness of the Treatment at Each Site in the Form of an Odds Ratio

Using Equation A-5 and Columns 13 and 27, calculate an estimate of the safety effectiveness of the treatment at each site in the form of an odds ratio. The results appear in Column 28.

(1)	(13)	(27)	(28)	(29)	(30)
Site No.	Observed crash frequency in after period	Expected average crash frequency in after period without treatment	Odds ratio	Safety effectiveness (%)	Variance term (Eq. A-10)
1	2	6.08	0.329	67.13	1.787
2	2	3.02	0.662	33.84	0.939
3	2	1.87	1.068	-6.75	0.582
4	1	5.40	0.185	81.47	1.440
5	1	0.79	1.274	-27.35	0.209
6	1	1.89	0.530	46.96	0.499
7	9	5.61	1.604	-60.44	1.486
8	0	3.50	0.000	100.00	0.817
9	0	2.71	0.000	100.00	0.627
10	0	1.40	0.000	100.00	0.323
11	5	2.84	1.758	-75.81	0.657
12	6	3.17	1.894	-89.44	0.732
13	1	4.60	0.217	78.26	1.063
Total	30	42.88			11.162

Step 7: Calculate the Safety Effectiveness as a Percentage Crash Change at Each Site

Using Equation A-6 and Column 28, calculate the safety effectiveness as a percentage crash change at each site. The results appear in Column 29 in the table presented in Step 6. A positive result indicates a reduction in crashes; conversely, a negative result indicates an increase in crashes.

Step 8: Calculate the Overall Effectiveness of the Treatment for all Sites Combined, in the Form of an Odds Ratio

Using Equation A-7 and the totals from Columns 13 and 27, calculate the overall effectiveness of the treatment for all sites combined, in the form of an odds ratio:

$$OR' = 30/42.88 = 0.700$$

Step 9: Calculate each Term of Equation A-9

Using Columns 26, 22, and 21, calculate each term of Equation A-9. The results appear in Column 30 in the table presented in Step 6. Sum the terms in Column 30. Next, using Equations A-8 and A-9, the value for OR' from Step 8, and the sums in Column 30 and 27, calculate the final adjusted odds ratio:

$$OR = \frac{0.700}{1 + \frac{11.162}{(42.88)^2}} = 0.695$$

Since the odds ratio is less than 1, it indicates a reduction in crash frequency due to the treatment.

Step 10: Calculate the Overall Unbiased Safety Effectiveness as a Percentage Change in Crash Frequency Across all Sites

Using Equation A-10 and the above result, calculate the overall unbiased safety effectiveness as a percentage change in crash frequency across all sites:

$$AMF = 100 \times (1-0.695) = 30.5\%$$

9.10.5. Estimation of the Precision of the Treatment Effectiveness

Step 11: Calculate the Variance of OR

Using Equation A-11, the value for OR' from Step 8, and the sums from Columns 13, 30, and 27, calculate the variance of OR:

$$Var(OR) = \frac{(0.700)^2\left[\frac{1}{30} + \frac{11.162}{(42.88)^2}\right]}{\left[1 + \frac{11.162)}{(42.88)^2}\right]} = 0.019$$

Step 12: Calculate the Standard Error of OR

Using Equation A-12 and the result from Step 11, calculate the standard error of OR:

$$SE(\mathrm{OR}) = \sqrt{0.019} = 0.138$$

Step 13: Calculate the Standard Error of AMF

Using Equation A-13 and the result from Step 12, calculate the standard error of AMF:

$$SE(AMF) = 100 \times 0.138 = 13.8\%$$

Step 14: Assess the Statistical Significance of the Estimated Safety Effectiveness

Assess the statistical significance of the estimated safety effectiveness by calculating the quantity:

$$Abs[AMF/SE(AMF)] = 30.5/13.85 = 2.20$$

Since Abs[AMF/SE(AMF)] ≥ 2.0, conclude that the treatment effect is significant at the (approximate) 95-percent confidence level. The positive estimate of AMF, 30.5%, indicates a positive effectiveness, i.e., a reduction, in total crash frequency.

In summary, the evaluation results indicate that the installation of passing lanes at the 13 rural two-lane highway sites reduced total crash frequency by 30.5% on average, and that this result is statistically significant at the 95-percent confidence level.

9.11. SAMPLE PROBLEM TO ILLUSTRATE THE COMPARISON-GROUP SAFETY EFFECTIVENESS EVALUATION METHOD

Passing lanes have been installed to increase passing opportunities at 13 rural two-lane highway sites. An evaluation is to be conducted to determine the overall effect of the installation of these passing lanes on total crashes at the 13 treatment sites.

9.11.1. Basic Input Data for Treatment Sites

Data for total crash frequencies are available for the 13 sites, including five years of data before and two years of data after installation of the passing lanes. Other available data include the site length (L) and the before- and after-period traffic volumes. To simplify the calculations for this sample problem, AADT is assumed to be constant across all years for both the before and after periods. The detailed step-by-step procedures in Appendix A show how to handle computations for sites with AADTs that vary from year to year.

Column numbers are shown in the first row of all the tables in this sample problem; the description of the calculations refers to these column numbers for clarity of explanation. When columns are repeated from table to table, the original column number is kept. Where appropriate, column totals are indicated in the last row of each table.

Organize the observed before- and after-period data for the 13 rural two-lane road segments as shown below based on the input data for the treatment sites shown in the sample problem in Section B.1:

(1)	(2)	(3)	(4)	(5)	(6)
Treatment Sites					
Site No.	Site length (L) (mi)	AADT (veh/day)		Observed crash frequency in before period (5 years) (K)	Observed crash frequency in after period (2 years) (L)
		Before	After		
1	1.114	8,858	8,832	16	2
2	0.880	11,190	11,156	6	2
3	0.479	11,190	11,156	4	2
4	1.000	6,408	6,388	16	1
5	0.459	6,402	6,382	1	1
6	0.500	6,268	6,250	5	1
7	0.987	6,268	6,250	17	9
8	0.710	5,503	5,061	12	0
9	0.880	5,523	5,024	8	0
10	0.720	5,523	5,024	3	0
11	0.780	5,523	5,024	9	5
12	1.110	5,523	5,024	9	6
13	0.920	5,523	5,024	16	1
Total	10.539			122	30

9.11.2. Basic Input Data for Comparison Group Sites

A comparison group of 15 similar, but untreated, rural two-lane highway sites has been selected. The length of each site is known. Seven years of before-period data and three years of after-period data (crash frequencies and before- and after-period AADTs) are available for each of the 15 sites in the comparison group. As above, AADT is assumed to be constant across all years in both the before and after periods for each comparison site. The same comparison group is assigned to each treatment site in this sample problem.

Organize the observed before- and after-period data for the 15 rural two-lane road segments as shown below:

(7)	(8)	(9)	(10)	(11)	(12)
Comparison Group					
Site No.	**Site length (L) (mi)**	**AADT (veh/day)**		**Observed crash frequency in before period (7 years)**	**Observed crash frequency in after period (3 years)**
		Before	After		
1	1.146	8,927	8,868	27	4
2	1.014	11,288	11,201	5	5
3	0.502	11,253	11,163	7	3
4	1.193	6,504	6,415	21	2
5	0.525	6,481	6,455	3	0
6	0.623	6,300	6,273	6	1
7	1.135	6,341	6,334	26	11
8	0.859	5,468	5,385	12	4
9	1.155	5,375	5,324	20	12
10	0.908	5,582	5,149	33	5
11	1.080	5,597	5,096	5	0
12	0.808	5,602	5,054	3	0
13	0.858	5,590	5,033	4	10
14	1.161	5,530	5,043	12	2
15	1.038	5,620	5,078	21	2
Total	14.004			205	61

9.11.3. Estimation of Mean Treatment Effectiveness

Equation 10-6 of Section 10.6.1 in *Chapter 10* provides the applicable SPF for total crashes on rural two-lane roads:

$$N_{spf\ rs} = AADT \times L \times 365 \times 10^{-6} \times e^{(-0.312)} \qquad (10\text{–}6)$$

The overdispersion parameter for this SPF is not relevant to the comparison group method.

Equation 10-1 of Section 10.2 in *Chapter 10* presents the predicted average crash frequency for a specific site type x (roadway, rs, in this example). Note in this example all AMFs and the calibration factor are assumed to equal 1.0.

$$N_{predicted} = N_{spf\ x} \times (AMF_{1x} \times AMF_{2x} \times ... \times AMF_{yx}) \times C_x \qquad (10\text{-}1)$$

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*;

AMF_{yx} = Accident Modification Factors specific to site type *x* and specific geometric design and traffic control features *y*;

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

Step 1a: Calculate the Predicted Average Crash Frequency at each Treatment Site in the 5-year Before Period

Using the above SPF and Columns 2 and 3, calculate the predicted average crash frequency at each treatment site in the 5-year before period. The results appear in Column 13 in the table below. For use in later calculations, sum these predicted average crash frequencies over the 13 treatment sites.

Step 1b: Calculate the Predicted Average Crash Frequency at each Treatment Site in the 2-year After Period

Similarly, using the above SPF and Columns 2 and 4, calculate the predicted average crash frequency at each treatment site in the 2-year after period. The results appear in Column 14. Sum these predicted average crash frequencies over the 13 treatment sites.

(1)	(13)	(14)
Treatment Sites		
Site No.	Predicted average crash frequency at treatment site in before period (5 years)	Predicted average crash frequency at treatment site in after period (2 years)
1	13.18	5.26
2	13.15	5.25
3	7.16	2.86
4	8.56	3.41
5	3.93	1.57
6	4.19	1.67
7	8.26	3.30
8	5.22	1.92
9	6.49	2.36
10	5.31	1.93
11	5.75	2.09
12	8.19	2.98
13	6.79	2.47

Total	96.19	37.06

Step 2a: Calculate the Predicted Average Crash Frequency for each Comparison Site in the 7-year Before Period

Using the above SPF and Columns 8 and 9, calculate the predicted average crash frequency for each comparison site in the 7-year before period. The results appear in Column 15 in the table below. Sum these predicted average crash frequencies over the 15 comparison sites.

Step 2b: Calculate the Predicted Average Crash Frequency for each Comparison Site in the 3-year After Period

Similarly, using the above SPF and Columns 8 and 10, calculate the predicted average crash frequency for each comparison site in the 3-year after period. The results appear in Column 16. Sum these predicted average crash frequencies over the 15 comparison sites.

(7)	(15)	(16)
Comparison Group		
Site No.	Predicted average crash frequency at comparison site in before period (7 years)	Predicted average crash frequency at comparison site in after period (3 years)
1	19.13	8.14
2	21.40	9.10
3	10.56	4.49
4	14.51	6.13
5	6.37	2.72
6	7.34	3.13
7	13.46	5.76
8	8.79	3.71
9	11.62	4.93
10	9.48	3.75
11	11.30	4.41
12	8.46	3.27
13	8.97	3.46
14	12.01	4.69
15	10.91	4.22
Total	174.29	71.93

Step 3a: Calculate the 13 Before Adjustment Factors for Each of the 15 Comparison Sites

Using Equation A-14, Columns 13 and 15, the number of before years for the treatment sites (5 years), and the number of before years for the comparison sites (7 years), calculate the 13 before adjustment factors for each of the 15 comparison sites. The results appear in Columns 17 through 29.

(7)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)
	Comparison Group—Before Adjustment Factors (Equation A-14)												
Site No.	1	2	3	4	5	6	7	8	9	10	11	12	13
1	0.49	0.49	0.27	0.32	0.15	0.16	0.31	0.19	0.24	0.20	0.21	0.31	0.25
2	0.44	0.44	0.24	0.29	0.13	0.14	0.28	0.17	0.22	0.18	0.19	0.27	0.23
3	0.89	0.89	0.48	0.58	0.27	0.28	0.56	0.35	0.44	0.36	0.39	0.55	0.46
4	0.65	0.65	0.35	0.42	0.19	0.21	0.41	0.26	0.32	0.26	0.28	0.40	0.33
5	1.48	1.48	0.80	0.96	0.44	0.47	0.93	0.59	0.73	0.60	0.65	0.92	0.76
6	1.28	1.28	0.70	0.83	0.38	0.41	0.80	0.51	0.63	0.52	0.56	0.80	0.66
7	0.70	0.70	0.38	0.45	0.21	0.22	0.44	0.28	0.34	0.28	0.31	0.43	0.36
8	1.07	1.07	0.58	0.70	0.32	0.34	0.67	0.42	0.53	0.43	0.47	0.67	0.55
9	0.81	0.81	0.44	0.53	0.24	0.26	0.51	0.32	0.40	0.33	0.35	0.50	0.42
10	0.99	0.99	0.54	0.65	0.30	0.32	0.62	0.39	0.49	0.40	0.43	0.62	0.51
11	0.83	0.83	0.45	0.54	0.25	0.26	0.52	0.33	0.41	0.34	0.36	0.52	0.43
12	1.11	1.11	0.60	0.72	0.33	0.35	0.70	0.44	0.55	0.45	0.49	0.69	0.57
13	1.05	1.05	0.57	0.68	0.31	0.33	0.66	0.42	0.52	0.42	0.46	0.65	0.54
14	0.78	0.78	0.43	0.51	0.23	0.25	0.49	0.31	0.39	0.32	0.34	0.49	0.40
15	0.86	0.86	0.47	0.56	0.26	0.27	0.54	0.34	0.43	0.35	0.38	0.54	0.44
Total	0.49	0.49	0.27	0.32	0.15	0.16	0.31	0.19	0.24	0.20	0.21	0.31	0.25

Step 3b: Calculate the 13 After Adjustment Factors for Each of the 15 Comparison Sites

Using Equation A-15, Columns 14 and 16, the number of after years for the treatment sites (2 years), and the number of after years for the comparison sites (3 years), calculate the 13 after adjustment factors for each of the 15 comparison site. The results appear in Columns 30 through 42.

(7)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)
Comparison Group—After Adjustment Factors (Equation A-15)													
Site No.	1	2	3	4	5	6	7	8	9	10	11	12	13
1	0.43	0.43	0.23	0.28	0.13	0.14	0.27	0.16	0.19	0.16	0.17	0.24	0.20
2	0.39	0.38	0.21	0.25	0.11	0.12	0.24	0.14	0.17	0.14	0.15	0.22	0.18
3	0.78	0.78	0.42	0.51	0.23	0.25	0.49	0.29	0.35	0.29	0.31	0.44	0.37
4	0.57	0.57	0.31	0.37	0.17	0.18	0.36	0.21	0.26	0.21	0.23	0.32	0.27
5	1.29	1.29	0.70	0.84	0.38	0.41	0.81	0.47	0.58	0.47	0.51	0.73	0.61
6	1.12	1.12	0.61	0.73	0.33	0.36	0.70	0.41	0.50	0.41	0.45	0.63	0.53
7	0.61	0.61	0.33	0.39	0.18	0.19	0.38	0.22	0.27	0.22	0.24	0.34	0.29
8	0.94	0.94	0.51	0.61	0.28	0.30	0.59	0.35	0.42	0.35	0.38	0.54	0.44
9	0.71	0.71	0.39	0.46	0.21	0.23	0.45	0.26	0.32	0.26	0.28	0.40	0.33
10	0.94	0.93	0.51	0.61	0.28	0.30	0.59	0.34	0.42	0.34	0.37	0.53	0.44
11	0.79	0.79	0.43	0.52	0.24	0.25	0.50	0.29	0.36	0.29	0.32	0.45	0.37
12	1.07	1.07	0.58	0.70	0.32	0.34	0.67	0.39	0.48	0.39	0.43	0.61	0.50
13	1.01	1.01	0.55	0.66	0.30	0.32	0.64	0.37	0.46	0.37	0.40	0.57	0.48
14	0.75	0.75	0.41	0.49	0.22	0.24	0.47	0.27	0.34	0.27	0.30	0.42	0.35
15	0.83	0.83	0.45	0.54	0.25	0.26	0.52	0.30	0.37	0.31	0.33	0.47	0.39
Total	0.43	0.43	0.23	0.28	0.13	0.14	0.27	0.16	0.19	0.16	0.17	0.24	0.20

Step 4a: Calculate the Expected Average Crash Frequencies in the Before Period for an Individual Comparison Site

Using Equation A-16, Columns 17 through 29, and Column 11, calculate the adjusted crash frequencies in the before period for an individual comparison site. The results appear in Columns 43 through 55.

(7)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)
Comparison Group—Before Adjusted Crash Frequencies (Equation A-16)													
Site No.	1	2	3	4	5	6	7	8	9	10	11	12	13
1	13.29	13.26	7.22	8.63	3.96	4.22	8.33	5.26	6.55	5.36	5.80	8.26	6.84
2	2.20	2.20	1.19	1.43	0.66	0.70	1.38	0.87	1.08	0.89	0.96	1.37	1.13
3	6.24	6.23	3.39	4.05	1.86	1.98	3.91	2.47	3.08	2.52	2.73	3.88	3.21
4	13.63	13.60	7.40	8.85	4.06	4.33	8.54	5.40	6.71	5.49	5.95	8.47	7.02
5	4.44	4.43	2.41	2.88	1.32	1.41	2.78	1.76	2.19	1.79	1.94	2.76	2.28
6	7.69	7.68	4.18	5.00	2.29	2.44	4.82	3.05	3.79	3.10	3.36	4.78	3.96
7	18.18	18.14	9.88	11.81	5.41	5.77	11.40	7.20	8.96	7.33	7.94	11.30	9.36
8	12.86	12.83	6.98	8.35	3.83	4.08	8.06	5.09	6.33	5.18	5.61	7.99	6.62
9	16.21	16.18	8.81	10.53	4.83	5.15	10.16	6.42	7.99	6.53	7.08	10.07	8.35
10	32.78	32.71	17.81	21.29	9.76	10.41	20.55	12.98	16.15	13.21	14.31	20.37	16.88
11	4.16	4.16	2.26	2.70	1.24	1.32	2.61	1.65	2.05	1.68	1.82	2.59	2.14
12	3.34	3.33	1.81	2.17	0.99	1.06	2.09	1.32	1.64	1.35	1.46	2.07	1.72
13	4.20	4.19	2.28	2.73	1.25	1.33	2.63	1.66	2.07	1.69	1.83	2.61	2.16
14	9.41	9.39	5.11	6.11	2.80	2.99	5.90	3.73	4.64	3.79	4.11	5.85	4.85
15	18.13	18.09	9.85	11.77	5.40	5.76	11.37	7.18	8.93	7.31	7.91	11.26	9.34
Total	166.77	166.42	90.59	108.30	49.66	52.97	104.55	66.03	82.14	67.21	72.81	103.61	85.87

Step 4b: Calculate the Expected Average Crash Frequencies in the After Period for an Individual Comparison Site

Similarly, using Equation A-17, Columns 30 through 42, and Column 12, calculate the adjusted crash frequencies in the after period for an individual comparison site. The results appear in Columns 56 through 68.

(7)	(56)	(57)	(58)	(58)	(60)	(61)	(62)	(63)	(64)	(65)	(66)	(67)	(68)
Comparison Group—After Adjusted Crash Frequencies (Equation A-17)													
Site No.	1	2	3	4	5	6	7	8	9	10	11	12	13
1	1.72	1.72	0.94	1.12	0.51	0.55	1.08	0.63	0.77	0.63	0.69	0.98	0.81
2	1.93	1.92	1.05	1.25	0.57	0.61	1.21	0.70	0.87	0.71	0.77	1.09	0.90
3	2.34	2.34	1.27	1.52	0.70	0.74	1.47	0.86	1.05	0.86	0.93	1.33	1.10
4	1.14	1.14	0.62	0.74	0.34	0.36	0.72	0.42	0.51	0.42	0.46	0.65	0.54
5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6	1.12	1.12	0.61	0.73	0.33	0.36	0.70	0.41	0.50	0.41	0.45	0.63	0.53
7	6.69	6.67	3.63	4.34	1.99	2.12	4.19	2.44	3.01	2.46	2.66	3.79	3.14
8	3.78	3.77	2.05	2.45	1.13	1.20	2.37	1.38	1.70	1.39	1.51	2.14	1.78
9	8.53	8.51	4.63	5.54	2.54	2.71	5.35	3.12	3.83	3.14	3.40	4.83	4.01
10	4.68	4.67	2.54	3.04	1.39	1.49	2.93	1.71	2.10	1.72	1.86	2.65	2.20
11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
13	10.13	10.11	5.50	6.58	3.02	3.22	6.35	3.70	4.55	3.72	4.03	5.74	4.76
14	1.49	1.49	0.81	0.97	0.44	0.47	0.94	0.55	0.67	0.55	0.60	0.85	0.70
15	1.66	1.66	0.90	1.08	0.49	0.53	1.04	0.61	0.75	0.61	0.66	0.94	0.78
Total	45.21	45.11	24.56	29.35	13.46	14.36	28.35	16.51	20.32	16.62	18.01	25.63	21.24

Step 5: Calculate the Total Expected Comparison Group Crash Frequencies in the Before Period for each Treatment Site.

Applying Equation A-18, sum the crash frequencies in each of the Columns 43 through 55 obtained in Step 4a. These are the 13 total comparison-group adjusted crash frequencies in the *before* period for each treatment site. The results appear in the final row of the table presented with Step 4a.

Step 6: Calculate the Total Expected Comparison Group Crash Frequencies in the After Period for each Treatment Site

Similarly, applying Equation A-19, sum the crash frequencies in each of the Columns 56 through 68 obtained in Step 4b. These are the 13 total comparison-group adjusted crash frequencies in the *after* period for each treatment site. The results appear in the final row of the table presented with Step 4b.

Step 7: Reorganize the Treatment Site Data by Transposing the Column Totals (last row) of the Tables Shown in Steps 4a and 4b

For ease of computation, reorganize the treatment site data (M and N) as shown below by transposing the column totals (last row) of the tables shown in Steps 4a and 4b.

Using Equation A-20, Columns 69 and 70, calculate the comparison ratios. The results appear in Column 71.

(1)	(69)	(70)	(71)	(72)	(6)	(73)
Treatment Sites						
Site No.	Comparison-group adjusted crash frequency in before period	Comparison-group adjusted crash frequency in after period	Comparison ratio	Expected average crash frequency in after period without treatment	Observed crash frequency in after period	Odds ratio
1	166.77	45.21	0.271	4.34	2	0.461
2	166.42	45.11	0.271	1.63	2	1.230
3	90.59	24.56	0.271	1.08	2	1.845
4	108.30	29.35	0.271	4.34	1	0.231
5	49.66	13.46	0.271	0.27	1	3.689
6	52.97	14.36	0.271	1.36	1	0.738
7	104.55	28.35	0.271	4.61	9	1.953
8	66.03	16.51	0.250	3.00	0	0.000
9	82.14	20.32	0.247	1.98	0	0.000
10	67.21	16.62	0.247	0.74	0	0.000
11	72.81	18.01	0.247	2.23	5	2.246
12	103.61	25.63	0.247	2.23	6	2.695
13	85.87	21.24	0.247	3.96	1	0.253
Total	1,216.93	318.72		31.75	30	

Step 8: Calculate the Expected Average Crash Frequency for Each Treatment Site in the After Period had no Treatment Been Implemented

Using Equation A-21, Columns 5 and 71, calculate the expected average crash frequency for each treatment site in the after period had no treatment been implemented. The results appear in Column 72 in the table presented in Step 7. Sum the frequencies in Column 72.

Step 9: Calculate the Safety Effectiveness, Expressed as an Odds Ratio, OR, at an Individual Treatment Site

Using Equation A-22, Columns 6 and 72, calculate the safety effectiveness, expressed as an odds ratio, OR, at an individual treatment site. The results appear in Column 73 in the table presented in Step 7.

9.11.4. Estimation of the Overall Treatment Effectiveness and its Precision

Step 10: Calculate the Log Odds Ratio (R) for Each Treatment Site

Using Equation A-24 and Column 73, calculate the log odds ratio (R) for each treatment site. The results appear in Column 74.

(1)	(74)	(75)	(76)	(77)
Treatment Sites				
Site No.	**Log odds ratio, R**	**Squared standard error of log odds ratio**	**Weighted Adjustment, w**	**Weighted product**
1	-0.774	0.591	1.69	-1.31
2	0.207	0.695	1.44	0.30
3	0.612	0.802	1.25	0.76
4	-1.467	1.106	0.90	-1.33
5	1.305	2.094	0.48	0.62
6	-0.304	1.289	0.78	-0.24
7	0.669	0.215	4.66	3.12
8	NC	NC	NC	NC
9	NC	NC	NC	NC
10	NC	NC	NC	NC
11	0.809	0.380	2.63	2.13
12	0.992	0.326	3.06	3.04
13	-1.376	1.121	0.89	-1.23
Total			17.78	5.86

NC: Quantities cannot be calculated because zero crashes were observed in after period at these treatment sites

NC: Quantities cannot be calculated because zero crashes were observed in after period at these treatment sites

Step 11: Calculate the Squared Standard Error of the Log Odds Ratio at Each Treatment Site

Using Equation A-26, Columns 5, 6, 69, and 70, calculate the squared standard error of the log odds ratio at each treatment site. The results appear in Column 75 of the table presented with Step 10.

Using Equation A-25 and Column 75, calculate the weight w for each treatment site. The results appear in Column 76 of the table presented with Step 10. Calculate the product of Columns 75 and 76. The results appear in Column 77 of the table presented with Step 10. Sum each of Columns 76 and 77.

Step 12: Calculate the Weighted Average Log Odds ratio, R, Across all Treatment Sites

Using Equation A-27 and the sums from Columns 76 and 77, calculate the weighted average log odds ratio (R) across all treatment sites:

$$R = 5.86/17.78 = 0.33$$

Step 13: Calculate the Overall Effectiveness of the Treatment Expressed as an Odds Ratio

Using Equation A-28 and the result from Step 12, calculate the overall effectiveness of the treatment, expressed as an odds ratio, OR, averaged across all sites:

$$OR = \exp(0.33) = 1.391$$

Step 14: Calculate the Overall Safety Effectiveness, Expressed as a Percentage Change in Crash Frequency, AMF, Averaged across all Sites

Using Equation A-29 and the results from Step 13, calculate the overall safety effectiveness, expressed as a percentage change in crash frequency, AMF, averaged across all sites:

$$AMF = 100 \times (1-1.391) = -39.1\%$$

Note: The negative estimate of AMF indicates a negative effectiveness, i.e. an increase in total crashes.

Step 15: Calculate the Precision of the Treatment Effectiveness

Using Equation A-30 and the results from Step 13 and the sum from Column 76, calculate the precision of the treatment effectiveness:

$$SE(AMF) = 100\frac{1.391}{\sqrt{17.78}} = 33.0\%$$

Step 16: Assess the Statistical Significance of the Estimated Safety Effectiveness

Assess the statistical significance of the estimated safety effectiveness by calculating the quantity:

$$Abs[AMF/SE(AMF)] = 39.1/33.0 = 1.18$$

Since Abs[AMF/SE(AMF)] < 1.7, conclude that the treatment effect is not significant at the (approximate) 90-percent confidence level.

In summary, the evaluation results indicate that an average increase in total crash frequency of 39.1 percent was observed after the installation of passing lanes at the rural two-lane highway sites, but this increase was not statistically significant at the 90-percent confidence level. This sample problem provided different results than the EB evaluation in Section B.1 for two primary reasons. First, a comparison group rather than an SPF was used to estimate future changes in crash frequency at the treatment sites. Second, the three treatment sites at which zero crashes were observed in the period after installation of the passing lanes could not be considered in the comparison group method because of division by zero. These three sites were considered in the EB method. This illustrates a weakness of the comparison group method which has no mechanism for considering these three sites where the treatment appears to have been most effective.

9.12. SAMPLE PROBLEM TO ILLUSTRATE THE SHIFT OF PROPORTIONS SAFETY EFFECTIVENESS EVALUATION METHOD

Passing lanes have been installed to increase passing opportunities at 13 rural two-lane highway sites. An evaluation is to be conducted to determine the overall effect of the installation of these passing lanes on the proportion of fatal-and-injury crashes at the 13 treatment sites.

Data are available for both fatal-and-injury and total crash frequencies for each of the 13 rural two-lane highway sites for five years before and two years after installation of passing lanes. These data can be used to estimate fatal-and-injury crash frequency as a proportion of total crash frequency for the periods before and after implementation of the treatment.

As before, column numbers are shown in the first row of all the tables in this sample problem; the description of the calculations refers to these column numbers for clarity of explanation. When columns are repeated from table to table, the original column number is kept. Where appropriate, column totals are indicated in the last row of each table.

9.12.1. Basic Input Data

Organize the observed before- and after-period total and fatal-and-injury (FI) crash frequencies for the 13 rural two-lane road segments as follows in Columns 1 through 5:

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Site No.	Crash frequency in before period (5 years)		Crash frequency in after period (2 years)		Proportion of FI/TOTAL crashes		Difference in proportions
	Total	FI	Total	FI	Before	After	
1	17	9	3	3	0.53	1.000	0.471
2	6	3	3	2	0.50	0.667	0.167
3	6	2	3	2	0.33	0.667	0.333
4	17	6	3	2	0.35	0.667	0.314
5	1	1	2	1	1.00	0.500	-0.500
6	5	2	3	0	0.40	0.000	-0.400
7	18	12	10	3	0.67	0.300	-0.367
8	12	3	2	1	0.25	0.500	0.250
9	8	1	1	1	0.13	1.000	0.875
10	4	3	1	0	0.75	0.000	-0.750
11	10	1	6	2	0.10	0.333	0.233
12	10	3	7	1	0.30	0.143	-0.157
13	18	4	1	1	0.22	1.000	0.778
Total	132	50	45	19			1.247

9.12.2. Estimate the Average Shift in Proportion of the Target Collision Type

Step 1: Calculate the Before Treatment Proportion

Using Equation A-31 and Columns 2 and 3, calculate the before treatment proportion. The results appear in Column 6 above.

Step 2: Calculate the After Treatment Proportion

Similarly, using Equation A-32 and Columns 4 and 5, calculate the after treatment proportion. The results appear in Column 7 above.

Step 3: Calculate the Difference Between the After and Before Proportions at Each Treatment Site

Using Equation A-33 and Columns 6 and 7, calculate the difference between the after and before proportions at each treatment site. The results appear in Column 8 above. Sum the entries in Column 8.

Step 4: Calculate the Average Difference Between After and Before Proportions over all n Treatment Sites

Using Equation A-34, the total from Column 8, and the number of sites (13), calculate the average difference between after and before proportions over all n treatment sites:

$$AvgP(FI)Diff = 1.247/13 = 0.10$$

This result indicates that the treatment resulted in an observed change in the proportion of fatal-and-injury crashes of 0.10, i.e., a 10-percent increase in proportion.

9.12.3. Assess the Statistical Significance of the Average Shift in Proportion of the Target collision type

Step 5: Obtain the Absolute Value of the Differences in Proportion in Column 8

Using Equation A-35, obtain the absolute value of the differences in proportion in Column 8. The results appear in Column 9 in the table presented in Step 6.

Step 6: Sort the Data in Ascending Order of the Absolute Values in Column 9.

Sort the data in ascending order of the absolute values in Column 9. Assign the corresponding rank to each site. The results appear in Column 10. [Note: sum the numbers in Column 10; this is the maximum total rank possible based on 13 sites.] Organize the data as shown below:

(1)	(8)	(9)	(10)	(11)
Site No.	Difference in proportions	Absolute difference in proportions	Rank	Rank corresponding to positive difference
12	-0.157	0.157	1	0
2	0.167	0.167	2	2
11	0.233	0.233	3	3
8	0.250	0.250	4	4
4	0.314	0.314	5	5
3	0.333	0.333	6	6
7	-0.367	0.367	7	0
6	-0.400	0.400	8	0
1	0.471	0.471	9	9
5	-0.500	0.500	10	0
10	-0.750	0.750	11	0
13	0.778	0.778	12	12
9	0.875	0.875	13	13
Total			91	54

Step 7: Calculate the Value of the T+ Statistic

Replace all ranks (shown in Column 10) associated with negative difference (shown in Column 8) with zero. The results appear in Column 11 in the table presented in Step 6. Sum the ranks in Column 11. This is the value of the T+ statistic in Equation A-36:

$$T+ = 54$$

Step 8: Assess the Statistical Significance of T+ Using a Two-sided Significance Test at the 0.10 Level (90-percent confidence level)

Assess the statistical significance of T+ using a two-sided significance test at the 0.10 level (90-percent confidence level). Using Equation A-37 and Exhibit 9-17, obtain the upper and lower critical limits as:

- Upper limit: t(α2,13) = 70; this corresponds to an α2 of 0.047, the closest value to 0.10/2
- Lower limit: 91 - t(α1,13) = 91 - 69 = 22; here 69 corresponds to an α1 of 0.055, for a total α of 0.047 + 0.055 = 0.102, the closest value to the significance level of 0.10

Since the calculated T+ of 54 is between 22 and 70, conclude that the treatment has not significantly affected the proportion of fatal-and-injury crashes relative to total crashes.

In summary, the evaluation results indicate that an increase in proportion of fatal-and-injury crashes of 0.10 (i.e., 10%) was observed after the installation of passing lanes at the 13 rural two-lane highway sites, but this increase was not statistically significant at the 90-percent confidence level.

9.13. REFERENCES

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APPENDIX A– COMPUTATIONAL PROCEDURES FOR SAFETY EFFECTIVENESS EVALUATION METHODS

This appendix presents computational procedures for three observational before/after safety evaluation methods presented in this chapter, including the EB method, the comparison-group method, and the shift in proportions method.

A.1 COMPUTATIONAL PROCEDURE FOR IMPLEMENTING THE EB BEFORE/AFTER SAFETY EFFECTIVENESS EVALUATION METHOD

A computational procedure using the EB method to determine the safety effectiveness of the treatment being evaluated, expressed as a percentage change in crashes, θ, and to assess its precision and statistical significance, is presented below.

All calculations are shown in Steps 1 through 13 in this section for the total crash frequencies for the before period and after periods, respectively, at a given site. The computational procedure can also be adapted to consider crash frequencies on a year-by-year basis for each site [e.g., see the computational procedure used in the FHWA *SafetyAnalyst* software[(3)].]

EB Estimation of the Expected Average Crash Frequency in the Before Period

Step 1: Using the applicable SPF, calculate the predicted average crash frequency, $N_{predicted}$, for site type x during each year of the before period. For roadway segments, the predicted average crash frequency will be expressed as crashes per site per year; for intersections, the predicted average crash frequency is expressed as crashes per intersection per year. Note that:

$$N_{predicted} = N_{spf\ x} \times (AMF_{1x} \times AMF_{2x} \times \ldots \times AMF_{yx}) \times C_x$$

However for this level of evaluation it may be assumed that all AMFs and C_x are equal to 1.0.

Step 2: Calculate the expected average crash frequency, $N_{expected}$, for each site i, summed over the entire before period. For roadway segments, the expected average crash frequency will be expressed as crashes per site; for intersections, the expected average crash frequency is expressed as crashes per intersection.

$$N_{expected,B} = w_{i,B} N_{predicted,B} + (1 - w_{i,B}) N_{observed,B} \quad \text{(A-1)}$$

Where the weight, $w_{i,B}$, for each site i, is determined as:

$$w_{i,B} = \frac{1}{1 + k \sum_{\substack{Before \\ years}} N_{predicted}} \quad \text{(A-2)}$$

and:

$N_{expected}$ = Expected average crash frequency at site i for the entire before period

$N_{spf\,x}$ = Predicted average crash frequency determined with the applicable SPF (from Step 1)

$N_{observed,B}$ = Observed crash frequency at site i for the entire before period

k = Overdispersion parameter for the applicable SPF

NOTE: If no SPF is available for a particular crash severity level or crash type being evaluated, but that crash type is a subset of another crash severity level or crash type for which an SPF is available, the value of $PR_{i,y,B}$ can be determined by multiplying the SPF-predicted average crash frequency by the average proportion represented by the crash severity level or crash type of interest. This approach is an approximation that is used when a SPF for the crash severity level or crash type of interest cannot be readily developed. If an SPF from another jurisdiction is available, consider calibrating that SPF to local conditions using the calibration procedure presented in the Appendix to *Part C*.

EB Estimation of the Expected Average Crash Frequency in the After Period in the Absence of the Treatment

Step 3: Using the applicable SPF, calculate the predicted average crash frequency, $PR_{i,y,A}$, for each site i during each year y of the after period.

Step 4: Calculate an adjustment factor, r_i, to account for the differences between the before and after periods in duration and traffic volume at each site i as:

$$r_i = \frac{\sum_{\substack{After\\years}} N_{predicted,A}}{\sum_{\substack{Before\\years}} N_{predicted,B}} \quad (A\text{-}3)$$

Step 5: Calculate the expected average crash frequency, $E_{i,A}$, for each site i, over the entire after period in the absence of the treatment as:

$$N_{expected,A} = N_{expected,B} \times r_i \quad (A\text{-}4)$$

Estimation of Treatment Effectiveness

Step 6: Calculate an estimate of the safety effectiveness of the treatment at each site i in the form of an odds ratio, OR_i, as:

$$OR_i = \frac{N_{observed,A}}{N_{expected,A}} \quad (A\text{-}5)$$

Where,

OR_i = Odd ration at site i

$N_{observed,A}$ = Observed crash frequency at site i for the entire after period

Step 7: Calculate the safety effectiveness as a percentage crash change at site i, AMF_i, as:

$$AMF_i = 100 \times (1 - OR_i) \qquad \textit{(A-6)}$$

Step 8: Calculate the <u>overall</u> effectiveness of the treatment for all sites combined, in the form of an odds ratio, OR', as follows:

$$OR' = \frac{\sum_{All\ sites} N_{observed,A}}{\sum_{All\ sites} N_{expected,A}} \qquad \textit{(A-7)}$$

Step 9: The odds ratio, OR', calculated in Equation A-7 is potentially biased; therefore, an adjustment is needed to obtain an unbiased estimate of the treatment effectiveness in terms of an adjusted odds ratio, OR. This is calculated as follows:

$$OR = \frac{OR'}{1 + \dfrac{Var(\sum_{All\ sites} N_{expected,A})}{(\sum_{All\ sites} N_{expected,A})^2}} \qquad \textit{(A-8)}$$

Where,

$$Var(\sum_{All\ sites} N_{expected,A}) = \sum_{All\ sites} [(r_i)^2 \times N_{expected,B} \times (1 - w_{i,B})] \qquad \textit{(A-9)}$$

and $w_{i,B}$ is defined in Equation A-2 and r_i is defined in Equation A-3.

Step 10: Calculate the <u>overall unbiased</u> safety effectiveness as a percentage change in crash frequency across all sites, AMF, as:

$$AMF = 100 \times (1 - OR) \qquad \textit{(A-10)}$$

Estimation of the Precision of the Treatment Effectiveness

To assess whether the estimated safety effectiveness of the treatment, AMF, is statistically significant, one needs to determine its precision. This is done by first calculating the precision of the odds ratio, OR, in Equation A-8. The following steps show how to calculate the variance of this ratio to derive a precision estimate and present criteria assessing the statistical significance of the treatment effectiveness estimate.

Step 11: Calculate the variance of the unbiased estimated safety effectiveness, expressed as an odds ratio, OR, as follows:

$$Var(OR) = \frac{(OR')^2 \left[\frac{1}{N_{observed,A}} + \frac{Var(\sum_{All\ sites} N_{expected,A})}{(\sum_{All\ sites} N_{expected,A})^2} \right]}{\left[1 + \frac{Var(\sum_{All\ sites} N_{expected,A})}{(\sum_{All\ sites} N_{expected,A})^2} \right]} \tag{A-11}$$

Step 12: To obtain a measure of the precision of the odds ratio, OR, calculate its standard error as the square root of its variance:

$$SE(\mathrm{OR}) = \sqrt{Var(\mathrm{OR})} \tag{A-12}$$

Step 13: Using the relationship between OR and AMF shown in Equation A-10, the standard error of AMF, SE(AMF), is calculated as:

$$SE(AMF) = 100 \times SE(OR) \tag{A-13}$$

Step 14: Assess the statistical significance of the estimated safety effectiveness by making comparisons with the measure Abs[AMF/SE(AMF)] and drawing conclusions based on the following criteria:

- If Abs[AMF/SE(AMF)] < 1.7, conclude that the treatment effect is not significant at the (approximate) 90-percent confidence level.
- If Abs[AMF/SE(AMF)] ≥ 1.7, conclude that the treatment effect is significant at the (approximate) 90-percent confidence level.
- If Abs[AMF/SE(AMF)] ≥ 2.0, conclude that the treatment effect is significant at the (approximate) 95-percent confidence level.

A.2 COMPUTATIONAL PROCEDURE FOR IMPLEMENTING THE COMPARISON-GROUP SAFETY EFFECTIVENESS EVALUATION METHOD

A computational procedure using the comparison-group evaluation study method to determine the safety effectiveness of the treatment being evaluated, expressed as a percentage change in crashes, θ, and to assess its precision and statistical significance, is presented below.

Notation: The following notation will be used in presenting the computational procedure for the comparison-group method. Each individual treatment site has a corresponding comparison group of sites, each with their own ADT and number of before and after years. The notation is as follows:

- Subscript i denotes a treatment site, i=1,…,n, where n denotes the total number of treatment sites
- Subscript j denotes a comparison site, j=1,…,m, where m denotes the total number of comparison sites
- Each treatment site i has a number of before years, Y_{BT}, and a number of after years, Y_{AT}
- Each comparison site j has a number of before years, Y_{BC}, and a number of after years, Y_{AC}
- It is assumed for this section that Y_{BT} is the same across all treatment sites; that Y_{AT} is the same across all treatment sites; that Y_{BC} is the same across all comparison sites; and that Y_{AC} is the same across all comparison sites. Where this is not the case, computations involving the durations of the before and after periods may need to vary on a site-by-site basis.

The following symbols are used for <u>observed</u> crash frequencies, in accordance with Hauer's notation [5]:

	Before Treatment	After Treatment
Treatment Site	$N_{observed,T,B}$	$N_{observed,T,A}$
Comparison Group	$N_{observed,C,B}$	$N_{observed,C,A}$

Estimation of Mean Treatment Effectiveness

Step 1a: Using the applicable SPF and site-specific ADT, calculate $\Sigma N_{predicted,T,B}$, the sum of the <u>predicted average crash</u> frequencies at <u>treatment</u> site i in <u>before</u> period.

Step 1b: Using the applicable SPF and site-specific AADT, calculate $\Sigma N_{predicted,T,A}$, the sum of the <u>predicted average crash</u> frequencies at <u>treatment</u> site i in <u>after</u> period.

Step 2a: Using the applicable SPF and site-specific AADT, calculate $\Sigma N_{predicted,C,B}$, the sum of the <u>predicted average crash</u> frequencies at <u>comparison</u> site j in <u>before</u> period.

Step 2b: Using the applicable SPF and site-specific AADT, calculate $\Sigma N_{predicted,C,A}$, the sum of the predicted average crash frequencies at comparison site j in after period.

Step 3a: For each treatment site i and comparison site j combination, calculate an adjustment factor to account for differences in traffic volumes and number of years between the treatment and comparison sites during the before period as follows:

$$Adj_{i,j,B} = \frac{N_{predicted,T,B}}{N_{predicted,C,B}} \times \frac{Y_{BT}}{Y_{BC}} \qquad \text{(A-14)}$$

Where,

$N_{predicted,T,B}$ = Sum of predicted average crash frequencies at treatment site i in before period using the appropriate SPF and site-specific AADT;

$N_{predicted,C,B}$ = Sum of predicted average crash frequencies at comparison site j in before period using the same SPF and site-specific AADT;

Y_{BT} = Duration (years) of before period for treatment site i; and

Y_{BC} = Duration (years) of before period for comparison site j.

Step 3b: For each treatment site i and comparison site j combination, calculate an adjustment factor to account for differences in AADTs and number of years between the treatment and comparison sites during the after period as follows:

$$Adj_{i,j,A} = \frac{N_{predicted,T,A}}{N_{predicted,C,A}} \times \frac{Y_{AT}}{Y_{AC}} \qquad \text{(A-15)}$$

Where,

$N_{predicted,T,A}$ = Sum of predicted average crash frequencies at treatment site i in after period using the appropriate SPF and site-specific AADT;

$N_{predicted,C,A}$ = Sum of predicted average crash frequencies at comparison site j in the after period using the same SPF and site-specific AADT;

Y_{AT} = Duration (years) of after period for treatment site i; and

Y_{AC} = Duration (years) of after period for comparison site j

Step 4a: Using the adjustment factors calculated in Equation A-14, calculate the expected average crash frequencies in the before period for each comparison site j and treatment site i combination, as follows:

$$N_{expected,C,B} = \sum_{All\ sites} N_{observed,C,B} \times Adj_{i,j,B} \qquad \text{(A-16)}$$

Where,

$\Sigma N_{observed,C,B}$ = Sum of <u>observed</u> crash frequencies at comparison site j in the <u>before</u> period

Step 4b: Using the adjustment factor calculated in Equation A-15, calculate the <u>expected average</u> crash frequencies in the <u>after</u> period for each comparison site j and treatment site i combination, as follows:

$$N_{expected,C,A} = \sum_{All\ sites} N_{observed,C,A} \times Adj_{i,j,A} \qquad (A\text{-}17)$$

Where,

N_j = Sum of <u>observed</u> crash frequencies at comparison site j in the <u>after</u> period

Step 5: For each treatment site i, calculate the <u>total comparison-group expected</u> average crash frequency in the <u>before</u> period as follows:

$$N_{expected,C,B,total} = \sum_{All\ comparison\ sites} N_{expected,C,B} \qquad (A\text{-}18)$$

Step 6: For each treatment site i, calculate the <u>total comparison-group expected</u> average crash frequency in the <u>after</u> period as follows:

$$N_{expected,C,A,total} = \sum_{All\ comparison\ sites} N_{expected,C,A} \qquad (A\text{-}19)$$

Step 7: For each treatment site i, calculate the comparison ratio, r_{iC}, as the ratio of the <u>comparison-group expected average</u> crash frequency <u>after</u> period to the <u>comparison-group expected</u> average crash frequency in the <u>before</u> period at the comparison sites as follows:

$$r_{iC} = \frac{N_{expected,C,A,total}}{N_{expected,C,B,total}} \qquad (A\text{-}20)$$

Step 8: Using the comparison ratio calculated in Equation A-20, calculate the <u>expected</u> average crash frequency for a treatment site i in the <u>after</u> period, had no treatment been implement as follows:

$$N_{expected,T,A} = \sum_{All\ sites} N_{observed,T,B} \times r_{iC} \qquad (A\text{-}21)$$

Step 9: Using Equation A-22, calculate the safety effectiveness, expressed as an odds ratio, OR_i, at an individual treatment site i as the ratio of the expected average crash frequency with the treatment over the expected average crash frequency had the treatment not been implemented, as follows:

$$OR_i = \sum_{All\ sites} N_{observed,T,A} / N_{expected,T,A} \qquad (A\text{-}22)$$

Or alternatively,

$$OR_i = \frac{N_{observed,T,A,total}}{N_{observed,T,B,total}} \times \frac{N_{expected,C,B,total}}{N_{expected,C,A,total}} \quad \text{(A-23)}$$

Where,

$N_{observed,T,A,total}$ and $N_{observed,T,B,total}$ represent the total treatment group observed crash frequencies at treatment site i calculated as the sum of $N_{observed,T,A}$ and $N_{observed,T,B}$ for all sites;

The next steps show how to estimate weighted average safety effectiveness and its precision based on individual site data.

Step 10: For each treatment site i, calculate the log odds ratio, R_i, as follows:

$$R_i = \ln(OR_i) \quad \text{(A-24)}$$

Where the ln function represents the natural logarithm.

Step 11: For each treatment site i, calculate the weight w_i as follows:

$$w_i = 1 / R_{i(se)}^2 \quad \text{(A-25)}$$

Where,

$$R_{i(se)}^2 = \frac{1}{N_{observed,T,B,total}} + \frac{1}{N_{observed,T,A,total}} + \frac{1}{N_{expected,C,B,total}} + \frac{1}{N_{expected,C,A,total}} \quad \text{(A-26)}$$

Step 12: Using Equation A-27, calculate the weighted average log odds ratio, R, across all n treatment sites as:

$$R = \frac{\sum_n w_i R_i}{\sum_n w_i} \quad \text{(A-27)}$$

Step 13: Exponentiating the result from Equation A-27, calculate the overall effectiveness of the treatment, expressed as an odds ratio, OR, averaged across all sites, as follows:

$$OR = e^R \quad \text{(A-28)}$$

Step 14: Calculate the overall safety effectiveness, expressed as a percentage change in crash frequency, AMF, averaged across all sites as:

$$AMF = 100 \times (1 - OR) \quad \text{(A-29)}$$

Step 15: To obtain a measure of the precision of the treatment effectiveness, AMF, calculate its standard error, SE(AMF), as follows:

$$SE(AMF) = 100\frac{OR}{\sqrt{\sum_{n} w_i}} \qquad \textit{(A-30)}$$

Step 16: Assess the statistical significance of the estimated safety effectiveness by making comparisons with the measure Abs[AMF/SE(AMF)] and drawing conclusions based on the following criteria:

- If Abs[AMF/SE(AMF)] < 1.7, conclude that the treatment effect is not significant at the (approximate) 90-percent confidence level.
- If Abs[AMF/SE(AMF)] ≥ 1.7, conclude that the treatment effect is significant at the (approximate) 90-percent confidence level.
- If Abs[AMF/SE(AMF)] ≥ 2.0, conclude that the treatment effect is significant at the (approximate) 95-percent confidence level.

A.3 COMPUTATIONAL PROCEDURE FOR IMPLEMENTING THE SHIFT OF PROPORTIONS SAFETY EFFECTIVENESS EVALUATION METHOD

A computational procedure using the evaluation study method for assessing shifts in proportions of target collision types to determine the safety effectiveness of the treatment being evaluated, $AvgP_{(CT)Diff}$, and to assess its statistical significance, is presented below.

This step-by-step procedure uses the same notation as that used in the traditional comparison-group safety evaluation method. All proportions of specific crash types (subscript SCT) are relative to total crashes (subscript TOT).

- $N_{observed,B,TOT}$ denotes the observed number of TOT crashes at treatment site i over the entire before treatment period.
- $N_{observed,B.CT}$ denotes the observed number of CT crashes of a specific crash type at treatment site i over the entire before treatment period.
- $N_{observed,A,TOT}$ denotes the observed number of TOT crashes at treatment site i over the entire after treatment period.
- $N_{observed,A,CT}$ denotes the observed number of CT crashes of a specific crash type at treatment site i over the entire after treatment period.

Estimate the Average Shift in Proportion of the Target Collision Type

Step 1: Calculate the before treatment proportion of observed crashes of a specific target collision type (CT) relative to total crashes (TOT) at treatment site i, $P_{i(CT)B}$, across the entire before period as follows:

$$P_{i(CT)B} = \frac{N_{observed,B,CT}}{N_{observed,B,TOT}} \qquad \textit{(A-31)}$$

Step 2: Similarly, calculate the after treatment proportion of observed crashes of a specific target collision type of total crashes at treatment site i, $P_{i(CT)A}$, across the entire after period as follows:

$$P_{i(CT)A} = \frac{N_{observed,A,CT}}{N_{observed,A,TOT}} \quad \textit{(A-32)}$$

Step 3: Determine the difference between the after and before proportions at each treatment site i as follows:

$$P_{i(CT)Diff} = P_{i(CT)A} - P_{i(CT)B} \quad \textit{(A-33)}$$

Step 4: Calculate the average difference between after and before proportions over all n treatment sites as follows:

$$AvgP_{(CT)Diff} = \frac{1}{n} \sum_{\substack{Treat \\ sites}} P_{i(CT)Diff} \quad \textit{(A-34)}$$

Assess the Statistical Significance of the Average Shift in Proportion of the Target Collision Type

The following steps demonstrate how to assess whether the treatment significantly affected the proportion of crashes of the collision type under consideration. Because the site-specific differences in Equation A-34 do not necessarily come from a normal distribution and because some of these differences may be equal to zero, a nonparametric statistical method, the Wilcoxon signed rank test, is used to test whether the average difference in proportions calculated in Equation A-34 is significantly different from zero at a predefined confidence level.

Step 5: Take the absolute value of the <u>non-zero</u> $P_{i(CT)Diff}$ calculated in Equation A-33. For simplicity of notation, let Z_i denote the absolute value of $P_{i(CT)Diff}$, thus:

$$Z_i = abs(P_{i(CT)Diff}) \quad \textit{(A-35)}$$

Where,

i= 1,…,n*, with n* representing the (reduced) number of treatment sites with non-zero differences in proportions.

Step 6: Arrange the n* Z_i values in ascending rank order. When multiple Z_i have the same value (i.e., ties are present), use the average rank as the rank of each tied value of Z_i. For example, if three Z_i values are identical and would rank, say, 12, 13, and 14, use 13 as the rank for each. If the ranks would be, say, 15 and 16, use 15.5 as the rank for each. Let R_i designate the rank of the Z_i value.

Step 7: Using only the ranks associated with <u>positive</u> differences (i.e., positive values of $P_{i(CT)Diff}$), calculate the statistic T^+ as follows:

$$T^+ = \sum_{n^*} R_i^+ \quad \textit{(A-36)}$$

Step 8: Assess the statistical significance of T^+ using a two-sided significance test at the α level of significance (i.e. [1- α] confidence level) as follows:

- Conclude that the treatment is statistically significant if:

$$T^+ \geq t(\alpha_2, n^*) \text{ or } T^+ \leq \frac{n^*(n^*+1)}{2} - t(\alpha_1, n^*) \tag{A-37}$$

Where,

$\alpha = \alpha_1 + \alpha_2$

- Otherwise, conclude that the treatment is not statistically significant

The quantities $t(\alpha_1,n^*)$ and $t(\alpha_2,n^*)$ are obtained from the table of critical values for the Wilcoxon signed rank test, partially reproduced in Exhibit 9-12. Generally, α_1 and α_2 are approximately equal to $\alpha/2$. Choose the values for α_1 and α_2 so that $\alpha_1 + \alpha_2$ is closest to α in Exhibit 9-12 and α_1 and α_2 are each closest to $\alpha/2$. Often, $\alpha_1 = \alpha_2$ are the closest values to $\alpha/2$.

Exhibit 9-12 presents only an excerpt of the full table of critical values shown in Hollander and Wolfe (*8*). A range of significance levels (α) has been selected to test a change in proportion of a target collision type: approximately 10 to 20 percent. Although 5 to 10 percent are more typical significance levels used in statistical tests, the a 20-percent significance level has been included here because the Wilcoxon signed rank test is a conservative test (i.e., it is difficult to detect a significant effect when it is present). Exhibit 9-12 shows one-sided probability levels; since the test performed here is a two-sided test, the values in Exhibit 9-12 correspond to $\alpha/2$, with values ranging from 0.047 to 0.109 (corresponding to 0.094/2 to 0.218/2).

Example for Using Exhibit 9-12

Assume $T^+ = 4$, $n^* = 9$, and $\alpha = 0.10$ (i.e., 90-percent confidence level). The value of $t(\alpha_2,n^*) = t(0.049,9) = 37$ from Exhibit 9-12, the closest value corresponding to $\alpha = 0.10/2$ in the column for $n^* = 9$. In this case, $t(\alpha_1,n^*) = t(\alpha_2,n^*)$. Thus, the two critical values are 37 and 8 [$=9\times(9+1)/2 - 37 = 45 - 37 = 8$]. Since $T^+ = 4 < 8$, the conclusion would be that the treatment was statistically significant (i.e., effective) at the 90.2% confidence level [where $90.2 = 1 - 2 \times 0.049$] based on Equation A-37.

Exhibit 9-12: Upper Tail Probabilities for the Wilcoxon Signed Rank T+ Statistic (n* = 4 to 10)a (8)

x	Number of sites (n*)						
	4	**5**	**6**	**7**	**8**	**9**	**10**
10	0.062						
13		0.094					
14		0.062					
17			0.109				
18			0.078				
19			0.047				
22				0.109			
23				0.078			
24				0.055			
28					0.098		
29					0.074		
30					0.055		
34						0.102	
35						0.082	
36						0.064	
37						0.049	
41							0.097
42							0.080
43							0.065
44							0.053

[a] For a given n*, the table entry for the point x is $P(T^+ \geq x)$. Thus if x is such that $P(T^+ \geq x) = \alpha$, then $t(\alpha, n^*) = x$.

Exhibit 9-12 (Continued): Upper Tail Probabilities for the Wilcoxon Signed Rank T^+ Statistic (n* = 11 to 15)[a] (*8*)

x	Number of sites (n*)				
	11	12	13	14	15
48	0.103				
49	0.087				
50	0.074				
51	0.062				
52	0.051				
56		0.102			
57		0.088			
58		0.076			
59		0.065			
60		0.055			
64			0.108		
65			0.095		
66			0.084		
67			0.073		
68			0.064		
69			0.055		
70			0.047		
73				0.108	
74				0.097	
75				0.086	
76				0.077	
77				0.068	
78				0.059	
79				0.052	
83					0.104
84					0.094
85					0.084
86					0.076
87					0.068
88					0.060
89					0.053
90					0.047

[a] For a given n*, the table entry for the point x is $P(T^+ \geq x)$. Thus if x is such that $P(T^+ \geq x) = \alpha$, then $t(\alpha,n^*) = x$.

Large Sample Approximation (n > 15)*

Exhibit 9-12 provides critical values for T^+ for values of n* = 4 to 15 in increments of 1. Thus a minimum n* of 4 sites is required to perform this test. In those cases where n* exceeds 15, a large sample approximation is used to test the significance of T^+. The following steps show the approach to making a large sample approximation *8*):

Step 9: Calculate the quantity T* as follows:

$$T^* = \frac{T^+ - E_0(T^+)}{\sqrt{Var_0(T^+)}} \qquad \text{(A-38)}$$

Where,

$$E_0(T^+) = n^*(n^*+1)/4 \qquad \text{(A-39)}$$

And

$$Var_0(T^+) = \left[n^*(n^*+1)(2n^*+1) - \frac{1}{2}\sum_{j=1}^{g} t_j(t_j-1)(t_j+1) \right] / 24 \qquad \text{(A-40)}$$

Where,

g = number of tied groups and t_j = size of tied group j.

Step 10: For the large-sample approximation procedure, assess the statistical significance of T* using a two-sided test at the α level of significance as follows:

- Conclude that the treatment is statistically significant if:

$$T^* \geq z_{\alpha/2} \; or \; T^* \leq -z_{\alpha/2} \qquad \text{(A-41)}$$

Where,

$z_{(\alpha/2)}$ = the upper tail probability for the standard normal distribution.

Selected values of $z_{(\alpha/2)}$ are as follows:

α		$z_{(\alpha/2)}$
0.05		1.960
0.10		1.645
0.15		1.440
0.20		1.282

- Otherwise, conclude that the treatment is not statistically significant

PART C—PREDICTIVE METHOD

INTRODUCTION AND APPLICATIONS GUIDANCE

C.1. Introduction to the HSM Predictive Method C-1
C.2. Relationship to Parts A, B, and D of the HSM C-2
C.3. Part C and the Project Development Process C-2
C.4. Overview of the HSM Predictive Method C-3
C.5. The HSM Predictive Method C-6
C.6. Predictive Method Concepts C-14
- C.6.1. Roadway Limits and Facility Types C-14
- C.6.2. Definition of Roadway Segments and Intersections C-15
- C.6.3. Safety Performance Functions C-16
- C.6.4. Accident Modification Factors C-18
- C.6.5. Calibration of Safety Performance Functions to Local Conditions C-21
- C.6.6. Weighting Using the Empirical Bayes Method C-21

C.7. Methods for Estimating the Safety Effectiveness of a Proposed Project C-22
C.8. Limitations of the HSM Predictive Method C-23
C.9. Guide to Applying Part C C-24
C.10. Summary C-25

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PART C INTRODUCTION AND APPLICATIONS GUIDANCE

C.1. INTRODUCTION TO THE HSM PREDICTIVE METHOD

Part C of the HSM provides a predictive method for estimating expected average crash frequency (including by crash severity and collision types) of a network, facility, or individual site. The estimate can be made for existing conditions, alternatives to existing conditions (e.g., proposed upgrades or treatments), or proposed new roadways. The predictive method is applied to a given time period, traffic volume, and constant geometric design characteristics of the roadway.

Part C of the HSM provides a predictive method for estimating expected average crash frequency (including by crash severity and collision types) of a network, facility, or individual site.

The predictive method provides a quantitative measure of expected average crash frequency under both existing conditions and conditions which have not yet occurred. This allows proposed roadway conditions to be quantitatively assessed along with other considerations such as community needs, capacity, delay, cost, right-of-way, and environmental considerations.

The predictive method can be used for evaluating and comparing the expected average crash frequency of situations like:

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

Part C Introduction and Applications Guidance presents the predictive method in general terms for the first time user to understand the concepts applied in each of the *Part C* chapters. Each chapter in *Part C* provides the detailed steps of the predictive method and the predictive models required to estimate the expected average crash frequency for a specific facility type. The following roadway facility types are included in *Part C*:

- **Chapter 10** - Rural Two-Lane Two-Way Roads
- **Chapter 11** – Rural Multilane Highways
- **Chapter 12** – Urban and Suburban Arterials

The Part C Introduction and Applications Guidance provides:

- Relationships between *Part C* and *Parts A*, *B* and *D* of the HSM;
- Relationship between *Part C* and the Project Development Process;
- An overview of the predictive method;
- A summary of the predictive method;

- Detailed information needed to understand the concepts and elements in each of the steps of the predictive method;
- Methods for estimating the change in crash frequency due to a treatment;
- Limitations of the predictive method;
- Guidance for applying the predictive method.

C.2. RELATIONSHIP TO PARTS A, B, AND D OF THE HSM

All information needed to apply the predictive method is presented in *Part C*. The relationships of the predictive method in *Part C* to the contents of *Parts A*, *B*, and *D* are summarized below.

Chapter 3 of the HSM includes fundamental concepts in Part C.

- *Part A* introduces concepts that are fundamental to understanding the methods provided in the HSM to analyze and evaluate crash frequencies. *Part A* introduces the key components of the predictive method, including Safety Performance Functions (SPFs) and Accident Modification Factors (AMFs). Prior to using the information in *Part C*, an understanding of the material in *Part A*, *Chapter 3 Fundamentals* is recommended.

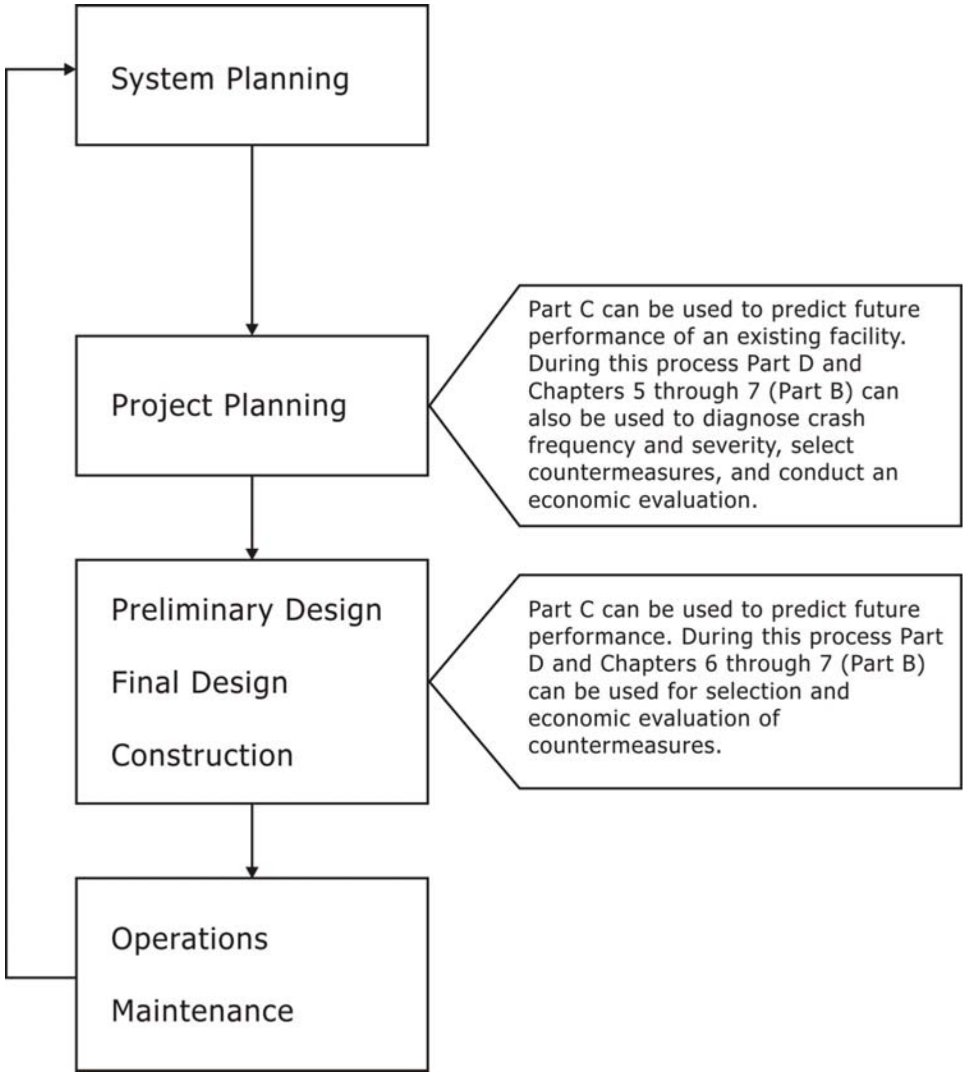
The predictive method in Part C in used to estimate expected average crash frequency for application in Part B.

- *Part B* presents the six basic components of a roadway safety management process. The material is useful for monitoring, improving, and maintaining an existing roadway network. Applying the methods and information presented in *Part B* can help to identify sites most likely to benefit from an improvement, diagnose accident patterns at specific sites, select appropriate countermeasures likely to reduce crashes, and anticipate the benefits and costs of potential improvements. In addition, it helps agencies determine whether potential improvements are economically justified, establish priorities for potential improvements, and assess the effectiveness of improvements that have been implemented. The predictive method in *Part C* provides tools to estimate the expected average crash frequency for application in *Part B Chapter 4 Network Screening* and *Chapter 7 Economic Appraisal*.
- *Part D* contains all AMFs in the HSM. The AMFs in *Part D* are used to estimate the change in expected average crash frequency as a result of implementing a countermeasure(s). Some *Part D* AMFs are included in *Part C* for use with specific SPFs. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7.

C.3. PART C AND THE PROJECT DEVELOPMENT PROCESS

Exhibit C-1 illustrates the relationship of the *Part C* predictive method to the project development process. As discussed in *Chapter 1*, the project development process is the framework used in the HSM to relate crash analysis to activities within planning, design, construction, operations, and maintenance.

Exhibit C-1: Relation between Part C Predictive Method and the Project Development Process



Chapter 1 provides a summary of the Project Development Process.

C.4. OVERVIEW OF THE HSM PREDICTIVE METHOD

The result from the predictive method is the "expected average crash frequency", $N_{expected}$, which is an estimate of a site's long term average crash frequency.

The predictive method provides an 18 step procedure to estimate the "expected average crash frequency" (by total crashes, crash severity or collision type) of a roadway network, facility, or site. In the predictive method the roadway is divided into individual sites, which are either homogenous roadway segments or intersections. A facility consists of a contiguous set of individual intersections and roadway segments, each referred to as "sites." Different facility types are determined by surrounding land use, roadway cross-section, and degree of access. For each facility type a number of different site types may exist, such as divided and undivided roadway segments, and unsignalized and signalized intersections. A roadway network consists of a number of contiguous facilities.

The predictive method is used to estimate the expected average crash frequency of an individual site. The cumulative sum of all sites is used as the estimate for an entire facility or network. The estimate is for a given time period of interest (in years) during which the geometric design and traffic control features are unchanged and traffic volumes (AADT) are known or forecast. The estimate relies upon regression models developed from observed crash data for a number of similar sites.

The predicted average crash frequency of an individual site, $N_{predicted}$, is estimated based on the geometric design, traffic control features, and traffic volumes of that site. For an existing site or facility, the observed crash frequency, $N_{observed}$, for that specific site or facility is then combined with $N_{predicted}$, to improve the statistical reliability of the estimate. The result from the predictive method is the expected average crash frequency, $N_{expected}$. This is an estimate of the long term average crash frequency that would be expected, given sufficient time to make a controlled observation, which is rarely possible. Once the expected average crash frequencies have been determined for all the individual sites that make up a facility or network, the sum of the crash frequencies for all of the sites is used as the estimate of the expected average crash frequency for an entire facility or network.

Chapter 3 provides information about regression-to-the-mean bias.

As discussed in Section 3.3.3 in *Chapter 3*, the observed crash frequency (number of crashes per year) will fluctuate randomly over any period and, therefore, using averages based on short term periods (e.g., 1 to 3 years) may give misleading estimates and create problems associated with regression-to-the-mean bias. The predictive method addresses these concerns by providing an estimate of long-term average crash frequency, which allows for sound decisions about improvement programs.

"Base conditions" are the specific geometric design and traffic control features of the Safety Performance Function.

In the HSM, predictive models are used to estimate the predicted average crash frequency, $N_{predicted}$, for a particular site type using a regression model developed from data for a number of similar sites. These regression models, called Safety Performance Functions (SPFs), have been developed for specific site types and "base conditions" which are the specific geometric design and traffic control features of a "base" site. SPFs are typically a function of only a few variables, primarily AADT.

AMFs adjust the SPF from "base conditions" to local conditions. AMFs are described in Chapter 3.

Adjustment to the prediction made by a SPF is required to account for the difference between base conditions, specific site conditions, and local/state conditions. Accident Modification Factors (AMFs) are used to account for the specific site conditions which vary from the base conditions. For example, the SPF for roadway segments in *Chapter 10* has a base condition of 12-ft lane width, but the specific site may be a roadway segment with a 10-ft lane width. A general discussion of AMFs is provided in Section C.6.4.

AMFs included in *Part C* chapters have the same base conditions as the SPFs in *Part C* and, therefore, the AMF = 1.00 when the specific site conditions are the same as the SPF base conditions.

A calibration factor (C_x) is used to account for differences between the jurisdiction(s) for which the models were developed and the jurisdiction for which the predictive method is applied. The use of calibration factors is described in Section C.6.5 and the procedure to determine calibration factors for a specific jurisdiction is described in the *Part C* Appendix.

The predictive models used in *Part C* to determine the predicted average crash frequency, $N_{predicted}$, are of the general form shown in Equation C-1.

$$N_{predicted} = N_{spf\ x} \times (AMF_{1x} \times AMF_{2x} \times \ldots \times AMF_{yx}) \times C_x \qquad \text{(C-1)}$$

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*;

AMF_{yx} = Accident Modification Factors specific to SPF for site type *x*;

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

For existing sites, facilities, or roadway networks, the empirical Bayes (EB) Method is applied within the predictive method to combine predicted average crash frequency determined using a predictive model, $N_{predicted}$, with the observed crash frequency, $N_{observed}$ (where applicable). A weighting is applied to the two estimates which reflects the statistical reliability of the SPF. The EB Method applies only when observed crash data are available. A discussion of the EB Method is presented in the *Part C* Appendix. The EB Method may be applied at the site-specific level when crashes can be assigned to individual sites (i.e., detailed geographic location of the observed crashes is known). Alternatively, the EB Method can be applied at the project-specific level (i.e., to an entire facility or network) when crashes cannot be assigned to individual sites but are known to occur within general geographic limits (i.e., detailed geographic locations of crashes are not available). As part of the EB Method, the expected average crash frequency can also be estimated for a future time period, when AADT may have changed or specific treatments or countermeasures may have been implemented.

The predictive method combines predicted average crash frequency determined using a predictive model, $N_{predicted}$, with the observed crash frequency $N_{observed}$, using the EB Method

The EB Method is presented in the Part C Appendix.

Advantages of the predictive method are that:

- Regression-to-the-mean bias is addressed as the method concentrates on long term expected average crash frequency rather than short-term observed crash frequency.
- Reliance on availability of crash data for any one site is reduced by incorporating predictive relationships based on data from many similar sites.
- The SPF models in the HSM are based on the negative binomial distribution, which are better suited to modeling the high natural variability of crash data than traditional modeling techniques, which are based on the normal distribution.
- The predictive method provides a method of crash estimation for sites or facilities that have not been constructed or have not been in operation long enough to make an estimate based on observed crash data.

The following sections provide the general 18 steps of the predictive method and detailed information about each of the concepts or elements presented in the predictive method. The information in the Part C Introduction and Applications Guidance chapter provides a brief summary of each step. Detailed information on each step and the associated predictive models are provided in the *Part C* chapters for each of the following facility types:

- **Chapter 10** - Rural Two-Lane Two-Way Roads
- **Chapter 11** – Rural Multilane Highways
- **Chapter 12** – Urban and Suburban Arterials

C.5. THE HSM PREDICTIVE METHOD

While the general form of the predictive method is consistent across the chapters, the predictive models vary by chapter and therefore the detailed methodology for each step may vary. The generic overview of the predictive method presented here is intended to provide the first time or infrequent user with a high level review of the steps in the method and the concepts associated with the predictive method. The detailed information for each step and the associated predictive models for each facility type are provided in *Chapters 10*, *11*, and *12*. Exhibit C-2 identifies the specific facility and site types for which Safety Performance Functions have been developed for the HSM.

Exhibit C-2: Safety Performance Functions by Facility Type and Site Types in Part C

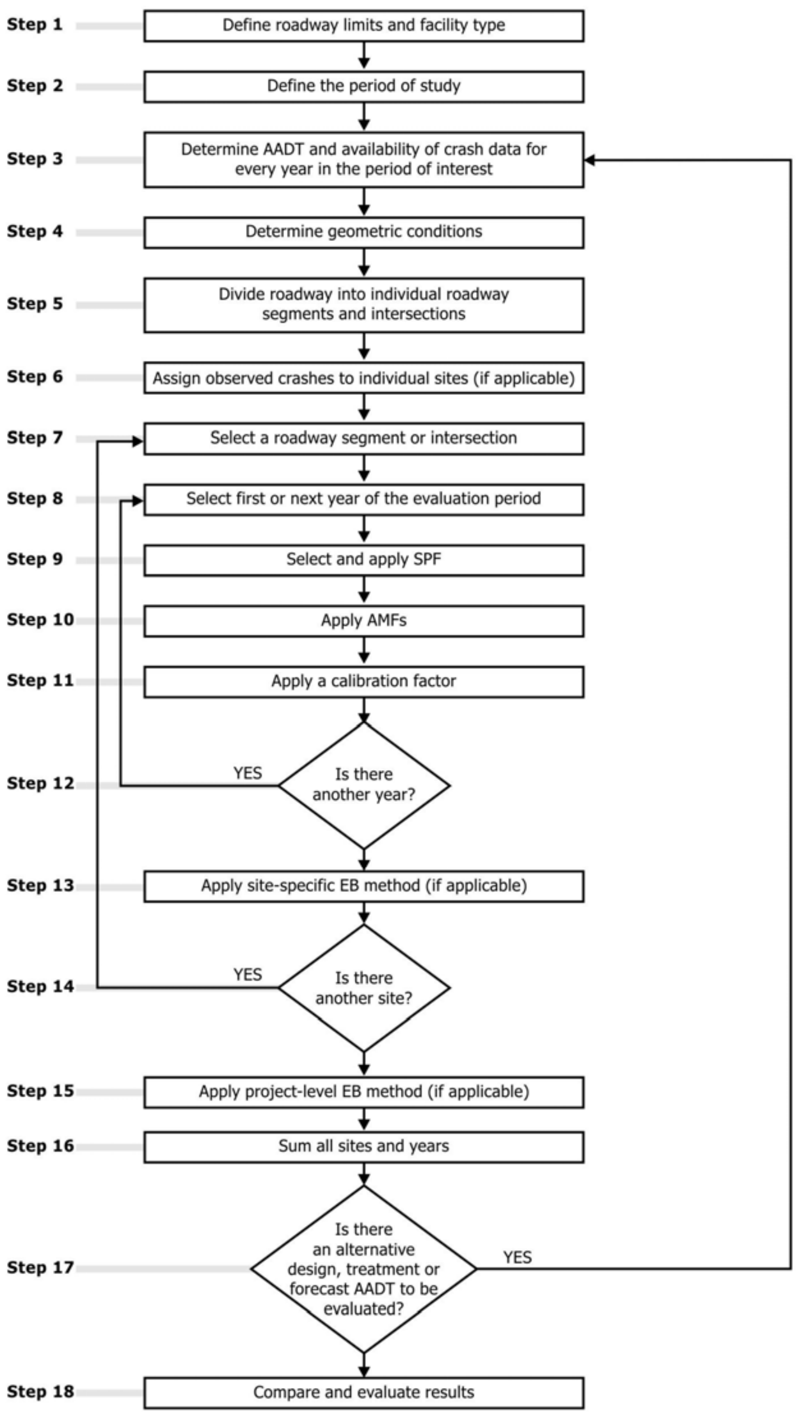
HSM Chapter / Facility Type	Undivided Roadway Segments	Divided Roadway Segments	Intersections			
			Stop Control on Minor Leg(s)		Signalized	
			3-Leg	4-Leg	3-Leg	4-Leg
10 - Rural Two-Lane Two-Way Roads	✓	-	✓	✓	-	✓
11 - Rural Multilane Highways	✓	✓	✓	✓	-	✓
12 - Urban and Suburban Arterials	✓	✓	✓	✓	✓	✓

The predictive method in *Chapters 10, 11,* and *12* consists of 18 steps. The elements of the predictive models that were discussed in Section C.4 are determined and applied in Steps 9, 10, and 11 of the predictive method. The 18 steps of the HSM predictive method are detailed below and shown graphically in Exhibit C-3. Brief detail is provided for each step, and material outlining the concepts and elements of the predictive method is provided in the following sections of the Part C Introduction and Applications Guidance or in the *Part C* Appendix. In some situations, certain steps will not require any action. For example, a new site or facility will not have observed crash data and, therefore, steps relating to the EB Method are not performed.

Section C.5 describes each of the 18 steps in the predictive method.

Where a facility consists of a number of contiguous sites or crash estimation is desired for a period of several years, some steps are repeated. The predictive method can be repeated as necessary to estimate crashes for each alternative design, traffic volume scenario or proposed treatment option within the same period to allow for comparison.

Exhibit C-3: The HSM Predictive Method



Step 1 - Define the limits of the roadway and facility types in the study network, facility, or site for which the expected average crash frequency, severity, and collision types are to be estimated.

The predictive method can be undertaken for a roadway network, a facility, or an individual site. The facility types included in the HSM are outlined in Section C.6.1. A site is either an intersection or homogeneous roadway segment. There are a number of different types of sites, such as signalized and unsignalized intersections or undivided and divided roadway segments. The site types included in the HSM are indicated in Exhibit C-2.

The predictive method can be applied to an existing roadway, a design alternative for an existing roadway, or a design alternative for new roadway (which may be either unconstructed or yet to experience enough traffic to have observed crash data).

The limits of the roadway of interest will depend on the nature of the study. The study may be limited to only one specific site or a group of contiguous sites. Alternatively, the predictive method can be applied to a long corridor for the purposes of network screening (determining which sites require upgrading to reduce crashes) which is discussed in *Chapter 4*.

Step 2 - Define the period of interest.

The predictive method can be undertaken for a past period or a future period. All periods are measured in years. Years of interest will be determined by the availability of observed or forecast AADTs, observed crash data, and geometric design data. Whether the predictive method is used for a past or future period depends upon the purpose of the study. The period of study may be:

A past period (based on observed AADTs) for:

- An existing roadway network, facility, or site. If observed crash data are available, the period of study is the period of time for which the observed crash data are available and for which (during that period) the site geometric design features, traffic control features, and traffic volumes are known.
- An existing roadway network, facility, or site for which alternative geometric design features or traffic control features are proposed (for near term conditions).

A future period (based on forecast AADTs) for:

- An existing roadway network, facility, or site for a future period where forecast traffic volumes are available.
- An existing roadway network, facility, or site for which alternative geometric design or traffic control features are proposed for implementation in the future.
- A new roadway network, facility, or site that does not currently exist, but is proposed for construction during some future period.

Step 3 – For the study period, determine the availability of annual average daily traffic volumes and, for an existing roadway network, the availability of observed crash data to determine whether the EB Method is applicable.

Determining Traffic Volumes

The SPFs used in Step 9 (and some AMFs in Step 10), require AADT volumes (vehicles per day). For a past period, the AADT may be determined by automated recording or estimated by a sample survey. For a future period, the AADT may be a forecast estimate based on appropriate land use planning and traffic volume forecasting models, or based on the assumption that current traffic volumes will remain relatively constant.

The predictive models require AADT data/volumes. If AADT are not available, although not the same, average daily traffic (ADT) volumes/data can be used.

For each roadway segment, the AADT is the average daily two-way 24 hour traffic volume on that roadway segment in each year of the period to be evaluated (selected in Step 8).

For each intersection, two values are required in each predictive model. These are the AADT of the major street, $AADT_{maj}$, and the AADT of the minor street, $AADT_{min}$. The method for determining $AADT_{maj}$ and $AADT_{min}$ varies between chapters because the predictive models in *Chapters 10*, *11*, and *12* were developed independently.

In many cases, it is expected that AADT data will not be available for all years of the evaluation period. In that case, an estimate of AADT for each year of the evaluation period is determined by interpolation or extrapolation as appropriate. If there is not an established procedure for doing this, the following default rules can be applied:

- If AADT data are available for only a single year, that same value is assumed to apply to all years of the before period;
- If two or more years of AADT data are available, the AADTs for intervening years are computed by interpolation;
- The AADTs for years before the first year for which data are available are assumed to be equal to the AADT for that first year;
- The AADTs for years after the last year for which data are available are assumed to be equal to the last year.

If the EB Method is to be used (discussed below), AADT data are needed for each year of the period for which observed crash frequency data are available. If the EB Method will not be used, AADT data for the appropriate time period—past, present, or future—determined in Step 2 are used.

Determining Availability of Observed Crash Data

Where an existing site or alternative conditions to an existing site are being considered, the EB Method is used. The EB Method is only applicable when reliable, observed crash data are available for the specific study roadway network, facility, or site. Observed data may be obtained directly from the jurisdiction's crash report system. At least two years of observed crash frequency data are desirable to apply the EB Method. The EB Method and criteria to determine whether the EB Method is applicable are presented in Section A.2.1 in the Appendix to *Part C*.

The EB Method and criteria to determine whether the EB Method is applicable are presented in Section A.2.1 in the Appendix to Part C.

The EB Method can be applied at the site-specific level (i.e., observed crashes are assigned to specific intersections or roadway segments in Step 6) or at the project level (i.e., observed crashes are assigned to a facility as a whole). The site-specific EB Method is applied in Step 13. Alternatively, if observed crash data are available but can not be assigned to individual roadway segments and intersections, the project level EB Method is applied (in Step 15).

If observed crash frequency data are not available, then Steps 6, 13, and 15 of the predictive method would not be performed. In this case the estimate of expected average crash frequency is limited to using a predictive model (i.e. the predicted average crash frequency).

Step 4 - Determine geometric design features, traffic control features, and site characteristics for all sites in the study network.

In order to determine the relevant data required and avoid unnecessary collection of data, it is necessary to understand the base conditions of the SPFs in Step 9, and the AMFs in Step 10. The base conditions for the SPFs for each of the facility types in the HSM are detailed in *Chapters 10*, *11*, and *12*.

Step 5 – Divide the roadway network or facility under consideration into individual roadway segments and intersections, which are referred to as sites.

Using the information from Step 1 and Step 4, the roadway is divided into individual sites, consisting of individual homogenous roadway segments and intersections. Section C.6.2 provides the general definitions of roadway segments and intersections used in the predictive method. When dividing roadway facilities into small homogenous roadway segments, limiting the segment length to no less than 0.10 miles will minimize calculation efforts and not affect results.

Step 6 – Assign observed crashes to the individual sites (if applicable).

Step 6 only applies if it was determined in Step 3 that the site-specific EB Method was applicable. If the site-specific EB Method is not applicable, proceed to Step 7. In Step 3, the availability of observed data and whether the data could be assigned to specific locations was determined. The specific criteria for assigning accidents to individual roadway segments o r intersections are presented in Section A.2.3 of the Appendix to *Part C*.

Crashes that occur at an intersection or on an intersection leg, and are related to the presence of an intersection, are assigned to the intersection and used in the EB Method together with the predicted average crash frequency for the intersection. Crashes that occur between intersections and are not related to the presence of an intersection are assigned to the roadway segment on which they occur, this includes crashes that occur within the intersection limits but are unrelated to the presence of the intersection. Such crashes are used in the EB Method together with the predicted average crash frequency for the roadway segment.

Step 7 – Select the first or next individual site in the study network. If there are no more sites to be evaluated, go to Step 15.

In Step 5 the roadway network within the study limits is divided into a number of individual homogenous sites (intersections and roadway segments). At each site, all geometric design features, traffic control features, AADTs, and observed crash data are determined in Steps 1 through 4. For studies with a large number of sites, it may be practical to assign a number to each site.

The outcome of the HSM predictive method is the expected average crash frequency of the entire study network, which is the sum of the all of the individual sites, for each year in the study. Note that this value will be the total number of crashes expected to occur over all sites during the period of interest. If a crash frequency is desired, the total can be divided by the number of years in the period of interest.

The estimate for each site (roadway segments or intersection) is undertaken one at a time. Steps 8 through 14, described below, are repeated for each site.

Step 8 – For the selected site, select the first or next year in the period of interest. If there are no more years to be evaluated for that site, proceed to Step 15.

Steps 8 through 14 are repeated for each site in the study and for each year in the study period.

The individual years of the evaluation period may have to be analyzed one year at a time for any particular roadway segment or intersection because SPFs and some AMFs (e.g., lane and shoulder widths) are dependent on AADT, which may change from year to year.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type and traffic control features.

Steps 9 through 13, described below, are repeated for each year of the evaluation period as part of the evaluation of any particular roadway segment or intersection.

Each predictive model in the HSM consists of a Safety Performance Function (SPF), which is adjusted to site specific conditions (in Step 10) using Accident Modification Factors (AMFs) and adjusted to local jurisdiction conditions (in Step 11) using a calibration factor (C). The SPFs, AMFs and calibration factor obtained in Steps 9, 10, and 11 are applied to calculate the predicted average crash frequency for the selected year of the selected site. The resultant value is the predicted average crash frequency for the selected year.

The SPF (which is a statistical regression model based on observed crash data for a set of similar sites) estimates the predicted average crash frequency for a site with the base conditions (i.e., a specific set of geometric design and traffic control features). The base conditions for each SPF are specified in each of the *Part C* chapters. A detailed explanation and overview of the SPFs in *Part C* is provided in Section C.6.3.

The facility types for which SPFs were developed for the HSM are shown in Exhibit C-2. The predicted average crash frequency for base conditions is calculated using the traffic volume determined in Step 3 (AADT for roadway segments or $AADT_{maj}$ and $AADT_{min}$ for intersections) for the selected year.

The predicted average crash frequency may be separated into components by crash severity level and collision type. Default distributions of crash severity and collision types are provided in the *Part C* chapters. These default distributions can benefit from being updated based on local data as part of the calibration process presented in Appendix A.1.1.

To account for differences between the base geometric design and the specific geometric design of the site, Accident Modification Factors (AMFs) adjust the SPF estimate.

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust the predicted average crash frequency to site-specific geometric design and traffic control features.

Each SPF is applicable to a set of base geometric design and traffic control features, which are identified for each site type in the *Part C* chapters. In order to account for differences between the base geometric design and the specific geometric design of the site, AMFs are used to adjust the SPF estimate. An overview of AMFs and guidance for their use is provided in Section C.6.4 including the limitations of current knowledge regarding the effects of simultaneous application of multiple AMFs. In using multiple AMFs, engineering judgment is required to assess the interrelationships and/or independence of individual elements or treatments being considered for implementation within the same project

Only the AMFs presented in Part C may be used as part of the Part C predictive method.

All AMFs used in *Part C* have the same base conditions as the SPFs used in the *Part C* chapter which the AMF is presented (i.e. when the specific site has the same condition as the SPF base condition, the AMF value for that condition is 1.00). Only the AMFs presented in *Part C* may be used as part of the *Part C* predictive method.

Part D contains all AMFs in the HSM. Some *Part D* AMFs are included in *Part C* for use with specific SPFs. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7.

For urban and suburban arterials (*Chapter 12*) the average crash frequency for pedestrian and bicycle base crashes is calculated at the end of this step.

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

The calibration factor adjusts the SPF accounting for jurisdictional differences such as weather, time periods, or driver demographics.

The SPFs used in the predictive method have each been developed with data from specific jurisdictions and time periods. Calibration of SPFs to local conditions will account for differences. A calibration factor (C_r for roadway segments or C_i for intersections) is applied to each SPF in the predictive method. An overview of the use of calibration factors is provided in Section C.6.5. Detailed guidance for the development of calibration factors is included in *Part C* Appendix A.1.1

Step 12 – If there is another year to be evaluated in the study period for the selected site, return to Step 8. Otherwise, proceed to Step 13.

This step creates a loop through Steps 8 to 12 that is repeated for each year of the evaluation period for the selected site.

Step 13 – Apply site-specific EB Method (if applicable).

The overdispersion parameter provides an indication of the statistical reliability of the SPF. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF.

Whether the site-specific EB Method is applicable is determined in Step 3 using criteria in *Part C* Appendix A.2.1. If it is not applicable then proceed to Step 14.

If the site-specific EB Method is applicable, Step 6 EB Method criteria (detailed in *Part C* Appendix A.2.4.) is used to assign observed crashes to each individual site.

The site-specific EB Method combines the predictive model estimate of predicted average crash frequency, $N_{predicted,}$ with the observed crash frequency of the specific site, $N_{observed}$. This provides a more statistically reliable estimate of the expected average crash frequency of the selected site.

In order to apply the site-specific EB Method, in addition to the material in *Part C* Appendix A.2.4, the overdispersion parameter, k, for the SPF is also used. The overdispersion parameter provides an indication of the statistical reliability of the SPF. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. This parameter is used in the site-specific EB Method to provide a weighting to $N_{predicted}$ and $N_{observed}$. Overdispersion parameters are provided for each SPF in the *Part C* chapters.

Apply the site-specific EB Method to a future time period, if appropriate.

The estimated expected average crash frequency obtained above applies to the time period in the past for which the observed crash data were obtained. Section A.2.6 in the Appendix to *Part C* provides a method to convert the estimate of expected average crash frequency for a past time period to a future time period.

Step 14 – If there is another site to be evaluated, return to step 7, otherwise, proceed to Step 15.

This step creates a loop for Steps 7 to 13 that is repeated for each roadway segment or intersection within the study area.

Step 15 – Apply the project level EB Method (if the site-specific EB Method is not applicable).

This step is applicable to existing conditions when observed crash data are available, but can not be accurately assigned to specific sites (e.g., the crash report may identify crashes as occurring between two intersections, but is not accurate to determine a precise location on the segment). The EB Method is discussed in Section C.6.6. Detailed description of the project level EB Method is provided in *Part C* Appendix A.2.5.

Step 16 – Sum all sites and years in the study to estimate total crashes or average crash frequency for the network

The total estimated number of crashes within the network or facility limits during the study period years is calculated using Equation C-2:

$$N_{total} = \sum_{\substack{\text{all} \\ \text{roadway} \\ \text{segments}}} N_{rs} + \sum_{\substack{\text{all} \\ \text{intersections}}} N_{int} \tag{C-2}$$

Where,

N_{total} = total expected number of crashes within the roadway limits of the study for all years in the period of interest. Or, the sum of the expected average crash frequency for each year for each site within the defined roadway limits within the study period;

N_{rs} = expected average crash frequency for a roadway segment using the predictive method for one year;

N_{int} = expected average crash frequency for an intersection using the predictive method for one year.

Equation C-2 represents the total expected number of crashes estimated to occur during the study period. Equation C-3 is used to estimate the total expected average crash frequency within the network or facility limits during the study period.

$$N_{total\ average} = \frac{N_{total}}{n} \qquad (C\text{-}3)$$

Where,

$N_{total\ average}$ = total expected average crash frequency estimated to occur within the defined roadway limits during the study period;

n = number of years in the study period.

Regardless of whether the total or the total average is used, a consistent approach in the methods will produce reliable comparisons.

Step 17 – Determine if there is an alternative design, treatment, or forecast AADT to be evaluated.

Steps 3 through 16 of the predictive method are repeated as appropriate for the same roadway limits but for alternative geometric design, treatments, or periods of interest or forecast AADTs.

Step 18 – Evaluate and compare results.

The predictive method is used to provide a statistically reliable estimate of the expected average crash frequency within defined network or facility limits over a given period of time for given geometric design and traffic control features and known or estimated AADT. The predictive method results may be used for a number of different purposes. Methods for estimating the effectiveness of a project are presented in Section C.7. *Part B* of the HSM includes a number of methods for effectiveness evaluation and network screening, many of which use of the predictive method. Example uses include:

- Screening a network to rank sites and identify those sites likely to respond to a safety improvement;
- Evaluating the effectiveness of countermeasures after a period of implementation;
- Estimating the effectiveness of proposed countermeasures on an existing facility.

C.6. PREDICTIVE METHOD CONCEPTS

The 18 steps of the predictive method have been summarized in section C.5. Section C.6 provides additional explanation of the some of the steps of the predictive method. Detail regarding the procedure for determining a calibration factor to apply in Step 11 is provided in the *Part C* Appendix A.1. Detail regarding the EB Method, which is required in Steps 6, 13, and 15, is provided in the *Part C* Appendix A.2

C.6.1. Roadway Limits and Facility Types

Section C.6.1 provides information about identifying facility types and establishing roadway limits.

In Step 1 of the predictive method the extent or limits of the roadway network under consideration are defined and the facility type or types within those limits is determined. *Part C* provides three facility types; Rural Two-Lane Two-Way Roads, Rural Multilane Highways, and Urban and Suburban Arterials. In Step 5 of the predictive method, the roadway within the defined roadway limits is divided into individual sites, which are either homogenous roadway segments or intersections. A facility consists of a contiguous set of individual intersections and roadway segments, referred to as "sites." A roadway network consists of a number of contiguous facilities.

Classifying an area as urban, suburban or rural is subject to the roadway characteristics, surrounding population and land uses and is at the user's discretion. In the HSM, the definition of "urban" and "rural" areas is based on Federal Highway Administration (FHWA) guidelines which classify "urban" areas as places inside urban boundaries where the population is greater than 5,000 persons. "Rural" areas are defined as places outside urban areas which have with population greater than 5,000 persons. The HSM uses the term "suburban" to refer to outlying portions of an urban area; the predictive method does not distinguish between urban and suburban portions of a developed area.

For each facility type, SPFs and AMFs for specific individual site types (i.e., intersections and roadway segments) are provided. The predictive method is used to determine the expected average crash frequency for each individual site in the study, for all years in the period of interest, and the overall crash estimation is the cumulative sum of all sites for all years.

The facility types and facility site types in the HSM *Part C* are defined below. Exhibit C-2 summarizes the site types for each of the facility types that are included in each of the *Part C* chapters:

- **Chapter 10 - Rural Two-Lane Two-Way Roads:** includes all rural highways with two-lanes and two-way traffic operation. *Chapter 10* also addresses two-lane two-way highways with center two-way left-turn lanes and two-lane highways with added passing or climbing lanes or with short segments of four-lane cross-sections (up to two miles in length) where the added lanes in each direction are provided specifically to enhance passing opportunities. Short lengths of highway with four-lane cross-sections essentially function as two-lane highways with side-by-side passing lanes and, therefore, are within the scope of the two-lane two-way highway methodology. Rural highways with longer sections of four-lane cross-sections can be addressed with the rural multilane highway procedures in *Chapter 11*. *Chapter 10* includes three- and four-leg intersections with minor-road stop control and four-leg signalized intersections on all the roadway cross-sections to which the chapter applies.

- **Chapter 11 - Rural Multilane Highways:** includes rural multilane highways without full access control. This includes all rural nonfreeways with four through travel lanes, except for two-lane highways with side-by-side passing lanes, as described above. *Chapter 11* includes three- and four-leg intersections with minor-road stop control and four-leg signalized intersections on all the roadway cross-sections to which the chapter applies.

- **Urban and Suburban Arterial Highways:** includes arterials without full access control, other than freeways, with two, or four through lanes in urban and suburban areas. *Chapter 12* includes three- and four-leg intersections with minor-road stop control or traffic signal control and roundabouts on all of the roadway cross-sections to which the chapter applies.

C.6.2. Definition of Roadway Segments and Intersections

The predictive models for roadway segments estimate the frequency of crashes that would occur on the roadway if no intersection were present. The predictive models for an intersection estimate the frequency of additional crashes that occur because of the presence of the intersection.

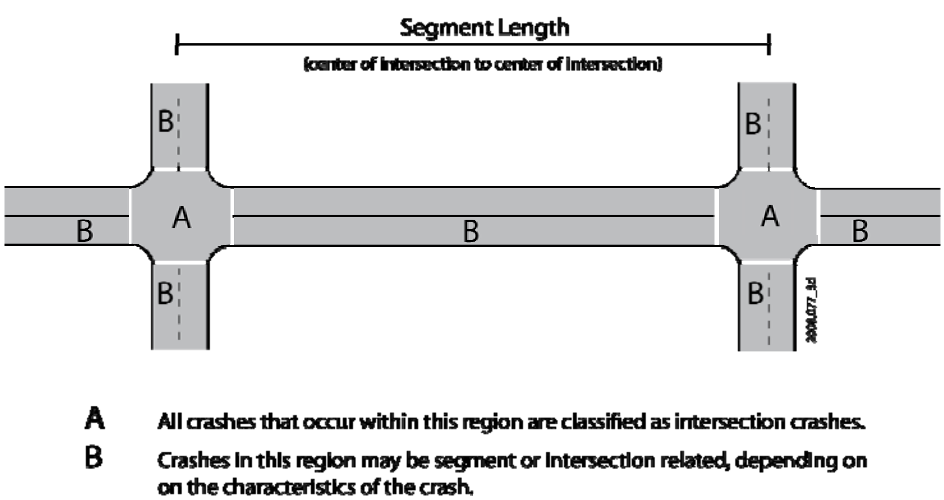
A roadway segment is a section of continuous traveled way that provides two-way operation of traffic, that is not interrupted by an intersection, and consists of homogenous geometric and traffic control features. A roadway segment begins at the center of an intersection and ends at either the center of the next intersection, or where there is a change from one homogeneous roadway segment to another homogenous segment. The roadway segment model estimates the frequency of roadway segment related crashes which occur in Region B in Exhibit C-4. When a roadway segments begins or ends at an intersection, the length of the roadway segment is measured from the center of the intersection.

Intersections are defined as the junction of two or more roadway segments. The intersection models estimate the predicted average frequency of crashes that occur within the limits of an intersection (Region A of Exhibit C-4) and intersection-related crashes that occur on the intersection legs (Region B in Exhibit C-4).

When the EB Method is applicable at the site-specific level (see Section C.6.6), observed crashes are assigned to individual sites. Some observed crashes that occur at intersections may have characteristics of roadway segment crashes and some roadway segment crashes may be attributed to intersections. These crashes are individually assigned to the appropriate site. The method for assigning and classifying crashes as individual roadway segment crashes and intersection crashes for use with the EB Method is described in *Part C* Appendix A.2.3. In Exhibit C-4, all observed crashes that occur in Region A are assigned as intersection crashes, but crashes that occur in Region B may be assigned as either roadway segment crashes or intersection crashes depending on the characteristics of the crash.

Using these definitions, the roadway segment predictive models estimate the frequency of crashes that would occur on the roadway if no intersection were present. The intersection predictive models estimate the frequency of additional crashes that occur because of the presence of the intersection.

Exhibit C-4: Definition of Roadway Segments and Intersections



Section C.6.3 provides information about Safety Performance Functions.

C.6.3. Safety Performance Functions

SPFs are regression models for estimating the predicted average crash frequency of individual roadway segments or intersections. In Step 9 of the predictive method, the appropriate SPFs are used to determine the predicted average crash frequency for the selected year for specific base conditions. Each SPF in the predictive method was developed with observed crash data for a set of similar sites. In the SPFs developed for the HSM, the dependent variable estimated is the predicted average crash frequency for a roadway segment or intersection under base conditions and the independent variables are the AADTs of the roadway segment or intersection legs (and, in some cases a few additional variables such as the length of the roadway segment).

An example of a SPF (for rural two-way two-lane roadway segments from *Chapter 10*) is shown in Equation C-4.

$$N_{spf\ rs} = (AADT) \times (L) \times (365) \times 10^{(-6)} \times e^{(-0.4865)} \quad \text{(C-4)}$$

Where,

$N_{spf\ rs}$= predicted average crash frequency estimated for base conditions using a statistical regression model;

AADT= annual average daily traffic volume (vehicles/day) on roadway segment;

L= length of roadway segment (miles).

SPFs are developed through statistical multiple regression techniques using historic crash data collected over a number of years at sites with similar characteristics and covering a wide range of AADTs. The regression parameters of the SPFs are determined by assuming that crash frequencies follow a negative binomial distribution. The negative binomial distribution is an extension of the Poisson distribution which is typically used for crash frequencies. However, the mean and the variance of the Poisson distribution are equal. This is often not the case for crash frequencies where the variance typically exceeds the mean.

The negative binomial distribution incorporates an additional statistical parameter, the overdispersion parameter that is estimated along with the parameters of the regression equation. The overdispersion parameter has positive values. The greater the overdispersion parameter, the more that crash data vary as compared to a Poisson distribution with the same mean. The overdispersion parameter is used to determine a weighted adjustment factor for use in the EB Method described in Section C.6.6.

Accident Modification Factors (AMFs) are applied to the SPF estimate to account for geometric or geographic differences between the base conditions of the model and local conditions of the site under consideration. AMFs and their application to SPFs are described in Section C.6.4.

In order to apply a SPF, the following information relating to the site under consideration is necessary:

- Basic geometric design and geographic information of the site to determine the facility type and whether a SPF is available for that site type;
- AADT information for estimation of past periods, or forecast estimates of AADT for estimation of future periods;
- Detailed geometric design of the site and base conditions (detailed in each of the *Part C* chapters) to determine whether the site conditions vary from the base conditions and therefore an AMF is applicable.

The HSM provides default distributions of crash severity and collision type. These distributions can benefit from calibration to local conditions.

Updating Default Values of Crash Severity and Collision Type Distribution for Local Conditions

In addition to estimating the predicted average crash frequency for all crashes, SPFs can be used to estimate the distribution of crash frequency by crash severity types and by collision types (such as single-vehicle or driveway crashes). The distribution models in the HSM are default distributions.

Where sufficient and appropriate local data are available, the default values (for crash severity types and collision types and the proportion of night-time accidents) can be replaced with locally derived values when it is explicitly stated in *Chapters 10*, *11*, and *12*. Calibration of default distributions to local conditions is described in detail in the *Part C* Appendix A.1.1.

Development of Local SPFs

If possible, development of local SPFs is encouraged.

Some HSM users may prefer to develop SPFs with data from their own jurisdiction for use with the predictive method rather than calibrating the SPFs presented in the HSM. The Appendix to *Part C* provides guidance on developing jurisdiction-specific SPFs that are suitable for use with the predictive method. Development of jurisdiction-specific SPFs is not required

C.6.4. Accident Modification Factors

In Step 10 of the predictive method, AMFs are determined and applied to the results of Step 9. The AMFs are used in *Part C* to adjust the predicted average crash frequency estimated by the SPF for a site with base conditions to the predicted average crash frequency for the specific conditions of the selected site.

AMFs are the ratio of the estimated average crash frequency of a site under two different conditions. Therefore, an AMF represents the relative change in estimated average crash frequency due to a change in one specific condition (when all other conditions and site characteristics remain constant).

Equation C-5 shows the calculation of an AMF for the change in estimated average crash frequency from site condition 'a' to site condition 'b'.

$$AMF = \frac{estimated\ average\ crash\ frequency\ with\ condition\ 'b'}{estimated\ average\ crash\ frequency\ with\ condition\ 'a'} \qquad (C\text{-}5)$$

Section C.6.4 describes application of AMFs.

AMFs defined in this way for expected crashes can also be applied to the comparison of predicted crashes between site condition 'a' and site condition 'b'.

AMFs are an estimate of the effectiveness of the implementation of a particular treatment, also known as a countermeasure, intervention, action, or alternative design. Examples include; illuminating an unlighted road segment, paving gravel shoulders, signalizing a stop-controlled intersection, increasing the radius of a horizontal curve, or choosing a signal cycle time of 70 seconds instead of 80 seconds. AMFs have also been developed for conditions that are not associated with the roadway, but represent geographic or demographic conditions surrounding the site or with users of the site, for example, the number of liquor outlets in proximity to a site.

The values of AMFs in the HSM are determined for a specified set of base conditions. These base conditions serve the role of site condition 'a' in Equation C-5. This allows comparison of treatment options against a specified reference condition. For example, AMF values for the effect of lane width changes are determined in comparison to a base condition of 12-ft lane width. Under the base conditions (i.e., with no change in the conditions), the value of an AMF is 1.00. AMF values less than 1.00 indicate the alternative treatment reduces the estimated average crash frequency in comparison to the base condition. AMF values greater than 1.00 indicate the alternative treatment increases the estimated crash frequency in comparison to the base condition. The relationship between an AMF and the expected percent change in crash frequency is shown in Equation C-6.

$$\text{Percent Reduction in Accidents} = 100\% \times (1.00 - AMF) \quad \text{(C-6)}$$

For example,

- If an AMF = 0.90 then the expected percent change is 100% × (1 - 0.90)= 10%, indicating a 10% change in estimated average crash frequency.
- If an AMF = 1.20 then the expected percent change is 100% × (1 - 1.20)= -20%, indicating a -20% change in estimated average crash frequency.

Application of AMFs to Adjust Crash Frequencies for Specific Site Conditions

In the *Part C* predictive models, a SPF estimate is multiplied by a series of AMFs to adjust the estimate of average crash frequency from the base conditions to the specific conditions present at that site (see, for example, Equation C-1). The AMFs are multiplicative because the most reasonable assumption based on current knowledge is to assume independence of the effects of the features they represent. Little research exists regarding the independence of these effects. The use of observed crash data in the EB Method (see Section C.6.6 and the Appendix to *Part C*) can help to compensate for any bias which may be caused by lack of independence of the AMFs. As new research is completed, future HSM editions may be able to address the independence (or lack thereof) of AMF effects more fully.

Application of AMFs in Estimating the Effect on Crash Frequencies of Proposed Treatments or Countermeasures

AMFs are also used in estimating the anticipated effects of proposed future treatments or countermeasures (e.g., in some of the methods discussed in Section C.7). Where multiple treatments or countermeasures will be applied concurrently and are presumed to have independent effects, the AMFs for the combined treatments are multiplicative. As discussed above, limited research exists regarding the independence of the effects of individual treatments from one another. However, in the case of proposed treatments that have not yet been implemented, there are no observed crash data for the future condition to provide any compensation for overestimating forecast effectiveness of multiple treatments. Thus, engineering judgment is required to assess the interrelationships and independence for multiple treatments at a site.

The limited understanding of interrelationships among various treatments requires consideration, especially when several AMFs are being multiplied. It is possible to overestimate the combined effect of multiple treatments when it is expected that more than one of the treatments may affect the same type of crash. The implementation of wider lanes and shoulders along a corridor is an example of a combined treatment where the independence of the individual treatments is unclear, because both treatments are expected to reduce the same crash types. When implementing potentially interdependent treatments, users should exercise engineering judgment to assess the interrelationship and/or independence of individual elements or treatments being considered for implementation within the same project. These assumptions may or may not be met by multiplying the AMFs under consideration together with either a SPF or with observed crash frequency of an existing site.

Engineering judgment is also necessary in the use of combined AMFs where multiple treatments change the overall nature or character of the site. In this case, certain AMFs used in the analysis of the existing site conditions and the proposed treatment may not be compatible. An example of this concern is the installation of a roundabout at an urban two-way stop-controlled or signalized intersection. The procedure for estimating the crash frequency after installation of a roundabout (see *Chapter 12*) is to estimate the average crash frequency for the existing site conditions (as a SPF for roundabouts is currently unavailable) and then apply an AMF for conversion of a conventional intersection to a roundabout. Clearly, the installation of a roundabout changes the nature of the site so that other AMFs which may be applied to address other conditions at the two-way stop-controlled location may no longer be relevant.

AMFs and Standard Error

Standard error is defined as the estimated standard deviation of the difference between estimated values and values from sample data. It is a method of evaluating the error of an estimated value or model. The smaller the standard error, the more reliable (less error) the estimate. All AMF values are estimates of the change in expected average crash frequency due to a change in one specific condition plus or minus a standard error. Some AMFs in the HSM include a standard error value, indicating the variability of the AMF estimation in relation to sample data values.

Standard error can also be used to calculate a confidence interval for the estimated change in expected average crash frequency. Confidence intervals can be calculated using multiples of standard error using Equation C-7 and values from Exhibit C-5.

$$CI(X\%) = AMF \pm (SE \times MSE) \quad \text{(C-7)}$$

Where,

CI(X%) = confidence interval, or range of estimate values within which it is X% probable the true value will occur;

AMF = Accident Modification Factor;

SE = Standard Error of the AMF;

MSE = Multiple of Standard Error.

Exhibit C-5: Constructing Confidence Intervals Using AMF Standard Error

Desired Level of Confidence	Confidence Interval (probability that the true value is within the estimated intervals)	Multiple of Standard Error (MSE) to use in Equation C-7
Low	65-70%	1
Medium	95%	2
High	99.9%	3

AMFs in the HSM Part C

AMF values in the HSM are either explained in the text (typically where there are a limited range of options for a particular treatment), in a formula (where treatment options are continuous variables) or in tables (where the AMF values vary by facility type or are in discrete categories). The differences between AMFs in *Part C* and *D* AMFs are explained below.

Part D contains all AMFs in the HSM. Some *Part D* AMFs are included in *Part C* for use with specific SPFs. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7.

C.6.5. Calibration of Safety Performance Functions to Local Conditions

Section C.6.5 presents calibration concepts. The calibration method is described completely in the Part C Appendix.

The predictive models in *Chapters 10*, *11*, and *12* have three basic elements, Safety Performance Functions, Accident Modification Factors and a calibration factor. The SPFs were developed as part of HSM-related research from the most complete and consistent available data sets. However, the general level of crash frequencies may vary substantially from one jurisdiction to another for a variety of reasons including crash reporting thresholds, and crash reporting system procedures. These variations may result in some jurisdictions experiencing substantially more reported traffic accidents on a particular facility type than in other jurisdictions. In addition, some jurisdictions may have substantial variations in conditions between areas within the jurisdiction (e.g. snowy winter driving conditions in one part of the state and only wet winter driving conditions in another part of the state). Therefore, for the predictive method to provide results that are reliable for each jurisdiction that uses them, it is important that the SPFs in *Part C* be calibrated for application in each jurisdiction. Methods for calculating calibration factors for roadway segments C_r and intersections C_i are included in the *Part C* Appendix to allow highway agencies to adjust the SPF to match local conditions.

The calibration factors will have values greater than 1.0 for roadways that, on average, experience more accidents than the roadways used in developing the SPFs. Roadways that, on average, experience fewer accidents than the roadways used in the development of the SPF, will have calibration factors less than 1.0.

C.6.6. Weighting Using the Empirical Bayes Method

Section C.6.6 introduces more information about the EB Method.

Step 13 or Step 15 of the predictive method are optional steps that are applicable only when observed crash data are available for either the specific site or the entire facility of interest. Where observed crash data and a predictive model are available, the reliability of the estimation is improved by combining both estimates. The predictive method in *Part C* uses the Empirical Bayes method, herein referred to as the EB Method.

The EB Method can be used to estimate expected average crash frequency for past and future periods, and used at either the site-specific level or the project-specific level (where observed data may be known for a particular facility, but not at the site-specific level).

For an individual site (i.e., the site-specific EB Method) the EB Method combines the observed crash frequency with the predictive model estimate using Equation C-8. The EB Method uses a weighted factor, w, which is a function of the SPFs overdispersion parameter, k, to combine the two estimates. The weighted adjustment is therefore dependant only on the variance of the SPF model. The weighted adjustment factor, w, is calculated using Equation C-9.

$$N_{expected} = w \times N_{predicted} + (1.00 - w) \times N_{observed} \quad (C\text{-}8)$$

$$w = \frac{1}{1 + k \times \left(\sum_{\substack{\text{all study}\\ \text{years}}} N_{predicted} \right)} \quad (C\text{-}9)$$

Where,

$N_{expected}$ = estimate of expected average crash frequency for the study period;

$N_{predicted}$ = predictive model estimate of predicted average crash frequency for the study period;

$N_{observed}$ = observed crash frequency at the site over the study period;

w = weighted adjustment to be placed on the SPF prediction;

k = overdispersion parameter from the associated SPF.

As the value of the overdispersion parameter increases, the value of the weighted adjustment factor decreases, and thus more emphasis is placed on the observed rather than the SPF predicted crash frequency. When the data used to develop a model are greatly dispersed, the precision of the resulting SPF is likely to be lower; in this case, it is reasonable to place less weight on the SPF estimation and more weight on the observed crash frequency. On the other hand, when the data used to develop a model have little overdispersion, the reliability of the resulting SPF is likely to be higher; in this case, it is reasonable to place more weight on the SPF estimation and less weight on the observed crash frequency. A more detailed discussion of the EB Method is included in the Appendix to *Part C*.

The EB Method cannot be applied without an applicable SPF and observed crash data. There may be circumstances where a SPF may not be available or cannot be calibrated to local conditions or circumstances where crash data are not available or applicable to current conditions. If the EB Method is not applicable, Steps 6, 13, and 15 are not conducted.

C.7. METHODS FOR ESTIMATING THE SAFETY EFFECTIVENESS OF A PROPOSED PROJECT

Section C.7 provides methods for estimating effectiveness of projects.

The *Part C* Predictive Method provides a structured methodology to estimate the expected average crash frequency where geometric design and traffic control features are specified. There are four methods for estimating the change in expected average crash frequency of a proposed project or project design alternative (i.e., the effectiveness of the proposed changes in terms of crash reduction). In order of predictive reliability (high to low) these are:

- Method 1 – Apply the *Part C* predictive method to estimate the expected average crash frequency of both the existing and proposed conditions.
- Method 2 – Apply the *Part C* predictive method to estimate the expected average crash frequency of the existing condition and apply an appropriate project AMF from *Part D* (i.e., an AMF that represents a project which changes the character of a site) to estimate the safety performance of the proposed condition.
- Method 3 – If the *Part C* predictive method is not available, but a Safety Performance Function (SPF) applicable to the existing roadway condition is available (i.e., a SPF developed for a facility type that is not included in *Part C* of the HSM), use that SPF to estimate the expected average crash frequency of the existing condition. Apply an appropriate project AMF from *Part D* to estimate the expected average crash frequency of the proposed condition. A locally-derived project AMF can also be used in Method 3.
- Method 4 – Use observed crash frequency to estimate the expected average crash frequency of the existing condition and apply an appropriate project AMF from *Part D* to the estimated expected average crash frequency of the existing condition to obtain the estimated expected average crash frequency for the proposed condition.

In all four of the above methods, the difference in estimated expected average crash frequency between the existing and proposed conditions/projects is used as the project effectiveness estimate.

C.8. LIMITATIONS OF THE HSM PREDICTIVE METHOD

The predictive method is based on research using available data bases describing geometric and traffic characteristics of road systems in the United States. The predictive models incorporate the effects of many, but not all, geometric designs and traffic control features of potential interest. The absence of a factor from the predictive models does not necessarily mean that the factor has no effect on crash frequency; it may merely indicate that the effect is not fully known or has not been quantified at this time.

The major limitation of the predictive method is that the predictive models incorporate the effect of many, but not all, geometric designs and traffic control features of potential interest.

While the predictive method addresses the effects of physical characteristics of a facility, it considers effect of non-geometric factors only in a general sense. Primary examples of this limitation are:

- Driver populations vary substantially from site to site in age distribution, years of driving experience, seat belt usage, alcohol usage, and other behavioral factors. The predictive method accounts for the statewide or community-wide influence of these factors on crash frequencies through calibration, but not site-specific variations in these factors, which may be substantial.
- The effects of climate conditions may be addressed indirectly through the calibration process, but the effects of weather are not explicitly addressed.
- The predictive method considers annual average daily traffic volumes, but does not consider the effects of traffic volume variations during the day or

the proportions of trucks or motorcycles; the effects of these traffic factors are not fully understood.

Furthermore, the predictive method treats the effects of individual geometric design and traffic control features as independent of one another and ignores 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.

C.9. GUIDE TO APPLYING PART C

The HSM provides a predictive method for crash estimation which can be used for the purposes of making decisions relating to designing, planning, operating and maintaining roadway networks.

These methods focus on the use of statistical methods in order to address the inherent randomness in crashes. Users do not need to have detailed knowledge of statistical analysis methods in order to understand and use the HSM. However, use of the HSM does require understanding the following general principles:

- Observed crash frequency is an inherently random variable. It is not possible to precisely predict the value for a specific one year period – the estimates in the HSM refer to the expected average crash frequency that would be observed if the site could be maintained under consistent conditions for a long-term period, which is rarely possible.
- Calibration of an SPF to local state conditions is an important step in the predictive method.
- Engineering judgment is required in the use of all HSM procedures and methods, particularly selection and application of SPFs and AMFs to a given site condition.
- Errors and limitations exist in all crash data which affects both the observed crash data for a specific site, and also the models developed. *Chapter 3* provides additional explanation on this subject.
- Development of SPFs and AMFs requires understanding of statistical regression modeling and crash analysis techniques. Appendix to *Part C* provides guidance on developing jurisdiction-specific SPFs that are suitable for use with the predictive method. Development of jurisdiction-specific SPFs is not required
- In general, a new roadway segment is applicable when there is a change in the condition of a roadway segment that requires application of a new or different AMF value, but where a value changes frequently within a minimum segment length, engineering judgment is required to determine an appropriate average value across the minimum segment length. When dividing roadway facilities into small homogenous roadway segments, limiting the segment length to greater then or equal to 0.10 miles will decrease data collection and management efforts
- Where the EB Method is applied, a minimum of two years of observed data is recommended. The use of observed data is only applicable if geometric design and AADTs are known during the period for which observed data are available.

C.10. SUMMARY

The predictive method consists of 18 steps which provide detailed guidance for dividing a facility into individual sites, selecting an appropriate period of interest, obtaining appropriate geometric data, traffic volume data and observed crash data, and applying the predictive models and the EB Method. By following the predictive method steps, the expected average crash frequency of a facility can be estimated for a given geometric design, traffic volumes and period of time. This allows comparison to be made between alternatives in design and traffic volume forecast scenarios. The HSM predictive method allows the estimate to be made between crash frequency and treatment effectiveness to be considered along with community needs, capacity, delay, cost, right-of-way and environmental considerations in decision making for highway improvement projects.

The predictive method can be applied to either a past or a future period of time and used to estimate total expected average crash frequency, or crash frequencies by crash severity and collision type. The estimate may be for an existing facility, for proposed design alternatives for an existing facility, or for a new (unconstructed) facility. Predictive models are used to determine the predicted average crash frequencies based on site conditions and traffic volumes. The predictive models in the HSM consist of three basic elements: safety performance functions, accident modification factors and a calibration factor. These are applied in Steps 9, 10, and 11 of the predictive method to determine the predicted average crash frequency of a specific individual intersection or homogenous roadway segment for a specific year.

Where observed crash data are available, observed crash frequencies are combined with the predictive model estimates using the EB Method, to obtain a statistically reliable estimate. The EB Method may be applied in Step 13 or 15 of the predictive method. The EB Method can be applied at the site-specific level (Step 13) or at the project-specific level (Step 15). It may also be applied to a future time period if site conditions will not change in the future period. The EB Method is described in the *Part C* Appendix A.2.

The following Chapters in *Part C* provide the detailed predictive method steps for estimating expected average crash frequency for the following facility types:

- **Chapter 10** - Rural Two-Lane Two-Way Roads
- **Chapter 11** – Rural Multilane Highways
- **Chapter 12** – Urban and Suburban Arterials

PART C— PREDICTIVE METHOD

CHAPTER 10—PREDICTVE METHOD FOR RURAL TWO-LANE TWO-WAY ROADS

10.1. Introduction 10-4
10.2. Overview of the Predictive Method 10-4
10.3. Rural Two-Lane Two-Way Roads – Definitions and Predictive Models In Chapter 10 10-5
10.3.1. Definition of Chapter 10 Facility and Site Types 10-5
10.3.2. Predictive Models for Rural Two-Lane Two-Way Roadway Segments 10-6
10.3.3. Predictive Models for Rural Two-Lane Two-Way Intersections 10-7
10.4. Predictive Method for Rural Two-Lane Two-Way roads 10-7
10.5. Roadway Segments and Intersections 10-17
10.6. Safety Performance Functions 10-20
10.6.1. Safety Performance Functions for Rural Two-Lane Two-Way Roadway Segments 10-21
10.6.2. Safety Performance Functions for Intersections 10-23
10.7. Accident Modification Factors 10-28
10.7.1. Accident Modification Factors for Roadway Segments 10-30
10.7.2. Accident Modification Factors for Intersections 10-39
10.8. Calibration of the SPFs to Local Conditions 10-42
10.9. Limitations of Predictive Method in Chapter 10 10-43
10.10. Application of Chapter 10 Predictive method 10-43
10.11. Summary 10-43
10.12. Sample Problems 10-44
10.12.1. Sample Problem 1 10-44
10.12.2. Sample Problem 2 10-53
10.12.3. Sample Problem 3 10-62
10.12.4. Sample Problem 4 10-70
10.12.5. Sample Problem 5 10-77
10.12.6. Sample Problem 6 10-81
10.13. References 10-85

EXHIBITS

Exhibit 10-1: Rural Two-Lane Two-Way Road Site Type with SPFs in Chapter 10 10-6

Exhibit 10-2: The HSM Predictive Method 10-9

Exhibit 10-3: Definition of Segments and Intersections.. 10-17

Exhibit 10-4: Safety Performance Functions included in Chapter 10 10-20

Exhibit 10-5: Graphical Form of SPF for Rural Two-Lane Two-Way Roadway Segments (Equation 10-6) 10-22

Exhibit 10-6: Default Distribution for Crash Severity Level on Rural Two-Lane Two-Way Roadway Segments... 10-23

Exhibit 10-7: Default Distribution by Collision Type for Specific Crash Severity Levels on Rural Two-Lane Two-Way Roadway Segments. 10-23

Exhibit 10-8: Graphical Representation of the SPF for Three-leg STOP-controlled (3ST) Intersections (Equation 10-8) 10-25

Exhibit 10-9: Graphical Representation of the SPF for Four-leg STOP controlled (4ST) Intersections (Equation 10-9) 10-26

Exhibit 10-10: Graphical Representation of the SPF for Four-leg Signalized (4SG) Intersections (Equation 10-10)... 10-27

Exhibit 10-11: Default Distribution for Crash Severity Level at Rural Two-Lane Two-Way Intersections... 10-27

Exhibit 10-12: Default Distribution for Collision Type and Manner of Collision at Rural Two-Way Intersections 10-28

Exhibit 10-13: Summary of Accident Modification Factors (AMFs) in Chapter 10 and the Corresponding Safety Performance Functions (SPFs)............. 10-29

Exhibit 10-14: AMF for Lane Width on Roadway Segments (AMF_{ra}) 10-30

Exhibit 10-15: Accident Modification Factor for Lane Width on Roadway Segments 10-31

Exhibit 10-16: AMF for Shoulder Width on Roadway Segments (AMF_{wra}) 10-32

Exhibit 10-17: Accident Modification Factor for Shoulder Width on Roadway Segments 10-32

Exhibit 10-18: Accident Modification Factors for Shoulder Types and Shoulder Widths on Roadway Segments (AMF_{tra}) 10-33

Exhibit 10-19: Accident Modification Factors (AMF_{5r}) for Grade of Roadway Segments 10-35

Exhibit 10-20: Nighttime Accident Proportions for Unlighted Roadway Segments 10-39

Exhibit 10-21: Accident Modification Factors (AMF_{2i}) for Installation of Left-Turn Lanes on Intersection Approaches.. 10-41

Exhibit 10-22: Accident Modification Factors (AMF_{3i}) for Right-Turn Lanes on Approaches to an Intersection on Rural Two-Lane Two-Way Highways. 10-41

Exhibit 10-23: Nighttime Accident Proportions for Unlighted Intersections................ 10-42

Exhibit 10-24: List of Sample Problems in Chapter 10 10-44

APPENDIX A

A.1 Appendix A – Worksheets for Predictive Method for Rural Two-Lane Two-Way Roads.. 10-87

CHAPTER 10 PREDICTIVE METHOD FOR RURAL TWO-LANE TWO-WAY ROADS

10.1. INTRODUCTION

Chapter 10 explains the predictive method for rural two-lane two-way roads.

This chapter presents the predictive method for rural two-lane two-way roads. A general introduction to the Highway Safety Manual (HSM) Predictive Method is provided in the *Part C Introduction and Applications Guidance*.

The predictive method for rural two-lane two-way roads provides a structured methodology to estimate the expected average crash frequency, crash severity, and collision types for a rural two-lane two-way facility with known characteristics. All types of crashes involving vehicles of all types, bicycles, and pedestrians are included, with the exception of crashes between bicycles and pedestrians. The predictive method can be applied to existing sites, design alternatives to existing sites, new sites, or for alternative traffic volume projections. An estimate can be made for crash frequency of a prior time period (i.e. what did or would have occurred) or in the future (i.e., what is expected to occur). The development of the predictive method in Chapter 10 is documented by Harwood et al.[4]

This chapter presents the following information about the predictive method for rural two-lane two-way roads:

- A concise overview of the predictive method.
- The definitions of the facility types included in Chapter 10 and site types for which predictive models have been developed for Chapter 10.
- The steps of the predictive method in graphical and descriptive forms.
- Details for dividing a rural two-lane two-way facility into individual sites, consisting of intersections and roadway segments.
- Safety Performance Functions (SPFs) for rural two-lane two-way roads.
- Accident Modification Factors (AMFs) applicable to the SPFs in Chapter 10.
- Guidance for applying the Chapter 10 predictive method and limitations of the predictive method specific to Chapter 10.
- Sample problems illustrating the Chapter 10 predictive method for rural two-lane two-way roads.

10.2. OVERVIEW OF THE PREDICTIVE METHOD

The predictive method provides an 18 step procedure to estimate the "expected average crash frequency", $N_{expected}$ (by total crashes, crash severity or collision type), of a roadway network, facility, or site. In the predictive method the roadway is divided into individual sites, which are homogenous roadway segments and intersections. A facility consists of a contiguous set of individual intersections and roadway segments, referred to as "sites." Different facility types are determined by surrounding land use, roadway cross-section, and degree of access. For each facility type, a number of different site types may exist, such as divided and undivided roadway segments, and unsignalized and signalized intersections. A roadway network consists of a number of contiguous facilities.

The method is used to estimate the expected average crash frequency of an individual site, with the cumulative sum of all sites used as the estimate for an entire facility or network. The estimate is for a given time period of interest (in years) during which the geometric design and traffic control features are unchanged and traffic volumes (AADT) are known or forecasted. The estimate relies on estimates made using predictive models which are combined with observed crash data using the Empirical Bayes (EB) Method.

The EB Method is described in full detail in the Part C Appendix.

The predictive models used within the Chapter 10 predictive method are described in detail in Section 10.3.

The predictive models used in Chapter 10 to determine the predicted average crash frequency, $N_{predicted}$, are of the general form shown in Equation 10-1.

$$N_{predicted} = N_{spf\ x} \times (AMF_{1x} \times AMF_{2x} \times \ldots \times AMF_{yx}) \times C_x \qquad (10\text{-}1)$$

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*;

AMF_{yx} = Accident Modification Factors specific to site type *x* and specific geometric design and traffic control features *y*;

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

10.3. RURAL TWO-LANE TWO-WAY ROADS – DEFINITIONS AND PREDICTIVE MODELS IN CHAPTER 10

This section provides the definitions of the facility and site types included in Chapter 10, and the predictive models for each the site types included in Chapter 10. These predictive models are applied following the steps of the predictive method presented in Section 10.4.

10.3.1. Definition of Chapter 10 Facility and Site Types

The predictive method in Chapter 10 addresses all types of rural two-lane two-way highway facilities, including rural two-lane two-way highways with center two-way left-turn lanes or added passing lanes, and rural two-lane two-way highways containing short sections of rural four-lane highway that serve exclusively to increase passing opportunities (i.e., side-by-side passing lanes). Facilities with four or more lanes are not covered in Chapter 10.

The terms "highway" and "road" are used interchangeably in this chapter and apply to all rural two-way two-lane facilities independent of official state or local highway designation.

Classifying an area as urban, suburban or rural is subject to the roadway characteristics, surrounding population and land uses and is at the user's discretion. In the HSM, the definition of "urban" and "rural" areas is based on Federal Highway Administration (FHWA) guidelines which classify "urban" areas as places inside urban boundaries where the population is greater than 5,000 persons. "Rural" areas are defined as places outside urban areas which have a population greater than 5,000 persons. The HSM uses the term "suburban" to refer to outlying portions of an urban area; the predictive method does not distinguish between urban and suburban portions of a developed area.

Exhibit 10-1 identifies 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.

SPFs are available for: undivided roadway segments, three-leg intersections with STOP control, four-leg intersections with STOP control, and four-leg signalized intersections.

Exhibit 10-1: Rural Two-Lane Two-Way Road Site Type with SPFs in Chapter 10

Site 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)

These specific site types are defined as follows:

- Undivided roadway segment (2U) – a roadway consisting of two lanes with a continuous cross-section providing two directions of travel in which the lanes are not physically separated by either distance or a barrier. In addition, the definition includes a section with three lanes where the center lane is a two-way left-turn lane (TWLTL) or a section with added lanes in one or both directions of travel to provide increased passing opportunities (e.g., passing lanes, climbing lanes, and short four-lane sections).
- Three-leg intersection with STOP control (3ST) – 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.
- Four-leg intersection with STOP control (4ST) – 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.
- Four-leg signalized intersection (4SG) - 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.

10.3.2. Predictive Models for Rural Two-Lane Two-Way Roadway Segments

The predictive models can be used to estimate total predicted average crash frequency (i.e., all crash severities and collision types) or can be used to predict average crash frequency of specific crash severity types or specific collision types. The predictive model for an individual roadway segment or intersection combines a SPF with AMFs and a calibration factor.

For rural two-lane two-way undivided roadway segments the predictive model is shown in Equation 10-2:

$$N_{predicted\ rs} = N_{spf\ rs} \times C_r \times (AMF_{1r} \times AMF_{2r} \times \ldots \times AMF_{12r}) \qquad (10\text{-}2)$$

Where,

$N_{predicted\ rs}$ = predicted average crash frequency for an individual roadway segment for a specific year;

$N_{spf\ rs}$ = predicted average crash frequency for base conditions for an individual roadway segment;

C_r = calibration factor for roadway segments of a specific type developed for a particular jurisdiction or geographical area;

$AMF_{1r} \ldots AMF_{12r}$ = Accident Modification Factors for rural two-way two-lane roadway segments;

This model estimates the predicted average crash frequency of non-intersection related crashes (i.e. crashes that would occur regardless of the presence of an intersection).

10.3.3. Predictive Models for Rural Two-Lane Two-Way Intersections

The predictive models for intersections estimate the predicted average crash frequency of crashes occurring within the limits of an intersection (i.e., at-intersection crashes) and crashes that occur on the intersection legs and are attributed to the presence of an intersection (i.e., intersection-related crashes).

The SPFs for rural two-lane two-way roads are presented in Section 10.6. The associated AMFs for each of the SPFs are presented in Section 10.7 and summarized in Exhibit 10-13.

For all intersection types in Chapter 10 the predictive model is shown in Equation 10-3:

$$N_{predicted\ int} = N_{spf\ int} \times C_i \times (AMF_{1i} \times AMF_{2i} \times \ldots \times AMF_{4i}) \qquad (10\text{-}3)$$

Where,

$N_{predicted\ int}$ = predicted average crash frequency for an individual intersection for the selected year;

$N_{spf\ int}$ = predicted average crash frequency for an intersection with base conditions;

$AMF_{1i} \ldots AMF_{4i}$ = Accident Modification Factors for intersections;

C_i = calibration factor for intersections of a specific type developed for use for a particular jurisdiction or geographical area.

The SPFs for rural two-lane two-way roads are presented in Section 10.6. The associated AMFs for each of the SPFs are presented in Section 10.7, and summarized in Exhibit 10-13. Only the specific AMFs associated with each SPF are applicable to an SPF as these AMFs have base conditions which are identical to the base conditions. The calibration factors, C_r and C_i, are determined in the *Part C* Appendix A.1.1. Due to continual change in the crash frequency and severity distributions with time, the value of the calibration factors may change for the selected year of the study period.

10.4. PREDICTIVE METHOD FOR RURAL TWO-LANE TWO-WAY ROADS

The predictive method for rural two-lane two-way road is shown in Exhibit 10-2. Applying the predictive method yields an estimate of the expected average crash frequency (and/or crash severity and collision types) for a rural two-lane two-way facility. The components of the predictive models in Chapter 10 are determined and applied in Steps 9, 10, and 11 of the predictive method. The information that is needed to apply each step is provided in the following sections and in the *Part C* Appendix.

There are 18 steps in the predictive method. In some situations, certain steps will not be needed because the data is not available or the step is not applicable to the situation at hand. In other situations, steps may be repeated, if an estimate is desired for several sites or for a period of several years. In addition, the predictive method can be repeated as necessary to undertake crash estimation for each alternative design, traffic volume scenario or proposed treatment option (within the same period to allow for comparison).

The following explains the details of each step of the method as applied to two-lane two-way rural roads.

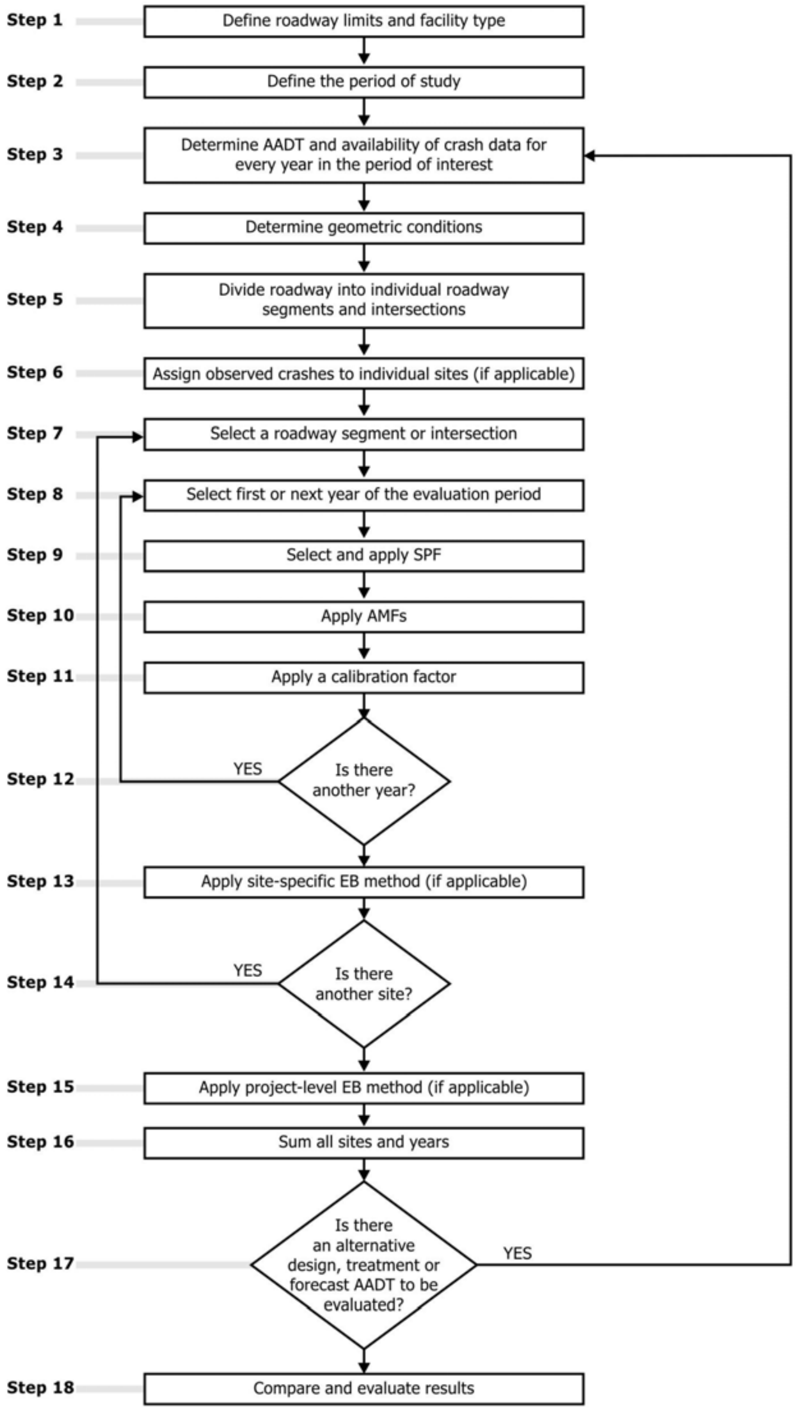
Step 1 - Define the limits of the roadway and facility types in the study network, facility, or site for which the expected average crash frequency, severity, and collision types are to be estimated.

The predictive method can be undertaken for a roadway network, a facility, or a individual site. A site is either an intersection or a homogeneous roadway segment. There are a number of different types of sites, such as signalized and unsignalized intersections. The definitions of a rural two-lane two-way road, an intersection, and a roadway segment and the site types for which SPFs are included in Chapter 10 are provided in Section 10.3.

The predictive method can be applied to an existing roadway, a design alternative for an existing roadway, or a design alternative for new roadway (which may be either unconstructed or yet to experience enough traffic to have observed crash data).

The limits of the roadway of interest will depend on the nature of the study. The study may be limited to only one specific site or a group of contiguous sites. Alternatively, the predictive method can be applied to a long corridor for the purposes of network screening (determining which sites require upgrading to reduce crashes) which is discussed in *Chapter 4*.

Exhibit 10-2: The HSM Predictive Method



Step 2 - Define the period of interest.

The predictive method can be undertaken for either a past period or a future period. All periods are measured in years. Years of interest will be determined by the availability of observed or forecast AADTs, observed crash data, and geometric design data. Whether the predictive method is used for a past or future period depends upon the purpose of the study. The period of study may be:

A past period (based on observed AADTs) for:

- An existing roadway network, facility, or site. If observed crash data are available, the period of study is the period of time for which the observed crash data are available and for which (during that period) the site geometric design features, traffic control features, and traffic volumes are known.
- An existing roadway network, facility, or site for which alternative geometric design features or traffic control features are proposed (for near term conditions).

A future period (based on forecast AADTs) for:

- An existing roadway network, facility, or site for a future period where forecast traffic volumes are available.
- An existing roadway network, facility, or site for which alternative geometric design or traffic control features are proposed for implementation in the future.
- A new roadway network, facility, or site that does not currently exist, but is proposed for construction during some future period.

Step 3 – For the study period, determine the availability of annual average daily traffic volumes and, for an existing roadway network, the availability of observed crash data to determine whether the EB Method is applicable.

Determining Traffic Volumes

The SPFs used in Step 9 (and some AMFs in Step 10), include AADT volumes (vehicles per day) as a variable. For a past period the AADT may be determined by automated recording or estimated from a sample survey. For a future period the AADT may be a forecast estimate based on appropriate land use planning and traffic volume forecasting models, or based on the assumption that current traffic volumes will remain relatively constant.

Roadway segments require two-way AADT.

For each roadway segment, the AADT is the average daily two-way 24 hour traffic volume on that roadway segment in each year of the period to be evaluated selected in Step 8.

Intersections require the major and minor road AADT.

For each intersection, two values are required in each predictive model. These are the AADT of the major street, $AADT_{maj}$,; and the two-way AADT of the minor street, $AADT_{min}$.

In Chapter 10, $AADT_{maj}$ and $AADT_{min}$ are determined as follows: if the AADTs on the two major road legs of an intersection differ, the larger of the two AADT values is used for the intersection. For a three-leg intersection, the minor road AADT is the AADT of the single minor road leg. For a four-leg intersection, if the AADTs of the two minor road legs differ, the larger of the two AADTs values is used for the intersection. If AADTs are available for every roadway segment along a facility, the major road AADTs for intersection legs can be determined without additional data.

In many cases, it is expected that AADT data will not be available for all years of the evaluation period. In that case, an estimate of AADT for each year of the evaluation period is interpolated or extrapolated as appropriate. If there is no established procedure for doing this, the following default rules may be applied within the predictive method to estimate the AADTs for years for which data are not available.

- If AADT data are available for only a single year, that same value is assumed to apply to all years of the before period;
- If two or more years of AADT data are available, the AADTs for intervening years are computed by interpolation;
- The AADTs for years before the first year for which data are available are assumed to be equal to the AADT for that first year;
- The AADTs for years after the last year for which data are available are assumed to be equal to the last year.

If the EB Method is used (discussed below), AADT data are needed for each year of the period for which observed crash frequency data are available. If the EB Method will not be used, AADT data for the appropriate time period—past, present, or future—determined in Step 2 are used.

Determining Availability of Observed Crash Data

Where an existing site or alternative conditions to an existing site are being considered, the EB Method is used. The EB Method is only applicable when reliable observed crash data are available for the specific study roadway network, facility, or site. Observed data may be obtained directly from the jurisdiction's accident report system. At least two years of observed crash frequency data are desirable to apply the EB Method. The EB Method and criteria to determine whether the EB Method is applicable are presented in Section A.2.1 in the Appendix to *Part C.*

The EB Method and criteria to determine whether the EB Method is applicable are presented in Section A.2.1 in the Appendix to Part C.

The EB Method can be applied at the site-specific level (i.e., observed crashes are assigned to specific intersections or roadway segments in Step 6) or at the project level (i.e., observed crashes are assigned to a facility as a whole). The site-specific EB Method is applied in Step 13. Alternatively, if observed crash data are available but can not be assigned to individual roadway segments and intersections, the project level EB Method is applied (in Step 15).

If observed crash data are not available, then Steps 6, 13, and 15 of the predictive method are not conducted. In this case, the estimate of expected average crash frequency is limited to using a predictive model (i.e. the predicted average crash frequency).

Step 4 - Determine geometric design features, traffic control features, and site characteristics for all sites in the study network.

In order to determine the relevant data needs and avoid unnecessary data collection, it is necessary to understand the base conditions of the SPFs in Step 9 and the AMFs in Step 10. The base conditions are defined in Section 10.6.1 for roadway segments and in Section 10.6.2 for intersections.

The base conditions for Chapter 10 SPFs are defined in Section 10.6.1 for roadway segments and in Section 10.6.2 for intersections.

The following geometric design and traffic control features are used to select a SPF and to determine whether the site specific conditions vary from the base conditions and, therefore, whether an AMF is applicable:

- Length of segment (miles)
- AADT (vehicles per day)
- Lane width (feet)
- Shoulder width (feet)
- Shoulder type (paved/gravel/composite/turf)
- Presence or absence of horizontal curve (curve/tangent). If the segment has one or more curve:
  - Length of horizontal curve (miles), (this represents the total length of the horizontal curve and includes spiral transition curves, even if the curve extends beyond the limits of the roadway segment being analyzed);.
  - Radius of horizontal curve (feet);
  - Presence or absence of spiral transition curve, (this represents the presence or absence of a spiral transition curve at the beginning and end of the horizontal curve, even if the beginning and/or end of the horizontal curve are beyond the limits of the segment being analyzed); and
  - Superelevation of horizontal curve and the maximum superelevation (e_{max}) used according to policy for the jurisdiction, if available.
- Grade (percent), considering each grade as a straight grade from Point of Vertical Intersection (PVI) to PVI (i.e., ignoring the presence of vertical curves)
- Driveway density (driveways per mile)
- Presence or absence of centerline rumble strips
- Presence or absence of a passing lane
- Presence or absence of a short four-lane section
- Presence or absence of a two-way left-turn lane
- Roadside hazard rating
- Presence or absence of roadway segment lighting
- Presence or absence of automated speed enforcement

For all intersections within the study area, the following geometric design and traffic control features are identified:

- Number of intersection legs (3 or 4)
- Type of traffic control (minor road stop or signal control)
- Intersection skew angle (degrees departure from 90 degrees)

- Number of approaches with intersection left-turn lanes (0, 1, 2, 3, or 4), not including stop-controlled approaches
- Number of approaches with intersection right-turn lanes (0, 1, 2, 3, or 4), not including stop-controlled approaches
- Presence or absence of intersection lighting

Step 5 – Divide the roadway network or facility under consideration into individual homogenous roadway segments and intersections, which are referred to as sites.

Using the information from Step 1 and Step 4, the roadway is divided into individual sites, consisting of individual homogenous roadway segments and intersections. The definitions and methodology for dividing the roadway into individual intersections and homogenous roadway segments for use with the Chapter 10 predictive models are provided in Section 10.5. When dividing roadway facilities into small homogenous roadway segments, limiting the segment length to a minimum of 0.10 miles will decrease data collection and management efforts.

The definitions and methodology for dividing the roadway into individual intersections and homogenous roadway segments for use with the Chapter 10 predictive models are provided in Section 10.5.

Step 6 – Assign observed crashes to the individual sites (if applicable).

Step 6 only applies if it was determined in Step 3 that the site-specific EB Method was applicable. If the site-specific EB Method is not applicable, proceed to Step 7. In Step 3, the availability of observed data and whether the data could be assigned to specific locations was determined. The specific criteria for assigning accidents to individual roadway segments or intersections are presented in Section A.2.3 of the Appendix to *Part C*.

The specific criteria for assigning crashes to individual roadway segments for intersections are presented in Section A.2.3 of the Appendix to Part C.

Crashes that occur at an intersection or on an intersection leg and are related to the presence of an intersection, are assigned to the intersection and used in the EB Method together with the predicted average crash frequency for the intersection. Crashes that occur between intersections and are not related to the presence of an intersection are assigned to the roadway segment on which they occur; such crashes are used in the EB Method together with the predicted average crash frequency for the roadway segment.

Step 7 – Select the first or next individual site in the study network. If there are no more sites to be evaluated, proceed to Step 15.

In Step 5, the roadway network within the study limits is divided into a number of individual homogenous sites (intersections and roadway segments).

The outcome of the HSM predictive method is the expected average crash frequency of the entire study network, which is the sum of the all of the individual sites, for each year in the study. Note that this value will be the total number of crashes expected to occur over all sites during the period of interest. If a crash frequency (crashes per year) is desired, the total can be divided by the number of years in the period of interest.

The estimation for each site (roadway segments or intersection) is conducted one at a time. Steps 8 through 14, described below, are repeated for each site.

Step 8 – For the selected site, select the first or next year in the period of interest. If there are no more years to be evaluated for that site, proceed to Step 15.

Steps 8 through 14 are repeated for each site in the study and for each year in the study period.

Expected average crashes for the study period are calculated for each year of the period.

The individual years of the evaluation period may have to be analyzed one year at a time for any particular roadway segment or intersection because SPFs and some AMFs (e.g., lane and shoulder widths) are dependent on AADT, which may change from year to year.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type and traffic control features.

Predictive models for rural two-lane two-way roads are provided in Section 10.3.

Steps 9 through 13 are repeated for each year of the evaluation period as part of the evaluation of any particular roadway segment or intersection. The predictive models in Chapter 10 follow the general form shown in Equation 10-1. Each predictive model consists of an SPF, which is adjusted to site specific conditions using AMFs (in Step 10) and adjusted to local jurisdiction conditions (in Step 11) using a calibration factor (C). The SPFs, AMFs and calibration factor obtained in Steps 9, 10, and 11 are applied to calculate the predicted average crash frequency for the selected year of the selected site. The resultant value is the predicted average crash frequency for the selected year. The SPFs available for rural two-lane two-way highways are presented in Section 10.6.

The SPF (which is a statistical regression model based on observed crash data for a set of similar sites) determines the predicted average crash frequency for a site with the base conditions (i.e., a specific set of geometric design and traffic control features). The base conditions for each SPF are specified in Section 10.6. A detailed explanation and overview of the SPFs in *Part C* is provided in Section C.6.3 of the *Part C Introduction and Applications Guidance*.

The SPFs for specific site types (and base conditions) developed for Chapter 10 are summarized in Exhibit 10-4 in Section 10.6. For the selected site, determine the appropriate SPF for the site type (roadway segment or one of three intersection types). The SPF is calculated using the AADT volume determined in Step 3 (AADT for roadway segments or $AADT_{maj}$ and $AADT_{min}$ for intersections) for the selected year.

Default distributions of crash severity and collision type are presented in Exhibit 10-6 and 10-7 for roadway segments and Exhibit 10-11 and 10-12 for intersections.

Each SPF determined in Step 9 is provided with default distributions of crash severity and collision type. The default distributions are presented in Exhibits 10-6 and 10-7 for roadway segments and in Exhibits 10-11 and 10-12 for intersections. These default distributions can benefit from being updated based on local data as part of the calibration process presented in Appendix A.1.1.

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust the estimated crash frequency for base conditions to the site specific geometric design and traffic control features.

An overview of AMFs and guidance for their use is provided in Section C.6.4 of the Part C Introduction and Applications Guidance

In order to account for differences between the base conditions (Section 10.6) and site specific conditions, AMFs are used to adjust the SPF estimate. An overview of AMFs and guidance for their use is provided in Section C.6.4 of the *Part C Introduction and Applications Guidance*, including the limitations of current knowledge related to the effects of simultaneous application of multiple AMFs. In using multiple AMFs, engineering judgment is required to assess the interrelationships and/or independence of individual elements or treatments being considered for implementation within the same project.

Only the AMFs presented in Section 10.7 may be used as part of the Chapter 10 predictive method.

All AMFs used in Chapter 10 have the same base conditions as the SPFs used in Chapter 10 (i.e., when the specific site has the same condition as the SPF base condition, the AMF value for that condition is 1.00). *Only the AMFs presented in Section 10.7 may be used as part of the Chapter 10 predictive method*. Exhibit 10-13 indicates which AMFs are applicable to the SPFs in Section 10.6.

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

The SPFs used in the predictive method have each been developed with data from specific jurisdictions and time periods. Calibration of the SPFs to local conditions will account for differences. A calibration factor (C_r for roadway segments or C_i for intersections) is applied to each SPF in the predictive method. An overview of the use of calibration factors is provided in the *Part C Introduction and Applications Guidance* Section C.6.5. Detailed guidance for the development of calibration factors is included in *Part C* Appendix A.1.1

Detailed guidance for the development of calibration factors is included in Part C Appendix A.1.1.

Steps 9, 10, and 11 together implement the predictive models in Equations 10-2 and 10-3 to determine predicted average crash frequency.

Step 12 – If there is another year to be evaluated in the study period for the selected site, return to Step 8. Otherwise, proceed to Step 13.

This step creates a loop through Steps 8 to 12 that is repeated for each year of the evaluation period for the selected site.

Step 13 – Apply site-specific EB Method (if applicable).

Whether the site-specific EB Method is applicable is determined in Step 3. The site-specific EB Method combines the Chapter 10 predictive model estimate of predicted average crash frequency, $N_{predicted,}$ with the observed crash frequency of the specific site, $N_{observed}$. This provides a more statistically reliable estimate of the expected average crash frequency of the selected site.

In order to apply the site-specific EB Method, in addition to the material in *Part C* Appendix A.2.4, overdispersion parameter, k, for the SPF is also used. The overdispersion parameter provides an indication of the statistical reliability of the SPF. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. This parameter is used in the site-specific EB Method to provide a weighting to $N_{predicted}$ and $N_{observed}$. Overdispersion parameters are provided for each SPF in Section 10.6.

Apply the site-specific EB Method to a future time period, if appropriate.

The estimated expected average crash frequency obtained above applies to the time period in the past for which the observed crash data were obtained. Section A.2.6 in the Appendix to *Part C* provides method to convert the past period estimate of expected average crash frequency into to a future time period.

Step 14 – If there is another site to be evaluated, return to Step 7, otherwise, proceed to Step 15.

This step creates a loop through Steps 7 to 13 that is repeated for each roadway segment or intersection within the facility.

Step 15 – Apply the project level EB Method (if the site-specific EB Method is not applicable).

This step is only applicable to existing conditions when observed crash data are available, but can not be accurately assigned to specific sites (e.g., the crash report may identify crashes as occurring between two intersections, but is not accurate to determine a precise location on the segment). Detailed description of the project level EB Method is provided in *Part C* Appendix A.2.5.

The project level EB Method is described in Part C Appendix A.2.5.

Step 16 – Sum all sites and years in the study to estimate total crash frequency.

The total estimated number of crashes within the network or facility limits during a study period of n years is calculated using Equation 10-4:

$$N_{total} = \sum_{\substack{all\\roadway\\segments}} N_{rs} + \sum_{\substack{all\\intersections}} N_{int} \qquad (10\text{-}4)$$

Where,

N_{total} = total expected number of crashes within the limits of a rural two-lane two-way facility for the period of interest. Or, the sum of the expected average crash frequency for each year for each site within the defined roadway limits within the study period;

N_{rs} = expected average crash frequency for a roadway segment using the predictive method for one specific year;

N_{int} = expected average crash frequency for an intersection using the predictive method for one specific year.

Equation 10-4 represents the total expected number of crashes estimated to occur during the study period. Equation 10-5 is used to estimate the total expected average crash frequency within the network or facility limits during the study period.

$$N_{total\ average} = \frac{N_{total}}{n} \qquad (10\text{-}5)$$

Where,

$N_{total\ average}$ = total expected average crash frequency estimated to occur within the defined network or facility limits during the study period;

n = number of years in the study period.

Step 17 – Determine if there is an alternative design, treatment or forecast AADT to be evaluated.

Steps 3 through 16 of the predictive method are repeated as appropriate for the same roadway limits but for alternative conditions, treatments, periods of interest, or forecast AADTs.

Step 18 – Evaluate and compare results.

The predictive method is used to provide a statistically reliable estimate of the expected average crash frequency within defined network or facility limits over a given period of time, for given geometric design and traffic control features, and known or estimated AADT. In addition to estimating total crashes, the estimate can be made for different crash severity types and different collision types. Default distributions of crash severity and collision type are provided with each SPF in Section 10.6. These default distributions can benefit from being updated based on local data as part of the calibration process presented in *Part C* Appendix A.1.1.

10.5. ROADWAY SEGMENTS AND INTERSECTIONS

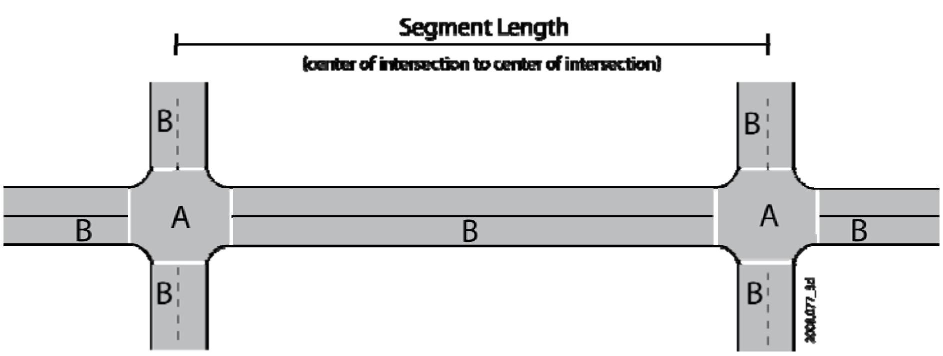
Section 10.4 provides an explanation of the predictive method. Sections 10.5 through 10.8 provide the specific detail necessary to apply the predictive method steps in a rural two-lane two-way road environment. Detail regarding the procedure for determining a calibration factor to apply in Step 11 is provided in the *Part C* Appendix A.1. Detail regarding the EB Method, which is applied in Steps 6, 13, and 15, is provided in the *Part C* Appendix A.2.

In Step 5 of the predictive method, the roadway within the defined roadway limits is divided into individual sites, which are homogenous roadway segments and intersections. A facility consists of a contiguous set of individual intersections and roadway segments, referred to as “sites.” A roadway network consists of a number of contiguous facilities. Predictive models have been developed to estimate crash frequencies separately for roadway segments and intersections. The definitions of roadway segments and intersections presented below are the same as those used in the FHWA Interactive Highway Safety Design Model (IHSDM) [(2)].

Roadway segments begin at the center of an intersection and end at either the center of the next intersection, or where there is a change from one homogeneous roadway segment to another homogenous segment. The roadway segment model estimates the frequency of roadway-segment-related crashes which occur in Region B in Exhibit 10-3. When a roadway segment begins or ends at an intersection, the length of the roadway segment is measured from the center of the intersection.

The Chapter 10 predictive method addresses stop controlled (three- and four-leg) and signalized (four-leg) intersections. The intersection models estimate the predicted average frequency of crashes that occur within the limits of an intersection (Region A of Exhibit 10-3) and intersection-related crashes that occur on the intersection legs (Region B in Exhibit 10-3).

Exhibit 10-3: Definition of Segments and Intersections



A All crashes that occur within this region are classified as intersection crashes.

B Crashes in this region may be segment or intersection related, depending on on the characteristics of the crash.

The roadway segment model estimates the frequency of roadway segment related crashes which occur in Region B in Exhibit 10-3. The intersection models estimate the frequency of all crashes in Region A plus intersection-related crashes that occur in Region B.

The segmentation process produces a set of roadway segments of varying length, each of which is homogeneous with respect to characteristics such as traffic volumes, roadway design characteristics, and traffic control features. Exhibit 10-3 shows the segment length, L, for a single homogenous roadway segment occurring between two intersections. However, it is likely that several homogenous roadway segments will occur between two intersections. A new (unique) homogeneous segment begins at the center of each intersection or at any of the following:

- Beginning or end of a horizontal curve (spiral transitions are considered part of the curve).
- Point of vertical intersection (PVI) for a crest vertical curve, a sag vertical curve, or an angle point at which two different roadway grades meet. Spiral transitions are considered part of the horizontal curve they adjoin and vertical curves are considered part of the grades they adjoin (i.e., grades run from PVI to PVI with no explicit consideration of any vertical curve that may be present).
- Beginning or end of a passing lane or short four-lane section provided for the purpose of increasing passing opportunities.
- Beginning or end of a center two-way left-turn lane.

Also, a new roadway segment starts where there is a change in at least one of the following characteristics of the roadway:

- Average annual daily traffic volume (vehicles per day)
- Lane width

  For lane widths measured to a 0.1-ft level of precision or similar, the following rounded lane widths are recommended before determining "homogeneous" segments:

Measured Lane Width	Rounded Lane Width
9.2-ft or less	9-ft or less
9.3-ft to 9.7-ft	9.5-ft
9.8-ft to 10.2-ft	10-ft
10.3-ft to 10.7-ft	10.5-ft
10.8-ft to 11.2-ft	11-ft
11.3-ft to 11.7-ft	11.5-ft
11.8-ft or more	12-ft or more

- Shoulder width

  For shoulder widths measures to a 0.1-ft level of precision or similar, the following rounded paved shoulder widths are recommended before determining "homogeneous" segments:

Measured Shoulder Width	Rounded Shoulder Width
0.5-ft or less	0-ft
0.6-ft to 1.5-ft	1-ft
1.6-ft to 2.5-ft	2-ft
2.6-ft to 3.5-ft	3-ft
3.6-ft to 4.5-ft	4-ft
4.6-ft to 5.5-ft	5-ft
5.6-ft to 6.5-ft	6-ft
6.6-ft to 7.5-ft	7-ft
7.6-ft or more	8-ft or more

- Shoulder type
- Driveway density (driveways per mile)

  For very short segment lengths (less than 0.5-miles), the use of driveway density for the single segment length may result in an inflated value since driveway density is determined based on length. As a result, the driveway density used for determining homogeneous segments should be for the facility (as defined in Section 10.2) length rather than the segment length.

- Roadside hazard rating

  As described later in Section 10.7.1, the roadside hazard rating (a scale from 1 to 7) will be used to determine a roadside design AMF. Since this rating is a subjective value and can differ marginally based on the opinion of the assessor, it is reasonable to assume that a “homogeneous” segment can have a roadside hazard rating that varies by as much as 2 rating levels. An average of the roadside hazard ratings can be used to compile a “homogeneous” segment as long as the minimum and maximum values are not separated by a value greater than 2. [For example, if the roadside hazard rating ranges from 5 to 7 for a specific road, an average value of 6 can be assumed and this would be considered one homogeneous roadside design condition. If, on the other hand, the roadside hazard ratings ranged from 2 to 5 (a range greater than 2) these would not be considered “homogeneous” roadside conditions and smaller segments may be appropriate.]

- Presence/absence of centerline rumble strip
- Presence/absence of lighting
- Presence/absence of automated speed enforcement

There is no minimum roadway segment length for application of the predictive models for roadway segments. When dividing roadway facilities into small homogenous roadway segments, limiting the segment length to a minimum of 0.10 miles will minimize calculation efforts and not affect results.

In order to apply the site-specific EB Method, observed crashes are assigned to the individual roadway segments and intersections. Observed crashes that occur between intersections are classified as either intersection-related or roadway segment-related. The methodology for assignment of crashes to roadway segments and intersections for use in the site-specific EB Method is presented in Section A.2.3 in the Appendix to *Part C*.

A detailed discussion of SPFs and their use in the HSM is presented in Chapter 3 Section 3.5.2 and the Part C Introduction and Applications Guidance Section C.6.3

10.6. SAFETY PERFORMANCE FUNCTIONS

In Step 9 of the predictive method, the appropriate Safety Performance Functions (SPFs) are used to predict average crash frequency for the selected year for specific base conditions. SPFs are regression models for estimating the predicted average crash frequency of individual roadway segments or intersections. Each SPF in the predictive method was developed with observed crash data for a set of similar sites. The SPFs, like all regression models, estimate the value of a dependent variable as a function of a set of independent variables. In the SPFs developed for the HSM, the dependent variable estimated is the predicted average crash frequency for a roadway segment or intersection under base conditions and the independent variables are the AADTs of the roadway segment or intersection legs (and, for roadway segments, the length of the roadway segment).

The Safety Performance Functions (SPFs) used in Chapter 10 were originally formulated by Vogt and Bared[12,13,14]. A few aspects of the Harwood et al.[4] and Vogt and Bared[12,13,14] work have been updated to match recent changes to the crash prediction module of the FHWA Interactive Highway Safety Design Model[2] software. The SPF coefficients, default crash severity and collision type distributions, and default nighttime crash proportions have been adjusted to a consistent basis by Srinivasan et al[11].

The predicted crash frequencies for base conditions are calculated from the predictive models in Equations 10-2 and 10-3. A detailed discussion of SPFs and their use in the HSM is presented in *Chapter 3* Section 3.5.2 and the *Part C Introduction and Applications Guidance* Section C.6.3.

Each SPF also has an associated overdispersion parameter, k. The overdispersion parameter provides an indication of the statistical reliability of the SPF. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. This parameter is used in the EB Method discussed in the *Part C* Appendix. The SPFs in Chapter 10 are summarized in Exhibit 10-4.

Exhibit 10-4: Safety Performance Functions included in Chapter 10

Chapter 10 SPFs for Rural Two-lane Two-way Roads	SPF Equations and Exhibits
Rural two-lane two-way roadway segments	Equation 10-6 , Exhibit 10-5
Three-leg STOP controlled intersections	Equation 10-8 , Exhibit 10-8
Four-leg STOP controlled intersections	Equation 10-9 , Exhibit 10-9
Four-leg signalized intersections	Equation 10-10 , Exhibit 10-10

Some highway agencies may have performed statistically-sound studies to develop their own jurisdiction-specific SPFs derived from local conditions and crash experience. These models may be substituted for models presented in this chapter. Criteria for the development of SPFs for use in the predictive method are addressed in the calibration procedure presented in the Appendix to *Part C*.

10.6.1. Safety Performance Functions for Rural Two-Lane Two-Way Roadway Segments

The predictive model for predicting average crash frequency for base conditions on a particular rural two-lane two-way roadway segment was presented in Equation 10-2. 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 AMFs.

The base conditions for roadway segments on rural two-lane two-way roads are:

■ Lane width (LW)	12 feet
■ Shoulder width (SW)	6 feet
■ Shoulder type	Paved
■ Roadside hazard rating (RHR)	3
■ Driveway density (DD)	5 driveways per mile
■ Horizontal curvature	None
■ Vertical curvature	None
■ Centerline rumble strips	None
■ Passing lanes	None
■ Two-way left-turn lanes	None
■ Lighting	None
■ Automated speed enforcement	None
■ Grade Level	0% (see note below)

A 0% grade is not allowed by most states and presents issues such as drainage. The SPF uses 0% as a numerical base condition that must always be modified based on the actual grade

The SPF for predicted average crash frequency for rural two-lane two-way roadway segments is shown in Equation 10-6 and presented graphically in Exhibit 10-5:

$$N_{spf\ rs} = AADT \times L \times 365 \times 10^{-6} \times e^{(-0.312)} \qquad (10\text{-}6)$$

Where,

$N_{spf\ rs}$ = predicted total crash frequency for roadway segment base conditions;

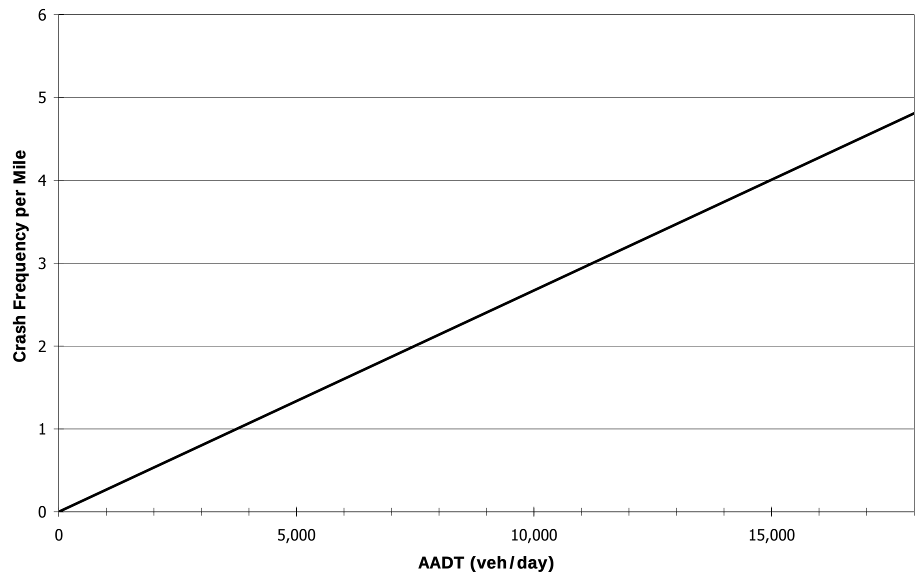
AADT = average annual daily traffic volume (vehicles per day);

L = length of roadway segment (miles).

Guidance on the estimation of traffic volumes for roadway segments for use in the SPFs is presented in Step 3 of the predictive method described in Section 10.4. The SPFs for roadway segments on rural two-lane highways are applicable to the

AADT range from 0 to 17,800 vehicles per day. Application to sites with AADTs substantially outside this range may not provide reliable results.

Exhibit 10-5: Graphical Form of SPF for Rural Two-Lane Two-Way Roadway Segments (Equation 10-6)



The value of the overdispersion parameter associated with the SPF for rural two-lane two-way roadway segments is determined as a function of the roadway segment length using Equation 10-7. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. The value is determined as:

$$k = \frac{0.236}{L} \tag{10-7}$$

Where,

k = overdispersion parameter;

L = length of roadway segment (miles).

Exhibits 10-6 and 10-7 provide the default proportions for crash severity and for collision type by crash severity level, respectively. These exhibits may be used to separate the crash frequencies from Equation 10-6 into components by crash severity level and collision type. Exhibits 10-6 and 10-7 are applied sequentially. First, Exhibit 10-6 is used to estimate crash frequencies by crash severity level and then Exhibit 10-7 is used to estimate accident frequencies by collision type for a particular crash severity level. The default proportions for severity levels and collision types shown in Exhibits 10-6 and 10-7 may be updated based on local data for a particular jurisdiction as part of the calibration process described in the Appendix to *Part C*.

Exhibit 10-6: Default Distribution for Crash Severity Level on Rural Two-Lane Two-Way Roadway Segments

Crash severity level	Percentage of total roadway segment crashes[a]
Fatal	1.3
Incapacitating Injury	5.4
Nonincapacitating injury	10.9
Possible injury	14.5
Total fatal plus injury	32.1
Property damage only	67.9
TOTAL	100.0

a Based on HSIS data for Washington (2002-2006)

Procedures to develop local proportions of crash severity and collision type are provided in the Appendix to Part C.

Exhibit 10-7: Default Distribution by Collision Type for Specific Crash Severity Levels on Rural Two-Lane Two-Way Roadway Segments.

Collision type	Percentage of total roadway segment crashes by crash severity level[a]		
	Total fatal and injury	Property damage only	TOTAL (all severity levels combined)
SINGLE-VEHICLE ACCIDENTS			
Collision with animal	3.8	18.4	12.1
Collision with bicycle	0.4	0.1	0.2
Collision with pedestrian	0.7	0.1	0.3
Overturned	3.7	1.5	2.5
Ran off road	54.5	50.5	52.1
Other single-vehicle accident	0.7	2.9	2.1
Total single-vehicle accidents	63.8	73.5	69.3
MULTIPLE-VEHICLE ACCIDENTS			
Angle collision	10.0	7.2	8.5
Head-on collision	3.4	0.3	1.6
Rear-end collision	16.4	12.2	14.2
Sideswipe collision[b]	3.8	3.8	3.7
Other multiple-vehicle collision	2.6	3.0	2.7
Total multiple-vehicle accidents	36.2	26.5	30.7
TOTAL ACCIDENTS	100.0	100.0	100.0

[a]Based on HSIS data for Washington (2002-2006)

[b]Includes approximately 70% opposite-direction sideswipe collisions and 30% same-direction sideswipe collisions

10.6.2. Safety Performance Functions for Intersections

The predictive model for predicting average crash frequency at particular rural two-lane two-way road intersections was presented in Equation 10-3. The effect of the major and minor road traffic volumes (AADTs) on crash frequency is incorporated through SPFs, while the effects of geometric design and traffic control features are incorporated through the AMFs. The SPFs for rural two-lane two-way highway intersections are presented in this section.

SPFs have been developed for three types of intersections on rural two-lane two-way roads. The three types of intersections are:

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

SPFs for three-leg signalized intersections on rural two-lane two-way roads are not available. Other types of intersections may be found on rural two-lane two-way highways but are not addressed by these procedures.

The SPFs for each of the intersection types listed above estimates total predicted average crash frequency for intersection-related accidents within the limits of a particular intersection and on the intersection legs. The distinction between roadway segment and intersection crashes is discussed in Section 10.5 and a detailed procedure for distinguishing between roadway-segment-related and intersection-related crashes is presented in Section A.2.3 in the Appendix to *Part C*. These SPFs address intersections that have only two lanes on both the major and minor road legs, not including turn lanes. The SPFs for each of the three intersection types are presented below in Equations 10-8, 10-9, and 10-10. Guidance on the estimation of traffic volumes for the major and minor road legs for use in the SPFs is presented in Section 10.4, Step 3.

The base conditions for the rural two-lane two-way road intersection models are presented here.

The base conditions which apply to the SPFs in Equations 10-8, 10-9, and 10-10 are:

▪ Intersection skew angle	0°
▪ Intersection left-turn lanes	None on approaches without stop control
▪ Intersection right-turn lanes	None on approaches without stop control
▪ Lighting	None

Three-Leg Stop-Controlled Intersections

The SPF for three-leg stop-controlled intersections is shown in Equation 10-8 and presented graphically in Exhibit 10-8:

$$N_{spf\ 3ST} = exp[-9.86 + 0.79 \times ln(AADT_{maj}) + 0.49 \times ln(AADT_{min})] \qquad (10\text{-}8)$$

Where,

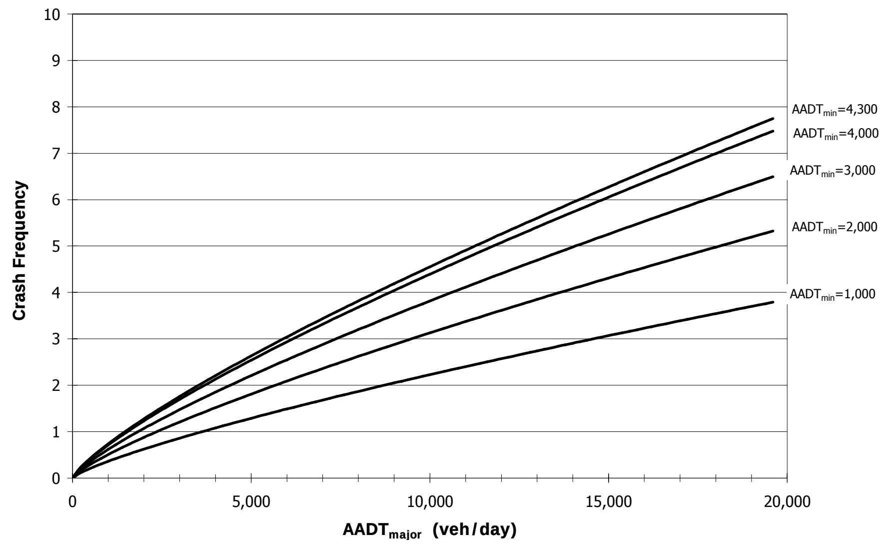
$N_{spf\,3ST}$ = estimate of intersection-related predicted average crash frequency for base conditions for three-leg stop-controlled intersections;

$AADT_{maj}$ = AADT (vehicles per day) on the major road;

$AADT_{min}$ = AADT (vehicles per day) on the minor road.

The overdispersion parameter (k) for this SPF is 0.54. This SPF is applicable to an $AADT_{maj}$ range from 0 to 19,500 vehicles per day and $AADT_{min}$ range from 0 to 4,300 vehicles per day. Application to sites with AADTs substantially outside these ranges may not provide reliable results.

Exhibit 10-8: Graphical Representation of the SPF for Three-leg STOP-controlled (3ST) Intersections (Equation 10-8)



Four-Leg Stop-Controlled Intersections

The SPF for four-leg stop controlled intersections is shown in Equation 10-9 and presented graphically in Exhibit 10-9:

$$N_{spf\ 4ST} = exp[-8.56 + 0.60 \times ln(AADT_{maj}) + 0.61 \times ln(AADT_{min})] \qquad (10\text{-}9)$$

Where,

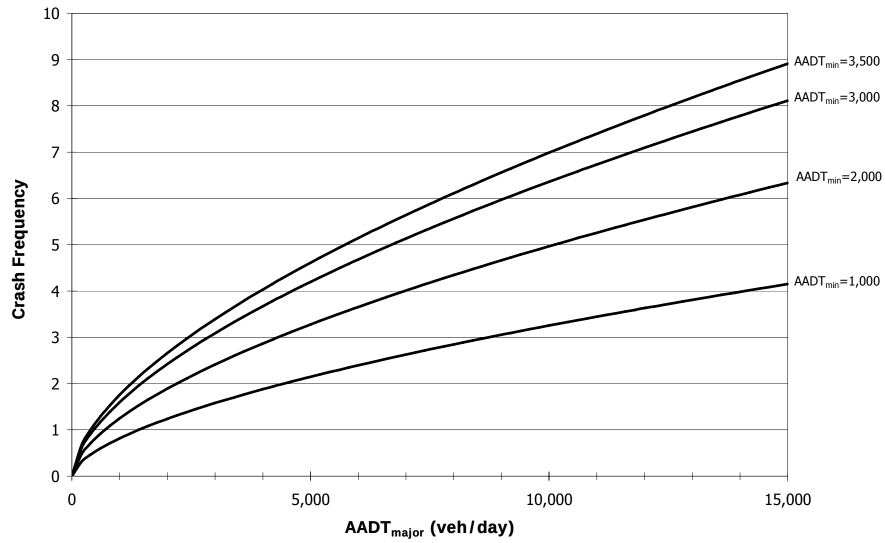
$N_{spf\,4ST}$ = estimate of intersection-related predicted average crash frequency for base conditions for four-leg STOP controlled intersections;

$AADT_{maj}$ = AADT (vehicles per day) on the major road;

$AADT_{min}$ = AADT (vehicles per day) on the minor road.

The overdispersion parameter (k) for this SPF is 0.24. This SPF is applicable to an $AADT_{maj}$ range from 0 to 14,700 vehicles per day and $AADT_{min}$ range from 0 to 3,500 vehicles per day. Application to sites with AADTs substantially outside these ranges may not provide accurate results.

Exhibit 10-9: Graphical Representation of the SPF for Four-leg STOP controlled (4ST) Intersections (Equation 10-9)



Four-Leg Signalized Intersections

The SPF for four-leg signalized intersections is shown below and presented graphically in Exhibit 10-10:

$$N_{spf\ 4SG} = exp[-5.13 + 0.60 \times ln(AADT_{maj}) + 0.20 \times ln(AADT_{min})] \tag{10-10}$$

Where,

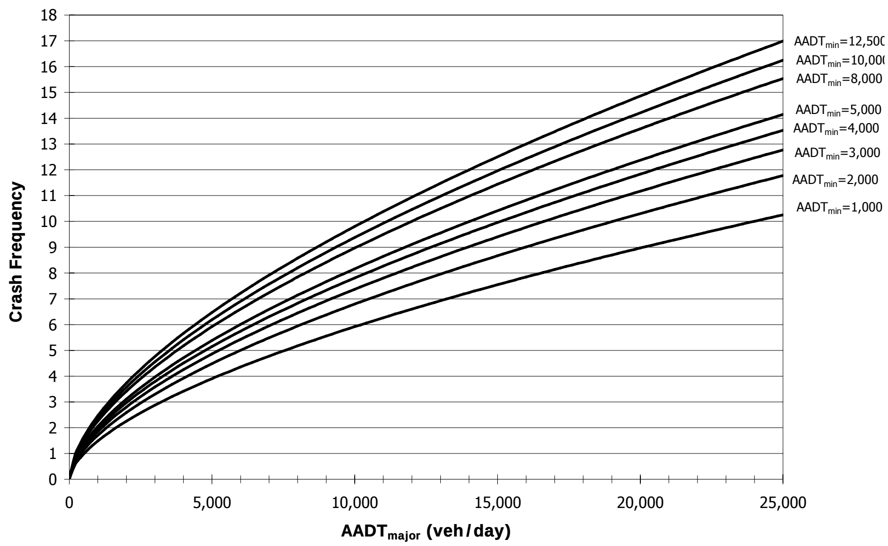
$N_{spf\ 4SG}$ = SPF estimate of intersection-related predicted average crash frequency for base conditions;

$AADT_{maj}$ = AADT (vehicles per day) on the major road;

$AADT_{min}$ = AADT (vehicles per day) on the minor road.

The overdispersion parameter (k) for this SPF is 0.11. This SPF is applicable to an $AADT_{maj}$ range from 0 to 25,200 vehicles per day and $AADT_{min}$ range from 0 to 12,500 vehicles per day. For instances when application is made to sites with AADT substantially outside these ranges, the reliability is unknown.

Exhibit 10-10: Graphical Representation of the SPF for Four-leg Signalized (4SG) Intersections (Equation 10-10)



Exhibits 10-11 and 10-12 provide the default proportions for accident severity levels and collision types, respectively. These exhibits may be used to separate the accident frequencies from Equations 10-8 through 10-10 into components by severity level and collision type. The default proportions for severity levels and collision types shown in Exhibits 10-11 and 10-12 may be updated based on local data for a particular jurisdiction as part of the calibration process described in the Appendix to *Part C*.

Exhibits 10-11 and 10-12 provide the default proportions for accident severity levels and collision types.

Exhibit 10-11: Default Distribution for Crash Severity Level at Rural Two-Lane Two-Way Intersections

	Percentage of total crashes		
Crash severity level	**Three-leg stop-controlled intersections**	**Four-leg stop-controlled intersections**	**Four-leg signalized intersections**
Fatal	1.7	1.8	0.9
Incapacitating Injury	4.0	4.3	2.1
Nonincapacitating injury	16.6	16.2	10.5
Possible injury	19.2	20.8	20.5
Total fatal plus injury	41.5	43.1	34.0
Property damage only	58.5	56.9	66.0
TOTAL	100.0	100.0	100.0

Based on HSIS data for California (2002-2006).

Exhibit 10-12: Default Distribution for Collision Type and Manner of Collision at Rural Two-Way Intersections

Collision Type	Percentage of total crashes by collision type								
	Three-leg stop-controlled intersections			Four-leg stop-controlled intersections			Four-leg signalized intersections		
	Fatal and injury	Property damage only	Total	Fatal and injury	Property damage only	Total	Fatal and injury	Property damage only	Total
SINGLE-VEHICLE ACCIDENTS									
Collision with animal	0.8	2.6	1.9	0.6	1.4	1.0	0.0	0.3	0.2
Collision with bicycle	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Collision with pedestrian	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Overturned	2.2	0.7	1.3	0.6	0.4	0.5	0.3	0.3	0.3
Ran off road	24.0	24.7	24.4	9.4	14.4	12.2	3.2	8.1	6.4
Other single-vehicle accident	1.1	2.0	1.6	0.4	1.0	0.8	0.3	1.8	0.5
Total single-vehicle accidents	28.3	30.2	29.4	11.2	17.4	14.7	4.0	10.7	7.6
MULTIPLE-VEHICLE ACCIDENTS									
Angle collision	27.5	21.0	23.7	53.2	35.4	43.1	33.6	24.2	27.4
Head-on collision	8.1	3.2	5.2	6.0	2.5	4.0	8.0	4.0	5.4
Rear-end collision	26.0	29.2	27.8	21.0	26.6	24.2	40.3	43.8	42.6
Sideswipe collision	5.1	13.1	9.7	4.4	14.4	10.1	5.1	15.3	11.8
Other multiple-vehicle collision	5.0	3.3	4.2	4.2	3.7	3.9	9.0	2.0	5.2
Total multiple-vehicle accidents	71.7	69.8	70.6	88.8	82.6	85.3	96.0	89.3	92.4
TOTAL ACCIDENTS	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

NOTE: Based on HSIS data for California (2002-2006).

10.7. ACCIDENT MODIFICATION FACTORS

In Step 10 of the predictive method shown in Section 10.4, Accident Modification Factors are applied to account for the effects of site-specific geometric design and traffic control features. AMFs are used in the predictive method in Equations 10-2 and 10-3. A general overview of Accident Modification Factors (AMFs) is presented in *Chapter 3* Section 3.5.3. The *Part C Introduction and Applications Guidance* provides further discussion on the relationship of AMFs to the predictive method. This section provides details of the specific AMFs applicable to the Safety Performance Functions presented in Section 10.6.

Accident Modification Factors (AMFs) are used to adjust the SPF estimate of predicted average crash frequency for the effect of individual geometric design and traffic control features, as shown in the general predictive model for Chapter 10 shown in Equation 10-1. The AMF for the SPF base condition of each geometric design or traffic control feature has a value of 1.00. Any feature associated with higher crash frequency than the base condition has an AMF with a value greater than 1.00. Any feature associated with lower crash frequency than the base condition has an AMF with a value less than 1.00.

A general overview of Accident Modification Factors (AMFs) is presented in Chapter 3 Section 3.5.3.

The AMFs used in Chapter 10 are consistent with the AMFs in *Part D*, although they have, in some cases, been expressed in a different form to be applicable to the base conditions. The AMFs presented in Chapter 10 and the specific site types to which they apply are summarized in Exhibit 10-13.

Exhibit 10-13: Summary of Accident Modification Factors (AMFs) in Chapter 10 and the Corresponding Safety Performance Functions (SPFs)

Facility Type	AMF	AMF Description	AMF Equations and Exhibits
Rural Two-Lane Two-Way Roadway Segments	AMF_{1r}	Lane Width	Exhibits 10-14, 10-15, Equation 10-11
	AMF_{2r}	Shoulder Width and Type	Exhibit 10-16, 10-17, 10-18, Equation 10-12
	AMF_{3r}	Horizontal Curves: Length, Radius, and Presence or Absence of Spiral Transitions	Equation 10-13
	AMF_{4r}	Horizontal Curves: Superelevation	Equation 10-14, 10-15, 10-16,
	AMF_{5r}	Grades	Exhibit 10-19
	AMF_{6r}	Driveway Density	Equation 10-17
	AMF_{7r}	Centerline Rumble Strips	See text
	AMF_{8r}	Passing Lanes	See text
	AMF_{9r}	Two-Way Left-Turn Lanes	Equation 10-18, 10-19
	AMF_{10r}	Roadside Design	Equation 10-20
	AMF_{11r}	Lighting	Equation 10-21, Exhibit 10-20
	AMF_{12r}	Automated Speed Enforcement	See text
Three- and four-leg STOP control intersections and four-leg signalized intersections	AMF_{1i}	Intersection Skew Angle	Equation 10-22, 10-23
	AMF_{2i}	Intersection Left-Turn Lanes	Exhibit 10-21
	AMF_{3i}	Intersection Right-Turn Lanes	Exhibit 10-22
	AMF_{4i}	Lighting	Equation 10-24, Exhibit 10-23

Section 10.7.1 provides the AMFs to be used with two-lane rural road segments.

10.7.1. Accident Modification Factors for Roadway Segments

The AMFs for geometric design and traffic control features of rural two-lane two-way roadway segments are presented below. These AMFs are applied in Step 10 of the predictive method and used in Equation 10-2 to adjust the SPF for rural two-lane two-way roadway segments presented in Equation 10-6, to account for differences between the base conditions and the local site conditions.

The first of 12 AMFs for use on rural road segments is an AMF for lane width.

AMF_{1r} - Lane Width

The AMF for lane width on two-lane highway segments is presented in Exhibit 10-14 and illustrated by the graph in Exhibit 10-15. This AMF was developed from the work of Zegeer et al.[(15)] and Griffin and Mak[(3)]. The base value for the lane width AMF is 12-ft. In other words, the roadway segment SPF will predict safety performance of a roadway segment with 12-ft lanes. To predict the safety performance of the actual segment in question (e.g. one with lane widths different than 12 feet), AMFs are used to account for differences between base and actual conditions. Thus, 12-ft lanes are assigned an AMF of 1.00. AMF_{1r} is determined from Exhibit 10-14 based on the applicable lane width and traffic volume range. The relationships shown in Exhibit 10-14 are illustrated in Exhibit 10-15. Lanes greater than 12-ft wide are assigned an AMF equal to that for 12-ft lanes.

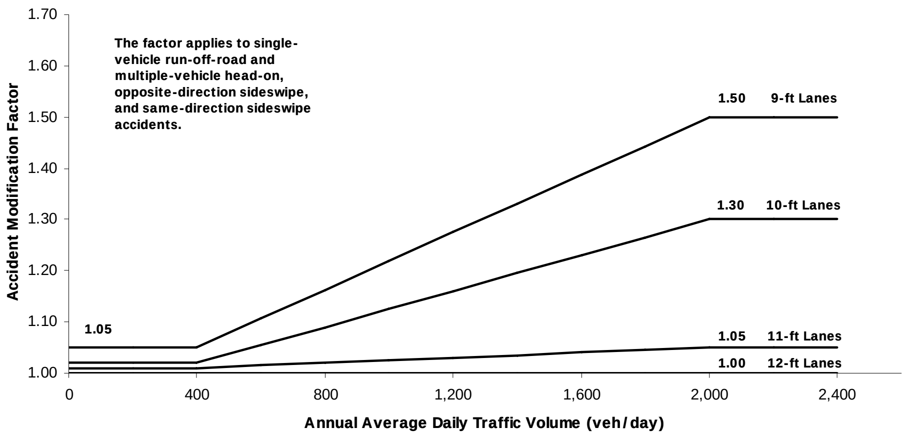
For lane widths with 0.5-ft increments that are not depicted specifically in Exhibit 10-14 or Exhibit 10-15, an AMF value can be interpolated using either of these exhibits since there is a linear transition between the various AADT effects.

Exhibit 10-14: AMF for Lane Width on Roadway Segments (AMF_{ra})

Lane Width	AADT (veh/day)		
	< 400	400 to 2000	> 2000
9-ft or less	1.05	$1.05+2.81 \times 10^{-4}(AADT-400)$	1.50
10-ft	1.02	$1.02+1.75 \times 10^{-4}(AADT-400)$	1.30
11-ft	1.01	$1.01+2.5 \times 10^{-5}(AADT-400)$	1.05
12-ft or more	1.00	1.00	1.00

NOTE: The collision types related to lane width to which this AMF applies include single-vehicle run-off the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents.

Exhibit 10-15: Accident Modification Factor for Lane Width on Roadway Segments



If the lane widths for the two directions of travel on a roadway segment differ, the AMF are determined separately for the lane width in each direction of travel and the resulting AMFs are then be averaged.

The AMFs shown in Exhibits 10-14 and 10-15 apply only to the accident types that are most likely to be affected by lane width: single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents. These are the only accident types assumed to be affected by variation in lane width, and other accident types are assumed to remain unchanged due to the lane width variation. The AMFs expressed on this basis are, therefore, adjusted to total accidents within the predictive method. This is accomplished using Equation 10-11:

$$AMF_{1r} = (AMF_{ra} - 1.0) \times p_{ra} + 1.0 \qquad (10\text{-}11)$$

Where,

AMF_{1r} = Accident Modification Factor for the effect of lane width on total accidents;

AMF_{ra} = Accident Modification Factor for the effect of lane width on related accidents (i.e., single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents), such as the Accident Modification Factor for lane width shown in Exhibit 10-14;

p_{ra} = proportion of total accidents constituted by related accidents.

The proportion of related accidents, p_{ra}, (i.e. single-vehicle run-off-road, and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipes accidents) is estimated as 0.574 (i.e., 57.4%) based on the default distribution of crash types presented in Exhibit 10-7. This default accident type distribution, and therefore the value of p_{ra}, may be updated from local data as part of the calibration process.

AMF_{2r} - Shoulder Width and Type

The second of 12 AMFs for use on two-lane rural road segments is an AMF for shoulder width and type.

The AMF for shoulders has an AMF for shoulder width (AMF_{wra}) and an AMF for shoulder type (AMF_{tra}). The AMFs for both shoulder width and shoulder type are based on the results of Zegeer et al.[15,16] The base value of shoulder width and type is a 6-foot paved shoulder, which is assigned an AMF value of 1.00.

AMF_{wra} for shoulder width on two-lane highway segments is determined from Exhibit 10-16 based on the applicable shoulder width and traffic volume range. The relationships shown in Exhibit 10-16 are illustrated in Exhibit 10-17.

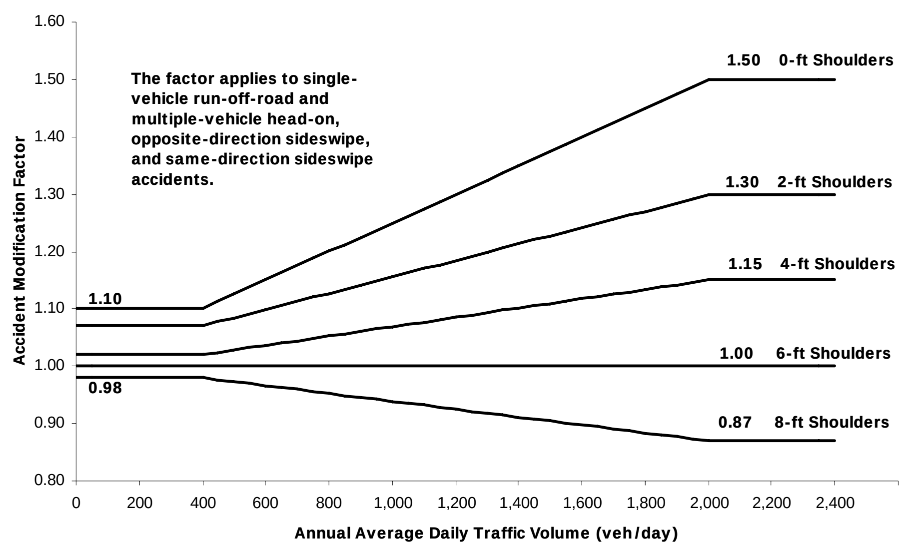
Shoulders over 8-ft wide are assigned an AMF_{wra} equal to that for 8–ft shoulders. The AMFs shown in Exhibits 10-16 and 10-17 apply only to single-vehicle run-off the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents.

Exhibit 10-16: AMF for Shoulder Width on Roadway Segments (AMF_{wra})

Shoulder Width	AADT (vehicles per day)		
	< 400	400 to 2000	> 2000
0-ft	1.10	$1.10 + 2.5 \times 10^{-4}$ (AADT - 400)	1.50
2-ft	1.07	$1.07 + 1.43 \times 10^{-4}$ (AADT - 400)	1.30
4-ft	1.02	$1.02 + 8.125 \times 10^{-5}$ (AADT - 400)	1.15
6-ft	1.00	1.00	1.00
8-ft or more	0.98	$0.98 + 6.875 \times 10^{-5}$ (AADT - 400)	0.87

NOTE: The collision types related to shoulder width to which this AMF applies include single-vehicle run-off the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents.

Exhibit 10-17: Accident Modification Factor for Shoulder Width on Roadway Segments



The base condition for shoulder type is paved. Exhibit 10-18 presents values for AMF_{tra} which adjusts for the safety effects of gravel, turf, and composite shoulders as a function of shoulder width.

Exhibit 10-18: Accident Modification Factors for Shoulder Types and Shoulder Widths on Roadway Segments (AMF_{tra})

Shoulder Type	Shoulder width (ft)						
	0	1	2	3	4	6	8
Paved	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Gravel	1.00	1.00	1.01	1.01	1.01	1.02	1.02
Composite	1.00	1.01	1.02	1.02	1.03	1.04	1.06
Turf	1.00	1.01	1.03	1.04	1.05	1.08	1.11

NOTE: The values for composite shoulders in this exhibit represent a shoulder for which 50 percent of the shoulder width is paved and 50 percent of the shoulder width is turf.

If the shoulder types and/or widths for the two directions of a roadway segment differ, the AMF are determined separately for the shoulder type and width in each direction of travel and the resulting AMFs are then be averaged.

The AMFs for shoulder width and type shown in Exhibits 10-16 through 10-18 apply only to the collision types that are most likely to be affected by shoulder width and type: single-vehicle run-off the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents. The AMFs expressed on this basis are, therefore, adjusted to total accidents using Equation 10-12:

$$AMF_{2r} = (AMF_{wra} \times AMF_{tra} - 1.0) \times p_{ra} + 1.0 \qquad (10\text{-}12)$$

Where,

AMF_{2r} = Accident Modification Factor for the effect of shoulder width and type on total accidents;

AMF_{wra} = Accident Modification Factor for related accidents (i.e., single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents), based on shoulder width (from Exhibit 10-16);

AMF_{tra} = Accident Modification Factor for related accidents based on shoulder type (from Exhibit 10-18);

p_{ra} = proportion of total accidents constituted by related accidents.

The proportion of related accidents, p_{ra}, (i.e. single-vehicle run-off-road, and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipes accidents) is estimated as 0.574 (i.e., 57.4%) based on the default distribution of accident types presented in Exhibit 10-7. This default accident type distribution, and therefore the value of p_{ra}, may be updated from local data by a highway agency as part of the calibration process.

AMF_{3r} - Horizontal Curves: Length, Radius, and Presence or Absence of Spiral Transitions

The third of 12 AMFs for use on two-lane rural road segments is an AMF for horizontal curve length, radius, and the presence or absence of spiral transitions.

The base condition for horizontal alignment is a tangent roadway segment. An AMF has been developed to represent the manner in which accident experience on curved alignments differs from that of tangents. This AMF applies to total roadway segment accidents.

The AMF for horizontal curves has been determined from the regression model developed by Zegeer et al[17].

Equation 10-13 is used to determine the AMF for horizontal curve length, radius, and the presence or absence of spiral transitions.

The AMF for horizontal curvature is in the form of an equation and yields a factor similar to the other AMFs in this chapter. The AMF for length, radius, and presence or absence of spiral transitions on horizontal curves is determined using Equation 10-13:

$$AMF_{3r} = \frac{(1.55 \times L_c) + (\frac{80.2}{R}) - (0.012 \times S)}{(1.55 \times L_c)} \quad (10\text{-}13)$$

Where,

AMF_{3r} = Accident Modification Factor for the effect of horizontal alignment on total accidents;

L_c = length of horizontal curve (miles) which includes spiral transitions, if present;

R = radius of curvature (feet);

S = 1 if spiral transition curve is present; 0 if spiral transition curve is not present; 0.5 if a spiral transition curve is present at one but not both ends of the horizontal curve.

Some roadway segments being analyzed may include only a portion of a horizontal curve. In this case, L_c represents the length of the entire horizontal curve, including portions of the horizontal curve that may lie outside the roadway segment of interest.

In applying Equation 10-13, if the radius of curvature (R) is less than 100-ft, R is set to equal to 100-ft. If the length of the horizontal curve (L_c) is less than 100 feet, L_c is set to equal 100ft.

AMF values are computed separately for each horizontal curve in a horizontal curve set (a curve set consists of a series of consecutive curve elements). For each individual curve, the value of L_c used in Equation 10-13 is the total length of the compound curve set and the value of R is the radius of the individual curve.

If the value of AMF_{3r} is less than 1.00, the value of AMF_{3r} is set equal to 1.00.

AMF_{4r} - Horizontal Curves: Superelevation

The fourth of 12 AMFs for two-lane rural road segments is an AMF for the superelevation of a horizontal curve.

The base condition for the AMF for the superelevation of a horizontal curve is the amount of superelevation identified in the AASHTO Green Book[18]. The superelevation in the AASHTO Green Book is determined by taking into account the value of maximum superelevation rate, e_{max}, established by highway agency policies. Policies concerning maximum superelevation rates for horizontal curves vary between highway agencies based on climate and other considerations.

The AMF for superelevation is based on the superelevation variance of a horizontal curve (i.e., the difference between the actual superelevation and the superelevation identified by AASHTO policy). When the actual superelevation meets or exceeds that in the AASHTO policy, the value of the superelevation AMF is 1.00. There is no effect of superelevation variance on crash frequency until the superelevation variance exceeds 0.01. The general functional form of an AMF for superelevation variance is based on the work of Zegeer et al[17,18].

The following relationships present the AMF for superelevation variance:

$$AMF_{4r} = 1.00 \text{ for } SV < 0.01 \qquad (10\text{-}14)$$

$$AMF_{4r} = 1.00 + 6 \times (SV - 0.01) \text{ for } 0.01 \leq SV < 0.02 \qquad (10\text{-}15)$$

$$AMF_{4r} = 1.06 + 3 \times (SV - 0.02) \text{ for } SV \geq 0.02 \qquad (10\text{-}16)$$

Where,

AMF_{4r} = Accident Modification Factor for the effect of superelevation variance on total accidents;

SV = superelevation variance (ft/ft), which represents the superelevation rate contained in the AASHTO Green Book minus the actual superelevation of the curve.

AMF_{4r} applies to total roadway segment accidents for roadway segments located on horizontal curves.

AMF_{5r} - Grades

The base condition for grade is a generally level roadway. Exhibit 10-19 presents the AMF for grades based on an analysis of rural two-lane two-way highway grades in Utah conducted by Miaou(7). The AMFs in Exhibit 10-19 are applied to each individual grade segment on the roadway being evaluated without respect to the sign of the grade. The sign of the grade is irrelevant because each grade on a rural two-lane two-way highway is an upgrade for one direction of travel and a downgrade for the other. The grade factors are applied to the entire grade from one point of vertical intersection (PVI) to the next (i.e., there is no special account taken of vertical curves). The AMFs in Exhibit 10-19 apply to total roadway segment accidents.

The fifth of 12 AMFs for two-lane rural road segments is an AMF for grades.

Exhibit 10-19: Accident Modification Factors (AMF_{5r}) for Grade of Roadway Segments

Approximate Grade (%)		
Level Grade (≤ 3%)	Moderate Terrain (3%< grade ≤ 6%)	Steep Terrain (> 6%)
1.00	1.10	1.16

The sixth of 12 AMFs for two-lane rural road segments is an AMF for driveway density.

AMF_{6r} - Driveway Density

The base condition for driveway density is five driveways per mile. As with the other AMFs, the model for the base condition was established for roadways with this driveway density. The AMF for driveway density is determined using Equation 10-17, derived from the work of Muskaug[8]:

Equation 10-17 is used to determine the AMF for driveway density.

For DD< 5

AMF = 1.0

$$AMF_{6r} = \frac{0.322 + DD \times [0.05 - 0.005 \times ln(AADT)]}{0.322 + 5 \times [0.05 - 0.005 \times ln(AADT)]} \qquad (10\text{-}17)$$

Where,

AMF_{6r} = Accident Modification Factor for the effect of driveway density on total accidents;

AADT = average annual daily traffic volume of the roadway being evaluated (vehicles per day);

DD = driveway density considering driveways on both sides of the highway (driveways/mile).

If driveway density is less than 5 driveways per mile, AMF_{6r} is 1.00. Equation 10-17 can be applied to total roadway accidents of all severity levels.

Driveways serving all types of land use are considered in determining the driveway density. All driveways that are used by traffic on at least a daily basis for entering or leaving the highway are considered. Driveways that receive only occasional use (less than daily), such as field entrancesare not considered.

The seventh of 12 AMFs for two-lane rural road segments is an AMF for centerline rumble strips.

AMF_{7r} - Centerline Rumble Strips

Centerline rumble strips are installed on undivided highways along the centerline of the roadway which divides opposing directions of traffic flow. Centerline rumble strips are incorporated in the roadway surface to alert drivers who unintentionally cross, or begin to cross, the roadway centerline. The base condition for centerline rumble strips is the absence of rumble strips.

The value of AMF_{7r} for the effect of centerline rumble strips for total crashes on rural two-lane two-way highways is derived as 0.94 from the AMF value presented in *Chapter 13* and crash type percentages found in Chapter 10. Details of this derivation are not provided.

The AMF for centerline rumble strips applies only to two-lane undivided highways with no separation other than a centerline marking between the lanes in opposite directions of travel. Otherwise the value of this AMF is 1.00.

AMF_{8r} - Passing Lanes

The base condition for passing lanes is the absence of a lane (i.e., the normal two-lane cross section). The AMF for a conventional passing or climbing lane added in one direction of travel on a rural two-lane two-way highway is 0.75 for total accidents in both directions of travel over the length of the passing lane from the upstream end of the lane addition taper to the downstream end of the lane drop taper. This value assumes that the passing lane is operationally warranted and that the length of the passing lane is appropriate for the operational conditions on the roadway. There may also be some safety benefit on the roadway downstream of a passing lane, but this effect has not been quantified.

The eighth of 12 AMFs for two-lane rural road segments is an AMF for passing lanes.

The AMF for short four-lane sections (i.e., side-by-side passing lanes provided in opposite directions on the same section of roadway) is 0.65 for total accidents over the length of the short four-lane section. This AMF applies to any portion of roadway where the cross section has four lanes and where both added lanes have been provided over a limited distance to increase passing opportunities. This AMF does not apply to extended four-lane highway sections.

The AMF for passing lanes is based primarily on the work of Harwood and St.John[5], with consideration also given to the results of Rinde[10] and Nettleblad[9]. The AMF for short four-lane sections is based on the work of Harwood and St. John [5].

AMF_{9r} - Two-Way Left-Turn Lanes

The installation of a center two-way left-turn lane (TWLTL) on a rural two-lane two-way highway to create a three-lane cross-section can reduce accidents related to turning maneuvers at driveways. The base condition for two-way left-turn lanes is the absence of a TWLTL. The AMF for installation of a TWLTL is:

The ninth of 12 AMFs for two-lane rural road segments is an AMF for two-way left-turn lanes.

$$AMF_{9r} = 1.0 - \left(0.7 \times p_{dwy} \times p_{LT/D}\right) \qquad (10\text{-}18)$$

Where,

AMF_{9r} = Accident Modification Factor for the effect of two-way left-turn lanes on total accidents;

p_{dwy} = driveway-related accidents as a proportion of total accidents;

$p_{LT/D}$ = left-turn accidents susceptible to correction by a TWLTL as a proportion of driveway-related accidents.

The value of p_{dwy} can be estimated using the following equation[6]

$$p_{dwy} = \frac{\left(0.0047 \times DD\right) + \left(0.0024 \times DD^{(2)}\right)}{1.199 + \left(0.0047 \times DD\right) + \left(0.0024 \times DD^{(2)}\right)} \qquad (10\text{-}19)$$

Where,

p_{dwy} = driveway-related accidents as a proportion of total accidents;

DD = driveway density considering driveways on both sides of the highway (driveways/mile).

The value of $p_{LT/D}$ is estimated as 0.5.[6]

Equation 10-18 provides the best estimate of the AMF for TWLTL installation that can be made without data on the left-turn volumes within the TWLTL.

Realistically, such volumes are seldom available for use in such analyses though Section A.1. of the Appendix to *Part C* describes how to appropriately calibrate this value. This AMF applies to total roadway segment accidents.

The AMF for TWLTL installation is not applied unless the driveway density is greater than or equal to five driveways per mile. If the driveway density is less than five driveways per mile, the AMF for TWLTL installation is 1.00.

The tenth of 12 AMFs for two-lane rural road segments is AMF for roadside design.

AMF_{10r} - Roadside Design

For purposes of the HSM predictive method, the level of roadside design is represented by the roadside hazard rating (1-7 scale) developed by Zegeer et al.[15]. The AMF for roadside design was developed in research by Harwood et al[4]. The base value of roadside hazard rating for roadway segmen ts is 3. The AMF is:

$$AMF_{10r} = \frac{e^{(-0.6869+0.0668 \times RHR)}}{e^{(-0.4865)}} \tag{10-20}$$

Where,

AMF_{10r} = Accident Modification Factor for the effect of roadside design;

RHR = roadside hazard rating.

This AMF applies to total roadway segment accidents. Photographic examples and quantitative definitions for each roadside hazard rating (1 through 7) as a function of roadside design features such as side slope and clear zone width are presented in *Chapter 13* Appendix A.

AMF_{11r} - Lighting

The eleventh of 12 AMFs for two-lane rural road segments is an AMF for lighting.

The base condition for lighting is the absence of roadway segment lighting. The AMF for lighted roadway segments is determined, based on the work of Elvik and Vaa[1], as:

$$AMF_{11r} = 1.0 - [(1.0 - 0.72 \times p_{inr} - 0.83 \times p_{pnr}) \times p_{nr}] \tag{10-21}$$

Where,

AMF_{11r} = Accident Modification Factor for the effect of lighting on total accidents;

p_{inr} = proportion of total nighttime accidents for unlighted roadway segments that involve a fatality or injury;

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

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

This AMF applies to total roadway segment accidents. Exhibit 10-20 presents default values for the nighttime accident proportions p_{inr}, p_{pnr}, and p_{nr}. HSM users are encouraged to replace the estimates in Exhibit 10-20 with locally derived values. If lighting installation increases the density of roadside fixed objects, the value of AMF_{10r} is adjusted accordingly.

Exhibit 10-20: Nighttime Accident Proportions for Unlighted Roadway Segments

Roadway Type	Proportion of total nighttime accidents by severity level		Proportion of accidents that occur at night
	Fatal and Injury p_{inr}	PDO p_{pnr}	p_{nr}
2U	0.382	0.618	0.370

NOTE: Based on HSIS data for Washington (2002-2006)

AMF_{12r} - Automated Speed Enforcement

The twelfth of 12 AMFs for two-lane rural road segments is an AMF for automated speed enforcement.

Automated speed enforcement systems use video or photographic identification in conjunction with radar or lasers to detect speeding drivers. These systems automatically record vehicle identification information without the need for police officers at the scene. The base condition for automated speed enforcement is that it is absent.

The value of AMF_{12r} for the effect of automated speed enforcement for total crashes on rural two-lane two-way highways is derived as 0.93 from the AMF value presented in *Chapter 17* and crash type percentages found in Chapter 10. Details of this derivation are not provided.

10.7.2. Accident Modification Factors for Intersections

Section 10.7.2 presents AMFs for intersections on two-lane rural roads.

The effects of individual geometric design and traffic control features of intersections are represented in the predictive models by AMFs. The AMFs for intersection skew angle, left-turn lanes, right-turn lanes and lighting are presented below. Each of the AMFs applies to total crashes.

AMF_{1i} - Intersection Skew Angle

The first of four AMFs for intersections on two-lane rural roads is an AMF for intersection skew angle.

The base condition for intersection skew angle is 0 degrees of skew (i.e., an intersection angle of 90 degrees). The skew angle for an intersection was defined as the absolute value of the deviation from an intersection angle of 90 degrees. The absolute value is used in the definition of skew angle because positive and negative skew angles are considered to have similar detrimental effect[4]. This is illustrated in *Chapter 14* Section 14.6.2.

Three-Leg Intersections with Stop-Control on the Minor Approach

The AMF for intersection angle at three-leg intersections with stop-control on the minor approach is:

$$AMF_{1i} = e^{(0.004 \times SKEW)} \tag{10-22}$$

Where,

AMF_{1i} = Accident Modification Factor for the effect of intersection skew on total accidents;

$SKEW$ = intersection skew angle (in degrees); the absolute value of the difference between 90 degrees and the actual intersection angle.

This AMF applies to total intersection accidents.

Four-Leg Intersections with Stop-Control on the Minor Approaches

The AMF for intersection angle at four-leg intersection with stop-control on the minor approaches is:

$$AMF_{1i} = e^{(0.0054 \times SKEW)} \tag{10-23}$$

Where,

AMF_{1i} = Accident Modification Factor for the effect of intersection skew on total accidents;

$SKEW$ = intersection skew angle (in degrees); the absolute value of the difference between 90 degrees and the actual intersection angle.

This AMF applies to total intersection accidents.

If the skew angle differs for the two minor road legs at a four-leg stop-controlled intersection, values of AMF_{1i} is computed separately for each minor road leg and then averaged.

Four-leg Signalized Intersections

Since the traffic signal separates most movements from conflicting approaches, the risk of collisions related to the skew angle between the intersecting approaches is limited at a signalized intersection. Therefore, the AMF for skew angle at four-leg signalized intersections is 1.00 for all cases.

The second of four AMFs for intersections on two-lane rural roads is an AMF for intersection left-turn lanes.

***AMF_{2i}* - Intersection Left-Turn Lanes**

The base condition for intersection left-turn lanes is the absence of left-turn lanes on the intersection approaches. The AMFs for the presence of left-turn lanes are presented in Exhibit 10-21. These AMFs apply to installation of left-turn lanes on any approach to a signalized intersection, but only on uncontrolled major road approaches to a stop-controlled intersection. The AMFs for installation of left-turn lanes on multiple approaches to an intersection are equal to the corresponding AMF for the installation of a left-turn lane on one approach raised to a power equal to the number of approaches with left-turn lanes. There is no indication of any safety effect of providing a left-turn lane on an approach controlled by a stop sign, so the presence of a left-turn lane on a stop-controlled approach is not considered in applying Exhibit 10-21. The AMFs for installation of left-turn lanes are based on research by Harwood et al.[(4)] and are consistent with the AMFs presented in *Chapter 14*. An AMF of 1.00 is always be used when no left-turn lanes are present.

Exhibit 10-21: Accident Modification Factors (AMF_{2i}) for Installation of Left-Turn Lanes on Intersection Approaches.

Intersection type	Intersection traffic control	Number of approaches with left-turn lanes [a]			
		One approach	Two approaches	Three approaches	Four approaches
Three-leg intersection	Minor road stop control[b]	0.56	0.31	—	—
Four-leg intersection	Minor road stop control[b]	0.72	0.52	—	—
	Traffic signal	0.82	0.67	0.55	0.45

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

AMF_{3i} - Intersection Right-Turn Lanes

The third of four AMFs for intersections on two-lane rural roads is an AMF for intersection right-turn lanes.

The base condition for intersection right-turn lanes is the absence of right-turn lanes on the intersection approaches. The AMF for the presence of right-turn lanes is based on research by Harwood et al.[(4)] and is consistent with the AMFs in *Chapter 14*. These AMFs apply to installation of right-turn lanes on any approach to a signalized intersection, but only on uncontrolled major road approaches to stop-controlled intersections. The AMFs for installation of right-turn lanes on multiple approaches to an intersection are equal to the corresponding AMF for installation of a right-turn lane on one approach raised to a power equal to the number of approaches with right-turn lanes. There is no indication of any safety effect for providing a right-turn lane on an approach controlled by a stop sign, so the presence of a right-turn lane on a stop-controlled approach is not considered in applying Exhibit 10-22. The AMFs in the exhibit apply to total intersection accidents. An AMF value of 1.00 is always be used when no right-turn lanes are present. This AMF applies only to right-turn lanes that are identified by marking or signing. The AMF is not applicable to long tapers, flares, or paved shoulders that may be used informally by right-turn traffic.

Exhibit 10-22: Accident Modification Factors (AMF_{3i}) for Right-Turn Lanes on Approaches to an Intersection on Rural Two-Lane Two-Way Highways.

Intersection type	Intersection traffic control	Number of approaches with right-turn lanes [a]			
		One approach	Two approaches	Three approaches	Four approaches
Three-leg intersection	Minor road stop control[b]	0.86	0.74	–	–
Four-leg intersection	Minor road stop control[b]	0.86	0.74	—	—
	Traffic signal	0.96	0.92	0.88	0.85

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

The fourth of four AMFs for intersections on two-lane rural roads is an AMF for lighting.

AMF_{4i} - Lighting

The base condition for lighting is the absence of intersection lighting. The AMF for lighted intersections is adapted from the work of Elvik and Vaa [(1)], as:

$$AMF_{4i} = 1 - 0.38 \times p_{ni} \qquad (10\text{-}24)$$

Where,

AMF_{4i} = Accident Modification Factor for the effect of lighting on total accidents;

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

This AMF applies to total intersection accidents. Exhibit 10-23 presents default values for the nighttime accident proportion p_{ni}. HSM users are encouraged to replace the estimates in Exhibit 10-23 with locally derived values.

Exhibit 10-23: Nighttime Accident Proportions for Unlighted Intersections

Intersection Type	Proportion of accidents that occur at night p_{ni}
3ST	0.260
4ST	0.244
4SG	0.286

Based on HSIS data for California (2002-2006)

10.8. CALIBRATION OF THE SPFS TO LOCAL CONDITIONS

In Step 10 of the predictive method, presented in Section 10.4, the predictive model is calibrated to local state or geographic conditions. Accident frequencies, even for nominally similar roadway segments or intersections, can vary widely from one jurisdiction to another. Geographic regions differ markedly in climate, animal population, driver populations, accident reporting threshold, and accident reporting practices. These variations may result in some jurisdictions experiencing a different number of reported traffic accidents on rural two-lane two-way roads than others. Calibration factors are included in the methodology to allow highway agencies to adjust the SPFs to match actual local conditions.

The calibration procedures are presented in the Appendix to Part C.

The calibration factors for roadway segments and intersections (defined as C_r and C_i, respectively) will have values greater than 1.0 for roadways that, on average, experience more accidents than the roadways used in the development of the SPFs. The calibration factors for roadways that experience fewer accidents on average than the roadways used in the development of the SPFs will have values less than 1.0. The calibration procedures are presented in the Appendix to *Part C*.

Calibration factors provide one method of incorporating local data to improve estimated accident frequencies for individual agencies or locations. Several other default values used in the predictive method, such as collision type distribution, can also be replaced with locally derived values. The derivation of values for these parameters is addressed in the calibration procedure in the Appendix to *Part C*.

10.9. LIMITATIONS OF PREDICTIVE METHOD IN CHAPTER 10

This section discusses limitations of the specific predictive models and the application of the predictive method in Chapter 10.

Limitations of the predictive method which apply generally across all of the Part C chapters are discussed in Section C.14 of the Part C Introduction and Applications Guidance chapter.

Where rural two-lane two-way roads intersect access-controlled facilities (i.e., freeways), the grade-separated interchange facility, including the two-lane road within the interchange area, cannot be addressed with the predictive method for rural two-lane two-way roads.

The SPFs developed for Chapter 10 do not include signalized three-leg intersection models. Such intersections are occasionally found on rural two-lane two-way roads.

10.10. APPLICATION OF CHAPTER 10 PREDICTIVE METHOD

The predictive method presented in Chapter 10 applies to rural two-lane two-way roads. The predictive method is applied to a rural two-lane two-way facility by following the 18 steps presented in Section 10.4. Appendix A provides a series of worksheets for applying the predictive method and the predictive models detailed in this chapter. All computations within these worksheets are conducted with values expressed to three decimal places. This level of precision is needed for consistency in computations. In the last stage of computations, rounding the final estimate of expected average crash frequency to one decimal place is appropriate.

10.11. SUMMARY

The predictive method can be used to estimate the expected average crash frequency for a series of contiguous sites (entire rural two-lane two-way facility), or a single individual site. A rural two-lane two-way facility is defined in Section 10.3, and consists of a two-lane two-way undivided road which does not have access control and is outside of cities or towns with a population greater than 5,000 persons. Two-lane two-way undivided roads that have occasional added lanes to provide additional passing opportunities can also be addressed with the Chapter 10 predictive method.

The predictive method for rural two-lane two-way roads is applied by following the 18 steps of the predictive method presented in Section 10.4. Predictive models, developed for rural two-lane two-way facilities, are applied in Steps 9, 10, and 11 of the method. These predictive models have been developed to estimate the predicted average crash frequency of an individual site which is an intersection or homogenous roadway segment. The facility is divided into these individual sites in Step 5 of the predictive method.

Each predictive model in Chapter 10 consists of a Safety Performance Function (SPF), Accident Modification Factors (AMFs), and a calibration factor. The SPF is selected in Step 9, and is used to estimate the predicted average crash frequency for a site with base conditions. The estimate can be for total crashes, or by crash severity or collision type distribution. In order to account for differences between the base conditions and the specific conditions of the site, AMFs are applied in Step 10, which adjust the prediction to account for the geometric design and traffic control features of the site. Calibration factors are also used to adjust the prediction to local conditions in the jurisdiction where the site is located. The process for determining calibration factors for the predictive models is described in the *Part C* Appendix A.1.

Section 10.12 presents 6 sample problems which detail the application of the predictive method. Appendix A contains worksheets which can be used in the calculations for the predictive method steps.

10.12. SAMPLE PROBLEMS

In this section, six sample problems are presented using the predictive method for rural two-lane two-way roads. Sample Problems 1 and 2 illustrate how to calculate the predicted average crash frequency for rural two-lane roadway segments. Sample Problem 3 illustrates how to calculate the predicted average crash frequency for a stop-controlled intersection. Sample Problem 4 illustrates a similar calculation for a signalized intersection. Sample Problem 5 illustrates how to combine the results from Sample Problems 1 through 3 in a case where site-specific observed crash data are available (i.e. using the site-specific EB Method). Sample Problem 6 illustrates how to combine the results from Sample Problems 1 through 3 in a case where site-specific observed crash data are not available but project-level observed crash data are available (i.e. using the project-level EB Method).

Exhibit 10-24: List of Sample Problems in Chapter 10

Problem No.	Page No.	Description
1	10-44	Predicted average crash frequency for a tangent roadway segment
2	10-53	Predicted average crash frequency for a curved roadway segment
3	10-62	Predicted average crash frequency for a three-leg stop-controlled intersection
4	10-70	Predicted average crash frequency for a four-leg signalized intersection
5	10-77	Expected average crash frequency for a facility when site-specific observed crash data are available
6	10-81	Expected average crash frequency for a facility when site-specific observed crash data are not available

10.12.1. Sample Problem 1

The Site/Facility

A rural two-lane tangent roadway segment.

The Question

What is the predicted average crash frequency of the roadway segment for a particular year?

The Facts

- 1.5-mi length
- Tangent roadway segment
- 10,000 veh/day
- 2% grade
- 6 driveways per mi
- 10-ft lane width
- 4-ft gravel shoulder
- Roadside hazard rating = 4

Assumptions

- Collision type distributions used are the default values presented in Exhibit 10-7.
- The calibration factor is assumed to be 1.10.

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the roadway segment in Sample Problem 1 is determined to be 6.1 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8

To determine the predicted average crash frequency of the roadway segment in Sample Problem 1, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year, and the EB Method is not applied.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type and traffic control features.

The SPF for a single roadway segment can be calculated from Equation 10-6 as follows:

$$N_{spr\ rt} = AADT \times L \times 365 \times 10^{-6} \times e^{(-0.312)}$$

$$= 10{,}000 \times 1.5 \times 365 \times 10^{-6} \times e^{(-0.312)}$$

$$= 4.008 \text{ crashes/year}$$

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust the estimated crash frequency for base conditions to the site-specific geometric design and traffic control features.

Each AMF used in the calculation of the predicted average crash frequency of the roadway segment is calculated below:

Lane Width (AMF_{1r})

AMF_{1r} can be calculated from Equation 10-11 as follows:

$$AMF_{1r} = (AMF_{ra} - 1.0) \times p_{ra} + 1.0$$

For a 10-ft lane width and AADT of 10,000, AMF_{ra} = 1.30 (see Exhibit 10-14).

The proportion of related crashes, p_{ra}, is 0.574 (see discussion below Equation 10-11).

$$AMF_{1r} = (1.3 - 1.0) \times 0.574 + 1.0$$

$$= 1.17$$

Shoulder Width and Type (AMF_{2r})

AMF_{2r} can be calculated from Equation 10-12, using values from Exhibit 10-16, Exhibit 10-18 and Exhibit 10-7 as follows:

$$AMF_{2r} = (AMF_{wra} \times AMF_{ra} - 1.0) \times p_{ra} + 1.0$$

For 4-ft shoulders and AADT of 10,000, AMF_{wra} = 1.15 (see Exhibit 10-16).

For 4-ft gravel shoulders, AMF_{tra} = 1.01 (see Exhibit 10-18).

The proportion of related crashes, p_{ra}, is 0.574 (see discussion below Equation 10-12).

$$AMF_{2r} = (1.15 \times 1.01 - 1.0) \times 0.574. + 1.0$$

$$= 1.09$$

Horizontal Curves: Length, Radius, and Presence or Absence of Spiral Transitions (AMF_{3r})

Since the roadway segment in Sample Problem 1 is a tangent, AMF_{3r} = 1.00 (i.e. the base condition for AMF_{3r} is no curve).

Horizontal Curves: Superelevation (AMF_{4r})

Since the roadway segment in Sample Problem 1 is a tangent, and therefore has no superelevation, AMF_{4r} = 1.00.

Grade (AMF_{5r})

From Exhibit 10-19, for a 2% grade, AMF_{5r} = 1.00

Driveway Density (AMF_{6r})

The driveway density, DD, is 6 driveways per mile. AMF_{6r} can be calculated using Equation 10-17 as follows:

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

$$= \frac{0.322 + 6 \times [0.05 - 0.005 \times ln(10,000)]}{0.322 + 5 \times [0.05 - 0.005 \times ln(10,000)]}$$

$$= 1.01$$

Centerline Rumble Strips (AMF_{7r})

Since there are no centerline rumble strips in Sample Problem 1, AMF_{7r} = 1.00 (i.e. the base condition for AMF_{7r} is no centerline rumble strips).

Passing Lanes (AMF_{8r})

Since there are no passing lanes in Sample Problem 1, AMF_{8r} = 1.00 (i.e. the base condition for AMF_{8r} is the absence of a passing lane).

Two-Way Left-Turn Lanes (AMF_{9r})

Since there are no two-way left-turn lanes in Sample Problem 1, AMF_{9r} = 1.00 (i.e. the base condition for AMF_{9r} is the absence of a two-way left-turn lane).

Roadside Design (AMF_{10r})

The roadside hazard rating, RHR, in Sample Problem 1 is 4. AMF_{10r} can be calculated from Equation 10-20 as follows:

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

$$= \frac{e^{(-0.6869+0.0668\times 4)}}{e^{(-0.4865)}}$$

$$= 1.07$$

Lighting (AMF_{11r})

Since there is no lighting in Sample Problem 1, AMF_{11r} = 1.00 (i.e. the base condition for AMF_{11r} is the absence of roadway lighting).

Automated Speed Enforcement (AMF_{12r})

Since there is no automated speed enforcement in Sample Problem 1, AMF_{12r} = 1.00 (i.e. the base condition for AMF_{12r} is the absence of automated speed enforcement).

The combined AMF value for Sample Problem 1 is calculated below.

$$AMF_{COMB} = 1.17 \times 1.09 \times 1.01 \times 1.07$$

$$= 1.38$$

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed a calibration factor, C_r, of 1.10 has been determined for local conditions. See *Part C* Appendix A.1 for further discussion on calibration of the predictive models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated using Equation 10-2 based on the results obtained in Steps 9 through 11 as follows:

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

$$= 4.008 \times 1.10 \times (1.38)$$

$$= 6.084 \text{ crashes/year}$$

Worksheets

The step-by-step instructions above are provided to illustrate the predictive method for calculating the predicted average crash frequency for a roadway segment. To apply the predictive method steps to multiple segments, a series of five worksheets are provided for determining predicted average crash frequency. The five worksheets include:

- Worksheet 1A – General Information and Input Data for Rural Two-Lane Two-Way Roadway Segments
- Worksheet 1B – Accident Modification Factors for Rural Two-Lane Two-Way Roadway Segments

- Worksheet 1C – Roadway Segment Crashes for Rural Two-Lane Two-Way Roadway Segments
- Worksheet 1D – Crashes by Severity Level and Collision Type for Rural Two-Lane Two-Way Roadway Segments
- Worksheet 1E – Summary Results for Rural Two-Lane Two-Way Roadway Segments

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 10 Appendix A.

Worksheet 1A – General Information and Input Data for Rural Two-Lane Two-Way Roadway Segments

Worksheet 1A is a summary of general information about the roadway segment, analysis, input data (i.e., "The Facts") and assumptions for Sample Problem 1.

<table>
<tr><th colspan="4">Worksheet 1A – General Information and Input Data for Rural Two-Lane Two-Way Roadway Segments</th></tr>
<tr><th colspan="2">General Information</th><th colspan="2">Location Information</th></tr>
<tr><td>Analyst</td><td></td><td>Roadway</td><td></td></tr>
<tr><td>Agency or Company</td><td></td><td>Roadway Section</td><td></td></tr>
<tr><td>Date Performed</td><td></td><td>Jurisdiction</td><td></td></tr>
<tr><td></td><td></td><td>Analysis Year</td><td></td></tr>
<tr><th colspan="2">Input Data</th><th>Base Conditions</th><th>Site Conditions</th></tr>
<tr><td colspan="2">Length of segment, L (mi)</td><td>-</td><td>1.5</td></tr>
<tr><td colspan="2">AADT (veh/day)</td><td>-</td><td>10,000</td></tr>
<tr><td colspan="2">Lane width (ft)</td><td>12</td><td>10</td></tr>
<tr><td colspan="2">Shoulder width (ft)</td><td>6</td><td>4</td></tr>
<tr><td colspan="2">Shoulder type</td><td>paved</td><td>Gravel</td></tr>
<tr><td colspan="2">Length of horizontal curve (mi)</td><td>0</td><td>not present</td></tr>
<tr><td colspan="2">Radius of curvature (ft)</td><td>0</td><td>not present</td></tr>
<tr><td colspan="2">Spiral transition curve (present/not present)</td><td>not present</td><td>not present</td></tr>
<tr><td colspan="2">Superelevation variance (ft/ft)</td><td><0.01</td><td>not present</td></tr>
<tr><td colspan="2">Grade (%)</td><td>0</td><td>2</td></tr>
<tr><td colspan="2">Driveway density (driveways/mile)</td><td>5</td><td>6</td></tr>
<tr><td colspan="2">Centerline rumble strips (present/not present)</td><td>not present</td><td>not present</td></tr>
<tr><td colspan="2">Passing lanes (present/not present)</td><td>not present</td><td>not present</td></tr>
<tr><td colspan="2">Two-way left-turn lane (present/not present)</td><td>not present</td><td>not present</td></tr>
<tr><td colspan="2">Roadside hazard rating (1-7 scale)</td><td>3</td><td>4</td></tr>
<tr><td colspan="2">Segment lighting (present/not present)</td><td>not present</td><td>not present</td></tr>
<tr><td colspan="2">Auto speed enforcement (present/not present)</td><td>not present</td><td>not present</td></tr>
<tr><td colspan="2">Calibration Factor, C_r</td><td>1.0</td><td>1.1</td></tr>
</table>

Worksheet 1B – Accident Modification Factors for Rural Two-Lane Two-Way Roadway Segments

In Step 10 of the predictive method, Accident Modification Factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 10.7 presents the tables and equations necessary for determining AMF values. Once the value for each AMF has been determined, all of the AMFs are multiplied together in Column 13 of Worksheet 1B which indicates the combined AMF value.

Worksheet 1B – Accident Modification Factors for Rural Two-Lane Two-Way Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
AMF for Lane Width	AMF for Shoulder Width and Type	AMF for Horizontal Curves	AMF for Superelevation	AMF for Grades	AMF for Driveway Density	AMF for Centerline Rumble Strips	AMF for Passing Lanes	AMF for Two-Way Left-Turn Lane	AMF for Roadside Design	AMF for Lighting	AMF for Automated Speed Enforcement	Combined AMF
AMF_{1r}	AMF_{2r}	AMF_{3r}	AMF_{4r}	AMF_{5r}	AMF_{6r}	AMF_{7r}	AMF_{8r}	AMF_{9r}	AMF_{10r}	AMF_{11r}	AMF_{12r}	AMF_{COMB}
from Equation 10-11	from Equation 10-12	from Equation 10-13	from Equations 10-14, 10-15, or 10-16	from Exhibit 10-19	from Equation 10-17	from Section 10.7.1	from Section 10.7.1	from Equation 10-18	from Equation 10-20	from Equation 10-21	from Section 10.7.1	(1)*(2)*… *(11)*(12)
1.17	1.09	1.00	1.00	1.00	1.01	1.00	1.00	1.00	1.07	1.00	1.00	1.38

Worksheet 1C – Roadway Segment Crashes for Rural Two-Lane Two-Way Roadway Segments

The SPF for the roadway segment in Sample Problem 1 is calculated using Equation 10-6 and entered into Column 2 of Worksheet 1C. The overdispersion parameter associated with the SPF can be entered into Column 3; however, the overdispersion parameter is not needed for Sample Problem 1 (as the EB Method is not utilized). Column 4 of the worksheet presents the default proportions for crash severity levels from Exhibit 10-6. These proportions may be used to separate the SPF (from Column 2) into components by crash severity level, as illustrated in Column 5. Column 6 represents the combined AMF (from Column 13 in Worksheet 1B), and Column 7 represents the calibration factor. Column 8 calculates the predicted average crash frequency using the values in Column 5, the combined AMF in Column 6, and the calibration factor in Column 7.

Worksheet 1C – Roadway Segment Crashes for Rural Two-Lane Two-Way Roadway Segments							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crash Severity Level	**$N_{spf\ rs}$**	**Overdispersion Parameter, k**	**Crash Severity Distribution**	**$N_{spf\ rs}$ by Severity Distribution**	**Combined AMFs**	**Calibration Factor, C_r**	**Predicted average crash frequency, $N_{predicted\ rs}$**
	from Equation 10-6	from Equation 10-7	from Exhibit 10-6	$(2)_{TOTAL}$ * (4)	(13) from Worksheet 1B		(5)*(6)*(7)
Total	4.008	0.16	1.000	4.008	1.38	1.10	6.084
Fatal and Injury (FI)	-	-	0.321	1.287	1.38	1.10	1.954
Property Damage Only (PDO)	-	-	0.679	2.721	1.38	1.10	4.131

Worksheet 1D – Crashes by Severity Level and Collision for Rural Two-Lane Two-Way Roadway Segments

Worksheet 1D presents the default proportions for collision type (from Exhibit 10-7) by crash severity level as follows:

- Total crashes (Column 2)
- Fatal and injury crashes (Column 4)
- Property damage only crashes (Column 6)

Using the default proportions, the predicted average crash frequency by collision type is presented in Columns 3 (Total), 5 (Fatal and Injury, FI) and 7 (Property Damage Only, PDO).

These proportions may be used to separate the predicted average crash frequency (from Column 8, Worksheet 1C) by crash severity and collision type.

Worksheet 1D – Crashes by Severity Level and Collision Type for Rural Two-Lane Two-Way Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Collision Type	**Proportion of Collision Type (TOTAL)**	**$N_{predicted\ rs\ (TOTAL)}$ (crashes/year)**	**Proportion of Collision Type (FI)**	**$N_{predicted\ rs\ (FI)}$ (crashes/year)**	**Proportion of Collision Type (PDO)**	**$N_{predicted\ rs\ (PDO)}$ (crashes/year)**
	from Exhibit 10-7	$(8)_{TOTAL}$ from Worksheet 1C	from Exhibit 10-7	$(8)_{FI}$ from Worksheet 1C	from Exhibit 10-7	$(8)_{PDO}$ from Worksheet 1C
Total	1.000	6.084	1.000	1.954	1.000	4.131
		$(2)*(3)_{TOTAL}$		$(4)*(5)_{FI}$		$(6)*(7)_{PDO}$
SINGLE-VEHICLE						
Collision with animal	0.121	0.736	0.038	0.074	0.184	0.760
Collision with bicycle	0.002	0.012	0.004	0.008	0.001	0.004
Collision with pedestrian	0.003	0.018	0.007	0.014	0.001	0.004
Overturned	0.025	0.152	0.037	0.072	0.015	0.062
Ran off road	0.521	3.170	0.545	1.065	0.505	2.086
Other single-vehicle collision	0.021	0.128	0.007	0.014	0.029	0.120
Total single-vehicle crashes	0.693	4.216	0.638	1.247	0.735	3.036
MULTIPLE-VEHICLE						
Angle collision	0.085	0.517	0.100	0.195	0.072	0.297
Head-on collision	0.016	0.097	0.034	0.066	0.003	0.012
Rear-end collision	0.142	0.864	0.164	0.320	0.122	0.504
Sideswipe collision	0.037	0.225	0.038	0.074	0.038	0.157
Other multiple-vehicle collision	0.027	0.164	0.026	0.051	0.030	0.124
Total multiple-vehicle crashes	0.307	1.868	0.362	0.707	0.265	1.095

1445

Worksheet 1E – Summary Results or Rural Two-Lane Two-Way Roadway Segments

Worksheet 1E presents a summary of the results. Using the roadway segment length, the worksheet presents the crash rate in miles per year (Column 5).

Worksheet 1E – Summary Results for Rural Two-Lane Two-Way Roadway Segments

(1)	(2)	(3)	(4)	(5)
Crash severity level	**Crash Severity Distribution**	**Predicted average crash frequency (crashes/year)**	**Roadway segment length (mi)**	**Crash rate (crashes/mi/year)**
	(4) from Worksheet 1C	(8) from Worksheet 1C		(3)/(4)
Total	1.000	6.084	1.5	4.1
Fatal and Injury (FI)	0.321	1.954	1.5	1.3
Property Damage Only (PDO)	0.679	4.131	1.5	2.8

10.12.2. Sample Problem 2

The Site/Facility

A rural two-lane curved roadway segment.

The Question

What is the predicted average crash frequency of the roadway segment for a particular year?

The Facts

- 0.1-mi length
- Curved roadway segment
- 8,000 veh/day
- 1% grade
- 1,200-ft horizontal curve radius
- No spiral transition
- 0 driveways per mi
- 11-ft lane width
- 2-ft gravel shoulder
- Roadside hazard rating = 5
- 0.1-mi horizontal curve length
- 0.04 superelevation rate

Assumptions

- Collision type distributions have been adapted to local experience. The percentage of total crashes representing single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes is 78%.
- The calibration factor is assumed to be 1.10.
- Design speed = 60 mph
- Maximum superelevation rate, e_{max} = 6%

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the roadway segment in Sample Problem 2 is determined to be 0.5 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8

To determine the predicted average crash frequency of the roadway segment in Sample Problem 2, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year, and the EB Method is not applied.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type and traffic control features.

The SPF for a single roadway segment can be calculated from Equation 10-6 as follows:

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

$$= 8{,}000 \times 0.1 \times 365 \times 10^{-6} \times e^{(-0.312)}$$

$$= 0.214 \text{ crashes/year}$$

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust the estimated crash frequency for base conditions to the site specific geometric design and traffic control features.

Each AMF used in the calculation of the predicted average crash frequency of the roadway segment is calculated below:

Lane Width (AMF_{1r})

AMF_{1r} can be calculated from Equation 10-11 as follows:

$$AMF_{1r} = (AMF_{ra} - 1.0) \times p_{ra} + 1.0$$

For an 11-ft lane width and AADT of 8,000 veh/day, AMF_{ra} = 1.05 (see Exhibit 10-14)

The proportion of related crashes, p_{ra}, is 0.78 (see assumptions)

$$AMF_{1r} = (1.05 - 1.0) \times 0.78 + 1.0$$

$$= 1.04$$

Shoulder Width and Type (AMF_{2r})

AMF_{2r} can be calculated from Equation 10-12, using values from Exhibit 10-16, Exhibit 10-18 and local data (p_{ra} = 0.78) as follows:

$$AMF_{2r} = (AMF_{wra} \times AMF_{ra} - 1.0) \times p_{ra} + 1.0$$

For 2-ft shoulders and AADT of 8,000 veh/day, AMF_{wra} = 1.30 (see Exhibit 10-16)

For 2-ft gravel shoulders, AMF_{tra} = 1.01 (see Exhibit 10-18)

The proportion of related crashes, p_{ra}, is 0.78 (see assumptions)

$$AMF_{2r} = (1.30 \times 1.01 - 1.0) \times 0.78 + 1.0$$

$$= 1.24$$

Horizontal Curves: Length, Radius, and Presence or Absence of Spiral Transitions (AMF_{3r})

For a 0.1 mile horizontal curve with a 1,200 ft radius and no spiral transition, AMF_{3r} can be calculated from Equation 10-13 as follows:

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

$$= \frac{(1.55 \times 0.1) + (\frac{80.2}{1200}) - (0.012 \times 0)}{(1.55 \times 0.1)}$$

$$= 1.43$$

Horizontal Curves: Superelevation (AMF_{4r})

AMF_{4r} can be calculated from Equation 10-16 as follows:

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

For a roadway segment with an assumed design speed of 60 mph and an assumed maximum superelevation (e_{max}) of 6%, AASHTO *Green Book* provides for a 0.06 superelevation rate. Since the superelevation in Sample Problem 2 is 0.04, the superelevation variance is 0.02 (0.06 - 0.04).

$$AMF_{4r} = 1.06 + 3 \times (0.02 - 0.02)$$

$$= 1.06$$

Grade (AMF_{5r})

From Exhibit 10-19, for a 1% grade, AMF_{5r} = 1.00.

Driveway Density (AMF_{6r})

Since the driveway density, DD, in Sample Problem 2 is less than 5 driveways per mile, AMF_{6r} = 1.00 (i.e. the base condition for AMF_{6r} is five driveways per mile. If driveway density is less than five driveways per mile, AMF_{6r} is 1.00).

Centerline Rumble Strips (AMF_{7r})

Since there are no centerline rumble strips in Sample Problem 2, AMF_{7r} = 1.00 (i.e. the base condition for AMF_{7r} is no centerline rumble strips).

Passing Lanes (AMF_{8r})

Since there are no passing lanes in Sample Problem 2, AMF_{8r} = 1.00 (i.e. the base condition for AMF_{8r} is the absence of a passing lane).

Two-Way Left-Turn Lanes (AMF_{9r})

Since there are no two-way left-turn lanes in Sample Problem 2, AMF_{9r} = 1.00 (i.e. the base condition for AMF_{9r} is the absence of a two-way left-turn lane).

Roadside Design (AMF_{10r})

The roadside hazard rating, RHR, is 5. Therefore, AMF_{10r} can be calculated from Equation 10-20 as follows:

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

$$= \frac{e^{(-0.6869+0.0668\times 5)}}{e^{(-0.4865)}}$$

$$= 1.14$$

Lighting (AMF_{11r})

Since there is no lighting in Sample Problem 2, AMF_{11r} = 1.00 (i.e. the base condition for AMF_{11r} is the absence of roadway lighting).

Automated Speed Enforcement (AMF_{12r})

Since there is no automated speed enforcement in Sample Problem 2, AMF_{12r} = 1.00 (i.e. the base condition for AMF_{12r} is the absence of automated speed enforcement).

The combined AMF value for Sample Problem 2 is calculated below.

$$AMF_{COMB} = 1.04 \times 1.24 \times 1.43 \times 1.06 \times 1.14$$

$$= 2.23$$

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed that a calibration factor, C_r, of 1.10 has been determined for local conditions. See *Part C* Appendix A.1 for further discussion on calibration of the predictive models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated using Equation 10-2 based on the results obtained in Steps 9 through 11 as follows:

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

$$= 0.214 \times 1.10 \times (2.23)$$

$$= 0.525 \text{ crashes/year}$$

Worksheets

The step-by-step instructions above are provided to illustrate the predictive method for calculating the predicted average crash frequency for a roadway segment. To apply the predictive method steps to multiple segments, a series of five worksheets are provided for determining predicted average crash frequency. The five worksheets include:

- Worksheet 1A – General Information and Input Data for Rural Two-Lane Two-Way Roadway Segments
- Worksheet 1B – Accident Modification Factors for Rural Two-Lane Two-Way Roadway Segments
- Worksheet 1C – Roadway Segment Crashes for Rural Two-Lane Two-Way Roadway Segments

- Worksheet 1D – Crashes by Severity Level and Collision Type for Rural Two-Lane Two-Way Roadway Segments
- Worksheet 1E – Summary Results for Rural Two-Lane Two-Way Roadway Segments

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 10 Appendix A.

Worksheet 1A – General Information and Input Data for Rural Two-Lane Two-Way Roadway Segments

Worksheet 1A is a summary of general information about the roadway segment, analysis, input data (i.e., "The Facts") and assumptions for Sample Problem 2.

Worksheet 1A – General Information and Input Data for Rural Two-Lane Two-Way Roadway Segments

General Information		Location Information	
Analyst		Roadway	
Agency or Company		Roadway Section	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	Base Conditions	Site Conditions
Length of segment, L (mi)	-	0.1
AADT (veh/day)	-	8,000
Lane width (ft)	12	11
Shoulder width (ft)	6	2
Shoulder type	paved	gravel
Length of horizontal curve (mi)	0	0.1
Radius of curvature (ft)	0	1,200
Spiral transition curve (present/not present)	not present	not present
Superelevation variance (ft/ft)	<0.01	0.02 (0.06-0.04)
Grade (%)	0	1
Driveway density (driveways/mile)	5	0
Centerline rumble strips (present/not present)	not present	not present
Passing lanes (present/not present)	not present	not present
Two-way left-turn lane (present/not present)	not present	not present
Roadside hazard rating (1-7 scale)	3	5
Segment lighting (present/not present)	not present	not present
Auto speed enforcement (present/not present)	not present	not present
Calibration Factor, C_r	1.0	1.1

Worksheet 1B – Accident Modification Factors for Rural Two-Lane Two-Way Roadway Segments

In Step 10 of the predictive method, Accident Modification Factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 10.7 presents the tables and equations necessary for determining AMF values. Once the value for each AMF has been determined, all of the AMFs are multiplied together in Column 13 of Worksheet 1B which indicates the combined AMF value.

Worksheet 1B – Accident Modification Factors for Rural Two-Lane Two-Way Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
AMF for Lane Width	AMF for Shoulder Width and Type	AMF for Horizontal Curves	AMF for Superelevation	AMF for Grades	AMF for Driveway Density	AMF for Centerline Rumble Strips	AMF for Passing Lanes	AMF for Two-Way Left-Turn Lane	AMF for Roadside Design	AMF for Lighting	AMF for Automated Speed Enforcement	Combined AMF
AMF_{1r}	AMF_{2r}	AMF_{3r}	AMF_{4r}	AMF_{5r}	AMF_{6r}	AMF_{7r}	AMF_{8r}	AMF_{9r}	AMF_{10r}	AMF_{11r}	AMF_{12r}	AMF_{COMB}
from Equation 10-11	from Equation 10-12	from Equation 10-13	from Equations 10-14, 10-15, or 10-16	from Exhibit 10-19	from Equation 10-17	from Section 10.7.1	from Section 10.7.1	from Equation 10-18	from Equation 10-20	from Equation 10-21	from Section 10.7.1	(1)*(2)*… *(11)*(12)
1.04	1.24	1.43	1.06	1.00	1.00	1.00	1.00	1.00	1.14	1.00	1.00	2.23

Worksheet 1C – Roadway Segment Crashes for Rural Two-Lane Two-Way Roadway Segments

The SPF for the roadway segment in Sample Problem 2 is calculated using Equation 10-6 and entered into Column 2 of Worksheet 1C. The overdispersion parameter associated with the SPF can be entered into Column 3; however, the overdispersion parameter is not needed for Sample Problem 2. Column 4 of the worksheet presents the default proportions for crash severity levels from Exhibit 10-6 (as the EB Method is not utilized). These proportions may be used to separate the SPF (from Column 2) into components by crash severity level, as illustrated in Column 5. Column 6 represents the combined AMF (from Column 13 in Worksheet 1B), and Column 7 represents the calibration factor. Column 8 calculates the predicted average crash frequency using the values in Column 5, the combined AMF in Column 6, and the calibration factor in Column 7.

Worksheet 1C – Roadway Segment Crashes for Rural Two-Lane Two-Way Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crash Severity Level	**$N_{spf\ rs}$**	**Overdispersion Parameter, k**	**Crash Severity Distribution**	**$N_{spf\ rs}$ by Severity Distribution**	**Combined AMFs**	**Calibration Factor, C_r**	**Predicted average crash frequency, $N_{predicted\ rs}$**
	from Equation 10-6	from Equation 10-7	from Exhibit 10-6	$(2)_{TOTAL}$ * (4)	(13) from Worksheet 1B		(5)*(6)*(7)
Total	0.214	2.36	1.000	0.214	2.23	1.10	0.525
Fatal and Injury (FI)	-	-	0.321	0.069	2.23	1.10	0.169
Property Damage Only (PDO)	-	-	0.679	0.145	2.23	1.10	0.356

Worksheet 1D – Crashes by Severity Level and Collision for Rural Two-Lane Two-Way Roadway Segments

Worksheet 1D presents the default proportions for collision type (from Exhibit 10-6) by crash severity level as follows:

- Total crashes (Column 2)
- Fatal and injury crashes (Column 4)
- Property damage only crashes (Column 6)

Using the default proportions, the predicted average crash frequency by collision type is presented in Columns 3 (Total), 5 (Fatal and Injury, FI), and 7 (Property Damage Only, PDO).

These proportions may be used to separate the predicted average crash frequency (from Column 8, Worksheet 1C) by crash severity and collision type.

Worksheet 1D – Crashes by Severity Level and Collision Type for Rural Two-Lane Two-Way Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Collision Type	**Proportion of Collision Type $_{(TOTAL)}$**	**$N_{predicted\ rs\ (TOTAL)}$ (crashes/year)**	**Proportion of Collision Type $_{(FI)}$**	**$N_{predicted\ rs\ (FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**$N_{predicted\ rs\ (PDO)}$ (crashes/year)**
	from Exhibit 10-7	$(8)_{TOTAL}$ from Worksheet 1C	from Exhibit 10-7	$(8)_{FI}$ from Worksheet 1C	from Exhibit 10-7	$(8)_{PDO}$ from Worksheet 1C
Total	1.000	0.525	1.000	0.169	1.000	0.356
		$(2)*(3)_{TOTAL}$		$(4)*(5)_{FI}$		$(6)*(7)_{PDO}$
SINGLE-VEHICLE						
Collision with animal	0.121	0.064	0.038	0.006	0.184	0.066
Collision with bicycle	0.002	0.001	0.004	0.001	0.001	0.000
Collision with pedestrian	0.003	0.002	0.007	0.001	0.001	0.000
Overturned	0.025	0.013	0.037	0.006	0.015	0.005
Ran off road	0.521	0.274	0.545	0.092	0.505	0.180
Other single-vehicle collision	0.021	0.011	0.007	0.001	0.029	0.010
Total single-vehicle crashes	0.693	0.364	0.638	0.108	0.735	0.262
MULTIPLE-VEHICLE						
Angle collision	0.085	0.045	0.100	0.017	0.072	0.026
Head-on collision	0.016	0.008	0.034	0.006	0.003	0.001
Rear-end collision	0.142	0.075	0.164	0.028	0.122	0.043
Sideswipe collision	0.037	0.019	0.038	0.006	0.038	0.014
Other multiple-vehicle collision	0.027	0.014	0.026	0.004	0.030	0.011
Total multiple-vehicle crashes	0.307	0.161	0.362	0.061	0.265	0.094

1610

Worksheet 1E – Summary Results for Rural Two-Lane Two-Way Roadway Segments

Worksheet 1E presents a summary of the results. Using the roadway segment length, the worksheet presents the crash rate in miles per year (Column 5).

Worksheet 1E – Summary Results for Rural Two-Lane Two-Way Roadway Segments				
(1)	(2)	(3)	(4)	(5)
Crash severity level	**Crash Severity Distribution**	**Predicted average crash frequency (crashes/year)**	**Roadway segment length (mi)**	**Crash rate (crashes/mi/year)**
	(4) from Worksheet 1C	(8) from Worksheet 1C		(3)/(4)
Total	1.000	0.525	0.1	5.3
Fatal and Injury (FI)	0.321	0.169	0.1	1.7
Property Damage Only (PDO)	0.679	0.356	0.1	3.6

10.12.3. Sample Problem 3

The Site/Facility

A three-leg stop-controlled intersection located on a rural two-lane roadway.

The Question

What is the predicted average crash frequency of the stop-controlled intersection for a particular year?

The Facts

- 3 legs
- Minor-road stop control
- No right-turn lanes on major road
- No left-turn lanes on major road
- 30-degree skew angle
- AADT of major road = 8,000 veh/day
- AADT of minor road = 1,000 veh/day
- Intersection lighting is present

Assumptions

- Collision type distributions used are the default values from Exhibit 10-12.
- The proportion of crashes that occur at night are not known, so the default proportion for nighttime crashes is assumed.
- The calibration factor is assumed to be 1.50.

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the intersection in Sample Problem 3 is determined to be 2.9 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8

To determine the predicted average crash frequency of the intersection in Sample Problem 3, only Steps 9 through 11 are conducted. No other steps are necessary because only one intersection is analyzed for one year, and the EB Method is not applied.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type and traffic control features.

The SPF for a single three-leg stop-controlled intersection can be calculated from Equation 10-8 as follows:

$$N_{spf\ 3ST} = exp[-9.86 + 0.79 \times ln(AADT_{maj}) + 0.49 \times ln(AADT_{min})]$$

$$= exp[-9.86 + 0.79 \times ln(8{,}000) + 0.49 \times ln(1{,}000)]$$

$$= 1.867 \text{ crashes/year}$$

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust the estimated crash frequency for base conditions to the site specific geometric design and traffic control features.

Each AMF used in the calculation of the predicted average crash frequency of the intersection is calculated below:

Intersection Skew Angle (AMF_{1i})

AMF_{1i} can be calculated from Equation 10-22 as follows:

$$AMF_{1i} = e^{(0.004 \times SKEW)}$$

The intersection skew angle for Sample Problem 3 is 30 degrees.

$$AMF_{1i} = e^{(0.004 \times 30)}$$

$$= 1.13$$

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

Since no left-turn lanes are present in Sample Problem 3, AMF_{2i} = 1.00 (i.e. the base condition for AMF_{2i} is the absence of left-turn lanes on the intersection approaches).

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

Since no right-turn lanes are present, AMF_{3i} = 1.00 (i.e. the base condition for AMF_{3i} is the absence of right-turn lanes on the intersection approaches).

Lighting (AMF_{4i})

AMF_{4i} can be calculated from Equation 10-24 using Exhibit 10-23.

$$AMF_{4i} = 1 - 0.38 \times p_{ni}$$

From Exhibit 10-23, for a three-leg stop-controlled intersection, the proportion of total accidents that occur at night (see assumption), p_{ni}, is 0.26.

$$AMF_{4i} = 1 - 0.38 \times 0.26$$

$$= 0.90$$

The combined AMF value for Sample Problem 3 is calculated below.

$$AMF_{COMB} = 1.13 \times 0.90$$

$$= 1.02$$

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed that a calibration factor, C_i, of 1.50 has been determined for local conditions. See *Part C* Appendix A.1 for further discussion on calibration of the predictive models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated using Equation 10-3 based on the results obtained in Steps 9 through 11 as follows:

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

$$= 1.867 \times 1.50 \times (1.02)$$

$$= 2.857 \text{ crashes/year}$$

Worksheets

The step-by-step instructions above are the predictive method for calculating the predicted average crash frequency for an intersection. To apply the predictive method steps to multiple intersections, a series of five worksheets are provided for determining predicted average crash frequency. The five worksheets include:

- Worksheet 2A – General Information and Input Data for Rural Two-Lane Two-Way Road Intersections
- Worksheet 2B – Accident Modification Factors for Rural Two-Lane Two-Way Road Intersections
- Worksheet 2C – Intersection Crashes for Rural Two-Lane Two-Way Road Intersections
- Worksheet 2D – Crashes by Severity Level and Collision Type for Rural Two-Lane Two-Way Road Intersections
- Worksheet 2E – Summary Results for Rural Two-Lane Two-Way Road Intersections

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 10 Appendix A.

Worksheet 2A – General Information and Input Data for Rural Two-Lane Two-Way Road Intersections

Worksheet 2A is a summary of general information about the intersection, analysis, input data (i.e., "The Facts") and assumptions for Sample Problem 3.

Worksheet 2A – General Information and Input Data for Rural Two-Lane Two-Way Road Intersections

General Information		**Location Information**	
Analyst		Roadway	
Agency or Company		Intersection	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	**Base Conditions**	**Site Conditions**
Intersection type (3ST, 4ST, 4SG)	-	3ST
$AADT_{major}$ (veh/day)	-	8,000
$AADT_{minor}$ (veh/day)	-	1,000
Intersection skew angle (degrees)	0	30
Number of signalized or uncontrolled approaches with a left turn lane (0,1,2,3,4)	0	0
Number of signalized or uncontrolled approaches with a right turn lane (0,1,2,3,4)	0	0
Intersection lighting (present/not present)	not present	present
Calibration Factor, C_i	1.0	1.50

Worksheet 2B – Accident Modification Factors for Rural Two-Lane Two-Way Road Intersections

In Step 10 of the predictive method, Accident Modification Factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 10.7 presents the tables and equations necessary for determining AMF values. Once the value for each AMF has been determined, all of the AMFs are multiplied together in Column 5 of Worksheet 2B which indicates the combined AMF value.

Worksheet 2B – Accident Modification Factors for Rural Two-Lane Two-Way Road Intersections

(1)	(2)	(3)	(4)	(5)
AMF for Intersection Skew Angle	AMF for Left-Turn Lanes	AMF for Right-Turn Lanes	AMF for Lighting	Combined AMF
AMF_{1i}	AMF_{2i}	AMF_{3i}	AMF_{4i}	AMF_{COMB}
from Equations 10-22 or 10-23	from Exhibit 10-21	from Exhibit 10-22	from Equation 10-24	(1)*(2)*(3)*(4)
1.13	1.00	1.00	0.90	1.02

Worksheet 2C – Intersection Crashes for Rural Two-Lane Two-Way Road Intersections

The SPF for the intersection in Sample Problem 3 is calculated using Equation 10-8 and entered into Column 2 of Worksheet 2C. The overdispersion parameter associated with the SPF can be entered into Column 3; however, the overdispersion parameter is not needed for Sample Problem 3 (as the EB Method is not utilized). Column 4 of the worksheet presents the default proportions for crash severity levels from Exhibit 10-11. These proportions may be used to separate the SPF (from Column 2) into components by crash severity level, as illustrated in Column 5. Column 6 represents the combined AMF (from Column 13 in Worksheet 2B), and Column 7 represents the calibration factor. Column 8 calculates the predicted average crash frequency using the values in Column 5, the combined AMF in Column 6, and the calibration factor in Column 7.

Worksheet 2C – Intersection Crashes for Rural Two-Lane Two-Way Road Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crash Severity Level	**$N_{spf\ 3ST,4ST\ or\ 4SG}$**	**Overdispersion Parameter, k**	**Crash Severity Distribution**	**$N_{spf\ 3ST,4ST\ or\ 4SG}$ by Severity Distribution**	**Combined AMFs**	**Calibration Factor, C_i**	**Predicted average crash frequency, $N_{predicted\ int}$**
	from Equations 10-8, 10-9, or 10-10	from Section 10.6.2	from Exhibit 10-11	$(2)_{TOTAL}$ * (4)	from (5) of Worksheet 2B		(5)*(6)*(7)
Total	1.867	0.54	1.000	1.867	1.02	1.50	2.857
Fatal and Injury (FI)	-	-	0.415	0.775	1.02	1.50	1.186
Property Damage Only (PDO)	-	-	0.585	1.092	1.02	1.50	1.671

Worksheet 2D – Crashes by Severity Level and Collision for Rural Two-Lane Two-Way Road Intersections

Worksheet 2D presents the default proportions for collision type (from Exhibit 10-12) by crash severity level as follows:

- Total crashes (Column 2)
- Fatal and injury crashes (Column 4)
- Property damage only crashes (Column 6)

Using the default proportions, the predicted average crash frequency by collision type is presented in Columns 3 (Total), 5 (Fatal and Injury, FI), and 7 (Property Damage Only, PDO).

These proportions may be used to separate the predicted average crash frequency (from Column 8, Worksheet 2C) by crash severity and collision type.

Worksheet 2D – Crashes by Severity Level and Collision Type for Rural Two-Lane Two-Way Road Intersections						
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Collision Type	**Proportion of Collision Type (TOTAL)**	**$N_{predicted\ int\ (TOTAL)}$ (crashes/year)**	**Proportion of Collision Type (FI)**	**$N_{predicted\ int\ (FI)}$ (crashes/year)**	**Proportion of Collision Type (PDO)**	**$N_{predicted\ int\ (PDO)}$ (crashes/year)**
	from Exhibit 10-12	$(8)_{TOTAL}$ from Worksheet 2C	from Exhibit 10-12	$(8)_{FI}$ from Worksheet 2C	from Exhibit 10-12	$(8)_{PDO}$ from Worksheet 2C
Total	1.000	2.857	1.000	1.186	1.000	1.671
		$(2)*(3)_{TOTAL}$		$(4)*(5)_{FI}$		$(6)*(7)_{PDO}$
SINGLE-VEHICLE						
Collision with animal	0.019	0.054	0.008	0.009	0.026	0.043
Collision with bicycle	0.001	0.003	0.001	0.001	0.001	0.002
Collision with pedestrian	0.001	0.003	0.001	0.001	0.001	0.002
Overturned	0.013	0.037	0.022	0.026	0.007	0.012
Ran off road	0.244	0.697	0.240	0.285	0.247	0.413
Other single-vehicle collision	0.016	0.046	0.011	0.013	0.020	0.033
Total single-vehicle crashes	0.294	0.840	0.283	0.336	0.302	0.505
MULTIPLE-VEHICLE						
Angle collision	0.237	0.677	0.275	0.326	0.210	0.351
Head-on collision	0.052	0.149	0.081	0.096	0.032	0.053
Rear-end collision	0.278	0.794	0.260	0.308	0.292	0.488
Sideswipe collision	0.097	0.277	0.051	0.060	0.131	0.219
Other multiple-vehicle collision	0.042	0.120	0.050	0.059	0.033	0.055
Total multiple-vehicle crashes	0.706	2.017	0.717	0.850	0.698	1.166

Worksheet 2E –Summary Results for Rural Two-Lane Two-Way Road Intersections

Worksheet 2E presents a summary of the results.

Worksheet 2E – Summary Results for Rural Two-Lane Two-Way Road Intersections

(1)	(2)	(3)
Crash severity level	**Crash Severity Distribution**	**Predicted average crash frequency (crashes/year)**
	(4) from Worksheet 2C	(8) from Worksheet 2C
Total	1.000	2.857
Fatal and Injury (FI)	0.415	1.186
Property Damage Only (PDO)	0.585	1.671

10.12.4. Sample Problem 4

A four-leg signalized intersection located on a rural two-lane roadway.

The Question

What is the predicted average crash frequency of the signalized intersection for a particular year?

The Facts

- 4 legs
- 1 right-turn lane on one approach
- Signalized intersection
- 90-degree intersection angle
- No lighting present
- AADT of major road = 10,000 veh/day
- AADT of minor road = 2,000 veh/day
- 1 left-turn lane on each of two approaches

Assumptions

- Collision type distributions used are the default values from Exhibit 10-12.
- The calibration factor is assumed to be 1.30.

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the intersection in Sample Problem 4 is determined to be 5.7 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8

To determine the predicted average crash frequency of the intersection in Sample Problem 4, only Steps 9 through 11 are conducted. No other steps are necessary because only one intersection is analyzed for one year, and the EB Method is not applied.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type and traffic control features.

The SPF for a signalized intersection can be calculated from Equation 10-10 as follows:

$$N_{spf\,4SG} = exp[-5.13 + 0.60 \times ln(AADT_{maj}) + 0.20 \times ln(AADT_{min})]$$
$$= exp[-5.13 + 0.60 \times ln(10{,}000) + 0.20 \times ln(2{,}000)]$$
$$= 6.796 \text{ crashes/year}$$

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust the estimated crash frequency for base conditions to the site specific geometric design and traffic control features.

Each AMF used in the calculation of the predicted average crash frequency of the intersection is calculated below:

Intersection Skew Angle (AMF_{1i})

The AMF for skew angle at four-leg signalized intersections is 1.00 for all cases.

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

From Exhibit 10-21 for a signalized intersection with left-turn lanes on two approaches, AMF_{2i} = 0.67.

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

From Exhibit 10-22 for a signalized intersection with a right-turn lane on one approach, AMF_{3i} = 0.96.

Lighting (AMF_{4i})

Since there is no intersection lighting present in Sample Problem 4, AMF_{4i} = 1.00 (i.e. the base condition for AMF_{4i} is the absence of intersection lighting).

The combined AMF value for Sample Problem 4 is calculated below.

$$AMF_{COMB} = 0.67 \times 0.96$$
$$= 0.64$$

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed that a calibration factor, C_i, of 1.30 has been determined for local conditions. See *Part C* Appendix A.1 for further discussion on calibration of the predictive models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated using the results obtained in Steps 9 through 11 as follows:

$$N_{predicted\ int} = N_{spf\ int} \times C_i \times (AMF_{1i} \times AMF_{2i} \times \ldots \times AMF_{4i})$$
$$= 6.796 \times 1.30 \times (0.64)$$
$$= 5.654 \text{ crashes/year}$$

Worksheets

The step-by-step instructions above are the predictive method for calculating the predicted average crash frequency for an intersection. To apply the predictive method steps to multiple intersections, a series of five worksheets are provided for determining predicted average crash frequency. The five worksheets include:

- Worksheet 2A – General Information and Input Data for Rural Two-Lane Two-Way Road Intersections
- Worksheet 2B – Accident Modification Factors for Rural Two-Lane Two-Way Road Intersections
- Worksheet 2C – Intersection Crashes for Rural Two-Lane Two-Way Road Intersections
- Worksheet 2D – Crashes by Severity Level and Collision for Rural Two-Lane Two-Way Road Intersections
- Worksheet 2E – Summary Results for Rural Two-Lane Two-Way Road Intersections

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 10 Appendix A.

Worksheet 2A – General Information and Input Data for Rural Two-Lane Two-Way Road Intersections

Worksheet 2A is a summary of general information about the intersection, analysis, input data (i.e., "The Facts") and assumptions for Sample Problem 4.

<table>
<tr><th colspan="4">Worksheet 2A – General Information and Input Data for Rural Two-Lane Two-Way Road Intersections</th></tr>
<tr><th colspan="2">General Information</th><th colspan="2">Location Information</th></tr>
<tr><td>Analyst</td><td></td><td>Roadway</td><td></td></tr>
<tr><td>Agency or Company</td><td></td><td>Intersection</td><td></td></tr>
<tr><td>Date Performed</td><td></td><td>Jurisdiction</td><td></td></tr>
<tr><td></td><td></td><td>Analysis Year</td><td></td></tr>
<tr><th colspan="2">Input Data</th><th>Base Conditions</th><th>Site Conditions</th></tr>
<tr><td colspan="2">Intersection type (3ST, 4ST, 4SG)</td><td>-</td><td>4SG</td></tr>
<tr><td colspan="2">$AADT_{major}$ (veh/day)</td><td>-</td><td>10,000</td></tr>
<tr><td colspan="2">$AADT_{minor}$ (veh/day)</td><td>-</td><td>2,000</td></tr>
<tr><td colspan="2">Intersection skew angle (degrees)</td><td>0</td><td>0</td></tr>
<tr><td colspan="2">Number of signalized or uncontrolled approaches with a left turn lane (0,1,2,3,4)</td><td>0</td><td>2</td></tr>
<tr><td colspan="2">Number of signalized or uncontrolled approaches with a right turn lane (0,1,2,3,4)</td><td>0</td><td>1</td></tr>
<tr><td colspan="2">Intersection lighting (present/not present)</td><td>not present</td><td>not present</td></tr>
<tr><td colspan="2">Calibration Factor, C_i</td><td>1.0</td><td>1.3</td></tr>
</table>

Worksheet 2B – Accident Modification Factors for Rural Two-Lane Two-Way Road Intersections

In Step 10 of the predictive method, Accident Modification Factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 10.7 presents the tables and equations necessary for determining AMF values. Once the value for each AMF has been determined, all of the AMFs are multiplied together in Column 5 of Worksheet 2B which indicates the combined AMF value.

Worksheet 2B – Accident Modification Factors for Rural Two-Lane Two-Way Road Intersections

(1)	(2)	(3)	(4)	(5)
AMF for Intersection Skew Angle	AMF for Left-Turn Lanes	AMF for Right-Turn Lanes	AMF for Lighting	Combined AMF
AMF_{1i}	AMF_{2i}	AMF_{3i}	AMF_{4i}	AMF_{COMB}
from Equations 10-22 or10-23	from Exhibit 10-21	from Exhibit 10-22	from Equation 10-24	(1)*(2)*(3)*(4)
1.00	0.67	0.96	1.00	0.64

Worksheet 2C – Intersection Crashes for Rural Two-Lane Two-Way Road Intersections

The SPF the intersection in Sample Problem 4 is calculated using Equation 10-8 and entered into Column 2 of Worksheet 2C. The overdispersion parameter associated with the SPF can be entered into Column 3; however, the overdispersion parameter is not needed for Sample Problem 4 (as the EB Method is not utilized). Column 4 of the worksheet presents the default proportions for crash severity levels from Exhibit 10-11. These proportions may be used to separate the SPF (from Column 2) into components by crash severity level, as illustrated in Column 5. Column 6 represents the combined AMF (from Column 13 in Worksheet 2B), and Column 7 represents the calibration factor. Column 8 calculates the predicted average crash frequency using the values in Column 5, the combined AMF in Column 6, and the calibration factor in Column 7.

Worksheet 2C – Intersection Crashes for Rural Two-Lane Two-Way Road Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crash Severity Level	**$N_{spf\ 3ST,4ST\ or\ 4SG}$**	**Overdispersion Parameter, k**	**Crash Severity Distribution**	**$N_{spf\ 3ST,4ST\ or\ 4SG}$ by Severity Distribution**	**Combined AMFs**	**Calibration Factor, C_i**	**Predicted average crash frequency, $N_{predicted\ int}$**
	from Equations 10-8, 10-9, or 10-10	from Section 10.6.2	from Exhibit 10-11	$(2)_{TOTAL}$ * (4)	from (5) of Worksheet 2B		(5)*(6)*(7)
Total	6.796	0.11	1.000	6.796	0.64	1.30	5.654
Fatal and Injury (FI)	-	-	0.340	2.311	0.64	1.30	1.923
Property Damage Only (PDO)	-	-	0.660	4.485	0.64	1.30	3.732

Worksheet 2D – Crashes by Severity Level and Collision for Rural Two-Lane Two-Way Road Intersections

Worksheet 2D presents the default proportions for collision type (from Exhibit 10-12) by crash severity level as follows:

- Total crashes (Column 2)
- Fatal and injury crashes (Column 4)
- Property damage only crashes (Column 6)

Using the default proportions, the predicted average crash frequency by collision type is presented in Columns 3 (Total), 5 (Fatal and Injury, FI), and 7 (Property Damage Only, PDO).

These proportions may be used to separate the predicted average crash frequency (from Column 8, Worksheet 2C) by crash severity and collision type.

Worksheet 2D – Crashes by Severity Level and Collision Type for Rural Two-Lane Two-Way Road Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Collision Type	**Proportion of Collision Type (TOTAL)**	**$N_{predicted\ int\ (TOTAL)}$ (crashes/year)**	**Proportion of Collision Type (FI)**	**$N_{predicted\ int\ (FI)}$ (crashes/year)**	**Proportion of Collision Type (PDO)**	**$N_{predicted\ int\ (PDO)}$ (crashes/year)**
	from Exhibit 10-12	$(8)_{TOTAL}$ from Worksheet 2C	from Exhibit 10-12	$(8)_{FI}$ from Worksheet 2C	from Exhibit 10-12	$(8)_{PDO}$ from Worksheet 2C
Total	1.000	5.654	1.000	1.923	1.000	3.732
		$(2)*(3)_{TOTAL}$		$(4)*(5)_{FI}$		$(6)*(7)_{PDO}$
SINGLE-VEHICLE						
Collision with animal	0.002	0.011	0.000	0.000	0.003	0.011
Collision with bicycle	0.001	0.006	0.001	0.002	0.001	0.004
Collision with pedestrian	0.001	0.006	0.001	0.002	0.001	0.004
Overturned	0.003	0.017	0.003	0.006	0.003	0.011
Ran off road	0.064	0.362	0.032	0.062	0.081	0.302
Other single-vehicle collision	0.005	0.028	0.003	0.006	0.018	0.067
Total single-vehicle crashes	0.076	0.430	0.040	0.077	0.107	0.399
MULTIPLE-VEHICLE						
Angle collision	0.274	1.549	0.336	0.646	0.242	0.903
Head-on collision	0.054	0.305	0.080	0.154	0.040	0.149
Rear-end collision	0.426	2.409	0.403	0.775	0.438	1.635
Sideswipe collision	0.118	0.667	0.051	0.098	0.153	0.571
Other multiple-vehicle collision	0.052	0.294	0.090	0.173	0.020	0.075
Total multiple-vehicle crashes	0.924	5.224	0.960	1.846	0.893	3.333

Worksheet 2E –Summary Results for Rural Two-Lane Two-Way Road Intersections

Worksheet 2E presents a summary of the results.

Worksheet 2E – Summary Results for Rural Two-Lane Two-Way Road Intersections		
(1)	(2)	(3)
Crash severity level	**Crash Severity Distribution**	**Predicted average crash frequency (crashes/year)**
	(4) from Worksheet 2C	(8) from Worksheet 2C
Total	1.000	5.654
Fatal and injury (FI)	0.340	1.923
Property Damage Only (PDO)	0.660	3.732

10.12.5. Sample Problem 5

The Project

A project of interest consists of three sites: a rural two-lane tangent segment; a rural two-lane curved segment; and a three-leg intersection with minor-road stop control. (This project is a compilation of roadway segments and intersections from Sample Problems 1, 2 and 3.)

The Question

What is the expected average crash frequency of the project for a particular year incorporating both the predicted average crash frequencies from Sample Problems 1, 2 and 3 and the observed crash frequencies using the **site-specific EB Method**?

The Facts

- 2 roadway segments (2U tangent segment, 2U curved segment)
- 1 intersection (3ST intersection)
- 15 observed crashes (2U tangent segment: 10 crashes; 2U curved segment: 2 crashes; 3ST intersection: 3 crashes)

Outline of Solution

To calculate the expected average crash frequency, site-specific observed crash frequencies are combined with predicted average crash frequencies for the project using the site-specific EB Method (i.e. observed crashes are assigned to specific intersections or roadway segments) presented in Section A.2.4 of *Part C* Appendix.

Results

The expected average crash frequency for the project is 12.3 crashes per year (rounded to one decimal place).

Worksheets

To apply the site-specific EB Method to multiple roadway segments and intersections on a rural two-lane two-way road combined, two worksheets are provided for determining the expected average crash frequency. The two worksheets include:

- Worksheet 3A – Predicted and Observed Crashes by Severity and Site Type Using the Site-Specific EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways
- Worksheet 3B – Site-Specific EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 10 Appendix A.

Worksheets 3A – Predicted and Observed Crashes by Severity and Site Type Using the Site-Specific EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways

The predicted average crash frequencies by severity type determined in Sample Problems 1 through 3 are entered into Columns 2 through 4 of Worksheet 3A. Column 5 presents the observed crash frequencies by site type, and Column 6 presents the overdispersion parameters. The expected average crash frequency is calculated by applying the site-specific EB Method which considers both the predicted model estimate and observed crash frequencies for each roadway segment and intersection. Equation A-5 from *Part C* Appendix is used to calculate the weighted adjustment and entered into Column 7. The expected average crash frequency is calculated using Equation A-4 and entered into Column 8. Detailed calculation of Columns 7 and 8 are provided below.

Worksheet 3A – Predicted and Observed Crashes by Severity and Site Type Using the Site-Specific EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Site type	**Predicted average crash frequency (crashes/year)**			**Observed crashes, $N_{observed}$ (crashes/year)**	**Overdispersion parameter, k**	**Weighted adjustment, w**	**Expected average crash frequency, $N_{expected}$**
	$N_{predicted\ (TOTAL)}$	$N_{predicted\ (FI)}$	$N_{predicted\ (PDO)}$			Equation A-5 from Part C Appendix	Equation A-4 from Part C Appendix
ROADWAY SEGMENTS							
Segment 1	6.084	1.954	4.131	10	0.16	0.507	8.015
Segment 2	0.525	0.169	0.356	2	2.36	0.447	1.341
INTERSECTIONS							
Intersection 1	2.857	1.186	1.671	3	0.54	0.393	2.944
COMBINED (sum of column)	9.466	3.309	6.158	15	-	-	12.300

Column 7 - Weighted Adjustment

The weighted adjustment, w, to be placed on the predictive model estimate is calculated using Equation A-5 from *Part C* Appendix as follows:

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

Segment 1 $$w = \frac{1}{1 + 0.16 \times (6.084)}$$

$$= 0.507$$

Segment 2 $$w = \frac{1}{1 + 2.36 \times (0.525)}$$

$$= 0.447$$

Intersection 1 $$w = \frac{1}{1 + 0.54 \times (2.857)}$$

$$= 0.393$$

Column 8 - Expected Average Crash Frequency

The estimate of expected average crash frequency, $N_{expected}$, is calculated using Equation A-4 from *Part C* Appendix as follows:

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

Segment 1 $$N_{expected} = 0.507 \times 6.084 + (1 - 0.507) \times 10$$

$$= 8.015$$

Segment 2 $$N_{expected} = 0.447 \times 0.525 + (1 - 0.447) \times 2$$

$$= 1.341$$

Intersection 1 $$N_{expected} = 0.393 \times 2.857 + (1 - 0.393) \times 3$$

$$= 2.944$$

Worksheet 3B – Site-Specific EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

Worksheet 3B presents a summary of the results. The expected average crash frequency by severity level is calculated by applying the proportion of predicted average crash frequency by severity level to the total expected average crash frequency (Column 3).

Worksheet 3B – Site-Specific EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

(1)	(2)	(3)
Crash severity level	**$N_{predicted}$**	**$N_{expected}$**
Total	$(2)_{COMB}$ from Worksheet 3A	$(8)_{COMB}$ from Worksheet 3A
	9.466	12.3
Fatal and injury (FI)	$(3)_{COMB}$ from Worksheet 3A	$(3)_{TOTAL}*(2)_{FI}/(2)_{TOTAL}$
	3.309	4.3
Property damage only (PDO)	$(4)_{COMB}$ from Worksheet 3A	$(3)_{TOTAL}*(2)_{PDO}/(2)_{TOTAL}$
	6.158	8.0

10.12.6. Sample Problem 6

The Project

A project of interest consists of three sites: a rural two-lane tangent segment; a rural two-lane curved segment; and a three-leg intersection with minor-road stop control. (This project is a compilation of roadway segments and intersections from Sample Problems 1, 2 and 3.)

The Question

What is the expected average crash frequency of the project for a particular year incorporating both the predicted average crash frequencies from Sample Problems 1, 2 and 3 and the observed crash frequencies using the **project-level EB Method**?

The Facts

- 2 roadway segments (2U tangent segment, 2U curved segment)
- 1 intersection (3ST intersection)
- 15 observed crashes (but no information is available to attribute specific crashes to specific sites within the project)

Outline of Solution

Observed crash frequencies for the project as a whole are combined with predicted average crash frequencies for the project as a whole using the project-level EB Method (i.e. observed crash data for individual roadway segments and intersections are not available, but observed crashes are assigned to a facility as a whole) presented in Section A.2.5 of *Part C* Appendix.

Results

The expected average crash frequency for the project is 11.7 crashes per year (rounded to one decimal place).

Worksheets

To apply the project-level EB Method to multiple roadway segments and intersections on a rural two-lane two-way road combined, two worksheets are provided for determining the expected average crash frequency. The two worksheets include:

- Worksheet 4A – Predicted and Observed Crashes by Severity and Site Type Using the Project-Level EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways
- Worksheet 4B – Project-Level EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 10 Appendix A.

Worksheets 4A – Predicted and Observed Crashes by Severity and Site Type Using the Project-Level EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways

The predicted average crash frequencies by severity type determined in Sample Problems 1 through 3 are entered in Columns 2 through 4 of Worksheet 4A. Column 5 presents the total observed crash frequencies combined for all sites, and Column 6 presents the overdispersion parameters. The expected average crash frequency is calculated by applying the project-level EB Method which considers both the predicted model estimate for each roadway segment and intersection and the project observed crashes. Column 7 calculates N_{w0} and Column 8 N_{w1}. Equations A-10 through A-14 from *Part C* Appendix are used to calculate the expected average crash frequency of combined sites. The results obtained from each equation are presented in Columns 9 through 14. Section A.2.5 in *Part C* Appendix defines all the variables used in this worksheet. Detailed calculations of Columns 9 through 13 are provided below.

Worksheet 4A – Predicted and Observed Crashes by Severity and Site Type Using the Project-Level EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Site type	**Predicted average crash frequency (crashes/year)**			**Observed crashes, $N_{observed}$ (crashes/year)**	**Overdispersion Parameter, k**	**$N_{predicted\ w0}$**	**$N_{predicted\ w1}$**	**w_0**	**N_0**	**w_1**	**N_1**	**$N_{expected/comb}$**
	$N_{predicted}$ (TOTAL)	$N_{predicted}$ (FI)	$N_{predicted}$ (PDO)			Equation A-8 (6)* (2)^2	Equation A-9 sqrt((6)*(2))	Equation A-10	Equation A-11	Equation A-12	Equation A-13	Equation A-14
ROADWAY SEGMENTS												
Segment 1	6.084	1.954	4.131	-	0.16	5.922	0.987	-	-	-	-	-
Segment 2	0.525	0.169	0.356	-	2.36	0.651	1.113	-	-	-	-	-
INTERSECTIONS												
Intersection 1	2.857	1.186	1.671	-	0.54	4.408	1.242	-	-	-	-	-
COMBINED (sum of column)	9.466	3.309	6.158	15	-	10.981	3.342	0.463	12.438	0.739	10.910	11.674

NOTE: $N_{predicted\ w0}$ = Predicted number of total accidents assuming that accidents frequencies are statistically independent

$$N_{predicted\ w0} = \sum_{j=1}^{5} k_{rmj} N_{rmj}^2 + \sum_{j=1}^{5} k_{rsj} N_{rsj}^2 + \sum_{j=1}^{5} k_{rdj} N_{rdj}^2 + \sum_{j=1}^{4} k_{imj} N_{imj}^2 + \sum_{j=1}^{4} k_{isj} N_{isj}^2 \quad \text{(A-8)}$$

$N_{predicted\ w1}$ = Predicted number of total accidents assuming that accidents frequencies are perfectly correlated

$$N_{predicted\ w1} = \sum_{j=1}^{5} \sqrt{k_{rmj} N_{rmj}} + \sum_{j=1}^{5} \sqrt{k_{rsj} N_{rsj}} + \sum_{j=1}^{5} \sqrt{k_{rdj} N_{rdj}} + \sum_{j=1}^{4} \sqrt{k_{imj} N_{imj}} + \sum_{j=1}^{4} \sqrt{k_{isj} N_{isj}} \quad \text{(A-9)}$$

Column 9 – w_0

The weight placed on predicted crash frequency under the assumption that accidents frequencies for different roadway elements are statistically independent, w_0, is calculated using Equation A-10 from *Part C* Appendix as follows:

$$w_0 = \frac{1}{1 + \frac{N_{predicted\ w0}}{N_{predicted\ (TOTAL)}}}$$

$$= \frac{1}{1 + \frac{10.981}{9.466}}$$

$$= 0.463$$

Column 10 – N_0

The expected crash frequency based on the assumption that different roadway elements are statistically independent, N_0, is calculated using Equation A-11 from *Part C* Appendix as follows:

$$N_0 = w_0\ N_{predicted\ (TOTAL)} + (1 - w_0)\ N_{observed\ (TOTAL)}$$

$$= 0.463 \times 9.466 + (1 - 0.463) \times 15$$

$$= 12.438$$

Column 11 – w_1

The weight placed on predicted crash frequency under the assumption that accidents frequencies for different roadway elements are perfectly correlated, w_1, is calculated using Equation A-12 from *Part C* Appendix as follows:

$$w_1 = \frac{1}{1 + \frac{N_{predicted\ w1}}{N_{predicted\ (TOTAL)}}}$$

$$= \frac{1}{1 + \frac{3.342}{9.466}}$$

$$= 0.739$$

Column 12 – N_1

The expected crash frequency based on the assumption that different roadway elements are perfectly correlated, N_1, is calculated using Equation A-13 from *Part C* Appendix as follows:

$$N_1 = w_1\ N_{predicted\ (TOTAL)} + (1 - w_1)\ N_{observed\ (TOTAL)}$$

$$= 0.739 \times 9.466 + (1 - 0.739) \times 15$$

$$= 10.910$$

Column 13 – $N_{expected/comb}$

The expected average crash frequency based of combined sites, $N_{p/comb}$, is calculated using Equation A-14 from *Part C* Appendix as follows:

$$N_{expected/comb} = \frac{N_0 + N_1}{2}$$

$$= \frac{12.438 + 10.910}{2}$$

$$= 11.674$$

Worksheet 4B – Project-Level EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

Worksheet 4B presents a summary of the results. The expected average crash frequency by severity level is calculated by applying the proportion of predicted average crash frequency by severity level to the total expected average crash frequency (Column 3).

Worksheet 4B – Project-Level EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

(1)	(2)	(3)
Crash severity level	$N_{predicted}$	$N_{expected/comb}$
Total	$(2)_{COMB}$ from Worksheet 4A	$(13)_{COMB}$ from Worksheet 4A
	9.466	11.7
Fatal and injury (FI)	$(3)_{COMB}$ from Worksheet 4A	$(3)_{TOTAL}*(2)_{FI}/(2)_{TOTAL}$
	3.309	4.1
Property damage only (PDO)	$(4)_{COMB}$ from Worksheet 4A	$(3)_{TOTAL}*(2)_{PDO}/(2)_{TOTAL}$
	6.158	7.6

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A.1 Appendix A – Worksheets for Predictive Method for Rural Two-Lane Two-Way Roads

Worksheet 1A – General Information and Input Data for Rural Two-Lane Two-Way Roadway Segments			
General Information		**Location Information**	
Analyst		Roadway	
Agency or Company		Roadway Section	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	**Base Conditions**	**Site Conditions**
Length of segment, L (mi)	-	
AADT (veh/day)	-	
Lane width (ft)	12	
Shoulder width (ft)	6	
Shoulder type	paved	
Length of horizontal curve (mi)	0	
Radius of curvature (ft)	0	
Spiral transition curve (present/not present)	not present	
Superelevation variance (ft/ft)	<0.01	
Grade (%)	0	
Driveway density (driveways/mile)	5	
Centerline rumble strips (present/not present)	not present	
Passing lanes (present/not present)	not present	
Two-way left-turn lane (present/not present)	not present	
Roadside hazard rating (1-7 scale)	3	
Segment lighting (present/not present)	not present	
Auto speed enforcement (present/not present)	not present	
Calibration Factor, C_r	1.0	

Worksheet 1B – Accident Modification Factors for Rural Two-Lane Two-Way Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
AMF for Lane Width	AMF for Shoulder Width and Type	AMF for Horizontal Curves	AMF for Superelevation	AMF for Grades	AMF for Driveway Density	AMF for Centerline Rumble Strips	AMF for Passing Lanes	AMF for Two-Way Left-Turn Lane	AMF for Roadside Design	AMF for Lighting	AMF for Automated Speed Enforcement	Combined AMF
AMF_{1r}	AMF_{2r}	AMF_{3r}	AMF_{4r}	AMF_{5r}	AMF_{6r}	AMF_{7r}	AMF_{8r}	AMF_{9r}	AMF_{10r}	AMF_{11r}	AMF_{12r}	AMF_{COMB}
from Equation 10-11	from Equation 10-12	from Equation 10-13	from Equations 10-14, 10-15, or 10-16	from Exhibit 10-19	from Equation 10-17	from Section 10.7.1	from Section 10.7.1	from Equation 10-18	from Equation 10-20	from Equation 10-21	from Section 10.7.1	(1)*(2)*… *(11)*(12)

2057

Worksheet 1C – Roadway Segment Crashes for Rural Two-Lane Two-Way Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crash Severity Level	$N_{spf\ rs}$	**Overdispersion Parameter, k**	**Crash Severity Distribution**	$N_{spf\ rs}$ **by Severity Distribution**	**Combined AMFs**	**Calibration Factor,** C_r	**Predicted average crash frequency,** $N_{predicted\ rs}$
	from Equation 10-6	from Equation 10-7	from Exhibit 10-6	$(2)_{TOTAL}$ * (4)	(13) from Worksheet 1B		(5)*(6)*(7)
Total			1.000				
Fatal and Injury (FI)	-	-	0.321				
Property Damage Only (PDO)	-	-	0.679				

Worksheet 1D – Crashes by Severity Level and Collision Type for Rural Two-Lane Two-Way Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Collision Type	**Proportion of Collision Type $_{(TOTAL)}$**	**$N_{predicted\ rs\ (TOTAL)}$ (crashes/year)**	**Proportion of Collision Type $_{(FI)}$**	**$N_{predicted\ rs\ (FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**$N_{predicted\ rs\ (PDO)}$ (crashes/year)**
	from Exhibit 10-7	$(8)_{TOTAL}$ from Worksheet 1C	from Exhibit 10-7	$(8)_{FI}$ from Worksheet 1C	from Exhibit 10-7	$(8)_{PDO}$ from Worksheet 1C
Total	1.000		1.000		1.000	
		$(2)*(3)_{TOTAL}$		$(4)*(5)_{FI}$		$(6)*(7)_{PDO}$
SINGLE-VEHICLE						
Collision with animal	0.121		0.038		0.184	
Collision with bicycle	0.002		0.004		0.001	
Collision with pedestrian	0.003		0.007		0.001	
Overturned	0.025		0.037		0.015	
Ran off road	0.521		0.545		0.505	
Other single-vehicle collision	0.021		0.007		0.029	
Total single-vehicle crashes	0.693		0.638		0.735	
MULTIPLE-VEHICLE						
Angle collision	0.085		0.100		0.072	
Head-on collision	0.016		0.034		0.003	
Rear-end collision	0.142		0.164		0.122	
Sideswipe collision	0.037		0.038		0.038	
Other multiple-vehicle collision	0.027		0.026		0.03	
Total multiple-vehicle crashes	0.307		0.362		0.265	

Worksheet 1E – Summary Results for Rural Two-Lane Two-Way Roadway Segments

(1)	(2)	(3)	(4)	(5)
Crash severity level	**Crash Severity Distribution**	**Predicted average crash frequency (crashes/year)**	**Roadway segment length (mi)**	**Crash rate (crashes/mi/year)**
	(4) from Worksheet 1C	(8) from Worksheet 1C		(3)/(4)
Total				
Fatal and Injury (FI)				
Property Damage Only (PDO)				

Worksheet 2A – General Information and Input Data for Rural Two-Lane Two-Way Road Intersections

General Information		**Location Information**	
Analyst		Roadway	
Agency or Company		Intersection	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	**Base Conditions**	**Site Conditions**
Intersection type (3ST, 4ST, 4SG)	-	
$AADT_{major}$ (veh/day)	-	
$AADT_{minor}$ (veh/day)	-	
Intersection skew angle (degrees)	0	
Number of signalized or uncontrolled approaches with a left turn lane (0,1,2,3,4)	0	
Number of signalized or uncontrolled approaches with a right turn lane (0,1,2,3,4)	0	
Intersection lighting (present/not present)	not present	
Calibration Factor, C_i	1.0	

Worksheet 2B – Accident Modification Factors for Rural Two-Lane Two-Way Road Intersections

(1)	(2)	(3)	(4)	(5)
AMF for Intersection Skew Angle	AMF for Left-Turn Lanes	AMF for Right-Turn Lanes	AMF for Lighting	Combined AMF
AMF_{1i}	AMF_{2i}	AMF_{3i}	AMF_{4i}	AMF_{COMB}
from Equations 10-22 or10-23	from Exhibit 10-21	from Exhibit 10-22	from Equation 10-24	(1)*(2)*(3)*(4)

Worksheet 2C – Intersection Crashes for Rural Two-Lane Two-Way Road Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crash Severity Level	**$N_{spf\ 3ST,4ST\ or\ 4SG}$**	**Overdispersion Parameter, k**	**Crash Severity Distribution**	**$N_{spf\ 3ST,4ST\ or\ 4SG}$ by Severity Distribution**	**Combined AMFs**	**Calibration Factor, C_i**	**Predicted average crash frequency, $N_{predicted\ int}$**
	from Equations 10-8, 10-9, or 10-10	from Section 10.6.2	from Exhibit 10-11	$(2)_{TOTAL}$ * (4)	from (5) of Worksheet 2B		(5)*(6)*(7)
Total							
Fatal and Injury (FI)	-	-					
Property Damage Only (PDO)	-	-					

Worksheet 2D – Crashes by Severity Level and Collision Type for Rural Two-Lane Two-Way Road Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Collision Type	**Proportion of Collision Type $_{(TOTAL)}$**	**$N_{predicted\ int\ (TOTAL)}$ (crashes/year)**	**Proportion of Collision Type $_{(FI)}$**	**$N_{predicted\ int\ (FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**$N_{predicted\ int\ (PDO)}$ (crashes/year)**
	from Exhibit 10-12	$(8)_{TOTAL}$ from Worksheet 2C	from Exhibit 10-12	$(8)_{FI}$ from Worksheet 2C	from Exhibit 10-12	$(8)_{PDO}$ from Worksheet 2C
Total	1.000		1.000		1.000	
		$(2)*(3)_{TOTAL}$		$(4)*(5)_{FI}$		$(6)*(7)_{PDO}$
SINGLE-VEHICLE						
Collision with animal						
Collision with bicycle						
Collision with pedestrian						
Overturned						
Ran off road						
Other single-vehicle collision						
Total single-vehicle crashes						
MULTIPLE-VEHICLE						
Angle collision						
Head-on collision						
Rear-end collision						
Sideswipe collision						
Other multiple-vehicle collision						
Total multiple-vehicle crashes						

Worksheet 2E – Summary Results for Rural Two-Lane Two-Way Road Intersections

(1)	(2)	(3)
Crash severity level	**Crash Severity Distribution**	**Predicted average crash frequency (crashes/year)**
	(4) from Worksheet 2C	(8) from Worksheet 2C
Total		
Fatal and injury (FI)		
Property Damage Only (PDO)		

Worksheet 3A – Predicted and Observed Crashes by Severity and Site Type Using the Site-Specific EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Site type	**Predicted average crash frequency (crashes/year)**			**Observed crashes, $N_{observed}$ (crashes/year)**	**Overdispersion parameter, k**	**Weighted adjustment, w**	**Expected average crash frequency, $N_{expected}$**
	$N_{predicted\ (TOTAL)}$	$N_{predicted\ (FI)}$	$N_{predicted\ (PDO)}$			Equation A-5 from Part C Appendix	Equation A-4 from Part C Appendix
ROADWAY SEGMENTS							
Segment 1							
Segment 2							
Segment 3							
Segment 4							
Segment 5							
Segment 6							
Segment 7							
Segment 8							
INTERSECTIONS							
Intersection 1							
Intersection 2							
Intersection 3							
Intersection 4							
Intersection 5							
Intersection 6							
Intersection 7							
Intersection 8							
COMBINED (sum of column)					-	-	

Worksheet 3B – Site-Specific EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

(1)	(2)	(3)
Crash severity level	$N_{predicted}$	$N_{expected}$
Total	$(2)_{COMB}$ from Worksheet 3A	$(8)_{COMB}$ from Worksheet 3A
Fatal and injury (FI)	$(3)_{COMB}$ from Worksheet 3A	$(3)_{TOTAL}*(2)_{FI}/(2)_{TOTAL}$
Property damage only (PDO)	$(4)_{COMB}$ from Worksheet 3A	$(3)_{TOTAL}*(2)_{PDO}/(2)_{TOTAL}$

Worksheet 4A – Predicted and Observed Crashes by Severity and Site Type Using the Project-Level EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Site type	**Predicted average crash frequency (crashes/year)**			**Observed crashes, $N_{observed}$ (crashes/year)**	**Overdispersion Parameter, k**	$N_{predicted\ w0}$	$N_{predicted\ w1}$	W_0	N_0	w_1	N_1	$N_{expected/comb}$
	$N_{predicted}$ (TOTAL)	$N_{predicted}$ (FI)	$N_{predicted}$ (PDO)			Equation A-8 $(6)*(2)^2$	Equation A-9 sqrt((6)*(2))	Equation A-10	Equation A-11	Equation A-12	Equation A-13	Equation A-14
ROADWAY SEGMENTS												
Segment 1				-				-	-	-	-	-
Segment 2				-				-	-	-	-	-
Segment 3				-				-	-	-	-	-
Segment 4				-				-	-	-	-	-
Segment 5				-				-	-	-	-	-
Segment 6				-				-	-	-	-	-
Segment 7				-				-	-	-	-	-
Segment 8				-				-	-	-	-	-
INTERSECTIONS												
Intersection 1				-				-	-	-	-	-
Intersection 2				-				-	-	-	-	-
Intersection 3				-				-	-	-	-	-
Intersection 4				-				-	-	-	-	-
Intersection 5				-				-	-	-	-	-
Intersection 6				-				-	-	-	-	-
Intersection 7				-				-	-	-	-	-
Intersection 8				-				-	-	-	-	-
COMBINED (sum of column)					-							

Worksheet 4B – Project-Level EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

(1)	(2)	(3)
Crash severity level	**$N_{predicted}$**	**$N_{expected/comb}$**
Total	$(2)_{COMB}$ from Worksheet 4A	$(13)_{COMB}$ from Worksheet 4A
Fatal and injury (FI)	$(3)_{COMB}$ from Worksheet 4A	$(3)_{TOTAL}*(2)_{FI}/(2)_{TOTAL}$
Property damage only (PDO)	$(4)_{COMB}$ from Worksheet 4A	$(3)_{TOTAL}*(2)_{PDO}/(2)_{TOTAL}$

2073

PART C —PREDICTIVE METHOD

CHAPTER 11 — PREDICTIVE METHOD FOR RURAL MULTILANE HIGHWAYS

11.1. INTRODUCTION 11-1
11.2. OVERVIEW OF THE PREDICTIVE METHOD 11-1
11.3. RURAL MULTILANE HIGHWAYS – DEFINITIONS AND PREDICTIVE MODELS IN CHAPTER 11 11-2
11.3.1. Definition of Chapter 11 Facility and Site Types 11-2
11.3.2. Predictive Models for Rural Multilane Roadway Segments 11-4
11.3.3. Predictive Models for Rural Multilane Highway Intersections 11-4
11.4. PREDICTIVE METHOD FOR RURAL MULTILANE HIGHWAYS 11-5
11.5. ROADWAY SEGMENTS AND INTERSECTIONS 11-13
11.6. SAFETY PERFORMANCE FUNCTIONS 11-16
11.6.1. Safety Performance Functions for Undivided Roadway Segments 11-17
11.6.2. Safety Performance Functions for Divided Roadway Segments 11-20
11.6.3. Safety Performance Functions for Intersections 11-22
11.7. ACCIDENT MODIFICATION FACTORS 11-27
11.7.1. Accident Modification Factors for Undivided Roadway Segments 11-28
11.7.2. Accident Modification Factors for Divided Roadway Segments 11-33
11.7.3. Accident Modification Factors for Intersections 11-36
11.8. CALIBRATION TO LOCAL CONDITIONS 11-39
11.9. LIMITATIONS OF PREDICTIVE METHODS IN CHAPTER 11 11-40
11.10. APPLICATION OF CHAPTER 11 PREDICTIVE METHOD 11-40
11.11. SUMMARY 11-40
11.12. SAMPLE PROBLEMS 11-42
11.12.1. Sample Problem 1 11-43
11.12.2. Sample Problem 2 11-50
11.12.3. Sample Problem 3 11-57
11.12.4. Sample Problem 4 11-64
11.12.5. Sample Problem 5 11-68
11.12.6. Sample Problem 6 11-72
11.13. REFERENCES 11-74

EXHIBITS

Exhibit 11-1: Rural Multilane Highway Site Type with SPFs in Chapter 11 11-3

Exhibit 11-2: The HSM Predictive Method 11-6

Exhibit 11-3: Definition of Segments and Intersections.. 11-14

Exhibit 11-4: Safety Performance Functions included in Chapter 11 11-17

Exhibit 11-5: SPF Coefficients for Total and Fatal-and-Injury Accidents on Undivided Roadway Segments (for use in Equations 11-7 and 11-8) ... 11-18

Exhibit 11-6: Graphical Form of the SPF for Undivided Roadway Segments (from Equation 11-7 and Exhibit 11-5).. 11-19

Exhibit 11-7: Default Distribution of Crashes by Collision Type and Crash Severity Level for Undivided Roadway Segments 11-20

Exhibit 11-8: SPF Coefficients for Total and Fatal-and-Injury Accidents on Divided Roadway Segments (for use in Equations 11-9 and 11-10) 11-21

Exhibit 11-9: Graphical Form of SPF for Rural Multilane Divided Roadway Segments (from Equation 11-9 and Exhibit 11-8).............................. 11-22

Exhibit 11-10: Default Distribution of Crashes by Collision Type and Crash Severity Level for Divided Roadway Segments 11-22

Exhibit 11-11: SPF Coefficients for Three- and Four-leg Intersections with Minor road Stop Control for Total and Fatal-and-Injury Accidents (for use in Equation 11-11) 11-24

Exhibit 11-12: SPF Coefficients for Four-leg Signalized Intersections for Total and Fatal-and-Injury Accidents (for use in Equations 11-11 and 11-12) 11-25

Exhibit 11-13: Graphical Form of SPF for three-leg STOP-controlled Intersections - for Total Crashes Only (from Equation 11-11 and Exhibit 11-11) 11-25

Exhibit 11-14: Graphical Form of SPF for Four-leg STOP-controlled Intersections - for Total Crashes only (from Equation 11-11 and Exhibit 11-11) 11-25

Exhibit 11-15: Graphical Form of SPF for Four-leg Signalized Intersections - for Total Crashes only (from Equation 11-11 and Exhibit 11-11)............... 11-26

Exhibit 11-16: Default Distribution of Intersection Crashes by Collision Type and Crash Severity 11-27

Exhibit 11-17: Summary of AMFs in Chapter 11 and the Corresponding SPFs 11-28

Exhibit 11-18: AMF_{RA} for Collision Types Related to Lane Width 11-29

Exhibit 11-19: AMF_{RA} for Lane Width on Undivided Segments 11-30

Exhibit 11-20: AMF for Collision Types Related to Shoulder Width (AMF_{WRA}).............. 11-31

Exhibit 11-21: AMF_{WRA} for Shoulder Width on Undivided Segments 11-31

Exhibit 11-22: AMF for Collision Types Related to Shoulder Type and Shoulder Width (AMF_{TRA}) 11-31

Exhibit 11-23: AMF for Side Slope on Undivided Roadway Segments (AMF_{3ru}) 11-32

Exhibit 11-24: Night-time Accident Proportions for Unlighted Roadway Segments...... 11-32

Exhibit 11-25: AMF for Collision Types Related to Lane Width (AMF_{RA}) 11-33

Exhibit 11-26: AMF_{RA} for Lane Width on Divided Roadway Segments 11-34

Exhibit 11-27: AMF for Right Shoulder Width on Divided Roadway Segments (AMF_{2rd}) 11-34

Exhibit 11-28: AMFs for Median Width on Divided Roadway Segments without a Median Barrier (AMF_{3rd}) 11-35

Exhibit 11-29: Nighttime Accident Proportions for Unlighted Roadway Segments 11-36

Exhibit 11-30: AMFs for Three-leg Intersections with Minor Road Stop Control (3ST) 11-36

Exhibit 11-31: AMFs for Four-leg Intersection with Minor Road Stop Control (4ST) 11-37

Exhibit 11-32: Accident Modification Factors (AMF_{2i}) for Installation of Left-Turn Lanes on Intersection Approaches. 11-38

Exhibit 11-33: Accident Modification Factors (AMF_{3i}) for Installation of Right-Turn Lanes on Intersections Approaches. 11-39

Exhibit 11-34: Default Nighttime Accident Proportions for Unlighted Intersections 11-39

Exhibit 11-35: List of Sample Problems In Chapter 11 11-42

Exhibit 11-39: SPFs for Selected Collision Types on Four-Lane Undivided Roadway Segments (Based on Equation 11-4) 11-91

Exhibit 11-40: Collision Type Models for Stop-Controlled Intersections without Specific Base Conditions (Based on Equations 11-11 and 11-12) 11-92

APPENDIX A

APPENDIX A — WORKSHEETS FOR APPLYING THE PREDICTIVE METHOD FOR RURAL MULTILANE ROADS 11-75

APPENDIX B – PREDICTIVE MODELS FOR SELECTED COLLISION TYPES 11-91

B.1 UNDIVIDED ROADWAY SEGMENTS 11-91

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CHAPTER 11 PREDICTIVE METHOD FOR RURAL MULTILANE HIGHWAYS

11.1. INTRODUCTION

Chapter 11 explains the predictive method for rural multilane highways.

This chapter presents for the predictive method for rural multilane highways. A general introduction to the Highway Safety Manual (HSM) predictive method is provided in the *Part C Introduction and Applications Guidance*.

The predictive method for rural multilane highways provides a structured methodology to estimate the expected average crash frequency, crash severity, and collision types for a rural multilane highway facility with known characteristics. All types of crashes involving vehicles of all types, bicycles, and pedestrians are included, with the exception of crashes between bicycles and pedestrians. The predictive method can be applied to existing sites, design alternatives to existing sites, new sites, or for alternative traffic volume projections. An estimate can be made for crash frequency in a period of time that occurred in the past (i.e. what did or would have occurred) or in the future (i.e., what is expected to occur). The development of the predictive models in Chapter 11 is documented in Lord et al[5]. The AMFs used in the predictive models have been reviewed and updated by Harkey et al[3] and in related work by Srinivasan et al[6]. The SPF coefficients, default collision type distributions, and default nighttime accident proportions have been adjusted to a consistent basis by Srinivasan et al[7].

This chapter presents the following information about the predictive method for rural multilane highways:

- A concise overview of the predictive method.
- The definitions of the facility types included in Chapter 11 and site types for which predictive models have been developed for Chapter 11.
- The steps of the predictive method in graphical and descriptive forms.
- Details for dividing a rural multilane facility into individual sites, consisting of intersections and roadway segments.
- Safety Performance Functions (SPFs) for rural multilane highways.
- Accident Modification Factors (AMFs) applicable to the SPFs in Chapter 11.
- Guidance for application of the Chapter 11 predictive method and limitations of the predictive method specific to Chapter 11.
- Sample problems illustrating the application of the Chapter 11 predictive method for rural multilane highways.

11.2. OVERVIEW OF THE PREDICTIVE METHOD

The predictive method provides an 18 step procedure to estimate the "expected average crash frequency", $N_{expected}$ (by total crashes, crash severity or collision type), of a roadway network, facility, or site. In the predictive method the roadway is divided into individual sites, which are homogenous roadway segments and intersections. A facility consists of a contiguous set of individual intersections and roadway segments, referred to as "sites." Different facility types are determined by surrounding land use, roadway cross-section, and degree of access. For each facility

The EB Method is described in full detail in the Part C Appendix.

type, a number of different site types may exist, such as divided and undivided roadway segments, and unsignalized and signalized intersections. A roadway network consists of a number of contiguous facilities.

The method is used to estimate the expected average crash frequency of an individual site, with the cumulative sum of all sites used as the estimate for an entire facility or network. The estimate is for a given time period of interest (in years) during which the geometric design and traffic control features are unchanged and traffic volumes (AADT) are known or forecasted. The estimate relies on estimates made using predictive models which are combined with observed crash data using the Empirical Bayes (EB) Method.

The predictive models used in Chapter 11 to determine the predicted average crash frequency $N_{predicted}$, are of the general form shown in Equation 11-1.

$$N_{predicted} = N_{spf\ x} \times (AMF_{1x} \times AMF_{2x} \times \ldots \times AMF_{yx}) \times C_x \quad (11\text{-}1)$$

Where,

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

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

AMF_{yx} = Accident Modification Factors specific to site type *x* and specific geometric design and traffic control features *y*;

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

11.3. RURAL MULTILANE HIGHWAYS – DEFINITIONS AND PREDICTIVE MODELS IN CHAPTER 11

This section provides the definitions of the facility and site types included in Chapter 11, and the predictive models for each the site types included in Chapter 11. These predictive models are applied following the steps of the predictive method presented in Section 11.4.

11.3.1. Definition of Chapter 11 Facility and Site Types

Chapter 11 applies to rural multilane highway facilities. The term "multilane" refers to facilities with four through lanes. Rural multilane highway facilities may have occasional grade-separated interchanges, but these are not be the primary form of access and egress. The predictive method does not apply to any section of a multilane highway within the limits of an interchange which has free-flow ramp terminals on the multilane highway of interest. Facilities with six or more lanes are not covered in Chapter 11

The terms "highway" and "road" are used interchangeably in this chapter and apply to all rural multilane facilities independent of official state or local highway designation.

Classifying an area as urban, suburban or rural is subject to the roadway characteristics, surrounding population and land uses and is at the user's discretion. In the HSM, the definition of "urban" and "rural" areas is based on Federal Highway Administration (FHWA) guidelines which classify "urban" areas as places inside urban boundaries where the population is greater than 5,000 persons. "Rural" areas are defined as places outside urban areas which have with population greater than 5,000 persons. The HSM uses the term "suburban" to refer to outlying portions of an urban area; the predictive method does not distinguish between urban and suburban portions of a developed area.

Exhibit 11-1 identifies the specific site types on rural multilane highways for which predictive models have been developed for estimating expected average crash frequency, severity and collision type. The four-leg signalized intersection models do not have base conditions and, therefore, can be used only for generalized predictions of crash frequencies. No predictive models are available for roadway segments with more than four lanes or for other intersection types such as all-way stop-controlled intersections, yield-controlled intersections, or uncontrolled intersections.

Exhibit 11-1: Rural Multilane Highway Site Type with SPFs in Chapter 11

Site Type	Site Types with SPFs in 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)*

* The four-leg signalized intersection models do not have base conditions and, therefore, can be used only for generalized predictions of crash frequency.

SPFs are available for: undivided roadway segments, three-leg intersections with STOP control, four-leg intersections with STOP control, and four-leg signalized intersections.

These specific site types are defined as follows:

- Undivided four lane roadway segment (4U) – a roadway consisting of four lanes with a continuous cross-section which provides two directions of travel in which the lanes are not physically separated by either distance or a barrier. While multilane roadways whose opposing lanes are separated by a flush median (i.e., a painted median) are considered undivided facilities, not divided facilities, the predictive models in Chapter 11 do not address rural multilane highways with flush separators.

- Divided four lane roadway segment (4D) – Divided highways are non-freeway facilities (i.e., facilities without full control of access) that have the lanes in the two directions of travel separated by a raised, depressed or flush median which is not designed to be traversed by a vehicle; this may include raised or depressed medians, with or without a physical median barrier, or flush medians with physical median barriers.

- Three-leg intersection with STOP control (3ST) – an intersection of a rural multilane highway (i.e., 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 (i.e., 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 (i.e., 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.

11.3.2. Predictive Models for Rural Multilane Roadway Segments

The predictive models can be used to estimate total crashes (i.e., all crash severities and collision types) or can be used to estimate the expected average frequency of specific crash severity types or specific collision types. The predictive model for an individual roadway segment or intersection combines a SPF with AMFs and a calibration factor.

The predictive models for roadway segments estimate the predicted average crash frequency of non-intersection-related crashes. In other words, the roadway segment predictive models estimate crashes that would occur regardless of the presence of an intersection.

The predictive models for undivided roadway segments, divided roadway segments and intersections are presented in Equations 11-2, 11-3 and 11-4 below.

For undivided roadway segments the predictive model is:

$$N_{predicted\ rs} = N_{spf\ ru} \times C_r \times (AMF_{1ru} \times AMF_{2ru} \times \ldots \times AMF_{5ru}) \tag{11-2}$$

For divided roadway segments the predictive model is:

$$N_{predicted\ rs} = N_{spf\ rd} \times C_r \times (AMF_{1rd} \times AMF_{2rd} \times \ldots \times AMF_{5rd}) \tag{11-3}$$

Where,

$N_{predicted\ rs}$ = predictive model estimate of expected average crash frequency for an individual roadway segment for the selected year;

$N_{spf\ ru}$ = expected average crash frequency for an undivided roadway segment with base conditions;

C_r = calibration factor for roadway segments of a specific type developed for a particular jurisdiction or geographical area;

$AMF_{1ru} \ldots AMF_{5ru}$ = Accident Modification Factors for undivided roadway segments;

$N_{spf\ rd}$ = expected average crash frequency for a divided roadway segment with base conditions;

$AMF_{1rd} \ldots AMF_{5rd}$ = Accident Modification Factors for divided roadway segments.

11.3.3. Predictive Models for Rural Multilane Highway Intersections

The predictive models for intersections estimate the predicted average crash frequency of crashes within the limits of an intersection, or crashes that occur on the intersection legs, and are a result of the presence of the intersection (i.e., intersection-related crashes).

For all intersection types in Chapter 11 the predictive model is:

$$N_{predicted\ int} = N_{spf\ int} \times C_i \times (AMF_{1i} \times AMF_{2i} \times \ldots \times AMF_{4i}) \tag{11-4}$$

Where,

$N_{predicted\ int}$ = predicted average crash frequency for an individual intersection for the selected year;

$N_{spf\ int}$ = predicted average crash frequency for an intersection with base conditions;

$AMF_{1i} \ldots AMF_{4i}$ = Accident Modification Factors for intersections – however, these AMFs are only applicable to three and four-leg STOP controlled intersections. No AMFs are available for four-leg signalized intersections; and

C_i = calibration factor for intersections of a specific type developed for use for a particular jurisdiction of geographical area.

The SPFs for rural multilane highways are presented in Section 11.6. The associated AMFs for each of the SPFs are presented in Section 11.7, and summarized in Exhibit 11-17. Only the specific AMFs associated with each SPF are applicable to an SPF (as these AMFs have base conditions which are identical the base conditions of the SPF). The calibration factors, C_r and C_i are determined in the *Part C* Appendix A.1.1. Due to continual change in the crash frequency and severity distributions with time, the value of the calibration factors may change for the selected year of the study period.

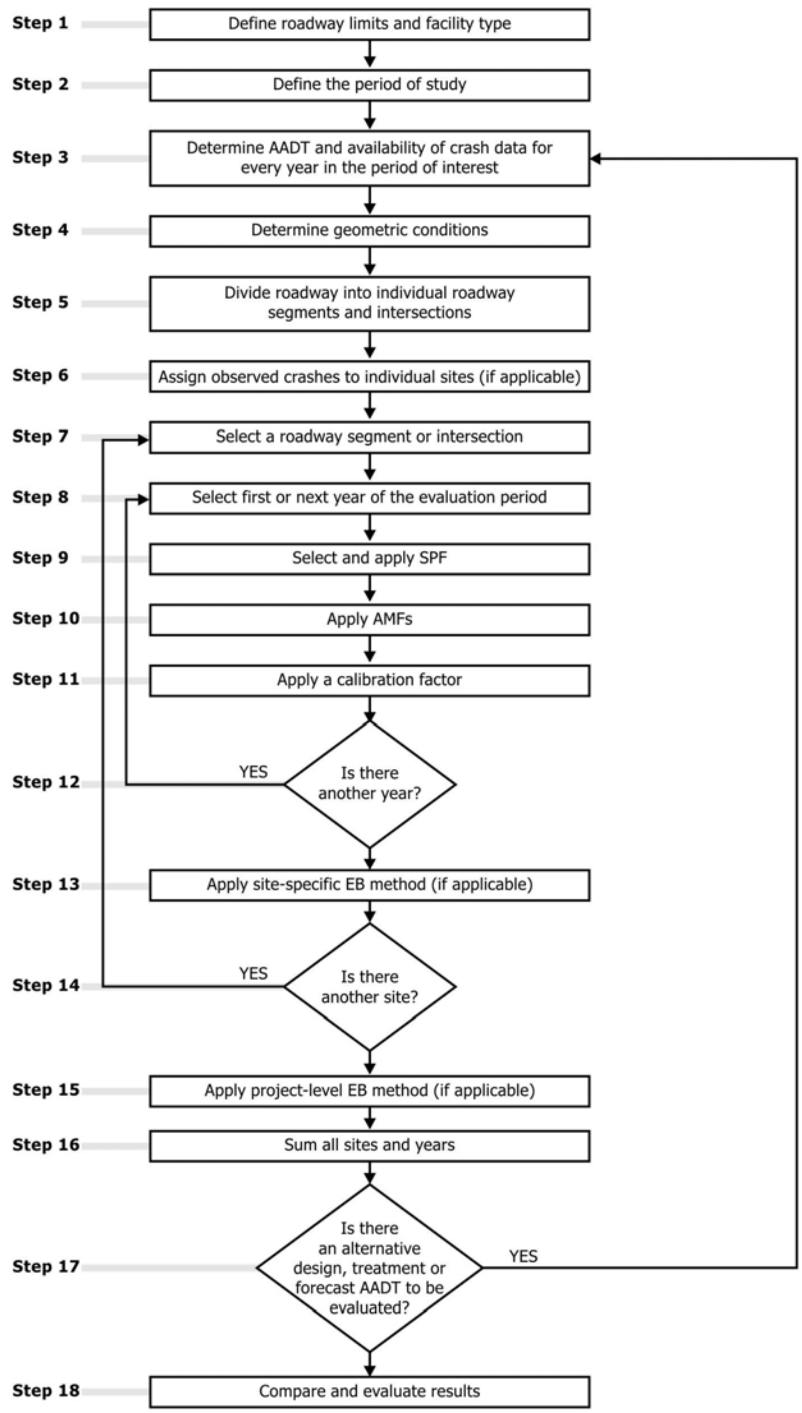
11.4. PREDICTIVE METHOD FOR RURAL MULTILANE HIGHWAYS

The predictive method for rural multilane highways is shown in Exhibit 11-2. Applying the predictive method yields an estimate of the expected average crash frequency (and/or crash severity and collision types) for a rural multilane highway facility. The components of the predictive models in Chapter 11 are determined and applied in Steps 9, 10, and 11 of the predictive method. Further information needed to apply each step is provided in the following sections and in the *Part C* Appendix.

There are 18 steps in the predictive method. In some situations, certain steps will not be needed because the data is not available or the step is not applicable to the situation at hand. In other situations, steps may be repeated if an estimate is desired for several sites or for a period of several years. In addition, the predictive method can be repeated as necessary to undertake crash estimation for each alternative design, traffic volume scenario or proposed treatment option (within the same period to allow for comparison).

The following explains the details of each step of the method as applied to rural multilane highways.

Exhibit 11-2: The HSM Predictive Method



Step 1 - Define the limits of the roadway and facility types in the study network, facility, or site for which the expected average crash frequency, severity, and collision types are to be estimated.

The predictive method can be undertaken for a roadway network, a facility, or an individual site. A site is either an intersection or a homogeneous roadway segment. Sites may consist of a number of types, such as signalized and unsignalized intersections. The definitions of a rural multilane highway, an intersection and roadway segments, and the specific site types included in Chapter 11 are provided in Section 11.3.

The predictive method can be undertaken for an existing roadway, a design alternative for an existing, or a new roadway (which may be either unconstructed or yet to experience enough traffic to have observed crash data).

The limits of the roadway of interest will depend on the nature of the study. The study may be limited to only one specific site or a group of contiguous sites. Alternatively, the predictive method can be applied to a very long corridor for the purposes of network screening (determining which sites require upgrading to reduce crashes) which is discussed in *Chapter 4*.

Step 2 - Define the period of interest.

The predictive method can be undertaken for either a past period or a future period. All periods are measured in years. Years of interest will be determined by the availability of observed or forecast AADTs, observed crash data, and geometric design data. Whether the predictive method is used for a past or future period depends upon the purpose of the study. The period of study may be:

A past period (based on observed AADTs) for:

- An existing roadway network, facility, or site. If observed crash data are available, the period of study is the period of time for which the observed crash data are available and for which (during that period) the site geometric design features, traffic control features, and traffic volumes are known.

- An existing roadway network, facility, or site for which alternative geometric design features or traffic control features are proposed (for near term conditions).

A future period (based on forecast AADTs) for:

- An existing roadway network, facility, or site for a future period where forecast traffic volumes are available.

- An existing roadway network, facility, or site for which alternative geometric design or traffic control features are proposed for implementation in the future.

- A new roadway network, facility, or site that does not currently exist, but is proposed for construction during some future period.

Step 3 – For the study period, determine the availability of annual average daily traffic volumes and, for an existing roadway network, the availability of observed crash data to determine whether the EB Method is applicable.

Determining Traffic Volumes

The SPFs used in Step 9 (and some AMFs in Step 10), include AADT volumes (vehicles per day) as a variable. For a past period, the AADT may be determined by automated recording or estimated from a sample survey. For a future period, the AADT may be a forecast estimate based on appropriate land use planning and traffic volume forecasting models, or based on the assumption that current traffic volumes will remain relatively constant.

Roadway segments require two-way AADT.

For each roadway segment, the AADT is the average daily two-way 24 hour traffic volume on that roadway segment in each year of the period to be evaluated, selected in Step 8.

Intersections require the major and minor road AADT.

For each intersection, two values are required in each predictive model. These are the AADT of the major street, $AADT_{maj}$; and the two-way AADT of the minor street, $AADT_{min}$.

In Chapter 11, $AADT_{maj}$ and $AADT_{min}$ are determined as follows: if the AADTs on the two major road legs of an intersection differ, the larger of the two AADT values are used for $AADT_{maj}$. For a three-leg intersection, the AADT of the minor road leg is used for $AADT_{min}$. For a four-leg intersection, the larger of the AADTs for the two minor road legs should be used for $AADT_{min}$. If a highway agency lacks data on the entering traffic volumes, but has two-way AADT data for the major and minor road legs of the intersection, these may be used as a substitute for the entering volume data. Where needed, $AADT_{total}$ can be estimated as the sum of $AADT_{maj}$ and $AADT_{min}$.

In many cases, it is expected that AADT data will not be available for all years of the evaluation period. In that case, an estimate of AADT for each year of the evaluation period is interpolated or extrapolated as appropriate. If there is no established procedure for doing this, the following may be applied within the predictive method to estimate the AADTs for years for which data are not available.

- If AADT data are available for only a single year, that same value is assumed to apply to all years of the before period;
- If two or more years of AADT data are available, the AADTs for intervening years are computed by interpolation;
- The AADTs for years before the first year for which data are available are assumed to be equal to the AADT for that first year;
- The AADTs for years after the last year for which data are available are assumed to be equal to the last year.

If the EB Method is to be used (discussed below), AADT data are needed for each year of the period for which observed crash frequency data are available. If the EB Method will not be used, AADT for the appropriate time period—is past, present, or future—determined in Step 2 are used.

Determining availability of Observed Crash Data

Where an existing site or alternative conditions to an existing site are being considered, the EB Method is used. The EB Method is only applicable when reliable observed crash data are available for the specific study roadway network, facility, or site. Observed data may be obtained directly from the jurisdiction's accident report system. At least two years of observed crash frequency data are desirable to apply the EB Method. The EB Method and criteria to determine whether the EB Method is applicable are presented in Section A.2.1 in the Appendix to *Part C*.

The EB Method can be applied at the site-specific level (i.e., observed crashes are assigned to specific intersections or roadway segments in Step 6) or at the project level (i.e., observed crashes are assigned to a facility as a whole). The site-specific EB Method is applied in Step 13. Alternatively, if observed crash data are available but can not be assigned to individual roadway segments and intersections, the project level EB Method is applied (in Step 15).

If observed crash data are not available, then Steps 6, 13, and 15 of the predictive method are not conducted. In this case, the estimate of expected average crash frequency is limited to using a predictive model (i.e. the predicted average crash frequency).

The EB Method and criteria to determine whether the EB Method is applicable are presented in Section A.2.1 in the Appendix to Part C.

Step 4 - Determine geometric design features, traffic control features and site characteristics for all sites in the study network.

In order to determine the relevant data needs and avoid unnecessary data collection, it is necessary to understand the base conditions of the SPFs in Step 9 and the AMFs in Step 10. The base conditions are defined in Section 11.6.1 and 11.6.2 for roadway segments and in Section 11.6.3 for intersections.

The following geometric design and traffic control features are used to select a SPF and to determine whether the site specific conditions vary from the base conditions and, therefore, whether an AMF is applicable:

- Length of roadway segment (miles)
- AADT (vehicles per day)
- Presence of median and median width (feet) (for divided roadway segments)
- Side slope (for undivided roadway segments)
- Shoulder widths (feet)
- Lane width (feet)
- Presence of lighting
- Presence of automated speed enforcement

For each intersection in the study area, the following geometric design and traffic control features are identified:

- Number of intersection legs (3 or 4)
- Type of traffic control (minor road STOP or signalized)
- Intersection skew angle (stop controlled intersections)
- Presence of left-turn and right-turn lanes (Stop controlled intersections)
- Presence or absence of lighting (Stop controlled intersections)

Step 5 – Divide the roadway network or facility under consideration into individual homogenous roadway segments and intersections, which are referred to as sites.

Using the information from Step 1 and Step 4, the roadway is divided into individual sites, consisting of individual homogenous roadway segments and intersections. The definitions and methodology for dividing the roadway into individual intersections and homogenous roadway segments for use with the Chapter 11 predictive models are provided in Section 11.5. When dividing roadway facilities into small homogenous roadway segments, limiting the segment length to a minimum of 0.10 miles will minimize calculation efforts and not affect results.

Step 6 – Assign observed crashes to the individual sites (if applicable).

Step 6 only applies if it was determined in Step 3 that the site-specific EB Method was applicable. If the site-specific EB Method is not applicable, proceed to Step 7. In Step 3, the availability of observed data and whether the data could be assigned to specific locations was determined. The specific criteria for assigning accidents to individual roadway segments or intersections are presented in Section A.2.3 of the Appendix to *Part C*.

The specific criteria for assigning crashes to individual roadway segments for intersections are presented in Section A.2.3 of Appendix to Part C.

Crashes that occur at an intersection or on an intersection leg, and are related to the presence of an intersection, are assigned to the intersection and used in the EB Method together with the predicted average crash frequency for the intersection. Crashes that occur between intersections and are not related to the presence of an intersection are assigned to the roadway segment on which they occur; such crashes are used in the EB Method together with the predicted average crash frequency for the roadway segment.

Step 7 – Select the first or next individual site in the study network. If there are no more sites to be evaluated, proceed to Step 15.

In Step 5, the roadway network within the study limits has been divided into a number of individual homogenous sites (intersections and roadway segments).

The outcome of the HSM Predictive Method is the expected average crash frequency of the entire study network, which is the sum of the all of the individual sites, for each year in the study. Note that this value will be the total number of crashes expected to occur over all sites during the period of interest. If a crash frequency is desired (crashes per year), the total can be divided by the number of years in the period of interest.

The estimation for each site (roadway segments or intersection) is conducted one at a time. Steps 8 through 14, described below, are repeated for each site.

Expected average crashes for the study period are calculated for each year of the period.

Step 8 – For the selected site, select the first or next year in the period of interest. If there are no more years to be evaluated for that site, proceed to Step 14.

Steps 8 through 14 are repeated for each site in the study and for each year in the study period.

The individual years of the evaluation period may have to be analyzed one year at a time for any particular roadway segment or intersection because SPFs and some AMFs (e.g., lane and shoulder widths) are dependent on AADT, which may change from year to year.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type and traffic control features.

Steps 9 through 13, described below, are repeated for each year of the evaluation period as part of the evaluation of any particular roadway segment or intersection. The predictive models in Chapter 11 follow the general form shown in Equation 11-1. Each predictive model consists of a SPF, which is adjusted to site specific conditions using AMFs (in Step 10) and adjusted to local jurisdiction conditions (in Step 11) using a calibration factor (C). The SPFs, AMFs and calibration factor obtained in Steps 9, 10 and 11 are applied to calculate the predictive model estimate of predicted average crash frequency for the selected year of the selected site. The SPFs available for rural multilane highways are presented in Section 11.6

The SPF (which is a statistical regression model based on observed crash data for a set of similar sites) determines the predicted average crash frequency for a site with the base conditions (i.e., a specific set of geometric design and traffic control features). The base conditions for each SPF are specified in Section 11.6. A detailed explanation and overview of the SPFs in *Part C* is provided in Section C.6.3 of the *Part C Introduction and Applications Guidance*.

The SPFs (and base conditions) developed for Chapter 11 are summarized in Exhibit 11-4 in Section 11.6. For the selected site, determine the appropriate SPF for the site type (intersection or roadway segment) and geometric and traffic control features (undivided roadway, divided roadway, stop controlled intersection, signalized intersection). The SPF for the selected site is calculated using the AADT determined in Step 3 (or $AADT_{maj}$ and $AADT_{min}$ for intersections) for the selected year.

Each SPF determined in Step 9 is provided with default distributions of crash severity and collision type (presented in Section 11.6). These default distributions can benefit from being updated based on local data as part of the calibration process presented in Appendix A.1.1.

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust base conditions to site specific geometric conditions and traffic control features.

In order to account for differences between the base conditions (Section 11.6) and the site specific conditions, AMFs are used to adjust the SPF estimate. An overview of AMFs and guidance for their use is provided in Section C.6.4 of the *Part C Introduction and Applications Guidance*, including the limitations of current knowledge related to the effects of simultaneous application of multiple AMFs. In using multiple AMFs, engineering judgment is required to assess the interrelationships and/or independence of individual elements or treatments being considered for implementation within the same project.

An overview of AMFs and guidance for their use is provided in Section C.6.4 of the Part C Introduction and Applications Guidance

All AMFs used in Chapter 11 have the same base conditions as the SPFs used in Chapter 11 (i.e., when the specific site has the same condition as the SPF base condition, the AMF value for that condition is 1.00). Only the AMFs presented in Section 11.7 may be used as part of the Chapter 11 predictive method. Exhibit 11-17 indicates which AMFs are applicable to the SPFs in Section 11.6.

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

Detailed guidance for the development of calibration factors is included in Part C Appendix A.1.1.

The SPFs used in the predictive method have each been developed with data from specific jurisdictions and time periods in the data sets. Calibration of the SPFs to local conditions will account for differences in the data set.. A calibration factor (C_r for roadway segments or C_i for intersections) is applied to each SPF in the predictive method. An overview of the use of calibration factors is provided in the *Part C Introduction and Applications Guidance* Section C.6.5. Detailed guidance for the development of calibration factors is included in *Part C* Appendix A.1.1.

Steps 9, 10, and 11 together implement the predictive models in Equations 11-2, 11-3 and 11-4 to determine predicted average crash frequency.

Step 12 – If there is another year to be evaluated in the study period for the selected site, return to Step 8. Otherwise, proceed to Step 14.

This step creates a loop through Steps 8 to 12 that is repeated for each year of the evaluation period for the selected site.

Step 13 – Apply site-specific EB Method (if applicable).

Whether the site-specific EB Method is applicable is determined in Step 3. The site-specific EB Method combines the Chapter 11 predictive model estimate of predicted average crash frequency, $N_{predicted,}$ with the observed crash frequency of the specific site, $N_{observed}$. This provides a more statistically reliable estimate of the expected average crash frequency of the selected site.

In order to apply the site-specific EB Method, in addition to the material in *Part C* Appendix A.2.4, the overdispersion parameter, k, for the SPF is also used. The overdispersion parameter provides an indication of the statistical reliability of the SPF. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. This parameter is used in the site-specific EB Method to provide a weighting to $N_{predicted}$ and $N_{observed}$. Overdispersion parameters are provided for each SPF in Section 11.6.

Apply the site-specific EB Method to a future time period, if appropriate.

The estimated expected average crash frequency obtained above applies to the time period in the past for which the observed crash data were obtained. Section A.2.6 in the Appendix to *Part C* provides a Method to convert the estimate of expected average crash frequency for a past time period to a future time period.

Step 14 – If there is another site to be evaluated, return to Step 7, otherwise, proceed to Step 15.

The project level EB Method is described in Part C Appendix A.2.5.

This step creates a loop through Steps 7 to 13 that is repeated for each roadway segment or intersection within the facility.

Step 15 – Apply the project level EB Method (if the site specific EB Method is not applicable).

This step is only applicable to existing conditions when observed crash data are available, but can not be accurately assigned to specific sites (e.g., the crash report may identify crashes as occurring between two intersections, but is not accurate to determine a precise location on the segment). Detailed description of the project level EB Method is provided in *Part C* Appendix A.2.5.

Step 16 – Sum all sites and years in the study to estimate total crash frequency.

The total estimated number of crashes within the network or facility limits during a study period of n years is calculated using Equation 11-5:

$$N_{total} = \sum_{\substack{all\\roadway\\segments}} N_{rs} + \sum_{\substack{all\\intersections}} N_{int} \tag{11-5}$$

Where,

N_{total} = total expected number of crashes within the limits of a rural two-lane two-way road facility for the period of interest. Or, the sum of the expected average crash frequency for each year for each site within the defined roadway limits within the study period;

N_{rs} = expected average crash frequency for a roadway segment using the predictive method for one specific year;

N_{int} = expected average crash frequency for an intersection using the predictive method for one specific year.

Equation 11-5 represents the total expected number of crashes estimated to occur during the study period. Equation 11-6 is used to estimate the total expected average crash frequency within the network or facility limits during the study period.

$$N_{total\ average} = \frac{N_{total}}{n} \tag{11-6}$$

Where,

$N_{total\ average}$ = total expected average crash frequency estimated to occur within the defined network or facility limits during the study period;

n = number of years in the study period.

Step 17 – Determine if there is an alternative design, treatment or forecast AADT to be evaluated.

Steps 3 through 16 of the predictive method are repeated as appropriate for the same roadway limits but for alternative conditions, treatments, periods of interest, or forecast AADTs.

Step 18 – Evaluate and compare results.

The predictive method is used to provide a statistically reliable estimate of the expected average crash frequency within defined network or facility limits over a given period of time, for given geometric design and traffic control features, and known or estimated AADT. In addition to estimating total crashes, the estimate can be made for different crash severity types and different collision types. Default distributions of crash severity and collision type are provided with each SPF in Section 11.6. These default distributions can benefit from being updated based on local data as part of the calibration process presented in *Part C* Appendix.A.1.

11.5. ROADWAY SEGMENTS AND INTERSECTIONS

Section 11.4 provides an explanation of the predictive method. Section 11.5 through to Section 11.8 provide the specific detail necessary to apply the predictive method steps on rural multilane roads. Detail regarding the procedure for determining a calibration factor to apply in Step 11 is provided in the *Part C*

Appendix A.1. Detail regarding the EB Method, which is applied in Steps 6, 13, and 15, is provided in the *Part C* Appendix A.2.

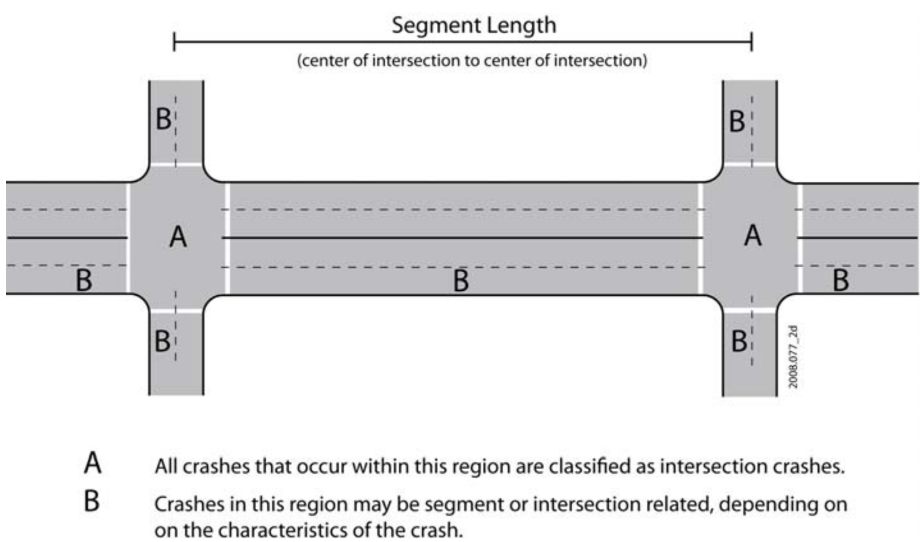
In Step 5 of the predictive method, the roadway within the defined roadway limits is divided into individual sites, which are homogenous roadway segments and intersections. A facility consists of a contiguous set of individual intersections and roadway segments, referred to as "sites." A roadway network consists of a number of contiguous facilities. Predictive models have been developed to estimate crash frequencies separately for roadway segments and intersections. The definitions of roadway segments and intersections presented below are the same as those for used in the FHWA Interactive Highway Safety Design Model (IHSDM)[2].

Roadway segments begin at the center of an intersection and end at either the center of the next intersection, or where there is a change from one homogeneous roadway segment to another homogenous segment. The roadway segment model estimates the frequency of roadway-segment-related crashes which occur in Region B in Exhibit 11-3. When a roadway segment begins or ends at an intersection, the length of the roadway segment is measured from the center of the intersection.

Chapter 11 provides predictive models for stop-controlled (three- and four-leg) and signalized (four-leg) intersections. The intersection models estimate the predicted average frequency of crashes that occur within the curbline limits of an intersection (Region A of Exhibit 11-3) and intersection-related crashes that occur on the intersection legs (Region B in Exhibit 11-3).

Exhibit 11-3: Definition of Segments and Intersections

The roadway segment model estimates the frequency of roadway segment related crashes which occur in Region B in Exhibit 11-3. The intersection models estimate the frequency of all crashes in Region A plus intersection-related crashes that occur in Region B.



The segmentation process produces a set of roadway segments of varying length, each of which is homogeneous with respect to characteristics such as traffic volumes, key roadway design characteristics, and traffic control features. Exhibit 11-3 shows the segment length, L, for a single homogenous roadway segment occurring between two intersections. However, it is likely that several homogenous roadway segments will occur between two intersections. A new (unique) homogeneous segment begins at the center of an intersection or where there is a change in at least one of the following characteristics of the roadway:

- Average annual daily traffic (vehicles per day)

- Presence of median and median width (feet)

The following rounded median widths are recommended before determining "homogeneous" segments:

Measured Median Width	Rounded Median Width
1-ft to 14-ft	10-ft
15-ft to 24-ft	20-ft
25-ft to 34-ft	30-ft
35-ft to 44-ft	40-ft
45-ft to 54-ft	50-ft
55-ft to 64-ft	60-ft
65-ft to 74-ft	70-ft
75-ft to 84-ft	80-ft
85-ft to 94-ft	90-ft
95 or more	100-ft

- Side slope (for undivided roadway segments)
- Shoulder type
- Shoulder width (feet)

For shoulder widths measures to a 0.1-ft level of precision or similar, the following rounded paved shoulder widths are recommended before determining "homogeneous" segments:

Measured Shoulder Width	Rounded Shoulder Width
0.5-ft or less	0-ft
0.6-ft to 1.5-ft	1-ft
1.6-ft to 2.5-ft	2-ft
2.6-ft to 3.5-ft	3-ft
3.6-ft to 4.5-ft	4-ft
4.6-ft to 5.5-ft	5-ft
5.6-ft to 6.5-ft	6-ft
6.6-ft to 7.5-ft	7-ft
7.6-ft or more	8-ft or more

- Lane width (feet)

For lane widths measured to a 0.1-ft level of precision or similar, the following rounded lane widths are recommended before determining "homogeneous" segments:

Measured Lane Width	Rounded Lane Width
9.2-ft or less	9-ft or less
9.3-ft to 9.7-ft	9.5-ft
9.8-ft to 10.2-ft	10-ft
10.3-ft to 10.7-ft	10.5-ft
10.8-ft to 11.2-ft	11-ft
11.3-ft to 11.7-ft	11.5-ft
11.8-ft or more	12-ft or more

- Presence of lighting
- Presence of automated speed enforcement

In addition, each individual intersection is treated as a separate site, for which the intersection-related crashes are estimated using the predictive method.

The methodology for assigning crashes to roadway segments and intersections for use in the site-specific EB Method is presented in Section A.2.3 in the Appendix to Part C.

There is no minimum roadway segment length, L, for application of the predictive models for roadway segments; However, as a practical matter, when dividing roadway facilities into small homogenous roadway segments, limiting the segment length to a minimum of 0.10 miles will minimize calculation efforts and not affect results.

In order to apply the site-specific EB Method, observed crashes are assigned to the individual roadway segments and intersections. Observed crashes that occur between intersections are classified as either intersection-related or roadway segment-related. The methodology for assignment of crashes to roadway segments and intersections for use in the site-specific EB Method is presented in Section A.2.3 in the Appendix to *Part C*.

11.6. SAFETY PERFORMANCE FUNCTIONS

A detailed discussion of SPFs and their use in the HSM is presented in Chapter 3 Section 3.5.2 and the Part C Introduction and Applications Guidance Section C.6.3

In Step 9 of the predictive method, the appropriate Safety Performance Functions (SPFs) are used to predict average crash frequency for the selected year for specific base conditions. SPFs are regression models for estimating the predicted average crash frequency of individual roadway segments or intersections. Each SPF in the predictive method was developed with observed crash data for a set of similar sites. The SPFs, like all regression models, estimate the value of a dependent variable as a function of a set of independent variables. In the SPFs developed for the HSM, the dependent variable estimated is the predicted average crash frequency for a roadway segment or intersection under base conditions and the independent variables are the AADTs of the roadway segment or intersection legs (and, for roadway segments, the length of the roadway segment).

The predicted crash frequencies for base conditions are calculated from the predictive method in Equations 11-2, 11-3 and 11-4. A detailed discussion of SPFs and their use in the HSM is presented in *Chapter 3* Section 3.5.2 and the *Part C Introduction and Applications Guidance* Section C.6.3.

Each SPF also has an associated overdispersion parameter, k. The overdispersion parameter provides an indication of the statistical reliability of the SPF. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. This parameter is used in the EB Method discussed in the *Part C* Appendix. The SPFs in Chapter 11 are summarized in Exhibit 11-4.

Exhibit 11-4: Safety Performance Functions included in Chapter 11

Chapter 11 SPFs for Rural Multilane Highways	SPF Equations and Exhibits
Undivided rural four-lane roadway segments	Equation 11-7, 11-8 , Exhibit 11-5, 11-6
Divided roadway segments	Equation 11-9, 11-10 , Exhibit 11-7, 11-8
Three and four-leg STOP controlled intersections	Equation 11-11 , Exhibit 11-11
Four-leg signalized intersections	Equation 11-11. 11-12 , Exhibit 11-11, 11-12

Some highway agencies may have performed statistically-sound studies to develop their own jurisdiction-specific SPFs derived from local conditions and crash experience. These models may be substituted for models presented in this chapter. Criteria for the development of SPFs for use in the predictive method are addressed in the calibration procedure presented in the Appendix to *Part C*.

Jurisdiction-specific SPFs can be used as substitutes to this chapter's models if statistically-sound models were developed consistent with HSM methods.

11.6.1. Safety Performance Functions for Undivided Roadway Segments

The predictive model for estimating predicted average crash frequency on a particular undivided rural multilane roadway segment was presented in Equation 11-2. The effect of traffic volume (AADT) on accident frequency is incorporated through the SPF, while the effects of geometric design and traffic control features are incorporated through the AMFs.

The base conditions of the SPF for undivided roadway segments on rural multilane highways are:

The base conditions for undivided rural multilane highways are summarized here.

- Lane width (LW) 12 feet
- Shoulder width 6 feet
- Shoulder type Paved
- Side slopes 1V:7H or flatter
- Lighting None
- Automated speed enforcement None

The SPF for undivided roadway segments on a rural multilane highway is shown in Equation 11-7 and presented graphically in Exhibit 11-6:

$$N_{spf\ ru} = e^{(a+b\times\ln(AADT)+\ln(L))} \qquad (11\text{-}7)$$

Where,

$N_{spf\ ru}$ = base total expected average crash frequency for a roadway segment;

AADT = annual average daily traffic (vehicles per day) on roadway segment;

L = length of roadway segment (miles);

a, b = regression coefficients.

Guidance on the estimation of traffic volumes for roadway segments for use in the SPFs is presented in Step 3 of the predictive method described in Section 11.4. The SPFs for undivided roadway segments on rural multilane highways are applicable to the AADT range from 0 to 33,200 vehicles per day. Application to sites with AADTs substantially outside this range may not provide accurate results.

The value of the overdispersion parameter associated with $N_{spf\ ru}$ is determined as a function of segment length. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. The value is determined as:

$$k = \frac{1}{e^{(c + ln(L))}} \qquad (11\text{-}8)$$

Where,

k = overdispersion parameter associated with the roadway segment;

L = length of roadway segment (miles);

c = a regression coefficient used to determine the overdispersion parameter.

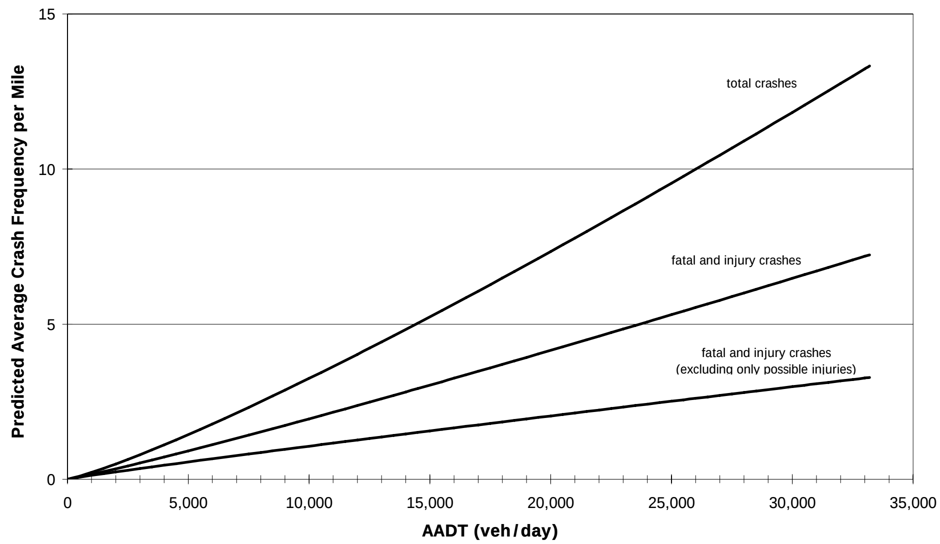
Exhibit 11-5 presents the values of the coefficients used for applying Equations 11-7 and 11-8 to determine the SPF for expected average crash frequency by total crashes, fatal and injury crashes, and fatal, injury and possible injury crashes.

Exhibit 11-5: SPF Coefficients for Total and Fatal-and-Injury Accidents on Undivided Roadway Segments (for use in Equations 11-7 and 11-8)

Crash Severity level	a	b	c
4-lane total	-9.653	1.176	1.675
4-lane fatal and injury	-9.410	1.094	1.796
4-lane fatal and injury[a]	-8.577	0.938	2.003

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

Exhibit 11-6: Graphical Form of the SPF for Undivided Roadway Segments (from Equation 11-7 and Exhibit 11-5)



The default proportions in Exhibit 11-5 are used to break down the accident frequencies from Equation 11-7 into specific collision types. To do so, the user multiplies the crash frequency for a specific severity level from Equation 11-7 by the appropriate collision type proportion for that severity level from Exhibit 11-7 to estimate the number of accidents for that collision type. Exhibit 11-7 is intended to separate the predicted frequencies for total accidents (all severity levels combined), fatal-and-injury accidents, and fatal-and-injury accidents (with possible injuries excluded) into components by collision type. Exhibit 11-7 cannot be used to separate predicted total accident frequencies into components by severity level. Ratios for PDO crashes are provided for application where the user has access to predictive models for that severity level. The default collision type proportions shown in Exhibit 11-7 may be updated with local data.

Procedures to develop local proportions of crash severity and collision type are provided in the Appendix to Part C.

There are a variety of factors that may affect the distribution of crashes among crash types and severity levels. To account for potential differences in these factors between jurisdictions, it is recommended that the values in Exhibit 11-7 be updated with local data. The values for total, fatal and injury, and fatal and injury (with possible injuries excluded) in this exhibit are used in the worksheets described in Appendix A.

Exhibit 11-7: Default Distribution of Crashes by Collision Type and Crash Severity Level for Undivided Roadway Segments

Collision type	Proportion of crashes by collision type and crash severity level			
	Severity level			
	Total	Fatal and injury	Fatal and injury[a]	PDO
Head-on	0.009	0.029	0.043	0.001
Sideswipe	0.098	0.048	0.044	0.120
Rear-end	0.246	0.305	0.217	0.220
Angle	0.356	0.352	0.348	0.358
Single	0.238	0.238	0.304	0.237
Other	0.053	0.028	0.044	0.064

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

Appendix B presents alternative SPFs that can be applied to predict accident frequencies for selected collision types for undivided roadway segments on rural multilane highways. Use of these alternative models may be considered when estimates are needed for a specific collision type rather than for all crash types combined. It should be noted that the alternative SPFs in Appendix B do not address all potential collision types of interest and there is no assurance that the estimates for individual collision types would sum to the estimate for all collision types combined provided by the models in Exhibit 11-5.

11.6.2. Safety Performance Functions for Divided Roadway Segments

The predictive model for estimating predicted average crash frequency on a particular divided rural multilane roadway segment was presented in Equation 11-3 in Section 11.3. The effect of traffic volume (AADT) on crash frequency is incorporated through the SPF, while the effects of geometric design and traffic control features are incorporated through the AMFs. The SPF for divided rural multilane highway segments is presented in this section. Divided rural multilane highway roadway segments are defined in Section 11.3.

Some divided highways have two roadways, built at different times, with independent alignments and distinctly different roadway characteristics, separated by a wide median. In this situation, it may be appropriate to apply the divided highway methodology twice, separately for the characteristics of each roadway but using the combined traffic volume, and then average the predicted accident frequencies.

The base conditions for divided rural multilane highways are summarized here.

The base conditions for the SPF for divided roadway segments on rural multilane highways are:

- Lane width (LW) 12 feet
- Right shoulder width 8 feet
- Median width 30 feet
- Lighting None
- Automated speed enforcement None

The SPF for expected average crash frequency for divided roadway segments on rural multilane highways is shown in Equation 11-9 and presented graphically in Exhibit 11-9:

$$N_{spf\ rd} = e^{(a+b\times\ln(AADT)+\ln(L))} \tag{11-9}$$

Where,

$N_{spf\ rd}$ = base total number of roadway segment accidents per year;

AADT = annual average daily traffic (vehicles/day) on roadway segment;

L = length of roadway segment (miles);

a, b = regression coefficients.

Guidance on the estimation of traffic volumes for roadway segments for use in the SPFs is presented in Step 3 of the predictive method described in Section 11.4. The SPFs for undivided roadway segments on rural multilane highways are applicable to the AADT range from 0 to 89,300 vehicles per day. Application to sites with AADTs substantially outside this range may not provide reliable results.

The value of the overdispersion parameter is determined as a function of segment length as:

$$k = \frac{1}{e^{(c+ln(L))}} \tag{11-10}$$

Where,

K = overdispersion parameter associated with the roadway segment;

L = length of roadway segment (mi); and

c = a regression coefficient used to determine the overdispersion parameter.

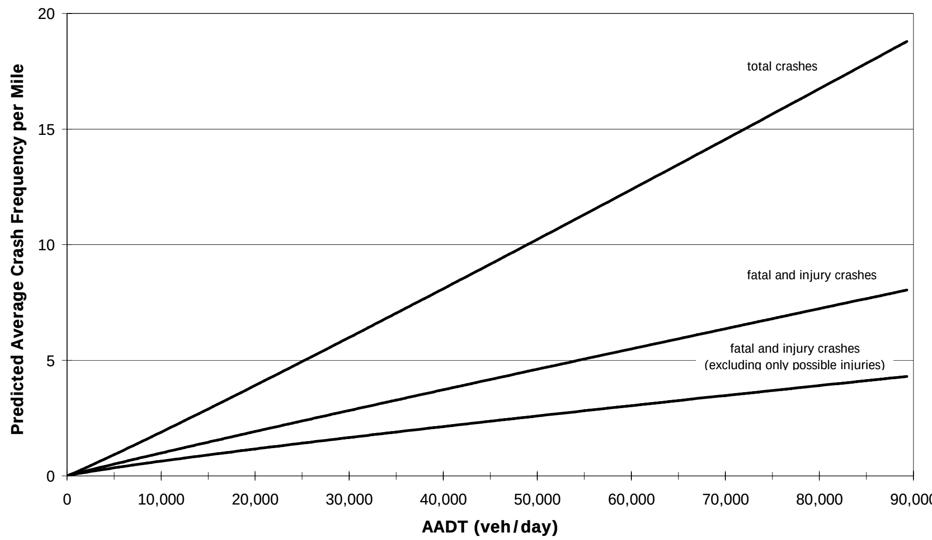
Exhibit 11-8 presents the values for the coefficients used in applying Equations 11-9 and 11-10.

Exhibit 11-8: SPF Coefficients for Total and Fatal-and-Injury Accidents on Divided Roadway Segments (for use in Equations 11-9 and 11-10)

Severity level	a	b	c
4-lane total	-9.025	1.049	1.549
4-lane fatal and injury	-8.837	0.958	1.687
4-lane fatal and injury[a]	-8.505	0.874	1.740

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

Exhibit 11-9: Graphical Form of SPF for Rural Multilane Divided Roadway Segments (from Equation 11-9 and Exhibit 11-8)



The default proportions in Exhibit 11-8 are used to break down the accident frequencies from Equation 11-9 into specific collision types. To do so, the user multiplies the accident frequency for a specific severity level from Equation 11-9 by the appropriate collision type proportion for that severity level from Exhibit 11-10 to estimate the number of accidents for that collision type. Exhibit 11-10 is intended to separate the predicted frequencies for total accidents (all severity levels combined), fatal-and-injury accidents, and fatal-and-injury accidents (with possible injuries excluded) into components by collision type. Exhibit 11-10 cannot be used to separate predicted total accident frequencies into components by severity level. Ratios for PDO crashes are provided for application where the user has access to predictive models for that severity level. The default collision type proportions shown in Exhibit 11-10 may be updated with local data.

Exhibit 11-10: Default Distribution of Crashes by Collision Type and Crash Severity Level for Divided Roadway Segments

Collision type	Proportion of crashes by collision type and crash severity level			
	Severity level			
	Total	Fatal and injury	Fatal and injury[a]	PDO
Head-on	0.006	0.013	0.018	0.002
Sideswipe	0.043	0.027	0.022	0.053
Rear-end	0.116	0.163	0.114	0.088
Angle	0.043	0.048	0.045	0.041
Single	0.768	0.727	0.778	0.792
Other	0.024	0.022	0.023	0.024

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

11.6.3. Safety Performance Functions for Intersections

The predictive model for estimating predicted average crash frequency at particular rural multilane intersection was presented in Equation 11-4. The effect of traffic volume (AADT) on accident frequency is incorporated through the SPF, while the effects of geometric design and traffic control features are incorporated through the AMFs. The SPFs for rural multilane highway intersection are presented in this section. Three and four-leg STOP controlled and four-leg signalized rural multilane highway intersections are defined in Section 11.3.

SPFs have been developed for three types of intersections on rural multilane highways. These models can be used for intersections located on both divided and undivided rural four-lane highways. The three types of intersections are:

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

The SPFs for four-leg signalized intersections (4SG) on rural multilane highways have no specific base conditions and, therefore, can only be applied for generalized predictions. No AMFs are provided for 4SG intersections and predictions of average crash frequency cannot be made for intersections with specific geometric design and traffic control features.

Models for three-leg signalized intersections on rural multilane roads are not available.

The SPFs for three- and four-leg stop-controlled intersections (3ST and 4ST) on rural multilane highways are applicable to the following base conditions:

The base conditions for three- and four-leg stop-controlled intersections on rural multilane highway are summarized here.

- Intersection skew angle 0°
- Intersection left-turn lanes 0, except on stop-controlled approaches
- Intersection right-turn lanes 0, except on stop-controlled approaches
- Lighting None

The SPFs for accident frequency have two alternative functional forms, shown in Equations 11-11 and 11-12, and presented graphically in Exhibit 11-13, 11-14 and 11-15 (for total crashes only):

$$N_{spfint} = exp[a + b \times ln(AADT_{maj}) + c \times ln(AADT_{min})] \qquad (11\text{-}11)$$

or

$$N_{spfint} = exp[a + d \times ln(AADT_{tot})] \qquad (11\text{-}12)$$

Where,

$N_{spf\,int}$ = SPF estimate of intersection-related expected average crash frequency for base conditions;

$AADT_{maj}$ = AADT (vehicles per day) for major road approaches;

$AADT_{min}$ = AADT (vehicles per day) for minor road approaches;

$AADT_{tot}$ = AADT (vehicles per day) for minor and major roads combined approaches;

a, b, c, d = regression coefficients.

The functional form shown in Equation 11-11 is used for most site types and crash severity levels; the functional form shown in Equation 11-12 is used for only one specific combination of site type and facility type – four-leg signalized intersections for fatal-and-injury accidents (excluding possible injuries) – as shown in Exhibit 11-12.

Guidance on the estimation of traffic volumes for the major- and minor road legs for use in the SPFs is presented in Step 3 of the predictive method described in Section 11.4. The intersection SPFs for rural multilane highways are applicable to the following AADT ranges:

- 3ST: $AADT_{maj}$ 0 to 78,300 vehicles per day and
  $AADT_{min}$ 0 to 23,000 vehicles per day
- 4ST: $AADT_{maj}$ 0 to 78,300 vehicles per day and
  $AADT_{min}$ 0 to 7,400 vehicles per day
- 4SG: $AADT_{maj}$ 0 to 43,500 vehicles per day and
  $AADT_{min}$ 0 to 18,500 vehicles per day

Application to sites with AADTs substantially outside these ranges may not provide reliable results.

Exhibit 11-11 presents the values of the coefficients a, b, and c used in applying Equation 11-11 for stop-controlled intersections, along with the overdispersion parameter and the base conditions.

Exhibit 11-12 presents the values of the coefficients a, b, c, and d used in applying Equations 11-11 and 11-12 for four-leg signalized intersections, along with the overdispersion parameter. Coefficients a, b, and c are provided for total accidents and are applied to the SPF shown in Equation 11-11. Coefficients a and d are provided for injury accidents and are applied to the SPF shown in Equation 11-12. SPFs for three-leg signalized intersections on rural multilane roads are not currently available.

Separate calibration of the models in Exhibits 11-11 and 11-12 for application to intersections on undivided and divided roadway segments would be desirable, if feasible. Calibration procedures are presented in the Appendix to *Part C*.

Exhibit 11-11: SPF Coefficients for Three- and Four-leg Intersections with Minor road Stop Control for Total and Fatal-and-Injury Accidents (for use in Equation 11-11)

Intersection type/severity level	a	b	c	Overdispersion parameter (fixed k)[a]
4ST Total	-10.008	0.848	0.448	0.494
4ST Fatal and injury	-11.554	0.888	0.525	0.742
4ST Fatal and injury[b]	-10.734	0.828	0.412	0.655
3ST Total	-12.526	1.204	0.236	0.460
3ST Fatal and injury	-12.664	1.107	0.272	0.569
3ST Fatal and injury[b]	-11.989	1.013	0.228	0.566

NOTE: [a] This value should be used directly as the overdispersion parameter; no further computation is required.
[b] Using the KABCO scale, these include only KAB accidents. Crashes with severity level C (possible injury) are not included.

Exhibit 11-12: SPF Coefficients for Four-leg Signalized Intersections for Total and Fatal-and-Injury Accidents (for use in Equations 11-11 and 11-12)

Intersection type/ severity level	a	b	c	d	Overdispersion parameter (fixed k)[a]
4SG Total	-7.182	0.722	0.337		0.277
4SG Fatal and injury	-6.393	0.638	0.232		0.218
4SG Fatal and injury[b]	-12.011			1.279	0.566

NOTE: [a] This value should be used directly as the overdispersion parameter; no further computation is required.
[b] Using the KABCO scale, these include only KAB accidents. Crashes with severity level C (possible injury) are not included.

Exhibit 11-13: Graphical Form of SPF for three-leg STOP-controlled Intersections - for Total Crashes Only (from Equation 11-11 and Exhibit 11-11)

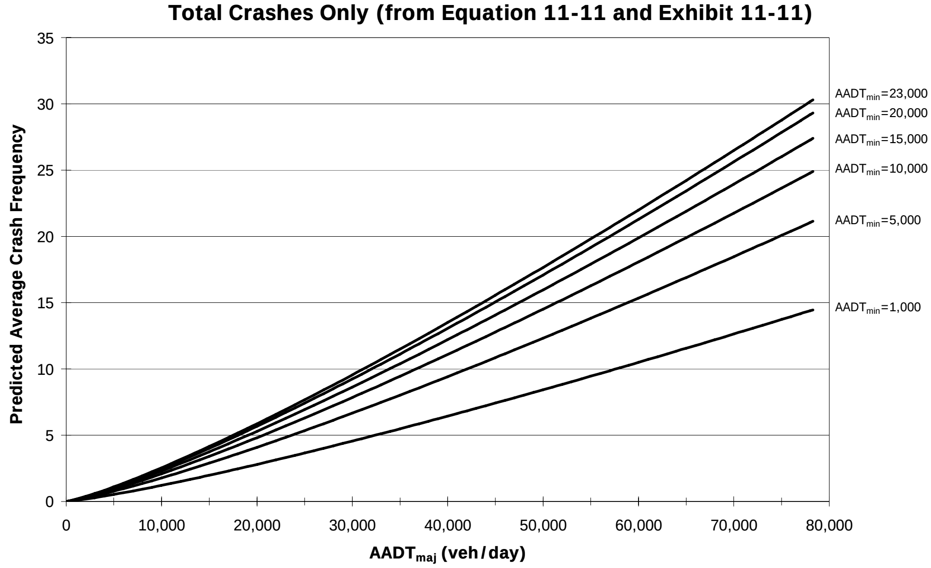


Exhibit 11-14: Graphical Form of SPF for Four-leg STOP-controlled Intersections - for Total Crashes only (from Equation 11-11 and Exhibit 11-11)

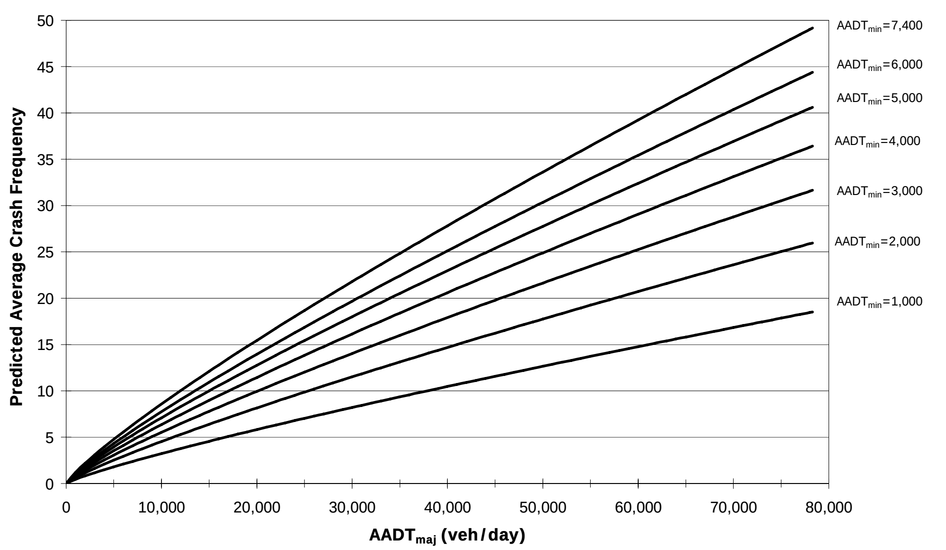
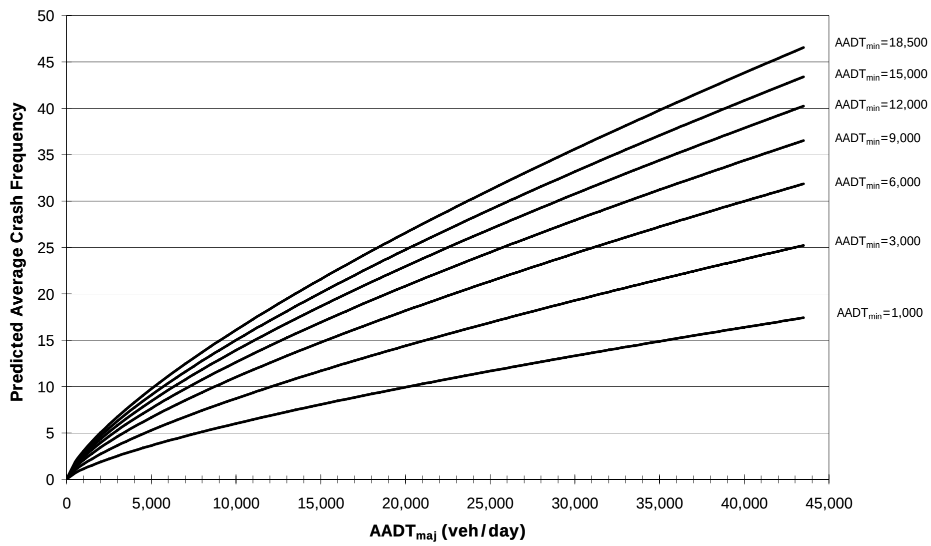


Exhibit 11-15: Graphical Form of SPF for Four-leg Signalized Intersections - for Total Crashes only (from Equation 11-11 and Exhibit 11-11)



The default proportions in Exhibit 11-16 are used to break down the accident frequencies from Equation 11-11 into specific collision types. To do so the user multiplies the predicted average frequency for a specific crash severity level from Equation 11-11 by the appropriate collision type proportion for that crash severity level from Exhibit 11-16 to estimate the predicted average crash frequency for that collision type. Exhibit 11-16 separates the predicted frequencies for total accidents (all severity levels combined), fatal-and-injury accidents, and fatal-and-injury accidents (with possible injuries excluded) into components by collision type. Exhibit 11-16 cannot be used to separate predicted total accident frequencies into components by crash severity level. Ratios for PDO crashes are provided for application where the user has access to predictive models for that crash severity level. The default collision type proportions shown in Exhibit 11-16 may be updated with local data.

There are a variety of factors that may affect the distribution of crashes among crash types and crash severity levels. To account for potential differences in these factors between jurisdictions, it is recommended that the values in Exhibit 11-16 be updated with local data. The values for total, fatal and injury, and fatal and injury (excluding accidents involving only possible injuries) in this exhibit are used in the worksheets described in Appendix A.

Exhibit 11-16: Default Distribution of Intersection Crashes by Collision Type and Crash Severity

Collision type	Proportion of crashes by severity level							
	Three-leg intersections with minor road stop control				Four-leg intersections with minor road stop control			
	Total	Fatal and injury	Fatal and injury[a]	PDO	Total	Fatal and injury	Fatal and injury[a]	PDO
Head-on	0.029	0.043	0.052	0.020	0.016	0.018	0.023	0.015
Sideswipe	0.133	0.058	0.057	0.179	0.107	0.042	0.040	0.156
Rear-end	0.289	0.247	0.142	0.315	0.228	0.213	0.108	0.240
Angle	0.263	0.369	0.381	0.198	0.395	0.534	0.571	0.292
Single	0.234	0.219	0.284	0.244	0.202	0.148	0.199	0.243
Other	0.052	0.064	0.084	0.044	0.051	0.046	0.059	0.055
Collision type	**Three-leg signalized intersections**				**Four-leg signalized intersections**			
	Total	**Fatal and injury**	**Fatal and injury[a]**	**PDO**	**Total**	**Fatal and injury**	**Fatal and injury[a]**	**PDO**
Head-on	--	--	--	--	0.054	0.083	0.093	0.034
Sideswipe	--	--	--	--	0.106	0.047	0.039	0.147
Rear-end	--	--	--	--	0.492	0.472	0.314	0.505
Angle	--	--	--	--	0.256	0.315	0.407	0.215
Single	--	--	--	--	0.062	0.041	0.078	0.077
Other	--	--	--	--	0.030	0.041	0.069	0.023

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

Appendix B presents alternative SPFs that can be applied to predict accident frequencies for selected collision types for intersections with minor road stop control on rural multilane highways. Use of these alternative models may be considered when safety predictions are needed for a specific collision type rather than for all crash types combined. Care must be exercised in using the alternative SPFs in Appendix B because they do not address all potential collision types of interest and because there is no assurance that the safety predictions for individual collision types would sum to the predictions for all collision types combined provided by the models in Exhibit 11-11.

11.7. ACCIDENT MODIFICATION FACTORS

In Step 10 of the predictive method shown in Section 11.4, Accident Modification Factors are applied to the selected Safety Performance Function, which was selected in Step 9. SPFs provided in Chapter 11 are presented in Section 11.6. A general overview of Accident Modification Factors (AMFs) is presented in *Chapter 3* Section 3.5.3. The *Part C Introduction and Applications Guidance* provides further discussion on the relationship of AMFs to the predictive method. This section provides details of the specific AMFs applicable to the Safety Performance Functions presented in Section 11.6.

A general overview of Accident Modification Factors (AMFs) is presented in Chapter 3 Section 3.5.3.

Accident Modification Factors (AMFs) are used to adjust the SPF estimate of expected average crash frequency for the effect of individual geometric design and traffic control features, as shown in the general predictive model for Chapter 11 shown in Equation 11-1. The AMF for the SPF base condition of each geometric design or traffic control feature has a value of 1.00. Any feature associated with higher average crash frequency than the SPF base condition has an AMF with a value greater than 1.00; any feature associated with lower average crash frequency than the SPF base condition has an AMF with a value less than 1.00.

The AMFs in Chapter 11 were determined from a comprehensive literature review by an expert panel(5). They represent the collective judgment of the expert panel concerning the effects of each geometric design and traffic control feature of interest. Others were derived by modeling data assembled for developing the predictive models rural multilane roads. The AMFs used in Chapter 11 are consistent with the AMFs in the *Part D*, although they have, in some cases, been expressed in a different form to be applicable to the base conditions. The AMFs presented in Chapter 11 and, the specific SPFs to which they apply, are summarized in Exhibit 11-17.

Summary of AMFs in Chapter 11 and the corresponding SPFs.

Exhibit 11-17: Summary of AMFs in Chapter 11 and the Corresponding SPFs

Applicable SPF	AMF	AMF Description	AMF Equations and Exhibits
Undivided Roadway Segment SPF	AMF_{1ru}	Lane Width on Undivided Segments	Equation 11-13, Exhibit 11-18, 11-19
	AMF_{2ru}	Shoulder Width and Shoulder Type	Equation 11-14, Exhibit 11-20, 11-21,11-22
	AMF_{3ru}	Side Slopes	Exhibit 11-23
	AMF_{4ru}	Lighting	Equation 11-15, Exhibit 11-24
	AMF_{5ru}	Automated Speed Enforcement	See text
Divided Roadway Segment SPF	AMF_{1rd}	Lane Width on Divided Segments	Equation 11-16, Exhibit 11-25, 11-26
	AMF_{2rd}	Right Shoulder Width on Divided Roadway Segment	Exhibit 11-27
	AMF_{3rd}	Median Width	Exhibit 11-28
	AMF_{4rd}	Lighting	Equation 11-17, Exhibit 11-29
	AMF_{5rd}	Automated Speed Enforcement	See text
Three- and four-leg STOP-controlled Intersection SPFs	AMF_{1i}	Intersection Angle	Exhibit 11-30, 11-31
	AMF_{2i}	Left-Turn Lane on Major Road	Exhibit 11-30, 11-31
	AMF_{3i}	Right-Turn Lane on Major Road	Exhibit 11-30, 11-31
	AMF_{4i}	Lighting	Exhibit 11-30, 11-31

11.7.1. Accident Modification Factors for Undivided Roadway Segments

Section 11.7.1 provides the AMFs to be used with undivided roadway segments.

The AMFs for geometric design and traffic control features of undivided roadway segments are presented below. These AMFs are applicable to the SPF presented in Section 11.6.1 for undivided roadway segments on rural multilane highways. Each of the AMFs applies to all of the crash severity levels shown in Exhibit 11-5.

AMF_{1ru} - Lane Width

The first of five AMFs for use on undivided roadway segments is an AMF for lane width.

The AMF for lane width on undivided segments is based on the work of Harkey et al.[3] and is determined as follows:

$$AMF_{1ru} = (AMF_{RA} - 1.0) \times p_{RA} + 1.0 \qquad (11\text{-}13)$$

Where,

AMF_{1ru} = Accident Modification Factor for total accidents

AMF_{RA} = Accident Modification Factor for related accidents (run-off-the-road, head-on, and sideswipe), from Exhibit 11-18

p_{RA} = proportion of total accidents constituted by related accidents (default is 0.27)

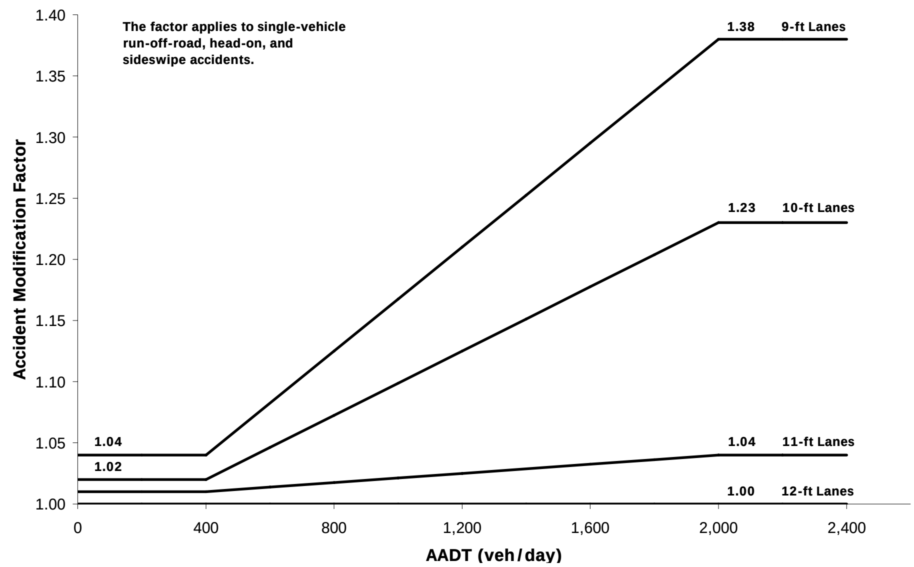
AMF_{RA} is determined from Exhibit 11-18 based on the applicable lane width and traffic volume range. The relationships shown in Exhibit 11-18 are illustrated in Exhibit 11-19. This effect represents 75% of the effect of lane width on rural two-lane roads shown in *Chapter 10*. The default value of p_{RA} for use in Equation 11-13 is 0.27, which indicates that run-off-road, head-on, and sideswipe accidents typically represent 27% of total accidents. This default value may be updated based on local data. The SPF base condition for the lane width is 12-ft. Where the lane widths on a roadway vary, the AMF is determined separately for the lane width in each direction of travel and the resulting AMFs are then averaged.

For lane widths with 0.5-ft increments that are not depicted specifically in Exhibit 11-18 or in Exhibit 11-19, an AMF value can be interpolated using either of these exhibits since there is a linear transition between the various AADT effects.

Exhibit 11-18: AMF_{RA} for Collision Types Related to Lane Width

Lane Width	Average Annual Daily Traffic (AADT) (vehicles/day)		
	< 400	400 to 2000	> 2000
9-ft or less	1.04	$1.04+2.13\times10^{-4}$(AADT-400)	1.38
10-ft	1.02	$1.02+1.31\times10^{-4}$(AADT-400)	1.23
11-ft	1.01	$1.01+1.88\times10^{-5}$(AADT-400)	1.04
12-ft or more	1.00	1.00	1.00

Exhibit 11-19: AMF_{RA} for Lane Width on Undivided Segments



AMF_{2ru} - Shoulder Width

The second of five AMFs for use on undivided roadway segments is an AMF for shoulder width and type.

The AMF for shoulder width on undivided segments is based on the work of Harkey et al. [3] and is determined as follows:

$$AMF_{2ru} = (AMF_{WRA} \times AMF_{TRA} - 1.0) \times p_{RA} + 1.0 \qquad (11\text{-}14)$$

Where,

AMF_{2ru} = Accident Modification Factor for total accidents

AMF_{WRA} = Accident Modification Factor for related accidents based on shoulder width from Exhibit 11-20

AMF_{TRA} = Accident Modification Factor for related accidents based on shoulder type from Exhibit 11-22

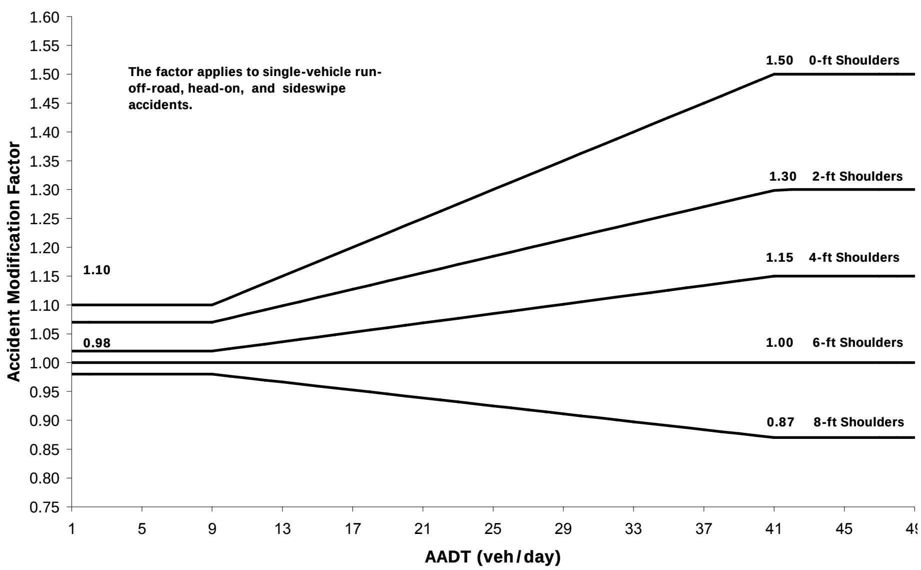
p_{RA} = proportion of total accidents constituted by related accidents (default is 0.27)

AMF_{WRA} is determined from Exhibit 11-20 based on the applicable shoulder width and traffic volume range. The relationships shown in Exhibit 11-20 are illustrated in Exhibit 11-21. The default value of p_{RA} for use in Equation 11-14 is 0.27, which indicates that run-off-road, head-on, and sideswipe accidents typically represent 27% of total accidents. This default value may be updated based on local data. The SPF base condition for shoulder width is 6-ft.

Exhibit 11-20: AMF for Collision Types Related to Shoulder Width (AMF_{WRA})

Shoulder Width	Annual Average Daily Traffic (AADT) (vehicles/day)		
	< 400	400 to 2000	> 2000
0-ft	1.10	$1.10 + 2.5 \times 10^{-4}$ (AADT - 400)	1.50
2-ft	1.07	$1.07 + 1.43 \times 10^{-4}$ (AADT - 400)	1.30
4-ft	1.02	$1.02 + 8.125 \times 10^{-5}$ (AADT - 400)	1.15
6-ft	1.00	1.00	1.00
8-ft or more	0.98	$0.98 + 6.875 \times 10^{-5}$ (AADT - 400)	0.87

Exhibit 11-21: AMF_{WRA} for Shoulder Width on Undivided Segments



AMF_{TRA} is determined from Exhibit 11-22 based on the applicable shoulder type and shoulder width.

Exhibit 11-22: AMF for Collision Types Related to Shoulder Type and Shoulder Width (AMF_{TRA})

Shoulder Type	Shoulder Width (ft)							
	0	1	2	3	4	6	8	10
Paved	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Gravel	1.00	1.00	1.01	1.01	1.01	1.02	1.02	1.03
Composite	1.00	1.01	1.02	1.02	1.03	1.04	1.06	1.07
Turf	1.00	1.01	1.03	1.04	1.05	1.08	1.11	1.14

If the shoulder types and/or widths for the two directions of a roadway segment differ, the AMF is determined separately for the shoulder type and width in each direction of travel and the resulting AMFs are then averaged.

AMF_{3ru} - Side Slopes

The third of five AMFs for use on undivided roadway segments is an AMF for slide slopes.

An AMF for the side slope for undivided roadway segments of rural multilane highways has been developed by Harkey et al.[3] from the work of Zegeer et al.[8] The AMF is presented in Exhibit 11-23. The base conditions are for a side slope of 1:7 or flatter.

Exhibit 11-23: AMF for Side Slope on Undivided Roadway Segments (AMF_{3ru})

1:2 or Steeper	1:4	1:5	1:6	1:7 or Flatter
1.18	1.12	1.09	1.05	1.00

AMF_{4ru} - Lighting

The fourth of five AMFs for use on undivided roadway segments is an AMF for lighting.

The SPF base condition for lighting of roadway segments is the absence of lighting. The AMF for lighted roadway segments is determined, based on the work of Elvik and Vaa [1], as:

$$AMF_{4ru} = 1 - [(1 - 0.72 \times p_{inr} - 0.83 \times p_{pnr}) \times p_{nr}] \qquad (11\text{-}15)$$

Where,

AMF_{4ru} = Accident Modification Factor for the effect of lighting on total accidents;

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

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

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

This AMF applies to total roadway segment accidents. Exhibit 11-24 presents default values for the nighttime accident proportions p_{inr}, p_{pnr}, and p_{nr}. HSM users are encouraged to replace the estimates in Exhibit 11-24 with locally derived values.

Exhibit 11-24: Night-time Accident Proportions for Unlighted Roadway Segments

Roadway Type	Proportion of total night-time accidents by severity level		Proportion of accidents that occur at night
	Fatal and injury p_{inr}	PDO p_{pnr}	p_{nr}
4U	0.361	0.639	0.255

AMF_{5ru} - Automated Speed Enforcement

The fifth of five AMFs for use on undivided roadway segments is an AMF for automated speed enforcement.

Automated speed enforcement systems use video or photographic identification in conjunction with radar or lasers to detect speeding drivers. These systems automatically record vehicle identification information without the need for police officers at the scene. The SPF base condition for automated speed enforcement is that it is absent. *Chapter 17* presents an AMF of 0.83 for the reduction of all types of injury accidents from implementation of automated speed enforcement. This AMF applies to roadway segments with fixed camera sites where the camera is always present or where drivers have no way of knowing whether the camera is present or not. Fatal and injury accidents constitute 31% of total accidents on rural two-lane highway segments. No information is available on the effect of automated speed enforcement on noninjury accidents. With the conservative assumption that automated speed enforcement has no effect on noninjury crashes, the value of AMF_{5ru} for automated speed enforcement would be 0.95 based on the injury accident proportion.

11.7.2. Accident Modification Factors for Divided Roadway Segments

The AMFs for geometric design and traffic control features of divided roadway segments for rural multilane highways are presented below. Each of the AMFs applies to all of the crash severity levels shown in Exhibit 11-8.

Section 11.7.2 presents AMFs for divided roadway segments on rural multilane highways.

AMF_{1rd} - Lane Width on Divided Roadway Segments

The AMF for lane width on divided segments is based on the work of Harkey et al.[3] and is determined as follows:

The first of five AMFs for divided roadway segments is an AMF for lane width.

$$AMF_{1rd} = (AMF_{RA} - 1.0) \times p_{RA} + 1.0 \qquad (11\text{-}16)$$

Where,

AMF_{1rd} = Accident Modification Factor for total accidents

AMF_{RA} = Accident Modification Factor for related accidents (run-off-the-road, head-on, and sideswipe), from Exhibit 11-25

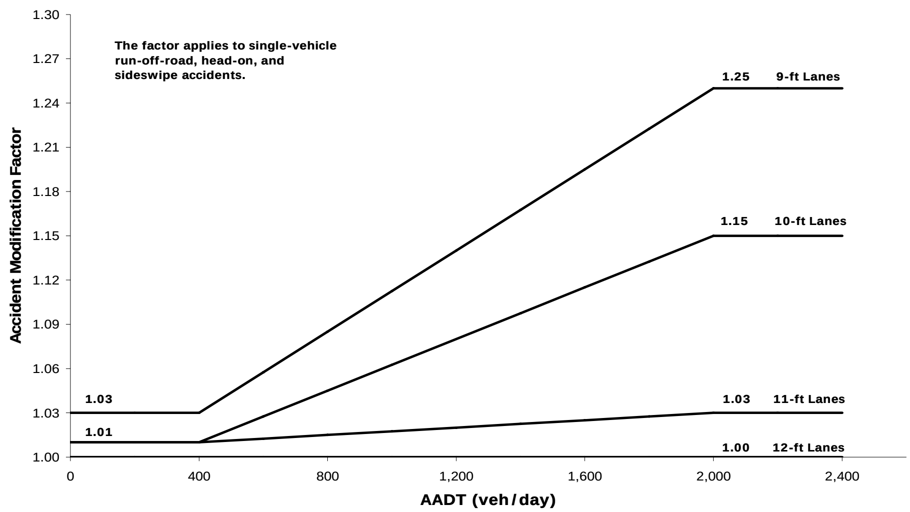
p_{RA} = proportion of total accidents constituted by related accidents (default is 0.50)

AMF_{RA} is determined from Exhibit 11-25 based on the applicable lane width and traffic volume range. The relationships shown in Exhibit 11-25 are illustrated in Exhibit 11-26. This effect represents 50% of the effect of lane width on rural two-lane roads shown in *Chapter 10*. The default value of p_{RA} for use in Equation 11-16 is 0.50, which indicates that run-off-road, head-on, and sideswipe accidents typically represent 50% of total accidents. This default value may be updated based on local data. The SPF base condition for lane width is 12-ft. Where the lane widths on a roadway vary, the AMF is determined separately for the lane width in each direction of travel and the resulting AMFs are then averaged.

Exhibit 11-25: AMF for Collision Types Related to Lane Width (AMF_{RA})

Lane Width	Annual Average Daily Traffic (AADT) (vehicles/day)		
	< 400	400 to 2000	> 2000
9-ft	1.03	$1.03+1.38\times10^{-4}$(AADT-400)	1.25
10-ft	1.01	$1.01+8.75\times10^{-5}$(AADT-400)	1.15
11-ft	1.01	$1.01+1.25\times10^{-5}$(AADT-400)	1.03
12-ft	1.00	1.00	1.00

Exhibit 11-26: AMF_{RA} for Lane Width on Divided Roadway Segments



AMF_{2rd} - Right Shoulder Width on Divided Roadway Segments

The second of five AMFs for divided roadway segments is an AMF for right shoulder width.

The AMF for right shoulder width on divided roadway segments was developed by Lord et al.[5] and is presented in Exhibit 11-27. The SPF base condition for the right shoulder width variable is 8 feet. If the shoulder widths for the two directions of travel differ, the AMF is based on the average of the shoulder widths. The safety effects of shoulder widths wider than 8-ft are unknown, but it is recommended that an AMF of 1.00 be used in this case.

The effects of unpaved right shoulders on divided roadway segments and of left (median) shoulders of any width or material are unknown. No AMFs are available for these cases.

Exhibit 11-27: AMF for Right Shoulder Width on Divided Roadway Segments (AMF_{2rd})

Average Shoulder Width (ft)				
0	**2**	**4**	**6**	**8 or more**
1.18	1.13	1.09	1.04	1.00

NOTE: This AMF applies to paved shoulders only.

AMF_{3rd} - Median Width

The third of five AMFs for divided roadway segments is an AMF for median width.

An AMF for median widths on divided roadway segments of rural multilane highways is presented in Exhibit 11-28 based on the work of Harkey et al.[3] The median width of a divided highway is measure d between the inside edges of the through travel lanes in the opposing direction of travel; thus, inside shoulder and turning lanes are included in the median width. The base condition for this AMF is a median width of 30-ft. The AMF applies to total crashes, but represents the effect of median width in reducing cross-median collisions; the AMF assumes that nonintersection collision types other than cross-median collisions are not affected by median width. The AMF in Exhibit 11-28 has been adapted from the AMF in Exhibit 13-15 based on the estimate by Harkey et al.[3] that cross-median collisions represent 12.2% of crashes on multilane divided highways.

This AMF applies only to traversable medians without traffic barriers. The effect of traffic barriers on safety would be expected to be a function of the barrier type and offset, rather than the median width; however, the effects of these factors on safety have not been quantified. Until better information is available, an AMF value of 1.00 is used for medians with traffic barriers.

Exhibit 11-28: AMFs for Median Width on Divided Roadway Segments without a Median Barrier (AMF_{3rd})

Median width (ft)	AMF
10	1.04
20	1.02
30	1.00
40	0.99
50	0.97
60	0.96
70	0.96
80	0.95
90	0.94
100	0.94

NOTE: This AMF applies only to medians without traffic barriers.

AMF_{4rd} - Lighting

The fourth of five AMFs for divided roadway segments is an AMF for lighting.

The SPF base condition for lighting is the absence of roadway segment lighting. The AMF for lighted roadway segments is determined, based on the work of Elvik and Vaa [1], as:

$$AMF_{5rd} = 1 - [(1 - 0.72 \times p_{inr} - 0.83 \times p_{pnr}) \times p_{nr}] \qquad (11\text{-}17)$$

Where,

AMF_{5rd} = Accident Modification Factor for the effect of lighting on total accidents;

p_{inr} = proportion of total night-time accidents for unlighted roadway segments that involve a fatality or injury;

p_{pnr} = proportion of total night-time accidents for unlighted roadway segments that involve property damage only;

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

This AMF applies to total roadway segment accidents. Exhibit 11-29 presents default values for the nighttime accident proportions p_{inr}, p_{pnr}, and p_{nr}. HSM users are encouraged to replace the estimates in Exhibit 11-29 with locally derived values.

Exhibit 11-29: Nighttime Accident Proportions for Unlighted Roadway Segments

Roadway Type	Proportion of total nighttime accidents by severity level		Proportion of accidents that occur at night
	Fatality and injury p_{inr}	PDO p_{pnr}	p_{nr}
4D	0.323	0.677	0.426

AMF_{5rd} - Automated Speed Enforcement

The fifth of five AMFs for divided roadway segments is an AMF for automated speed enforcement.

Automated speed enforcement systems use video or photographic identification in conjunction with radar or lasers to detect speeding drivers. These systems automatically record vehicle identification information without the need for police officers at the scene. The SPF base condition for automated speed enforcement is that it is absent. *Chapter 17* presents an AMF of 0.83 for the reduction of all types of fatal and injury accidents from implementation of automated speed enforcement. This AMF applies to roadway segments with fixed camera sites where the camera is always present or where drivers have no way of knowing whether the camera is present or not. Fatal and injury accidents constitute 37% of total accidents on rural multilane divided highway segments. No information is available on the effect of automated speed enforcement on noninjury accidents. With the conservative assumption that automated speed enforcement has no effect on noninjury crashes, the value of AMF_{5ru} for automated speed enforcement would be 0.94 based on the injury accident proportion.

11.7.3. Accident Modification Factors for Intersections

Section 11.7.3 presents AMFs for intersections on rural multilane highways.

The effects of individual geometric design and traffic control features of intersections are represented in the safety prediction procedure by AMFs. The equations and exhibits relating to AMFs for stop-controlled intersections are summarized in Exhibits 11-30 and 11-31 and presented below. Except where separate AMFs by crash severity level are shown, each of the AMFs applies to all of the crash severity levels shown in Exhibit 11-11. As noted earlier, AMFs are not available for signalized intersections.

Exhibit 11-30: AMFs for Three-leg Intersections with Minor Road Stop Control (3ST)

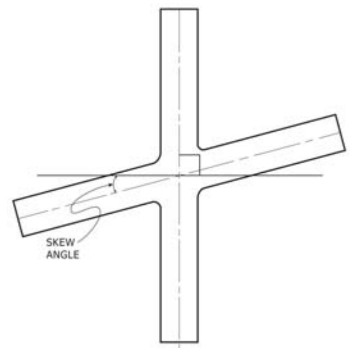
AMFs	Total	Fatal and injury
Intersection Angle	Equation 11-18	Equation 11-19
Left-Turn Lane on Major Road	Exhibit 11-32	Exhibit 11-32
Right-Turn Lane on Major Road	Exhibit 11-33	Exhibit 11-33
Lighting	Equation 11-22	Equation 11-22

Exhibit 11-31: AMFs for Four-leg Intersection with Minor Road Stop Control (4ST)

AMFs	Total	Fatal and injury
Intersection Angle	Equation 11-20	Equation 11-21
Left-turn Lane on Major Road	Exhibit 11-32	Exhibit 11-32
Right-turn Lane on Major Road	Exhibit 11-33	Exhibit 11-33
Lighting	Equation 11-22	Equation 11-22

AMF_{1i} - Intersection Skew Angle

The first of four AMFs for intersections is an AMF for intersection skew angle.



The SPF base condition for intersection skew angle is 0 degrees of skew (i.e., an intersection angle of 90 degrees). Reducing the skew angle of three- or four-leg stop-controlled intersections on rural multilane highways reduces total intersection accidents, as shown below. The skew angle is the deviation from an intersection angle of 90 degrees. Skew carries a positive or negative sign that indicates whether the minor road intersects the major road at an acute or obtuse angle, respectively

Three-Leg Intersections with Stop-Control on the Minor Approach

The AMF for total crashes for intersection skew angle at three-leg intersections with STOP-control on the minor approach is:

$$AMF_{1i} = \frac{0.016 \times SKEW}{(0.98 + 0.16 \times SKEW)} + 1.0 \qquad (11\text{-}18)$$

and the AMF for fatal-and-injury crashes is:

$$AMF_{1i} = \frac{0.017 \times SKEW}{(0.52 + 0.17 \times SKEW)} + 1.0 \qquad (11\text{-}19)$$

Where,

AMF_{1i}= Accident Modification Factor for the effect of intersection skew on total accidents;

SKEW = intersection skew angle (in degrees); the absolute value of the difference between 90 degrees and the actual intersection angle.

Four-Leg Intersections with Stop-Control on the Minor Approaches

The AMF for total crashes for intersection angle at four-leg intersection with STOP-control on the minor approaches is:

$$AMF_{1i} = \frac{0.053 \times SKEW}{(1.43 + 0.53 \times SKEW)} + 1.0 \qquad (11\text{-}20)$$

The AMF for fatal-and-injury crashes is:

$$AMF_{1i} = \frac{0.048 \times SKEW}{(0.72 + 0.48 \times SKEW)} + 1.0 \qquad (11\text{-}21)$$

The second of four AMFs for intersections is an AMF for intersection left-turn lanes.

AMF_{2i} - Intersection Left-Turn Lanes

The SPF base condition for intersection left-turn lanes is the absence of left-turn lanes on all of the intersection approaches. The AMFs for presence of left-turn lanes are presented in Exhibit 11-32 for total crashes and injury crashes. These AMFs apply only on uncontrolled major road approaches to STOP-controlled intersections. The AMFs for installation of left-turn lanes on multiple approaches to an intersection are equal to the corresponding AMF for installation of a left-turn lane on one approach raised to a power equal to the number of approaches with left-turn lanes (i.e., the AMFs are multiplicative and Equation 3-7 can be used). There is no indication of any effect of providing a left-turn lane on an approach controlled by a STOP sign, so the presence of a left-turn lane on a stop-controlled approach is not considered in applying Exhibit 11-32. The AMFs for installation of left-turn lanes are based on research by Harwood et al.[4] and are consistent with the AMFs presented in *Chapter 14*. An AMF of 1.00 is used when no left-turn lanes are present.

Exhibit 11-32: Accident Modification Factors (AMF_{2i}) for Installation of Left-Turn Lanes on Intersection Approaches.

Intersection type	Crash Severity Level	Number of non-stop-controlled approaches with left-turn lanes [a]	
		One approach	Two approaches
Three-leg minor road STOP control[b]	Total	0.56	-
	Fatal and Injury	0.45	-
Four-leg minor road STOP control[b]	Total	0.72	0.52
	Fatal and Injury	0.65	0.42

NOTE: [a] STOP-controlled approaches are not considered in determining the number of approaches with left-turn lanes

[b] STOP signs present on minor road approaches only.

The third of four AMFs for intersections is an AMF for intersection right-turn lanes.

AMF_{3i} - Intersection Right-Turn Lanes

The SPF base condition for intersection right-turn lanes is the absence of right-turn lanes on the intersection approaches. The AMFs for the presence of right-turn lanes are based on research by Harwood et al.[4] and are consistent with the AMFs in *Chapter 14*. These AMFs apply to installation of right-turn lanes on any approach to a signalized intersection, but only on uncontrolled major road approaches to stop-controlled intersections. The AMFs for installation of right-turn lanes on multiple approaches to an intersection are equal to the corresponding AMF for installation of a right-turn lane on one approach raised to a power equal to the number of approaches with right-turn lanes (i.e., the AMFs are multiplicative and Equation 3-7 can be used). There is no indication of any safety effect for providing a right-turn lane on an approach controlled by a STOP sign, so the presence of a right-turn lane on a stop-controlled approach is not considered in applying Exhibit 11-33. The AMFs for presence of right-turn lanes are presented in Exhibit 11-33 for total crashes and injury crashes. An AMF value of 1.00 is used when no right-turn lanes are present. This AMF applies only to right-turn lanes that are identified by marking or signing. The AMF is not applicable to long tapers, flares, or paved shoulders that may be used informally by right-turn traffic.

Exhibit 11-33: Accident Modification Factors (AMF_{3i}) for Installation of Right-Turn Lanes on Intersections Approaches.

Intersection type	Crash Severity Level	Number of non-stop-controlled approaches with left-turn lanes [a]	
		One approach	Two approaches
Three-leg minor road STOP control[b]	Total	0.86	-
	Fatal and Injury	0.77	-
Four-leg minor road STOP control[b]	Total	0.86	0.74
	Fatal and Injury	0.77	0.59

NOTE: [a] STOP-controlled approaches are not considered in determining the number of approaches with right-turn lanes.

[b] STOP signs present on minor road approaches only.

AMF_{4i} - Lighting

The SPF base condition for lighting is the absence of intersection lighting. The AMF for lighted intersections is adapted from the work of Elvik and Vaa [(1)], as:

The fourth of four AMFs for intersections is an AMF for lighting.

$$AMF_{4i} = 1.0 - 0.38 \times p_{ni} \tag{11-22}$$

Where,

AMF_{4i} = Accident Modification Factor for the effect of lighting on total accidents;

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

This AMF applies to total intersections accidents (not including vehicle-pedestrian and vehicle-bicycle collisions). Exhibit 11-34 presents default values for the nighttime accident proportion p_{ni}. HSM users are encouraged to replace the estimates in Exhibit 11-34 with locally derived values.

Exhibit 11-34: Default Nighttime Accident Proportions for Unlighted Intersections

Intersection Type	Proportion of accidents that occur at night, p_{ni}
3ST	0.276
4ST	0.273

11.8. CALIBRATION TO LOCAL CONDITIONS

In Step 10 of the predictive method, presented in Section 11.4, the predictive model is calibrated to local state or geographic conditions. Accident frequencies, even for nominally similar roadway segments or intersections, can vary widely from one jurisdiction to another. Geographic regions differ markedly in climate, animal population, driver populations, accident reporting threshold, and accident reporting practices. These variations may result in some jurisdictions experiencing a different number of traffic accidents on rural multilane highways than others. Calibration factors are included in the methodology to allow highway agencies to adjust the SPFs to match actual local conditions.

The calibration factors for roadway segments and intersections (defined below as C_r and C_i, respectively) will have values greater than 1.0 for roadways that, on average, experience more accidents than the roadways used in the development of the SPFs. The calibration factors for roadways that experience fewer accidents on average than the roadways used in the development of the SPFs will have values less than 1.0. The calibration procedures are presented in the Appendix to *Part C*.

Calibration factors provide one method of incorporating local data to improve estimated accident frequencies for individual agencies or locations. Several other default values used in the methodology, such as collision type distribution, can also be replaced with locally derived values. The derivation of values for these parameters is addressed in the calibration procedure in the Appendix to *Part C*.

11.9. LIMITATIONS OF PREDICTIVE METHODS IN CHAPTER 11

This section discusses limitations of the specific predictive models and the application of the predictive method in Chapter 11.

Where rural multilane highways intersect access-controlled facilities (i.e., freeways), the grade-separated interchange facility, including the rural multilane road within the interchange area, cannot be addressed with the predictive method for rural multilane highways.

The SPFs developed for Chapter 11 do not include signalized three-leg intersection models. Such intersections may be found on rural multilane highways.

AMFs have not been developed for the SPF for four-leg signalized intersections on rural multilane highways.

11.10. APPLICATION OF CHAPTER 11 PREDICTIVE METHOD

The predictive method presented in Chapter 11 applies to rural multilane highways. The predictive method is applied to a rural multilane highway facility by following the 18 steps presented in Section 11.4. Worksheets are presented in Appendix A for applying calculations in the predictive method steps specific to Chapter 11. All computations of accident frequencies within these worksheets are conducted with values expressed to three decimal places. This level of precision is needed only for consistency in computations. In the last stage of computations, rounding the final estimates of expected average crash frequency be to one decimal place is appropriate.

11.11. SUMMARY

The predictive method can be used to estimate the expected average crash frequency for an entire rural multilane highway facility, a single individual site, or series of contiguous sites. A rural multilane highway facility is defined in Section 11.3, and consists of a four lane highway facility which does not have access control and is outside of cities or towns with a population greater than 5,000 persons.

The predictive method for rural multilane highways is applied by following the 18 steps of the predictive method presented in Section 11.4. Predictive models, developed for rural multilane highway facilities, are applied in Steps 9, 10, and 11 of the method. These predictive models have been developed to estimate the predicted average crash frequency of an individual intersection or homogenous roadway segment. The facility is divided into these individual sites in Step 5 of the predictive method.

Each predictive model in Chapter 11 consists of a Safety Performance Function (SPF), Accident Modification Factors (AMFs), and a calibration factor. The SPF is selected in Step 9, and is used to estimate the predicted average crash frequency for a site with base conditions. The estimate can be for total crashes, or by crash severity or collision type distribution. In order to account for differences between the base conditions and the specific conditions of the site, AMFs are applied in Step 10, which adjust the prediction to account for the geometric design and traffic control features of the site. Calibration factors are also used to adjust the prediction to local conditions in the jurisdiction where the site is located. The process for determining calibration factors for the predictive models is described in the *Part C* Appendix A.1.

Where observed data are available, the EB Method is applied to improve the reliability of the estimate. The EB Method can be applied at the site-specific level or at the project-specific level. It may also be applied to a future time period if site conditions will not change in the future period. The EB Method is described in the *Part C* Appendix A.2.

Section 11.12 presents six sample problems which detail the application of the predictive method. Appendix A contains worksheets which can be used in the calculations for the predictive method steps.

11.12. SAMPLE PROBLEMS

In this section, six sample problems are presented using the predictive method for rural multilane highways. Sample Problem 1 illustrates how to calculate the predicted average crash frequency for a divided rural four-lane highway segment. Sample Problem 2 illustrates how to calculate the predicted average crash frequency for an undivided rural four-lane highway segment. Sample Problem 3 illustrates how to calculate the predicted average crash frequency for a three-leg stop-controlled intersection. Sample Problem 4 illustrates how to combine the results from Sample Problems 1 through 3 in a case where site-specific observed crash data are available (i.e. using the site-specific EB Method). Sample Problem 5 illustrates how to combine the results from Sample Problems 1 through 3 in a case where site-specific observed crash data are not available (i.e. using project level EB Method). Sample Problem 6 applies the Project Estimation Method 1 presented in Section C.7 of the *Part C Introduction and Applications Guidance*, to determine the effectiveness of a proposed upgrade from a rural two-lane roadway to a rural four-lane highway.

Exhibit 11-35: List of Sample Problems In Chapter 11

Problem No.	Page No.	Description
1	11-43	Predicted average crash frequency for a divided roadway segment
2	11-50	Predicted average crash frequency for an undivided roadway segment
3	11-57	Predicted average crash frequency for a three-leg STOP-controlled intersection
4	11-64	Expected average crash frequency for a facility when site-specific observed crash frequencies are available
5	11-68	Expected average crash frequency for a facility when site-specific observed crash frequencies are not available
6	11-72	Expected average crash frequency and the crash reduction for a proposed rural four-lane highway facility that will replace an existing rural two-lane roadway

11.12.1. Sample Problem 1

The Site/Facility

A rural four-lane divided highway segment.

The Question

What is the predicted average crash frequency of the roadway segment for a particular year?

The Facts

- 1.5-mi length
- 10,000 veh/day
- 12-ft lane width
- 6-ft paved right shoulder
- 20-ft traversable median
- No roadway lighting
- No automated enforcement

Assumptions

- Collision type distributions are the defaults values presented in Exhibit 11-10.
- The calibration factor is assumed to be 1.10.

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the roadway segment in Sample Problem 1 is determined to be 3.3 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8

To determine the predicted average crash frequency of the roadway segment in Sample Problem 1, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year, and the EB Method is not applied.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type and traffic control features.

The SPF for a divided roadway segment is calculated from Equation 11-9 and Exhibit 11-8 as follows:

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

$$= e^{(-9.025+1.049\times ln(10,000)+ln(1.5))}$$

$$= 2.835 \text{ crashes/year}$$

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust base conditions to site specific geometric conditions and traffic control features.

Each AMF used in the calculation of the predicted average crash frequency of the roadway segment is calculated below:

Lane Width (AMF_{1rd})

Since the roadway segment in Sample Problem 1 has 12-ft lanes, AMF_{1rd} = 1.00 (i.e. the base condition for AMF_{1rd} is 12-ft lane width).

Shoulder Width and Type (AMF_{2rd})

From Exhibit 11-27, for 6-ft paved shoulders, AMF_{2rd} =1.04.

Median Width (AMF_{3rd})

From Exhibit 11-28, for a traversable median width of 20 ft, AMF_{3rd} = 1.02.

Lighting (AMF_{4rd})

Since there is no lighting in Sample Problem 1, AMF_{4rd}=1.00 (i.e. the base condition for AMF_{4rd} is absence of roadway lighting).

Automated Speed Enforcement (AMF_{5rd})

Since there is no automated speed enforcement in Sample Problem 1, AMF_{5rd}=1.00 (i.e. the base condition for AMF_{5rd} is the absence of automated speed enforcement).

The combined AMF value for Sample Problem 1 is calculated below.

$$AMF_{COMB} = 1.04 \times 1.02$$

$$= 1.06$$

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed in Sample Problem 1 that a calibration factor, C_r, of 1.10 has been determined for local conditions. See *Part C* Appendix A.1 for further discussion on calibration of the predictive models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated using Equation 11-3 based on the results obtained in Steps 9 through 11 as follows:

$$N_{predicted\ rs} = N_{spf\ rd} \times C_r \times (AMF_{1rd} \times AMF_{2rd} \times \ldots \times AMF_{5rd})$$

$$= 2.835 \times 1.10 \times (1.06)$$

$$= 3.306 \text{ crashes/year}$$

Worksheets

The step-by-step instructions above are provided to illustrate the predictive method for calculating the predicted average crash frequency for a roadway segment. To apply the predictive method steps to multiple segments, a series of five worksheets are provided for determining the predicted average crash frequency. The five worksheets include:

- Worksheet 1A – General Information and Input Data for Rural Multilane Roadway Segments
- Worksheet 1B (a) – Accident Modification Factors for Rural Multilane Divided Roadway Segments
- Worksheet 1C (a) – Roadway Segment Crashes for Rural Multilane Divided Roadway Segments
- Worksheet 1D (a) – Crashes by Severity Level and Collision Type for Rural Multilane Divided Roadway Segments
- Worksheet 1E – Summary Results for Rural Multilane Roadway Segments

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 11 Appendix A.

Worksheet 1A – General Information and Input Data for Rural Multilane Roadway Segments

Worksheet 1A is a summary of general information about the roadway segment, analysis, input data (i.e., “The Facts”) and assumptions for Sample Problem 1.

Worksheet 1A – General Information and Input Data for Rural Multilane Roadway Segments

General Information		**Location Information**	
Analyst		Highway	
Agency or Company		Roadway Section	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	**Base Conditions**	**Site Conditions**
Roadway type (divided/undivided)	-	divided
Length of segment, L (mi)	-	1.5
AADT (veh/day)	-	10,000
Lane width (ft)	12	12
Shoulder width (ft) - right shoulder width for divided	8	6
Shoulder type - right shoulder type for divided	paved	paved
Median width (ft) - for divided only	30	20
Side Slopes - for undivided only	1:7 or flatter	N/A
Lighting (present/not present)	not present	not present
Auto speed enforcement (present/not present)	not present	not present
Calibration Factor, C_r	1.0	1.1

Worksheet 1B (a) – Accident Modification Factors for Rural Multilane Divided Roadway Segments

In Step 10 of the predictive method, Accident Modification Factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 11.7 presents the tables and equations necessary for determining the AMF values. Once the value for each AMF has been determined, all of the AMFs multiplied together in Column 6 of Worksheet 1B (a) which indicates the combined AMF value.

Worksheet 1B (a) – Accident Modification Factors for Rural Multilane Divided Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
AMF for Lane Width	AMF for Right Shoulder Width	AMF for Median Width	AMF for Lighting	AMF for Auto Speed Enforcement	Combined AMF
AMF_{1rd}	AMF_{2rd}	AMF_{3rd}	AMF_{4rd}	AMF_{5rd}	AMF_{COMB}
from Equation 11-16	from Exhibit 11-27	from Exhibit 11-28	from Equation 11-17	from Section 11.7.2	(1)*(2)*(3)*(4)*(5)
1.00	1.04	1.02	1.00	1.00	1.06

Worksheet 1C (a) – Roadway Segment Crashes for Rural Multilane Divided Roadway Segments

The SPF for the roadway segment in Sample Problem 1 is calculated using the coefficients found in Exhibit 11-8 (Column 2), which are entered into Equation 11-9 (Column 3). The overdispersion parameter associated with the SPF can be calculated using Equation 11-10 and entered into Column 4; however, the overdispersion parameter is not needed for Sample Problem 1 (as the EB Method is not utilized). Column 5 represents the combined AMF (from Column 6 in Worksheet 1B (a)), and Column 6 represents the calibration factor. Column 7 calculates predicted average crash frequency using the values in Column 4, the combined AMF in Column 5, and the calibration factor in Column 6.

(1)	(2)			(3)	(4)	(5)	(6)	(7)
Crash Severity Level	**SPF Coefficients**			**$N_{spf\ rd}$**	**Overdispersion Parameter, k**	**Combined AMFs**	**Calibration Factor, C_r**	**Predicted average crash frequency, $N_{predicted\ rs}$**
	from Exhibit 11-8			from Equation 11-9	from Equation 11-10	(6) from Worksheet 1B (a)		(3)*(5)*(6)
	a	b	c					
Total	-9.025	1.049	1.549	2.835	0.142	1.06	1.10	3.306
Fatal and Injury (FI)	-8.837	0.958	1.687	1.480	0.123	1.06	1.10	1.726
Fatal and Injury[a] (FI[a])	-8.505	0.874	1.740	0.952	0.117	1.06	1.10	1.110
Property damage only (PDO)								$(7)_{TOTAL}$ - $(7)_{FI}$
								1.580

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

Worksheet 1D (a) – Crashes by Severity Level and Collision Type for Rural Multilane Divided Roadway Segments

Worksheet 1D (a) presents the default proportions for collision type (from Exhibit 11-10) by crash severity level as follows:

- Total crashes (Column 2)
- Fatal and injury crashes (Column 4)
- Fatal and injury crashes, not including "possible injury" crashes (i.e., on a KABCO injury scale, only KAB crashes) (Column 6)
- Property damage only crashes (Column 8)

Using the default proportions, the predicted average crash frequency by collision type is presented in Columns 3 (Total), 5 (Fatal and Injury, FI), 7 (Fatal and Injury, not including "possible injury"), and 9 (Property Damage Only, PDO).

These proportions may be used to separate the predicted average crash frequency (from Column 7, Worksheet 1C (a)) by crash severity and collision type.

Worksheet 1D (a) – Crashes by Severity Level and Collision Type for Rural Multilane Divided Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Collision Type	**Proportion of Collision Type (TOTAL)**	$N_{predicted\ rs\ (TOTAL)}$ **(crashes/year)**	**Proportion of Collision Type (FI)**	$N_{predicted\ rs\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type (FI^a)**	$N_{predicted\ rs\ (FI^a)}$ **(crashes/year)**	**Proportion of Collision Type (PDO)**	$N_{predicted\ rs\ (PDO)}$
	from Exhibit 11-10	$(7)_{TOTAL}$ from Worksheet 1C (a)	from Exhibit 11-10	$(7)_{FI}$ from Worksheet 1C (a)	from Exhibit 11-10	$(7)_{FI}{}^a$ from Worksheet 1C (a)	from Exhibit 11-10	$(7)_{PDO}$ from Worksheet 1C (a)
Total	1.000	3.306	1.000	1.726	1.000	1.110	1.000	1.580
		$(2)*(3)_{TOTAL}$		$(4)*(5)_{FI}$		$(6)*(7)_{FI}{}^a$		$(8)*(9)_{PDO}$
Head-on collision	0.006	0.020	0.013	0.022	0.018	0.020	0.002	0.003
Sideswipe collision	0.043	0.142	0.027	0.047	0.022	0.024	0.053	0.084
Rear-end collision	0.116	0.383	0.163	0.281	0.114	0.127	0.088	0.139
Angle collision	0.043	0.142	0.048	0.083	0.045	0.050	0.041	0.065
Single-vehicle collision	0.768	2.539	0.727	1.255	0.778	0.864	0.792	1.251
Other collision	0.024	0.079	0.022	0.038	0.023	0.026	0.024	0.038

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

Worksheet 1E – Summary Results for Rural Multilane Roadway Segments

Worksheet 1E presents a summary of the results. Using the roadway segment length, the worksheet presents the crash rate in miles per year (Column 4).

Worksheet 1E – Summary Results for Rural Multilane Roadway Segments

(1)	(2)	(3)	(4)
Crash severity level	**Predicted average crash frequency (crashes/year)**	**Roadway segment length (mi)**	**Crash rate (crashes/mi/year)**
	(7) from Worksheet 1C (a)		(2)/(3)
Total	3.306	1.5	2.2
Fatal and injury (FI)	1.726	1.5	1.2
Fatal and Injury[a] (FI^a)	1.110	1.5	0.7
Property damage only (PDO)	1.580	1.5	1.1

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

11.12.2. Sample Problem 2

The Site/Facility

A rural four-lane undivided highway segment.

The Question

What is the predicted average crash frequency of the roadway segment for a particular year?

The Facts

- 0.1-mi length
- 8,000 veh/day
- 11-ft lane width
- 2-ft gravel shoulder
- Side slope of 1:6
- Roadside lighting present
- Automated enforcement present

Assumptions

- Collision type distributions have been adapted to local experience. The percentage of total crashes representing single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents is 33%.
- The proportion of crashes that occur at night are not known, so the default proportions for nighttime crashes will be used.
- The calibration factor is assumed to be 1.10.

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the roadway segment in Sample Problem 2 is determined to be 0.3 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8

To determine the predicted average crash frequency of the roadway segment in Sample Problem 2, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year, and the EB Method is not applied.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type and traffic control features.

The SPF for an undivided roadway segment is calculated from Equation 11-7 and Exhibit 11-5 as follows:

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

$$= e^{(-9.653+1.176\times\ln(8,000)+\ln(0.1))}$$

$$= 0.250 \text{ crashes/year}$$

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust base conditions to site specific geometric conditions and traffic control features.

Each AMF used in the calculation of the predicted average crash frequency of the roadway segment is calculated below:

Lane Width (AMF_{1ru})

AMF_{1ru} can be calculated from Equation 11-13 as follows:

$$AMF_{1ru} = (AMF_{RA} - 1.0) \times p_{RA} + 1.0$$

For 11-ft lane width and AADT of 8,000, AMF_{RA}= 1.04 (see Exhibit 11-18).

The proportion of related accidents, p_{RA}, is 0.33 (from local experience, see assumptions).

$$AMF_{1ru} = (1.04 - 1.0) \times 0.33 + 1.0$$

$$= 1.01$$

Shoulder Width and Type (AMF_{2ru})

AMF_{2ru} can be calculated from Equation 11-14 as follows:

$$AMF_{2ru} = (AMF_{WRA} \times AMD_{TRA} - 1.0) \times p_{RA} + 1.0$$

For 2-ft shoulders and AADT of 8,000, AMF_{WRA} = 1.30 (see Exhibit 11-20).

For 2-ft gravel shoulders, AMF_{TRA} = 1.01 (see Exhibit 11-22).

The proportion of related accidents, p_{RA}, is 0.33 (from local experience, see assumptions).

$$AMF_{2ru} = (1.30 \times 1.01 - 1.0) \times 0.33 + 1.0$$

$$= 1.10$$

Side Slopes (AMF_{3ru})

From Exhibit 11-23, for a side slope of 1:6, AMF_{3ru} = 1.05.

Lighting (AMF_{4ru})

AMF_{4ru} can be calculated from Equation 11-15 as follows:

$$AMF_{4ru} = 1 - [(1 - 0.72 \times p_{inr} - 0.83 \times p_{pnr}) \times p_{nr}]$$

Local values for nighttime crashes proportions are not known. The default nighttime crash proportions used are p_{inr}= 0.361, p_{pnr}= 0.639 and p_{nr}= 0.255 (see Exhibit 11-24).

$$AMF_{4ru} = 1 - [(1 - 0.72 \times 0.361 - 0.83 \times 0.639) \times 0.255]$$

$$= 0.95$$

Automated Speed Enforcement (AMF_{5ru})

For an undivided roadway segment with automated speed enforcement, AMF_{5ru}=0.95 (see Section 11.7.1).

The combined AMF value for Sample Problem 2 is calculated below.

$$AMF_{COMB} = 1.01 \times 1.10 \times 1.05 \times 0.95 \times 0.95$$

$$= 1.05$$

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed in Sample Problem 2 that a calibration factor, C_r, of 1.10 has been determined for local conditions. See *Part C* Appendix A.1 for further discussion on calibration of the predictive models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated using Equation 11-2 based on the results obtained in Steps 9 through 11 as follows:

$$N_{predicted\ rs} = N_{spf\ ru} \times C_r \times (AMF_{1ru} \times AMF_{2ru} \times \ldots \times AMF_{5ru})$$

$$= 0.250 \times 1.10 \times (1.05)$$

$$= 0.289 \text{ crashes/year}$$

Worksheets

The step-by-step instructions above are provided to illustrate the predictive method for calculating the predicted average crash frequency for a roadway segment. To apply the predictive method steps to multiple segments, a series of five worksheets are provided for determining the predicted average crash frequency. The five worksheets include:

- Worksheet 1A – General Information and Input Data for Rural Multilane Roadway Segments
- Worksheet 1B (b) – Accident Modification Factors for Rural Multilane Undivided Roadway Segments
- Worksheet 1C (b) – Roadway Segment Crashes for Rural Multilane Undivided Roadway Segments
- Worksheet 1D (b) – Crashes by Severity Level and Collision Type for Rural Multilane Undivided Roadway Segments
- Worksheet 1E – Summary Results for Rural Multilane Roadway Segments

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 11 Appendix A.

Worksheet 1A – General Information and Input Data for Rural Multilane Roadway Segments

Worksheet 1A is a summary of general information about the roadway segment, analysis, input data (i.e., "The Facts") and assumptions for Sample Problem 2.

Worksheet 1A – General Information and Input Data for Rural Multilane Roadway Segments			
General Information		**Location Information**	
Analyst		Highway	
Agency or Company		Roadway Section	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	**Base Conditions**	**Site Conditions**
Roadway type (divided/undivided)	-	undivided
Length of segment, L (mi)	-	0.1
AADT (veh/day)	-	8,000
Lane width (ft)	12	11
Shoulder width (ft) - right shoulder width for divided	6	2
Shoulder type - right shoulder type for divided	paved	gravel
Median width (ft) - for divided only	30	N/A
Side Slopes - for undivided only	1:7 or flatter	1:6
Lighting (present/not present)	not present	present
Auto speed enforcement (present/not present)	not present	present
Calibration Factor, C_r	1.0	1.1

Worksheet 1B (b) – Accident Modification Factors for Rural Multilane Undivided Roadway Segments

In Step 10 of the predictive method, Accident Modification Factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 11.7 presents the tables and equations necessary for determining the AMF values. Once the value for each AMF has been determined, all of the AMFs multiplied together in Column 6 of Worksheet 1B (b) which indicates the combined AMF value.

Worksheet 1B (b) – Accident Modification Factors for Rural Multilane Undivided Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
AMF for Lane Width	AMF for Shoulder Width	AMF for Side Slopes	AMF for Lighting	AMF for Automated Speed Enforcement	Combined AMF
$AMF1_{ru}$	AMF_{2ru}	AMF_{3ru}	AMF_{4ru}	AMF_{5ru}	AMF_{COMB}
from Equation 11-13	from Equation 11-14	from Exhibit 11-23	from Equation 11-15	from Section 11.7.1	(1)*(2)*(3)*(4)*(5)
1.01	1.10	1.05	0.95	0.95	1.05

Worksheet 1C (b) – Roadway Segment Crashes for Rural Multilane Undivided Roadway Segments

The SPF for the roadway segment in Sample Problem 2 is calculated using the coefficients found in Exhibit 11-5 (Column 2), which are entered into Equation 11-7 (Column 3). The overdispersion parameter associated with the SPF can be calculated using Equation 11-8 and entered into Column 4; however, the overdispersion parameter is not needed for Sample Problem 2 (as the EB Method is not utilized). Column 5 represents the combined AMF (from Column 6 in Worksheet 1B (b)), and Column 6 represents the calibration factor. Column 7 calculates the predicted average crash frequency using the values in Column 4, the combined AMF in Column 5, and the calibration factor in Column 6.

Worksheet 1C (b) – Roadway Segment Accidents for Rural Multilane Undivided Roadway Segments

(1)	(2)			(3)	(4)	(5)	(6)	(7)
Crash Severity Level	**SPF Coefficients**			**$N_{spf\ ru}$**	**Overdispersion Parameter, k**	**Combined AMFs**	**Calibration Factor, C_r**	**Predicted average crash frequency, $N_{predicted\ rs}$**
	from Exhibit 11-5			from Equation 11-7	from Equation 11-8	(6) from Worksheet 1B (b)		(3)*(5)*(6)
	a	b	c					
Total	-9.653	1.176	1.675	0.250	1.873	1.05	1.10	0.289
Fatal and Injury (FI)	-9.410	1.094	1.796	0.153	1.660	1.05	1.10	0.177
Fatal and Injury[a] (FI[a])	-8.577	0.938	2.003	0.086	1.349	1.05	1.10	0.099
Property damage only (PDO)	-	-	-	-	-	-	-	$(7)_{TOTAL} - (7)_{FI}$ 0.112

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

Worksheet 1D (b) – Crashes by Severity Level and Collision Type for Rural Multilane Undivided Roadway Segments

Worksheet 1D (b) presents the default proportions for collision type (from Exhibit 11-7) by crash severity level as follows:

- Total crashes (Column 2)
- Fatal and injury crashes (Column 4)
- Fatal and injury crashes, not including "possible injury" crashes (i.e., on a KABCO injury scale, only KAB crashes) (Column 6)
- Property damage only crashes (Column 8)

Using the default proportions, the predicted average crash frequency by collision type is presented in Columns 3 (Total), 5 (Fatal and Injury, FI), 7 (Fatal and Injury, not including "possible injury"), and 9 (Property Damage Only, PDO).

These proportions may be used to separate the predicted average crash frequency (from Column 7, Worksheet 1C (b)) by crash severity and collision type.

Worksheet 1D (b) – Accidents by Severity Level and Collision Type for Rural Multilane Undivided Roadway Segments								
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Collision Type	**Proportion of Collision Type (TOTAL)**	**$N_{predicted\ rs\ (TOTAL)}$ (crashes/year)**	**Proportion of Collision Type (FI)**	**$N_{predicted\ rs\ (FI)}$ (crashes/year)**	**Proportion of Collision Type (FI^a)**	**$N_{predicted\ rs\ (FI^a)}$ (crashes/year)**	**Proportion of Collision Type (PDO)**	**$N_{predicted\ rs\ (PDO)}$ (crashes/year)**
	from Exhibit 11-7	$(7)_{TOTAL}$ from Worksheet 1C (b)	from Exhibit 11-7	$(7)_{FI}$ from Worksheet 1C (b)	from Exhibit 11-7	$(7)_{FI}^{a}$ from Worksheet 1C (b)	from Exhibit 11-7	(7)PDO from Worksheet 1C (b)
Total	1.000	0.289	1.000	0.177	1.000	0.099	1.000	0.112
		$(2)*(3)_{TOTAL}$		$(4)*(5)_{FI}$		$(6)*(7)_{FI}^{a}$		$(8)*(9)_{PDO}$
Head-on collision	0.009	0.003	0.029	0.005	0.043	0.004	0.001	0.000
Sideswipe collision	0.098	0.028	0.048	0.008	0.044	0.004	0.120	0.013
Rear-end collision	0.246	0.071	0.305	0.054	0.217	0.021	0.220	0.025
Angle collision	0.356	0.103	0.352	0.062	0.348	0.034	0.358	0.040
Single-vehicle collision	0.238	0.069	0.238	0.042	0.304	0.030	0.237	0.027
Other collision	0.053	0.015	0.028	0.005	0.044	0.004	0.064	0.007

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

Worksheet 1E – Summary Results for Rural Multilane Roadway Segments

Worksheet 1E presents a summary of the results. Using the roadway segment length, the worksheet presents the crash rate in miles per year (Column 4).

Worksheet 1E – Summary Results for Rural Multilane Roadway Segments			
(1)	(2)	(3)	(4)
Crash severity level	**Predicted average crash frequency (crashes/year)**	**Roadway segment length (mi)**	**Crash rate (crashes/mi/year)**
	(7) from Worksheet 1C (b)		(2)/(3)
Total	0.289	0.1	2.9
Fatal and injury (FI)	0.177	0.1	1.8
Fatal and Injury[a] (FI^a)	0.099	0.1	1.0
Property damage only (PDO)	0.112	0.1	1.1

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

11.12.3. Sample Problem 3

The Site/Facility

A three-leg stop-controlled intersection located on a rural four-lane highway.

The Question

What is the predicted average crash frequency of the stop-controlled intersection for a particular year?

The Facts

- 3 legs
- Minor-road stop control
- 0 right-turn lanes on major road
- 1 left-turn lane on major road
- 30-degree skew angle
- AADT of major road = 8,000 veh/day
- AADT of minor road = 1,000 veh/day
- Calibration factor = 1.50
- Intersection lighting is present

Assumptions

- Collision type distributions are the default values from Exhibit 11-16.
- The calibration factor is assumed to be 1.50.

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the intersection in Sample Problem 3 is determined to be 0.8 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8

To determine the predicted average crash frequency of the intersection in Sample Problem 3, only Steps 9 through 11 are conducted. No other steps are necessary because only one intersection is analyzed for one year, and the EB Method is not applied.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type and traffic control features.

The SPF for a three-leg intersection with minor-road stop-control is calculated from Equation 11-11 and Exhibit 11-11 as follows:

$$N_{spfint} = exp[a + b \times ln(AADT_{maj}) + c \times ln(AADT_{min})]$$

$$= exp[-12.526 + 1.204 \times ln(8{,}000) + 0.236 \times ln(1{,}000)]$$

$$= 0.928 \text{ crashes/year}$$

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust base conditions to site specific geometric conditions and traffic control features

Each AMF used in the calculation of the predicted average crash frequency of the intersection is calculated below:

Intersection Skew Angle (AMF_{1i})

AMF_{1i} can be calculated from Equation 11-18 as follows:

$$AMF_{1i} = \frac{0.016 \times SKEW}{(0.98 + 0.16 \times SKEW)} + 1.0$$

The intersection skew angle for Sample Problem 3 is 30 degrees.

$$AMF_{1i} = \frac{0.016 \times 30}{(0.98 + 0.16 \times 30)} + 1.0$$

$$= 1.08$$

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

From Exhibit 11-32, for a left-turn lane on one non-stop-controlled approach at a three-leg STOP-controlled intersection, AMF_{2i} = 0.56.

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

Since no right-turn lanes are present, AMF_{3i} = 1.00 (i.e. the base condition for AMF_{3i} is the absence of right-turn lanes on the intersection approaches).

Lighting (AMF_{4i})

AMF_{4i} can be calculated from Equation 11-22 as follows:

$$AMF_{4i} = 1.0 - 0.38 \times p_{ni}$$

From Exhibit 11-34, for intersection lighting at a three-leg stop-controlled intersection, p_{ni} = 0.276.

$$AMF_{4i} = 1.0 - 0.38 \times 0.276$$

$$= 0.90$$

The combined AMF value for Sample Problem 3 is calculated below.

$$AMF_{COMB} = 1.08 \times 0.56 \times 0.90$$

$$= 0.54$$

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed that a calibration factor, C_i, of 1.50 has been determined for local conditions. See *Part C* Appendix A.1 for futher discussion on calibration of the predictive models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated using Equation 11-4 based on the results obtained in Steps 9 through 11 as follows:

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

$$= 0.928 \times 1.50 \times (0.54)$$

$$= 0.752 \text{ crashes/year}$$

Worksheets

The step-by-step instructions above are the predictive method for calculating the predicted average crash frequency for an intersection. To apply the predictive method steps, a series of five worksheets are provided for determining the predicted average crash frequency. The five worksheets include:

- Worksheet 2A – General Information and Input Data for Rural Multilane Highway Intersections
- Worksheet 2B – Accident Modification Factors for Rural Multilane Highway Intersections
- Worksheet 2C – Intersection Crashes for Rural Multilane Highway Intersections
- Worksheet 2D – Crashes by Severity Level and Collision Type for Rural Multilane Highway Intersections
- Worksheet 2E – Summary Results for Rural Multilane Highway Intersections

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 11 Appendix A.

Worksheet 2A – General Information and Input Data for Rural Multilane Highway Intersections

Worksheet 2A is a summary of general information about the intersection, analysis, input data (i.e., "The Facts") and assumptions for Sample Problem 3.

Worksheet 2A – General Information and Input Data for Rural Multilane Highway Intersections			
General Information		**Location Information**	
Analyst		Highway	
Agency or Company		Intersection	
Date Performed		Jurisdiction	
		Analysis Year	
Input Data		**Base Conditions**	**Site Conditions**
Intersection type (3ST, 4ST, 4SG)		-	3ST
$AADT_{major}$ (veh/day)		-	8,000
$AADT_{minor}$ (veh/day)		-	1,000
Intersection skew angle (degrees)		0	30
Number of signalized or uncontrolled approaches with a left turn lane (0,1,2,3,4)		0	1
Number of signalized or uncontrolled approaches with a right turn lane (0,1,2,3,4)		0	0
Intersection lighting (present/not present)		not present	present
Calibration Factor, C_i		1.0	1.5

Worksheet 2B – Accident Modification Factors for Rural Multilane Highway Intersections

In Step 10 of the predictive method, Accident Modification Factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 11.7 presents the tables and equations necessary for determining the AMF values. Once the value for each AMF has been determined, all of the AMFs are multiplied together in Column 6 of Worksheet 2B which indicates the combined AMF value.

Worksheet 2B – Accident Modification Factors for Rural Multilane Highway Intersections

(1)	(2)	(3)	(4)	(5)	(6)
Crash Severity Level	AMF for Intersection Skew Angle	AMF for Left-Turn Lanes	AMF for Right-Turn Lanes	AMF for Lighting	Combined AMF
	AMF_{1i}	AMF_{2i}	AMF_{3i}	AMF_{4i}	AMF_{COMB}
	from Equations 11-18 or 11-20 and 11-19 or 11-21	from Exhibit 11-32	from Exhibit 11-33	from Equation 11-22	(1)*(2)*(3)*(4)
Total	1.08	0.56	1.00	0.90	0.54
Fatal and Injury (FI)	1.09	0.45	1.00	0.90	0.44

Worksheet 2C – Intersection Crashes for Rural Multilane Highway Intersections

The SPF for the intersection in Sample Problem 3 is calculated using the coefficients shown in Exhibit 11-11 (Column 2), which are entered into Equation 11-11 (Column 3). The overdispersion parameter associated with the SPF is also found in Exhibit 11-11 and entered into Column 4; however, the overdispersion parameter is not needed for Sample Problem 3 (as the EB Method is not utilized). Column 5 represents the combined AMF (from Column 6 in Worksheet 2B), and Column 6 represents the calibration factor. Column 7 calculates the predicted average crash frequency using the values in Column 3, the combined AMF in Column 5, and the calibration factor in Column 6.

Worksheet 2C – Intersection Crashes for Rural Multilane Highway Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)
Crash Severity Level	**SPF Coefficients**			**$N_{spf\ int}$**	**Overdispersion Parameter, k**	**Combined AMFs**	**Calibration Factor, C_i**	**Predicted average crash frequency, $N_{predicted\ int}$**
	form Exhibit 11-11 or 11-12			from Equation 11-11 or 11-12	from Exhibit 11-11 or 11-12	from (6) of Worksheet 2B		(3)*(5)*(6)
	a	b	c					
Total	-12.526	1.204	0.236	0.928	0.460	0.54	1.50	0.752
Fatal and Injury (FI)	-12.664	1.107	0.272	0.433	0.569	0.44	1.50	0.286
Fatal and Injury[a] (FI[a])	-11.989	1.013	0.228	0.270	0.566	0.44	1.50	0.178
Property Damage Only (PDO)	-	-	-	-	-	-	-	$(7)_{TOTAL} - (7)_{FI}$ 0.466

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

Worksheet 2D – Crashes by Severity Level and Collision Type for Rural Multilane Highway Intersections

Worksheet 2D presents the default proportions for collision type (from Exhibit 11-16) by crash severity level as follows:

- Total crashes (Column 2)
- Fatal and injury crashes (Column 4)
- Fatal and injury crashes, not including "possible injury" crashes (i.e., on a KABCO injury scale, only KAB crashes) (Column 6)
- Property damage only crashes (Column 8)

Using the default proportions, the predicted average crash frequency by collision type in Columns 3 (Total), 5 (Fatal and Injury, FI), 7 (Fatal and Injury, not including "possible injury"), and 9(Property Damage Only, PDO).

These proportions may be used to separate the predicted average crash frequency (from Column 7, Worksheet 2C) by crash severity and collision type.

Worksheet 2D – Accidents by Severity Level and Collision Type for Rural Multilane Highway Intersections								
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Collision Type	**Proportion of Collision Type (TOTAL)**	**$N_{predicted\ int\ (TOTAL)}$ (crashes/year)**	**Proportion of Collision Type (FI)**	**$N_{predicted\ int\ (FI)}$ (crashes/year)**	**Proportion of Collision Type (FI^a)**	**$N_{predicted\ int\ (FI^a)}$ (crashes/year)**	**Proportion of Collision Type (PDO)**	**$N_{predicted\ int\ (PDO)}$ (crashes/year)**
	from Exhibit 11-16	$(7)_{TOTAL}$ from Worksheet 2C	from Exhibit 11-16	$(7)_{FI}$ from Worksheet 2C	from Exhibit 11-16	$(7)_{FI}{}^a$ from Worksheet 2C	from Exhibit 11-16	$(7)_{PDO}$ from Worksheet 2C
Total	1.000	0.752	1.000	0.286	1.000	0.178	1.000	0.466
		$(2)*(3)_{TOTAL}$		$(4)*(5)_{FI}$		$(6)*(7)_{FI}{}^a$		$(8)*(9)_{PDO}$
Head-on collision	0.029	0.022	0.043	0.012	0.052	0.009	0.020	0.009
Sideswipe collision	0.133	0.100	0.058	0.017	0.057	0.010	0.179	0.083
Rear-end collision	0.289	0.217	0.247	0.071	0.142	0.025	0.315	0.147
Angle collision	0.263	0.198	0.369	0.106	0.381	0.068	0.198	0.092
Single-vehicle collision	0.234	0.176	0.219	0.063	0.284	0.051	0.244	0.114
Other collision	0.052	0.039	0.064	0.018	0.084	0.015	0.044	0.021

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

Worksheet 2E – Summary Results for Rural Multilane Highway Intersections

Worksheet 2E presents a summary of the results.

Worksheet 2E – Summary Results for Rural Multilane Highway Intersections	
(1)	(2)
Crash severity level	**Predicted average crash frequency (crashes/year)**
	(7) from Worksheet 2C
Total	0.752
Fatal and Injury (FI)	0.286
Fatal and Injury[a] (FI^a)	0.178
Property Damage Only (PDO)	0.466

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

11.12.4. Sample Problem 4

The Project

A project of interest consists of three sites: a rural four-lane divided highway segment; a rural four-lane undivided highway segment; and a three-leg intersection with minor-road stop control. (This project is a compilation of roadway segments and intersections from Sample Problems 1, 2 and 3.)

The Question

What is the expected average crash frequency of the project for a particular year incorporating both the predicted crash frequencies from Sample Problems 1, 2 and 3 and the observed crash frequencies using the **site-specific EB Method**?

The Facts

- 2 roadway segments (4D segment, 4U segment)
- 1 intersection (3ST intersection)
- 9 observed crashes (4D segment: 4 crashes; 4U segment: 2 crashes; 3ST intersection: 3 crashes)

Outline of Solution

To calculate the expected average crash frequency, site-specific observed crash frequencies are combined with predicted average crash frequencies for the project using the site-specific EB Method (i.e. observed crashes are assigned to specific intersections or roadway segments) presented in Section A.2.4 of *Part C* Appendix.

Results

The expected average crash frequency for the project is 5.7 crashes per year (rounded to one decimal place).

Worksheets

To apply the site-specific EB Method to multiple roadways segments and intersections on a rural multilane highway combined, two worksheets are provided for determining the expected average crash frequency. The two worksheets include:

- Worksheet 3A – Predicted and Observed Crashes by Severity and Site Type Using the Site-Specific EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways
- Worksheet 3B – Site-Specific EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 11 Appendix A.

Worksheets 3A – Predicted and Observed Crashes by Severity and Site Type Using the Site-Specific EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways

The predicted average crash frequencies by severity type determined in Sample Problems 1 through 3 are entered into Columns 2 through 4 of Worksheet 3A. Column 5 presents the observed crash frequencies by site type, and Column 6 the overdispersion parameter. The expected average crash frequency is calculated by applying the site-specific EB Method which considers both the predicted model estimate and observed crash frequencies for each roadway segment and intersection. Equation A-5 from *Part C* Appendix is used to calculate the weighted adjustment and entered into Column 7. The expected average crash frequency is calculated using Equation A-4 and entered into Column 8.

Worksheet 3A – Predicted and Observed Crashes by Severity and Site Type Using the Site-Specific EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Site type	**Predicted average crash frequency (crashes/year)**			**Observed crashes, $N_{observed}$ (crashes/year)**	**Overdispersion parameter, k**	**Weighted adjustment, w**	**Expected average crash frequency, $N_{expected}$**
	$N_{predicted\ (TOTAL)}$	$N_{predicted\ (FI)}$	$N_{predicted\ (PDO)}$			Equation A-5 from Part C Appendix	Equation A-4 from Part C Appendix
ROADWAY SEGMENTS							
Segment 1	3.306	1.726	1.580	4	0.142	0.681	3.527
Segment 2	0.289	0.177	0.112	2	1.873	0.649	0.890
INTERSECTIONS							
Intersection 1	0.752	0.286	0.466	3	0.460	0.743	1.330
COMBINED (sum of column)	4.347	2.189	2.158	9	-	-	5.747

Column 7 - Weighted Adjustment

The weighted adjustment, w, to be placed on the predictive model estimate is calculated using Equation A-5 from *Part C* Appendix as follows:

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

Segment 1 $$w = \frac{1}{1 + 0.142 \times (3.306)}$$

$$= 0.681$$

Segment 2 $$w = \frac{1}{1 + 1.873 \times (0.289)}$$

$$= 0.649$$

Intersection 1 $$w = \frac{1}{1 + 0.460 \times (0.752)}$$

$$= 0.743$$

Column 8 - Expected Average Crash Frequency

The estimate of expected average crash frequency, $N_{expected}$, is calculated using Equation A-4 from *Part C* Appendix as follows:

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

Segment 1 $$N_{expected} = 0.681 \times 3.306 + (1 - 0.681) \times 4$$

$$= 3.527$$

Segment 2 $$N_{expected} = 0.649 \times 0.289 + (1 - 0.649) \times 2$$

$$= 0.890$$

Intersection 1 $$N_{expected} = 0.743 \times 0.752 + (1 - 0.743) \times 3$$

$$= 1.330$$

Worksheet 3B – Site-Specific EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highwways

Worksheet 3B presents a summary of the results. The expected average crash frequency by severity level is calculated by applying the proportion of predicted average crash frequency by severity level to the total expected average crash frequency (Column 3).

Worksheet 3B – Site-Specific EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

(1)	(2)	(3)
Crash severity level	**$N_{predicted}$**	**$N_{expected}$**
Total	$(2)_{COMB}$ from Worksheet 3A	$(8)_{COMB}$ from Worksheet 3A
	4.347	5.7
Fatal and injury (FI)	$(3)_{COMB}$ from Worksheet 3A	$(3)_{TOTAL}*(2)_{FI}/(2)_{TOTAL}$
	2.189	2.9
Property damage only (PDO)	$(4)_{COMB}$ from Worksheet 3A	$(3)_{TOTAL}*(2)_{PDO}/(2)_{TOTAL}$
	2.158	2.8

11.12.5. Sample Problem 5

The Project

A project of interest consists of three sites: a rural four-lane divided highway segment; a rural four-lane undivided highway segment; and a three-leg intersection with minor-road stop control. (This project is a compilation of roadway segments and intersections from Sample Problems 1, 2 and 3.)

The Question

What is the expected average crash frequency of the project for a particular year incorporating both the predicted crash frequencies from Sample Problems 1, 2 and 3 and the observed crash frequencies using the **project-level EB Method**?

The Facts

- 2 roadway segments (4D segment, 4U segment)
- 1 intersection (3ST intersection)
- 9 observed crashes (but no information is available to attribute specific crashes to specific sites within the project)

Outline of Solution

Observed crash frequencies for the project as a whole are combined with predicted average crash frequencies for the project as a whole using the project-level EB Method (i.e. observed crash data for individual roadway segments and intersections are not available, but observed crashes are assigned to a facility as a whole) presented in Section A.2.5 of *Part C* Appendix.

Results

The expected average crash frequency for the project is 5.8 crashes per year (rounded to one decimal place).

Worksheets

To apply the project-level EB Method to multiple roadway segments and intersections on a rural multilane highway combined, two worksheets are provided for determining the expected average crash frequency. The two worksheets include:

- Worksheet 4A – Predicted and Observed Crashes by Severity and Site Type Using the Project-Level EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways
- Worksheet 4B – Project-Level Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 11 Appendix A.

Worksheets 4A – Predicted and Observed Crashes by Severity and Site Type Using the Project-Level EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways

The predicted average crash frequencies by severity type determined in Sample Problems 1 through 3 are entered in Columns 2 through 4 of Worksheet 4A. Column 5 presents the observed crash frequencies by site type, and Column 6 the overdispersion parameter. The expected average crash frequency is calculated by applying the project-level EB Method which considers both the predicted model estimate for each roadway segment and intersection and the project observed crashes. Column 7 calculates N_{w0} and Column 8 N_{w1}. Equations A-10 through A-14 from *Part C* Appendix are used to calculate the expected average crash frequency of combined sites. The results obtained from each equation are presented in Columns 9 through 14. Section A.2.5 in *Part C* Appendix defines all the variables used in this worksheet.

Worksheet 4A – Predicted and Observed Crashes by Severity and Site Type Using the Project-Level EB Method for Rural Two-Lane Two-Way Roads and Multilane Highways

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Site type	**Predicted average crash frequency (crashes/year)**			**Observed crashes, $N_{observed}$ (crashes/year)**	**Overdispersion Parameter, k**	N_{w0}	N_{w1}	W_0	N_0	w_1	N_1	$N_{p/comb}$
	$N_{predicted}$ (TOTAL)	$N_{predicted}$ (FI)	$N_{predicted}$ (PDO)			Equation A-8 (6)* (2)^2	Equation A-9 sqrt((6)*(2))	Equation A-10	Equation A-11	Equation A-12	Equation A-13	Equation A-14
ROADWAY SEGMENTS												
Segment 1	3.306	1.726	1.580	-	0.142	1.552	0.685	-	-	-	-	-
Segment 2	0.289	0.177	0.112	-	1.873	0.156	0.736	-	-	-	-	-
INTERSECTIONS												
Intersection 1	0.752	0.286	0.466	-	0.460	0.260	0.588	-	-	-	-	-
COMBINED (sum of column)	4.347	2.189	2.158	9	-	1.968	2.009	0.688	5.799	0.684	5.817	5.808

NOTE: $N_{predicted\ w0}$ = Predicted number of total accidents assuming that accidents frequencies are statistically independent

$$N_{predicted\ w0} = \sum_{j=1}^{5} k_{rmj}\ N_{rmj}^2 + \sum_{j=1}^{5} k_{rsj}\ N_{rsj}^2 + \sum_{j=1}^{5} k_{rdj}\ N_{rdj}^2 + \sum_{j=1}^{4} k_{imj}\ N_{imj}^2 + \sum_{j=1}^{4} k_{isj}\ N_{isj}^2 \qquad \text{(A-8)}$$

$N_{predicted\ w1}$ = Predicted number of total accidents assuming that accidents frequencies are perfectly correlated

$$N_{predicted\ w1} = \sum_{j=1}^{5} \sqrt{k_{rmj}\ N_{rmj}} + \sum_{j=1}^{5} \sqrt{k_{rsj}\ N_{isj}} + \sum_{j=1}^{5} \sqrt{k_{rdj}\ N_{rdj}} + \sum_{j=1}^{4} \sqrt{k_{imj}\ N_{imj}} + \sum_{j=1}^{4} \sqrt{k_{isj}\ N_{isj}} \qquad \text{(A-9)}$$

Column 9 – w_0

The weight placed on predicted crash frequency under the assumption that accidents frequencies for different roadway elements are statistically independent, w_0, is calculated using Equation A-10 from *Part C* Appendix as follows:

$$w_0 = \frac{1}{1 + \frac{N_{predicted\ w0}}{N_{predicted\ (TOTAL)}}}$$

$$= \frac{1}{1 + \frac{1.968}{4.347}}$$

$$= 0.688$$

Column 10 – N_0

The expected crash frequency based on the assumption that different roadway elements are statistically independent, N_0, is calculated using Equation A-11 from *Part C* Appendix as follows:

$$N_0 = w_0\ N_{predicted\ (TOTAL)} + (1 - w_0)\ N_{observed\ (TOTAL)}$$

$$= 0.688 \times 4.347 + (1 - 0.688) \times 9$$

$$= 5.799$$

Column 11 – w_1

The weight placed on predicted crash frequency under the assumption that accidents frequencies for different roadway elements are perfectly correlated, w_1, is calculated using Equation A-12 from *Part C* Appendix as follows:

$$w_1 = \frac{1}{1 + \frac{N_{predicted\ w1}}{N_{predicted\ (TOTAL)}}}$$

$$= \frac{1}{1 + \frac{2.009}{4.347}}$$

$$= 0.684$$

Column 12 – N_1

The expected crash frequency based on the assumption that different roadway elements are perfectly correlated, N_1, is calculated using Equation A-13 from *Part C* Appendix as follows:

$$N_1 = w_1\ N_{predicted\ (TOTAL)} + (1 - w_1)\ N_{observed\ (TOTAL)}$$

$$= 0.684 \times 4.347 + (1 - 0.684) \times 9$$

$$= 5.817$$

Column 13 – $N_{expected/comb}$

The expected average crash frequency based of combined sites, $N_{p/comb}$, is calculated using Equation A-14 from *Part C* Appendix as follows:

$$N_{expected/comb} = \frac{N_0 + N_1}{2}$$

$$= \frac{5.799 + 5.817}{2}$$

$$= 5.808$$

Worksheet 4B – Project-Level EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

Worksheet 4B presents a summary of the results. The expected average crash frequency by severity level is calculated by applying the proportion of predicted average crash frequency by severity level to the total expected average crash frequency (Column 3).

Worksheet 4B – Project-Level EB Method Summary Results for Rural Two-Lane Two-Way Roads and Multilane Highways

(1)	(2)	(3)
Crash severity level	**$N_{predicted}$**	**$N_{expected}$**
Total	$(2)_{COMB}$ from Worksheet 4A	$(13)_{COMB}$ from Worksheet 4A
	4.347	5.8
Fatal and injury (FI)	$(3)_{COMB}$ from Worksheet 4A	$(3)_{TOTAL}*(2)_{FI}/(2)_{TOTAL}$
	2.189	2.9
Property damage only (PDO)	$(4)_{COMB}$ from Worksheet 4A	$(3)_{TOTAL}*(2)_{PDO}/(2)_{TOTAL}$
	2.158	2.9

11.12.6. Sample Problem 6

The Project

An existing rural two-lane roadway is proposed for widening to a four-lane highway facility. One portion of the project is planned as a four-lane divided highway, while another portion is planned as a four-lane undivided highway. There is one three-leg stop-controlled intersection located within the project limits.

The Question

What is the expected average crash frequency of the proposed rural four-lane highway facility for a particular year and what crash reduction is expected in comparison to the existing rural two-lane highway facility?

The Facts

- Existing rural two-lane roadway facility with two roadway segments and one intersection equivalent to the facilities in *Chapter 10* Sample Problems 1, 2 and 3.
- Proposed rural four-lane highway facility with two roadway segments and one intersection equivalent to the facilities in Sample Problems 1, 2 and 3 presented in this chapter.

Outline of Solution

Sample Problem 6 applies the Project Estimation Method 1 presented in Section C.7 of the *Part C Introduction and Applications Guidance* (i.e. the expected average crash frequency for existing conditions is compared to the predicted average crash frequency of proposed conditions). The expected average crash frequency for the existing rural two-lane roadway can be represented by the results from applying the site-specific EB Method in *Chapter 10* Sample Problem 5. The predicted average crash frequency for the proposed four-lane facility can be determined from the results of Sample Problems 1, 2 and 3 in this chapter. In this case, Sample Problems 1 through 3 are considered to represent a proposed facility rather than an existing facility; therefore, there is no observed crash frequency data, and the EB Method is not applicable.

Results

The predicted average crash frequency for the proposed four-lane facility project is 4.4 crashes per year and the predicted crash reduction from the project is 8.1 crashes per year. The table below presents a summary of the results.

Site	Expected Average Crash Frequency for the Existing Condition (crashes/year)[a]	Predicted average crash frequency for the Proposed Condition (crashes/year)[b]	Predicted Crash Reduction from Project Implementation (crashes/year)
Segment 1	8.2	3.3	4.9
Segment 2	1.4	0.3	1.1
Intersection 1	2.9	0.8	2.1
Total	12.5	4.4	8.1

[a] from Sample Problems 5 in Chapter 10
[b] from Sample Problems 1 through 3 in Chapter 11

11.13. REFERENCES

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APPENDIX A — WORKSHEETS FOR APPLYING THE PREDICTIVE METHOD FOR RURAL MULTILANE ROADS

Worksheet 1A – General Information and Input Data for Rural Multilane Roadway Segments

General Information		Location Information	
Analyst		Highway	
Agency or Company		Roadway Section	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	Base Conditions	Site Conditions
Roadway type (divided/undivided)	-	
Length of segment, L (mi)	-	
AADT (veh/day)	-	
Lane width (ft)	12	
Shoulder width (ft) - right shoulder width for divided	8	
Shoulder type - right shoulder type for divided	paved	
Median width (ft) - for divided only	30	
Side Slopes - for undivided only	1:7 or flatter	
Lighting (present/not present)	not present	
Auto speed enforcement (present/not present)	not present	
Calibration Factor, C_r	1.0	

Worksheet 1B (a) – Accident Modification Factors for Rural Multilane Divided Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
AMF for Lane Width	AMF for Right Shoulder Width	AMF for Median Width	AMF for Lighting	AMF for Auto Speed Enforcement	Combined AMF
AMF_{1rd}	AMF_{2rd}	AMF_{3rd}	AMF_{4rd}	AMF_{5rd}	AMF_{COMB}
from Equation 11-16	from Exhibit 11-27	from Exhibit 11-28	from Equation 11-17	from Section 11.7.2	(1)*(2)*(3)*(4)*(5)

Worksheet 1B (b) – Accident Modification Factors for Rural Multilane Undivided Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
AMF for Lane Width	AMF for Shoulder Width	AMF for Side Slopes	AMF for Lighting	AMF for Auto Speed Enforcement	Combined AMF
AMF_{1ru}	AMF_{2ru}	AMF_{3ru}	AMF_{4ru}	AMF_{5ru}	AMF_{COMB}
from Equation 11-13	from Equation 11-14	from Exhibit 11-23	from Equation 11-15	from Section 11.7.1	(1)*(2)*(3)*(4)*(5)

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Worksheet 1C (a) – Roadway Segment Crashes for Rural Multilane Divided Roadway Segments

(1)	(2)			(3)	(4)	(5)	(6)	(7)
Crash Severity Level	**SPF Coefficients**			**$N_{spf\ rd}$**	**Overdispersion Parameter, k**	**Combined AMFs**	**Calibration Factor, C_r**	**Predicted average crash frequency, $N_{predicted\ rs}$**
	from Exhibit 11-8			from Equation 11-9	from Equation 11-10	(6) from Worksheet 1B (a)		(3)*(5)*(6)
	a	b	c					
Total	-9.025	1.049	1.549					
Fatal and Injury (FI)	-8.837	0.958	1.687					
Fatal and Injury[a] (FI[a])	-8.505	0.874	1.740					
Property damage only (PDO)	-	-	-	-	-	-	-	$(7)_{TOTAL}$ - $(7)_{FI}$

1886 NOTE: [a] Using the KABCO scale, these include only KAB accidents. Crashes with severity level C (possible injury) are not included.

Worksheet 1C (b) – Roadway Segment Accidents for Rural Multilane Undivided Roadway Segments								
(1)	(2)			(3)	(4)	(5)	(6)	(7)
Crash Severity Level	**SPF Coefficients**			**$N_{spf\ ru}$**	**Overdispersion Parameter, k**	**Combined AMFs**	**Calibration Factor, C_r**	**Predicted average crash frequency, $N_{predicted\ rs}$**
	from Exhibit 11-5			from Equation 11-7	from Equation 11-8	(6) from Worksheet 1B (b)		(3)*(5)*(6)
	a	b	c					
Total	-9.653	1.176	1.675					
Fatal and Injury (FI)	-9.410	1.094	1.796					
Fatal and Injury[a] (FI[a])	-8.577	0.938	2.003					
Property damage only (PDO)	-	-	-	-	-	-	-	$(7)_{TOTAL}$ - $(7)_{FI}$

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

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Worksheet 1D (a) – Crashes by Severity Level and Collision Type for Rural Multilane Divided Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Collision Type	**Proportion of Collision Type (TOTAL)**	$N_{predicted\ rs\ (TOTAL)}$ **(crashes/year)**	**Proportion of Collision Type (FI)**	$N_{predicted\ rs\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type (FI[a])**	$N_{predicted\ rs\ (FI^a)}$ **(crashes/year)**	**Proportion of Collision Type (PDO)**	$N_{predicted\ rs\ (PDO)}$
	from Exhibit 11-10	$(7)_{TOTAL}$ from Worksheet 1C (a)	from Exhibit 11-10	$(7)_{FI}$ from Worksheet 1C (a)	from Exhibit 11-10	$(7)_{FI}{}^{a}$ from Worksheet 1C (a)	from Exhibit 11-10	$(7)_{PDO}$ from Worksheet 1C (a)
Total	1.000		1.000		1.000		1.000	
		$(2)*(3)_{TOTAL}$		$(4)*(5)_{FI}$		$(6)*(7)_{FI}{}^{a}$		$(8)*(9)_{PDO}$
Head-on collision	0.006		0.013		0.018		0.002	
Sideswipe collision	0.043		0.027		0.022		0.053	
Rear-end collision	0.116		0.163		0.114		0.088	
Angle collision	0.043		0.048		0.045		0.041	
Single-vehicle collision	0.768		0.727		0.778		0.792	
Other collision	0.024		0.022		0.023		0.024	

1890 NOTE: [a] Using the KABCO scale, these include only KAB accidents. Crashes with severity level C (possible injury) are not included.

Worksheet 1D (b) – Accidents by Severity Level and Collision Type for Rural Multilane Undivided Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Collision Type	**Proportion of Collision Type (TOTAL)**	**$N_{predicted\ rs\ (TOTAL)}$ (crashes/year)**	**Proportion of Collision Type (FI)**	**$N_{predicted\ rs\ (FI)}$ (crashes/year)**	**Proportion of Collision Type (FI^a)**	**$N_{predicted\ rs\ (FI^a)}$ (crashes/year)**	**Proportion of Collision Type (PDO)**	**$N_{predicted\ rs\ (PDO)}$ (crashes/year)**
	from Exhibit 11-7	$(7)_{TOTAL}$ from Worksheet 1C (b)	from Exhibit 11-7	$(7)_{FI}$ from Worksheet 1C (b)	from Exhibit 11-7	$(7)_{FI}^{a}$ from Worksheet 1C (b)	from Exhibit 11-7	(7)PDO from Worksheet 1C (b)
Total	1.000		1.000		1.000		1.000	
		$(2)*(3)_{TOTAL}$		$(4)*(5)_{FI}$		$(6)*(7)_{FI}^{a}$		$(8)*(9)_{PDO}$
Head-on collision	0.009		0.029		0.043		0.001	
Sideswipe collision	0.098		0.048		0.044		0.120	
Rear-end collision	0.246		0.305		0.217		0.220	
Angle collision	0.356		0.352		0.348		0.358	
Single-vehicle collision	0.238		0.238		0.304		0.237	
Other collision	0.053		0.028		0.044		0.064	

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

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Worksheet 1E – Summary Results for Rural Multilane Roadway Segments

(1)	(2)	(3)	(4)
Crash severity level	**Predicted average crash frequency (crashes/year)**	**Roadway segment length (mi)**	**Crash rate (crashes/mi/year)**
	(7) from Worksheet 1C (a) or (b)		(2)/(3)
Total			
Fatal and injury (FI)			
Fatal and Injury[a] (FI[a])			
Property damage only (PDO)			

1894 NOTE: [a] Using the KABCO scale, these include only KAB accidents. Crashes with severity level C (possible injury) are not included.

Worksheet 2A – General Information and Input Data for Rural Multilane Highway Intersections

General Information		**Local Information**	
Analyst		Highway	
Agency or Company		Intersection	
Date Performed		Jurisdiction	
		Analysis Year	
Input Data		**Base Conditions**	**Site Conditions**
Intersection type (3ST, 4ST, 4SG)		-	
$AADT_{major}$ (veh/day)		-	
$AADT_{minor}$ (veh/day)		-	
Intersection skew angle (degrees)		0	
Number of signalized or uncontrolled approaches with a left turn lane (0,1,2,3,4)		0	
Number of signalized or uncontrolled approaches with a right turn lane (0,1,2,3,4)		0	
Intersection lighting (present/not present)		not present	
Calibration Factor, C_i		1.0	

Worksheet 2B – Accident Modification Factors for Rural Multilane Highway Intersections					
(1)	(2)	(3)	(4)	(5)	(6)
Crash Severity Level	AMF for Intersection Skew Angle	AMF for Left-Turn Lanes	AMF for Right-Turn Lanes	AMF for Lighting	Combined AMF
	AMF_{1i}	AMF_{2i}	AMF_{3i}	AMF_{4i}	
	from Equations 11-18 or 11-20 and 11-19 or 11-21	from Exhibit 11-32	from Exhibit 11-33	from Equation 11-22	(1)*(2)*(3)*(4)
Total					
Fatal and Injury (FI)					

Worksheet 2C – Intersection Crashes for Rural Multilane Highway Intersections									
(1)	(2)			(3)	(4)	(5)	(6)	(7)	
Crash Severity Level	**SPF Coefficients**			**$N_{spf\ int}$**	**Overdispersion Parameter, k**	**Combined AMFs**	**Calibration Factor**	**Predicted average crash frequency, $N_{predicted\ int}$**	
	form Exhibit 11-11 or 11-12			from Equation 11-11 or 11-12	from Exhibit 11-11 or 11-12	from (6) of Worksheet 2B	C_i	(3)*(5)*(6)	
	a	b	c						
Total									
Fatal and Injury (FI)									
Fatal and Injury[a] (FI^a)									
Property Damage Only (PDO)	-	-	-	-	-	-	-	$(7)_{TOTAL} - (7)_{FI}$	

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

Worksheet 2D – Accidents by Severity Level and Collision Type for Rural Multilane Highway Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Collision Type	**Proportion of Collision Type (TOTAL)**	**$N_{predicted\ int\ (TOTAL)}$ (crashes/year)**	**Proportion of Collision Type (FI)**	**$N_{predicted\ int\ (FI)}$ (crashes/year)**	**Proportion of Collision Type (FI^{a})**	**$N_{predicted\ int\ (FI^{a})}$ (crashes/year)**	**Proportion of Collision Type (PDO)**	**$N_{predicted\ int\ (PDO)}$ (crashes/year)**
	from Exhibit 11-16	$(7)_{TOTAL}$ from Worksheet 2C	from Exhibit 11-16	$(7)_{FI}$ from Worksheet 2C	from Exhibit 11-16	$(7)_{FI}{}^{a}$ from Worksheet 2C	from Exhibit 11-16	$(7)_{PDO}$ from Worksheet 2C
Total	1.000		1.000		1.000		1.000	
		$(2)*(3)_{TOTAL}$		$(4)*(5)_{FI}$		$(6)*(7)_{FI}{}^{a}$		$(8)*(9)_{PDO}$
Head-on collision								
Sideswipe collision								
Rear-end collision								
Angle collision								
Single-vehicle collision								
Other collision								

1900 NOTE: [a] Using the KABCO scale, these include only KAB accidents. Crashes with severity level C (possible injury) are not included.

Worksheet 2E – Summary Results for Rural Multilane Highway Intersections

(1)	(2)
Crash severity level	**Predicted average crash frequency (crashes/year)**
	(7) from Worksheet 2C
Total	
Fatal and Injury (FI)	
Fatal and Injury[a] (FI^{a})	
Property Damage Only (PDO)	

1901 NOTE: [a] Using the KABCO scale, these include only KAB accidents. Crashes with severity level C (possible injury) are not included.

Worksheet 3A – Predicted and Observed Crashes by Severity and Site Type Using the Site-Specific EB Method

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Site type	**Predicted average crash frequency (crashes/year)**			**Observed crashes, $N_{observed}$ (crashes/year)**	**Overdispersion parameter, k**	**Weighted adjustment, w**	**Expected average crash frequency, $N_{expected}$**
	$N_{predicted\ (TOTAL)}$	$N_{predicted\ (FI)}$	$N_{predicted\ (PDO)}$			Equation A-5 from Part C Appendix	Equation A-4 from Part C Appendix
ROADWAY SEGMENTS							
Segment 1							
Segment 2							
Segment 3							
Segment 4							
Segment 5							
Segment 6							
Segment 7							
Segment 8							
INTERSECTIONS							
Intersection 1							
Intersection 2							
Intersection 3							
Intersection 4							
Intersection 5							
Intersection 6							
Intersection 7							
Intersection 8							
COMBINED (sum of column)					-	-	

Worksheet 3B – Site-Specific EB Method Summary Results

(1)	(2)	(3)
Crash severity level	$\mathbf{N}_{predicted}$	$\mathbf{N}_{expected}$
Total	$(2)_{COMB}$ from Worksheet 3A	$(8)_{COMB}$ from Worksheet 3A
Fatal and injury (FI)	$(3)_{COMB}$ from Worksheet 3A	$(3)_{TOTAL}*(2)_{FI}/(2)_{TOTAL}$
Property damage only (PDO)	$(4)_{COMB}$ from Worksheet 3A	$(3)_{TOTAL}*(2)_{PDO}/(2)_{TOTAL}$

Worksheet 4A – Predicted and Observed Crashes by Severity and Site Type Using the Project-Level EB Method

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Site type	**Predicted average crash frequency (crashes/year)**			**Observed crashes, $N_{observed}$ (crashes/year)**	**Overdispersion Parameter, k**	N_{w0}	N_{w1}	w_0	N_0	w_1	N_1	$N_{p/comb}$
	$N_{predicted}$ (TOTAL)	$N_{predicted}$ (FI)	$N_{predicted}$ (PDO)			Equation A-8 (6)* (2)^2	Equation A-9 sqrt((6)*(2))	Equation A-10	Equation A-11	Equation A-12	Equation A-13	Equation A-14
ROADWAY SEGMENTS												
Segment 1				-				-	-	-	-	-
Segment 2				-				-	-	-	-	-
Segment 3				-				-	-	-	-	-
Segment 4				-				-	-	-	-	-
Segment 5				-				-	-	-	-	-
Segment 6				-				-	-	-	-	-
Segment 7				-				-	-	-	-	-
Segment 8				-				-	-	-	-	-
INTERSECTIONS												
Intersection 1				-				-	-	-	-	-
Intersection 2				-				-	-	-	-	-
Intersection 3				-				-	-	-	-	-
Intersection 4				-				-	-	-	-	-
Intersection 5				-				-	-	-	-	-
Intersection 6				-				-	-	-	-	-
Intersection 7				-				-	-	-	-	-
Intersection 8				-				-	-	-	-	-
COMBINED (sum of column)					-							

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Worksheet 4B – Project-Level EB Method Summary Results		
(1)	(2)	(3)
Crash severity level	**$N_{predicted}$**	**$N_{expected}$**
Total	$(2)_{COMB}$ from Worksheet 4A	$(13)_{COMB}$ from Worksheet 4A
Fatal and injury (FI)	$(3)_{COMB}$ from Worksheet 4A	$(3)_{TOTAL}*(2)_{FI}/(2)_{TOTAL}$
Property damage only (PDO)	$(4)_{COMB}$ from Worksheet 4A	$(3)_{TOTAL}*(2)_{PDO}/(2)_{TOTAL}$

APPENDIX B – PREDICTIVE MODELS FOR SELECTED COLLISION TYPES

The main text of this chapter presents predictive models for accidents by severity level. Tables with accident proportions by collision type are also presented to allow estimates for accident frequencies by collision type to be derived from the accident predictions for specific severity levels. Safety prediction models are also available for some, but not all, collision types. These safety prediction models are presented in this appendix for application by HSM users, where appropriate. Users should generally expect that a more accurate safety prediction for a specific collision type can be obtained using a model developed specifically for that collision type than using a model for all collision types combined and multiplying the result by the proportion of that specific collision type of interest. However, prediction models are available only for selected collision types. And, such models must be used with caution by HSM users, because the results of a series of collision models for individual collision types will not necessarily sum to the predicted accident frequency for all collision types combined. In other words, when predicted accident frequencies for several collision types are used together, some adjustment of those predicted accident frequencies may be required to assure that their sum is consistent with results from the models presented in the main text of this chapter.

B.1 UNDIVIDED ROADWAY SEGMENTS

Exhibit 11-39 summarizes the values for the coefficients used in prediction models that apply Equation 11-4 for estimating accident frequencies by collision type for undivided roadway segments. Two specific collision types are addressed: single-vehicle and opposite-direction collisions without turning movements (SvOdn) and same-direction collisions without turning movements (SDN). These models are assumed to apply for base conditions represented as the average value of the variables in a jurisdiction. There are no AMFs for use with these models; the accident predictions provided by these models are assumed to apply to average conditions for these variables for which AMFs are provided in Section 11.7.

Exhibit 11-39: SPFs for Selected Collision Types on Four-Lane Undivided Roadway Segments (Based on Equation 11-4)

Severity level / collision type	a	b	Overdispersion parameter (fixed k)[a]
Total – SvOdn	-5.345	0.696	0.777
Fatal and injury-SvOdn	-7.224	0.821	0.946
Fatal and injury[b] – SvOdn	-7.244	0.790	0.962
Total – SDN	-14.962	1.621	0.525
Fatal and injury -SDN	-12.361	1.282	0.218
Fatal and injury[b] – SDN	-14.980	1.442	0.514

NOTE: SvOdn - Single Vehicle and Opposite Direction without Turning Movements Crashes (note: these two crash types were modeled together)
SDN - Same Direction without Turning Movement (note: this is a subset of all rear-end collisions)
[a] This value should be used directly as the overdispersion parameter; no further computation is required.
[b] Excluding accidents involving only possible injuries.

DIVIDED ROADWAY SEGMENTS

No models by collision type are available for divided roadway segments on rural multilane highways.

STOP-CONTROLLED INTERSECTIONS

Exhibit 11-40 summarizes the values for the coefficients used in prediction models that apply Equation 11-4 for estimating accident frequencies by collision type for stop-controlled intersections on rural multilane highways. Four specific collision types are addressed:

- Single-vehicle collisions
- Intersecting direction collisions (angle and left-turn-through collisions)
- Opposing-direction collisions (head-on collisions)
- Same-direction collisions (rear-end collisions)

Exhibit 11-40 presents values for the coefficients a, b, c, and d used in applying Equations 11-11 and 11-12 for predicting crashes by collision type for three- and four-leg intersections with minor-leg stop-control. The intersection types and severity levels for which values are shown for coefficients a, b, and c are addressed with the SPF shown in Equation 11-11. The intersection types and severity levels for which values are shown for coefficients a and d are addressed with the SPF shown in Equation 11-12. The models presented in this exhibit were developed for intersections without specific base conditions. Thus, when using these models for predicting accident frequencies, no AMFs should be used and it is assumed that the predictions apply to typical or average conditions for the AMFs presented in Section 11.7.

Exhibit 11-40: Collision Type Models for Stop-Controlled Intersections without Specific Base Conditions (Based on Equations 11-11 and 11-12)

Intersection type/ severity level/collision type	a	b	c	d	Overdispersion parameter (fixed k)[a]
4ST Total Single Vehicle	-9.999	-	-	0.950	0.452
4ST Fatal and injury Single Vehicle	-10.259			0.884	0.651
4ST Fatal and injury[b] Single Vehicle	-9.964	-	-	0.800	1.010
4ST Total Int. Direction	-7.095	0.458	0.462	-	1.520
4ST Fatal and injury Int. Direction	-7.807	0.467	0.505		1.479
4ST Fatal and injury[b] Int. Direction	-7.538	0.441	0.420	-	1.506
4ST Total Opp. Direction	-8.539	0.436	0.570	-	1.068
4ST Fatal and injury Opp. Direction	10.274	0.465	0.529		1.453
4ST Fatal and injury[b] Opp. Direction	-10.058	0.497	0.547	-	1.426

Intersection type/ severity level/collision type	a	b	c	d	Overdispersion parameter (fixed k)[a]
4ST Total Same Direction	-11.460	0.971	0.291	-	0.803
4ST Fatal and injury Same Direction	-11.602	0.932	0.246		0.910
4ST Fatal and injury[b] Same Direction	-13.223	1.032	0.184	-	1.283
3ST Total Single Vehicle	-10.986	-	-	1.035	0.641
3ST Fatal and injury Single Vehicle	-10.835			0.934	0.741
3ST Fatal and injury[b] Single Vehicle	-11.608	-	-	0.952	0.838
3ST Total Int. Direction	-10.187	0.671	0.529	-	1.184
3ST Fatal and injury Int. Direction	-11.171	0.749	0.487		1.360
3ST Fatal and injury[b] Int. Direction	-12.084	0.442	0.796	-	1.5375
3ST Total Opp. Direction	-13.808	1.043	0.425	-	1.571
3ST Fatal and injury Opp. Direction	-14.387	1.055	0.432		1.629
3ST Fatal and injury[b] Opp. Direction	-15.475	0.417	1.105		1.943
3ST Total Same Direction	-15.457	1.381	0.306		0.829
3ST Fatal and injury Same Direction	-14.838	1.278	0.227		0.754
3ST Fatal and injury[b] Same Direction	-14.736	1.199	0.147		0.654

NOTE: Int. Direction = Intersecting Direction (angle and left-turn-through crashes)
Opp. Direction = Opposing Direction (head-on)

[a] This value should be used directly as the overdispersion parameter; no further computation is required.

[b] Excluding accidents involving only possible injuries.

SIGNALIZED INTERSECTIONS

No models by collision type are available for signalized intersections on rural multilane highways.

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PART C— PREDICTIVE METHOD

CHAPTER 12— PREDICTIVE METHOD FOR URBAN AND SUBURBAN ARTERIALS

12.1. Introduction 12-6

12.2. Overview of the Predictive Method 12-6

12.3. Urban and Suburban Arterials – Definitions and Predictive Models In Chapter 12 12-7

12.3.1. Definition of Chapter 12 Facility Types 12-8

12.3.2. Predictive Models for Urban and Suburban Arterial Roadway Segments 12-10

12.3.3. Predictive Models for Urban and Suburban Arterial Intersections 12-11

12.4. Predictive Method Steps for Urban and suburban arterials 12-12

12.5. Roadway Segments and Intersections 12-22

12.6. Safety Performance Functions for Base Conditions 12-24

12.6.1. Safety Performance Functions for Urban and Suburban Arterial Roadway Segments 12-25

12.6.2. Safety Performance Functions for Urban and Suburban Arterial Intersections 12-36

12.7. Accident ModiFication Factors 12-46

12.7.1. Accident Modification Factors for Roadway Segments 12-47

12.7.2. Accident Modification Factors for Intersections 12-52

12.8. Calibration of the SPFs to Local Conditions 12-57

12.9. Interim Predictive Method for Roundabouts 12-58

12.10. Limitations of Predictive Method in Chapter 12 12-58

12.11. Application of Chapter 12 Predictive method 12-58

12.12. Summary 12-58

12.13. Sample Problems 12-59

12.13.1. Sample Problem 1 12-61

12.13.2. Sample Problem 2 12-79

12.13.3. Sample Problem 3 12-95

12.13.4. Sample Problem 4 12-109

12.13.5. Sample Problem 5 12-123

12.13.6. Sample Problem 6 12-130

12.14. References 12-136

EXHIBITS

Exhibit 12-1: Urban and Suburban Arterial Site Type SPFs included in Chapter 12......12-9

Exhibit 12-2: The HSM Predictive Method......12-13

Exhibit 12-3: Definition of Roadway Segments and Intersections......12-22

Exhibit 12-4: Safety Performance Functions included in Chapter 12......12-24

Exhibit 12-5: SPF Coefficients for Multiple-Vehicle Nondriveway Collisions on Roadway Segments......12-26

Exhibit 12-6: Graphical Form of the SPF for Multiple Vehicle Nondriveway collisions (from Equation 12-10 and Exhibit 12-5)......12-27

Exhibit 12-7: Distribution of Multiple-Vehicle Nondriveway Collisions for Roadway Segments by Manner of Collision Type......12-28

Exhibit 12-8: SPF Coefficients for Single-Vehicle Crashes on Roadway Segments......12-29

Exhibit 12-9: Graphical Form of the SPF for Single-Vehicle Crashes (from Equation 12-13 and Exhibit 12-8)......12-29

Exhibit 12-10: Distribution of Single-Vehicle Crashes for Roadway Segments by Collision Type......12-30

Exhibit 12-11: SPF Coefficients for Multiple-Vehicle Driveway Related Collisions......12-31

Exhibit 12-12: Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Two-Lane Undivided Arterials (2U) (from Equation 12-16 and Exhibit 12-11)......12-32

Exhibit 12-13: Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Three-Lane Undivided Arterials (3T) (from Equation 12-16 and Exhibit 12-11)......12-32

Exhibit 12-14: Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on on Four-Lane Undivided Arterials (4U)(from Equation 12-16 and Exhibit 12-11)......12-33

Exhibit 12-15: Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Four-Lane Divided Arterials(4D)(from Equation 12-16 and Exhibit 12-11)......12-33

Exhibit 12-16: Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Five-Lane Arterials Including a Center Two-Way Left-Turn Lane)(from Equation 12-16 and Exhibit 12-11)......12-34

Exhibit 12-17: Pedestrian Accident Adjustment Factor for Roadway Segments......12-35

Exhibit 12-18: Bicycle Accident Adjustment Factors for Roadway Segments......12-35

Exhibit 12-19: SPF Coefficients for Multiple-Vehicle Collisions at Intersections......12-38

Exhibit 12-20: Graphical Form of the Intersection SPF for Multiple Vehicle collisons on Three-Leg Intersections with Minor-Road Stop Control (3ST) (from Equation 12-21 and Exhibit 12-19)......12-38

Exhibit 12-21: Graphical Form of the Intersection SPF for Multiple Vehicle collisons on Three-Leg Signalized Intersections (3SG) (from Equation 12-21 and Exhibit 12-19)......12-39

Exhibit 12-22: Graphical Form of the Intersection SPF for Multiple Vehicle collisons on Four-Leg Intersections with Minor-Road Stop Control (4ST) (from Equation 12-21 and Exhibit 12-19)......12-39

Exhibit 12-23: Graphical Form of the Intersection SPF for Multiple Vehicle collisons on Four-Leg Signalized Intersections (4SG) (from Equation 12-21 and Exhibit 12-19) 12-40

Exhibit 12-24: Distribution of Multiple-Vehicle Collisions for Intersections by Collision Type 12-40

Exhibit 12-25: SPF Coefficients for Single-Vehicle Crashes at Intersections.......... 12-41

Exhibit 12-26: Graphical Form of the Intersection SPF for Single-Vehicle Crashes on Three-leg Intersections with Minor-Road Stop Control (3ST) (from Equation 12-24 and Exhibit 12-25) 12-41

Exhibit 12-27: Graphical Form of the Intersection SPF for Single-Vehicle Crashes on Three-Leg Signalized Intersections (3SG) (from Equation 12-24 and Exhibit 12-25) 12-42

Exhibit 12-28: Graphical Form of the Intersection SPF for Single-Vehicle Crashes on Four-leg Stop Controlled Intersections (4ST) (from Equation 12-24 and Exhibit 12-25) 12-42

Exhibit 12-29: Graphical Form of the Intersection SPF for Single-Vehicle Crashes on Four-Leg Signalized Intersections (4SG) (from Equation 12-24 and Exhibit 12-25) 12-43

Exhibit 12-30: Distribution of Single-Vehicle Crashes for Intersection by Collision Type 12-43

Exhibit 12-31: SPFs for Vehicle-Pedestrian Collisions at Signalized Intersections 12-45

Exhibit 12-32: Estimates of Pedestrian Crossing Volumes Based on General Level of Pedestrian Activity 12-45

Exhibit 12-33: Pedestrian Accident Adjustment Factors for STOP-controlled Intersections.......... 12-46

Exhibit 12-34: Bicycle Accident Adjustment Factors for Intersections 12-46

Exhibit 12-35: Summary of AMFs in Chapter 12 and the Corresponding SPFs 12-47

Exhibit 12-36: Values of f_{pk} Used in Determining the Accident Modification Factor for On-street Parking.......... 12-48

Exhibit 12-37: Fixed-Object Offset Factor.......... 12-49

Exhibit 12-38: Proportion of Fixed-Object Collisions 12-50

Exhibit 12-39: AMFs for Median Widths on Divided Roadway Segments without a Median Barrier (AMF_{3r}) 12-50

Exhibit 12-40: Nighttime Crash Proportions for Unlighted Roadway Segments 12-51

Exhibit 12-41: Accident Modification Factor (AMF_{1i}) for Installation of Left-Turn Lanes on Intersection Approaches.......... 12-52

Exhibit 12-42: Accident Modification Factor (AMF_{2i}) for Type of Left-Turn Signal Phasing 12-53

Exhibit 12-43: Accident Modification Factor (AMF_{3i}) for Installation of Right-Turn Lanes on Intersection Approaches.......... 12-53

Exhibit 12-44: Nighttime Crash Proportions for Unlighted Intersections.......... 12-54

Exhibit 12-45: Accident Modification Factor (AMF_{1p}) for the Presence of Bus Stops Near the Intersection 12-56

Exhibit 12-46: Accident Modification Factor (AMF_{2p}) for the Presence of Schools near the Intersection.......... 12-56

Exhibit 12-47: Accident Modification Factor (AMF_{3p}) for the Number of Alcohol Sales Establishments Near the Intersection..12-57

Exhibit 12-48: List of Sample Problems in Chapter 12..12-60

APPENDIX A

Appendix A - Worksheets for Predictive Method for Urban and Suburban Arterials... 12-138

CHAPTER 12 URBAN AND SUBURBAN ARTERIALS

12.1. INTRODUCTION

Chapter 12 presents the predictive method for urban and suburban arterials.

This chapter presents the predictive method for urban and suburban arterial facilities. A general introduction to the Highway Safety Manual (HSM) predictive method is provided in the *Part C Introduction and Applications Guidance*.

The predictive method for urban or suburban arterial facilities provides a structured methodology to estimate the expected average crash frequency, crash severity and collision types for facilities with known characteristics. All types of crashes involving vehicles of all types, bicycles, and pedestrians are included, with the exception of crashes between bicycles and pedestrians. The predictive method can be applied to existing sites, design alternatives to existing sites, new sites or for alternative traffic volume projections. An estimate can be made for crash frequency in a period of time that occurred in the past (i.e., what did or would have occurred) or in the future (i.e., what is expected to occur). The development of the SPFs in Chapter 12 is documented by Harwood et al.(1). The AMFs used in this chapter have been reviewed and updated by Harkey et al.(2) and in related work by Srinivasan et al.(3) The SPF coefficients, default collision type distributions, and default nighttime accident proportions have been adjusted to a consistent basis by Srinivasan et al.(4).

This chapter presents the following information about the predictive method for urban and suburban arterial facilities:

- A concise overview of the predictive method.
- The definitions of the facility types included in Chapter 12, and site types for which predictive models have been developed for Chapter 12.
- The steps of the predictive method in graphical and descriptive forms.
- Details for dividing an urban or suburban arterial facility into individual sites, consisting of intersections and roadway segments.
- Safety Performance Functions (SPFs) for urban and suburban arterials.
- Accident Modification Factors (AMFs) applicable to the SPFs in Chapter 12.
- Guidance for applying the Chapter 12 predictive method, and limitations of the predictive method specific to Chapter 12.
- Sample problems illustrating the application of the Chapter 12 predictive method for urban and suburban arterials.

12.2. OVERVIEW OF THE PREDICTIVE METHOD

The predictive method provides an 18 step procedure to estimate the "expected average crash frequency", $N_{expected}$ (by total crashes, crash severity or collision type) of a roadway network, facility or site. In the predictive method the roadway is divided into individual sites, which are homogenous roadway segments and intersections. A facility consists of a contiguous set of individual intersections and roadway segments, referred to as "sites." Different facility types are determined by surrounding land use, roadway cross-section, and degree of access. For each facility type, a number of different site types may exist, such as divided and undivided roadway segments, and unsignalized and signalized intersections. A roadway network consists of a number of contiguous facilities.

The method is used to estimate the expected average crash frequency of an individual site, with the cumulative sum of all sites used as the estimate for an entire facility or network. The estimate is for a given time period of interest (in years) during which the geometric design and traffic control features are unchanged and traffic volumes (AADT) are known or forecasted. The estimate relies on estimates made using predictive models which are combined with observed crash data using the Empirical Bayes (EB) Method.

The predictive models used within the Chapter 12 predictive method are described in detail in Section 12.3.

The predictive models used in Chapter 12 to predict average crash frequency $N_{predicted}$, are of the general form shown in Equation 12-1.

Section 12.6 provides the predictive models in Chapter 12.

$$N_{predicted} = (N_{spf\ x} \times (AMF_{1x} \times AMF_{2x} \times \ldots \times AMF_{yx}) + N_{pedx} + N_{bikex}) \times C_x \qquad (12\text{-}1)$$

Where,

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

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

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

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

AMF_{yx} = Accident Modification Factors specific to site type x and specific geometric design and traffic control features y;

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

The predictive models in Chapter 12 provide estimates of the crash severity and collision type distributions for roadway segments and intersections. The SPFs in Chapter 12 address two general crash severity levels: fatal-and-injury and property-damage-only crashes. Fatal-and-injury crashes include crashes involving all levels of injury severity including fatalities, incapacitating injuries, nonincapacitating injuries, and possible injuries. The relative proportions of crashes for the two severity levels are determined from separate SPFs for each severity level. The default estimates of the crash severity and crash type distributions are provided with the SPFs for roadway segments and intersections in Section 12.6.

12.3. URBAN AND SUBURBAN ARTERIALS – DEFINITIONS AND PREDICTIVE MODELS IN CHAPTER 12

This section provides the definitions of the facility and site types included in Chapter 12, and the predictive models for each of the site types included in Chapter 12. These predictive models are applied following the steps of the predictive method presented in Section 12.4.

12.3.1. Definition of Chapter 12 Facility Types

The predictive method in Chapter 12 addresses the following urban and suburban arterial facilities: two- and four-lane undivided facilities, four-lane divided facilities, and three- and five-lane facilities with center two-way left-turn lanes. Divided arterials are nonfreeway facilities (i.e., facilities without full control of access) that have lanes in the two directions of travel separated by a raised or depressed median. Such facilities may have occasional grade-separated interchanges, but these are not the primary form of access. The predictive models do not apply to any section of an arterial within the limits of an interchange which has free-flow ramp terminals on the arterial of interest. Arterials with a flush separator (i.e., a painted median) between the lanes in the two directions of travel are considered undivided facilities, not divided facilities. Separate prediction models are provided for arterials with a flush separator that serves as a center two-way left-turn lane. Chapter 12 does not address arterial facilities with six or more lanes.

The terms “highway” and “road” are used interchangeably in this chapter and apply to all urban and suburban arterials independent of official state or local highway designation.

Classifying an area as urban, suburban or rural is subject to the roadway characteristics, surrounding population and land uses and is at the user’s discretion. In the HSM, the definition of “urban” and “rural” areas is based on Federal Highway Administration (FHWA) guidelines which classify “urban” areas as places inside urban boundaries where the population is greater than 5,000 persons. “Rural” areas are defined as places outside urban areas with populations greater than 5,000 persons. The HSM uses the term “suburban” to refer to outlying portions of an urban area; the predictive method does not distinguish between urban and suburban portions of a developed area. The term “arterial” refers to facilities the meet the FHWA definition of “roads serving major traffic movements (high-speed, high volume) for travel between major points.”[(5)]

Exhibit 12-1 identifies the specific site types on urban and suburban arterial highways that have predictive models. In Chapter 12, separate SPFs are used for each individual site to predict multiple-vehicle nondriveway collisions, single-vehicle collisions, driveway-related collisions, vehicle-pedestrian collisions, and vehicle-bicycle collisions for both roadway segments and intersections. These are combined to predict the total average crash frequency at an individual site.

Exhibit 12-1: Urban and Suburban Arterial Site Type SPFs included in Chapter 12

Site Type	Site Types with SPFs in Chapter 12
Roadway Segments	Two-lane undivided arterials (2U)
	Three-lane arterials including a center two-way left-turn lane(TWLTL) (3T)
	Four-lane undivided arterials (4U)
	Four-lane divided arterials (i.e., including a raised or depressed median) (4D)
	Five-lane arterials including a center TWLTL (5T)
Intersections	Unsignalized three-leg intersection (Stop control on minor-road approaches) (3ST)
	Signalized three-leg intersections (3SG)
	Unsignalized four-leg intersection (Stop control on minor-road approaches) (4ST)
	Signalized four-leg intersection (4SG)

These specific site types are defined as follows:

This section defines urban and suburban arterial site types.

- Two-lane undivided arterial (2U) – a roadway consisting of two lanes with a continuous cross-section providing two directions of travel in which the lanes are not physically separated by either distance or a barrier.
- Three-lane arterials (3T) - a roadway consisting of three lanes with a continuous cross-section providing two directions of travel in which center lane is a two-way left turn lane (TWLTL).
- Four-lane undivided arterials (4U) – a roadway consisting of four lanes with a continuous cross-section providing two directions of travel in which the lanes are not physically separated by either distance or a barrier.
- Four-lane divided arterials (i.e., including a raised or depressed median) (4D) – a roadway consisting of two lanes with a continuous cross-section providing two directions of travel in which the lanes are physically separated by either distance or a barrier.
- Five-lane arterials including a center TWLTL (5T) - a roadway consisting of five lanes with a continuous cross-section providing two directions of travel in which the center lane is a two-way left-turn lane (TWLTL).
- Three-leg intersection with STOP control (3ST) – an intersection of a urban or suburban arterial and a minor road. A STOP sign is provided on the minor road approach to the intersection only.
- Four-leg intersection with STOP control (4ST) – an intersection of a urban or suburban arterial and two minor roads. A STOP sign is provided on both the minor road approaches to the intersection.

- Four-leg signalized intersection (4SG) - an intersection of a urban or suburban arterial and two minor roads. Signalized control is provided at the intersection by traffic lights.

- Three-leg signalized intersection (3SG) - an intersection of a urban or suburban arterial and one minor road. Signalized control is provided at the intersection by traffic lights.

12.3.2. Predictive Models for Urban and Suburban Arterial Roadway Segments

The predictive models can be used to estimate total average crashes (i.e., all crash severities and collision types) or can be used to predict average frequency of specific crash severity types or specific collision types. The predictive model for an individual roadway segment or intersection combines the SPF, AMFs, and a calibration factor. Chapter 12 contains separate predictive models for roadway segments and for intersections.

The predictive models for roadway segments estimate the predicted average crash frequency of non-intersection-related crashes. Non-intersection-related crashes may include crashes that occur within the limits of an intersection but are not related to the intersection. The roadway segment predictive models estimate crashes that would occur regardless of the presence of the intersection.

In contrast to the chapter 10 and 11 predictive method, in chapter 12 roadway segment crash frequency is estimated as the sum of three components: N_{br}, N_{pedr}, and N_{biker}.

The predictive models for roadway segments are presented in Equations 12-2 and 12-3 below.

$$N_{predicted\ rs} = C_r \times (N_{br} + N_{pedr} + N_{biker}) \qquad (12\text{-}2)$$

$$N_{br} = N_{spf\ rs} \times (AMF_{1r} \times AMF_{2r} \times \ldots \times AMF_{nr}) \qquad (12\text{-}3)$$

Where,

$N_{predicted\ rs}$ = predicted average crash frequency of an individual roadway segment for the selected year;

N_{br} = predicted average crash frequency of an individual roadway segment (excluding vehicle-pedestrian and vehicle-bicycle collisions);

$N_{spf\ rs}$ = predicted total average crash frequency of an individual roadway segment for base conditions (excluding vehicle-pedestrian and vehicle-bicycle collisions);

N_{pedr} = predicted average crash frequency of vehicle-pedestrian collisions for an individual roadway segment;

N_{biker} = predicted average crash frequency of vehicle-bicycle collisions for an individual roadway segment;

$AMF_{1r} \ldots AMF_{nr}$ = Accident Modification Factors for roadway segments;

C_r = calibration factor for roadway segments of a specific type developed for use for a particular geographical area.

Equation 12-2 shows that roadway segment crash frequency is estimated as the sum of three components: N_{br}, N_{pedr}, and N_{biker}. The following equation shows that the

SPF portion of N_{br}, designated as $N_{spf\ rs}$, is further separated into three components by collision type shown in Equation 12-4:

$$N_{spf\ rs} = N_{brmv} + N_{brsv} + N_{brdwy} \tag{12-4}$$

Where,

N_{brmv} = predicted average crash frequency of multiple-vehicle nondriveway collisions for base conditions;

N_{brsv} = predicted average crash frequency of single-vehicle crashes for base conditions; and

N_{brdwy} = predicted average crash frequency of multiple-vehicle driveway-related collisions.

Thus, the SPFs and adjustment factors are applied to determine five components: N_{brmv}, N_{brsv}, N_{brdwy}, N_{pedr}, and N_{biker}, which together provide a prediction of total average crash frequency for a roadway segment.

Equations 12-2 through 12-4 are applied to estimate roadway segment crash frequencies for all crash severity levels combined (i.e., total crashes) or for fatal-and-injury or property-damage-only crashes.

12.3.3. Predictive Models for Urban and Suburban Arterial Intersections

The predictive models for intersections estimate the predicted total average crash frequency including those crashes that occur within the limits of an intersection and are a result of the presence of the intersection. The predictive model for an urban or suburban arterial intersection is given by:

$$N_{predicted\ int} = C_i \times (N_{bi} + N_{pedi} + N_{bikei}) \tag{12-5}$$

$$N_{bi} = N_{spf\ int} \times (AMF_{1i} \times AMF_{2i} \times \ldots \times AMF_{6i}) \tag{12-6}$$

Where,

N_{int} = predicted average crash frequency of an intersection for the selected year;

N_{bi} = predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle collisions);

$N_{spf\ int}$ = predicted total average crash frequency of intersection-related crashes for base conditions (excluding vehicle-pedestrian and vehicle-bicycle collisions);

N_{pedi} = predicted average crash frequency of vehicle-pedestrian collisions;

N_{bikei} = predicted average crash frequency of vehicle-bicycle collisions;

$AMF_{1i} \ldots AMF_{6i}$ = Accident Modification Factors for intersections;

C_i = calibration factor for intersections developed for use for a particular geographical area.

The AMFs shown in Equation 12-6 do not apply to vehicle-pedestrian and vehicle-bicycle collisions. A separate set of AMFs that apply to vehicle-pedestrian collisions at signalized intersections is presented in Section 12.7.

Equation 12-5 shows that the intersection crash frequency is estimated as the sum of three components: N_{bi}, N_{pedi}, and N_{bikei}. The following equation shows that the SPF portion of N_{bi}, designated as $N_{spf\ int}$, is further separated into two components by collision type:

$$N_{spf\ int} = N_{bimv} + N_{bisv} \tag{12-7}$$

Where,

N_{bimv} = predicted average number of multiple-vehicle collisions for base conditions, and

N_{bisv} = predicted average number of single-vehicle collisions for base conditions.

The SPFs for urban and suburban arterial highways are presented in Section 12.6.

Thus, the SPFs and adjustment factors are applied to determine four components of total intersection average crash frequency: N_{bimv}, N_{bisv}, N_{pedi}, and N_{bikei}.

The SPFs for urban and suburban arterial highways are presented in Section 12.6. The associated AMFs for each of the SPFs are presented in Section 12.7, and summarized in Exhibit 12-35. Only the specific AMFs associated with each SPF are applicable to an SPF (as these AMFs have base conditions which are identical to the base conditions). The calibration factors, C_r and C_i are determined in the *Part C* Appendix A.1.1. Due to continual change in the crash frequency and severity distributions with time, the value of the calibration factors may change for the selected year of the study period.

12.4. PREDICTIVE METHOD STEPS FOR URBAN AND SUBURBAN ARTERIALS

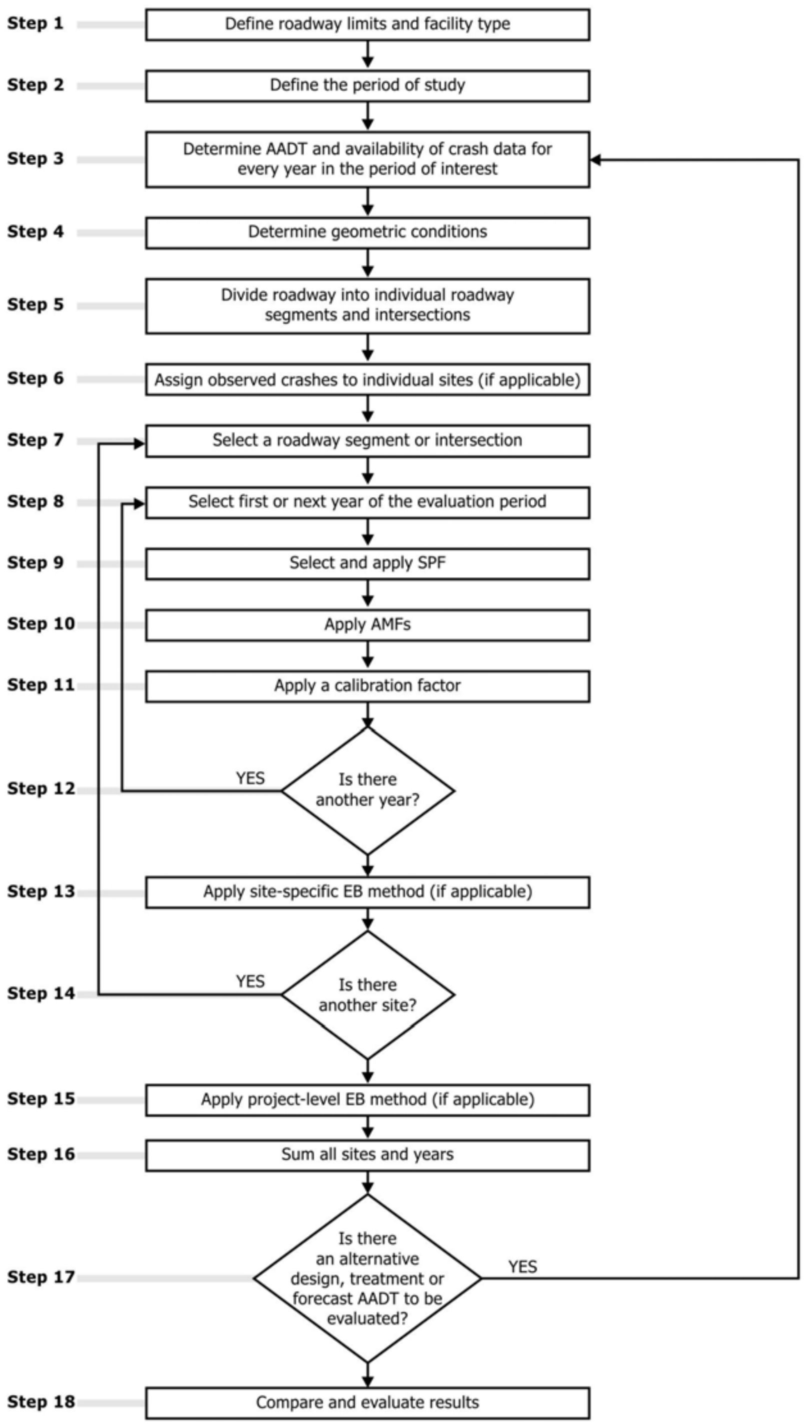
The predictive method is described in detail in the Part C Introduction and Applications Guide.

The predictive method for urban and suburban arterials is shown in Exhibit 12-2. Applying the predictive method yields an estimate of the expected average crash frequency (and/or crash severity and collision types) for an urban or suburban arterial facility. The components of the predictive models in Chapter 12 are determined and applied in Steps 9, 10 and 11 of the predictive method. The information to apply each step is provided in the following sections and in the *Part C* Appendix. In some situations, certain steps will not require any action. For example, a new facility will not have observed crash data and therefore steps relating to the EB Method require no action.

There are 18 steps in the predictive method. In some situations certain steps will not be needed because data is not available or the step is not applicable to the situation at hand. In other situations, steps may be repeated if an estimate is desired for several sites or for a period of several years. In addition, the predictive method can be repeated as necessary to undertake crash estimation for each alternative design, traffic volume scenario or proposed treatment option (within the same period to allow for comparison).

The following explains the details of each step of the method as applied to urban and suburban arterials.

Exhibit 12-2: The HSM Predictive Method



This section describes each step of the predictive method in the context of urban and suburban arterials.

Step 1 - Define the limits of the roadway and facility types in the study network, facility or site for which the expected average crash frequency, severity and collision types are to be estimated.

The predictive method can be undertaken for a roadway network, a facility, or an individual site. A site is either an intersection or a homogeneous roadway segment. Sites may consist of a number of types, such as signalized and unsignalized intersections. The definitions of urban and suburban arterials, intersections and roadway segments and the specific site types included in Chapter 12 are provided in Section 12-3.

The predictive method can be undertaken for an existing roadway, a design alternative for an existing, or a new roadway (which may be either unconstructed or yet to experience enough traffic to have observed crash data).

The limits of the roadway of interest will depend on the nature of the study. The study may be limited to only one specific site or a group of contiguous sites. Alternatively, the predictive method can be applied to a very long corridor for the purposes of network screening which is discussed in *Chapter 4*.

Step 2 - Define the period of interest.

The predictive method can be undertaken for either a past period or a future period. All periods are measured in years. Years of interest will be determined by the availability of observed or forecast AADTs, observed crash data and geometric design data. Whether the predictive method is used for a past or future period depends upon the purpose of the study. The period of study may be:

A past period (based on observed AADTs) for:

- An existing roadway network, facility or site. If observed crash data are available, the period of study is the period of time for which the observed crash data are available and for which (during that period) the site geometric design features, traffic control features and traffic volumes are known.
- An existing roadway network, facility or site for which alternative geometric design features or traffic control features are proposed (for near term conditions).

A future period (based on forecast AADTs) for:

- An existing roadway network, facility or site for a future period where forecast traffic volumes are available.
- An existing roadway network, facility or site for which alternative geometric design or traffic control features are proposed for implementation in the future.
- A new roadway network, facility or site that does not currently exist, but is proposed for construction during some future period.

Step 3 – For the study period, determine the availability of annual average daily traffic volumes, pedestrian crossing volumes and, for an existing roadway network, the availability of observed crash data (to determine whether the EB Method is applicable).

Determining Traffic Volumes

The SPFs used in Step 9 (and some AMFs in Step 10), include AADT volumes (vehicles per day) as a variable. For a past period the AADT may be determined by automated recording or estimated by a sample survey. For a future period the AADT may be a forecast estimate based on appropriate land use planning and traffic volume forecasting models, or based on the assumption that current traffic volumes will remain relatively constant.

For each roadway segment, the AADT is the average daily two-way 24 hour traffic volume on that roadway segment in each year of the period to be evaluated selected in Step 8.

For each intersection, two values are required in each predictive model. These are: the two-way AADT of the major street, $AADT_{maj}$ and the two-way AADT of the minor street, $AADT_{min}$.

$AADT_{maj}$ and $AADT_{min}$ are determined as follows: if the AADTs on the two major-road legs of an intersection differ, the larger of the two AADT values is used for the intersection. If the AADTs on the two minor road legs of a four-leg intersection differ, the larger of the AADTs for the two minor road legs is used. For a three-leg intersection, the AADT of the single minor road leg is used. If AADTs are available for every roadway segment along a facility, the major-road AADTs for intersection legs can be determined without additional data.

In many cases, it is expected that AADT data will not be available for all years of the evaluation period. In that case, an estimate of AADT for each year of the evaluation period is interpolated or extrapolated as appropriate. If there is not an established procedure for doing this, the following may be applied within the predictive method to estimate the AADTs for years for which data are not available.

- If AADT data are available for only a single year, that same value is assumed to apply to all years of the before period;
- If two or more years of AADT data are available, the AADTs for intervening years are computed by interpolation;
- The AADTs for years before the first year for which data are available are assumed to be equal to the AADT for that first year;
- The AADTs for years after the last year for which data are available are assumed to be equal to the last year.

If the EB Method is used (discussed below) AADT data are needed for each year of the period for which observed crash frequency data are available. If the EB Method will not be used, AADT data for the appropriate time period—past, present, or future—determined in Step 2 are used.

For signalized intersections, the pedestrian volumes crossing each intersection leg are determined for each year of the period to be evaluated. The pedestrian crossing volumes for each leg of the intersection are then summed to determine the total pedestrian crossing volume for the intersection. Where pedestrian volume counts are not available, they may be estimated using the guidance presented in Exhibit 12-32. Where pedestrian volume counts are not available for each year, they may be interpolated or extrapolated in the same manner as explained above for AADT data.

Determining Availability of Observed Crash Data

The EB Method and criteria to determine whether the EB Method is applicable are presented in Section A.2.1 in the Appendix to Part C.

Where an existing site or alternative conditions for an existing site are being considered, the EB Method is used. The EB Method is only applicable when reliable observed crash data are available for the specific study roadway network, facility, or site. Observed data may be obtained directly from the jurisdiction's accident report system. At least two years of observed crash frequency data are desirable to apply the EB Method. The EB Method and criteria to determine whether the EB Method is applicable are presented in Section A.2.1 in the Appendix to *Part C*.

The EB Method can be applied at the site-specific level (i.e., observed crashes are assigned to specific intersections or roadway segments in Step 6) or at the project level (i.e., observed crashes are assigned to a facility as a whole). The site-specific EB Method is applied in Step 13. Alternatively, if observed crash data are available but can not be assigned to individual roadway segments and intersections, the project level EB Method is applied (in Step 15).

If observed crash frequency data are not available, then Steps 6, 13, and 15 of the predictive method are not conducted. In this case the estimate of expected average crash frequency is limited to using a predictive model (i.e. the predictive average crash frequency).

Step 4 - Determine geometric design features, traffic control features and site characteristics for all sites in the study network.

In order to determine the relevant data needs and avoid unnecessary collection of data, it is necessary to understand the base conditions and AMFs in Step 9 and Step 10. The base conditions are defined in Section 12.6.1 for roadway segments and in Section 12.6.2 for intersections.

The following geometric design and traffic control features are used to determine whether the site specific conditions vary from the base conditions and therefore whether an AMF is applicable:

- Length of roadway segment (miles)
- AADT (vehicles per day)
- Number of through lanes
- Presence/type of median (undivided, divided by raised or depressed median, center TWLTL)
- Presence/type of on-street parking (parallel vs. angle; one side vs. both sides of street)
- Number of driveways for each driveway type (major commercial, minor commercial; major industrial/institutional; minor industrial/institutional; major residential; minor residential; other)
- Roadside fixed object density (fixed objects/mile, only obstacles 4-in or more in diameter that do not have a breakaway design are counted)
- Average offset to roadside fixed objects from edge of traveled way (feet)
- Presence/absence of roadway lighting

- Speed category (based on actual traffic speed or posted speed limit)
- Presence of automated speed enforcement

For all intersections within the study area, the following geometric and traffic control features are identified:

- Number of intersection legs (3 or 4)
- Type of traffic control (minor-road STOP or signal)
- Number of approaches with intersection left turn lane (all approaches, 0, 1, 2, 3, or 4 for signalized intersection; only major approaches, 0, 1, or 2, for Stop-controlled intersections)
- Number of major-road approaches with left-turn signal phasing (0, 1, or 2) (signalized intersections only) and type of left-turn signal phasing (permissive, protected/permissive, permissive/protected, or protected)
- Number of approaches with intersection right turn lane (all approaches, 0, 1, 2, 3, or 4 for signalized intersection; only major approaches, 0, 1, or 2, for Stop-controlled intersections)
- Number of approaches with right-turn-on-red operation prohibited (0, 1, 2, 3, or 4) (signalized intersections only)
- Presence/absence of intersection lighting
- Maximum number of traffic lanes to be crossed by a pedestrian in any crossing maneuver at the intersection considering the presence of refuge islands (for signalized intersections only)
- Proportions of nighttime crashes for unlighted intersections (by total, fatal, non-fatal injury and property damage only)

For signalized intersections, land use and demographic data used in the estimation of vehicle-pedestrian collisions include:

- Number of bus stops within 1,000 feet of the intersection
- Presence of schools within 1,000 feet of the intersection
- Number of alcohol sales establishments within 1,000 feet of the intersection
- Presence of Red Light Camera
- Number of approaches on which right turn on red is allowed

Step 5 – Divide the roadway network or facility into individual homogenous roadway segments and intersections, which are referred to as sites.

Using the information from Step 1 and Step 4, the roadway is divided into individual sites, consisting of individual homogenous roadway segments and intersections. The definitions and methodology for dividing the roadway into individual intersections and homogenous roadway segments for use with the Chapter 12 predictive models are provided in Section 12.5. When dividing roadway facilities into small homogenous roadway segments, limiting the segment length to a minimum of 0.10 miles will decrease data collection and management efforts.

Step 6 – Assign observed crashes to the individual sites (if applicable).

Step 6 only applies if it was determined in Step 3 that the site-specific EB Method was applicable. If the site-specific EB Method is not applicable, proceed to Step 7. In Step 3, the availability of observed data and whether the data could be assigned to specific locations was determined. The specific criteria for assigning accidents to individual roadway segments or intersections are presented in Section A.2.3 of the Appendix to *Part C*.

Crashes that occur at an intersection or on an intersection leg, and are related to the presence of an intersection, are assigned to the intersection and used in the EB Method together with the predicted average crash frequency for the intersection. Crashes that occur between intersections and are not related to the presence of an intersection are assigned to the roadway segment on which they occur; such crashes are used in the EB Method together with the predicted average crash frequency for the roadway segment.

Step 7 – Select the first or next individual site in the study network. If there are no more sites to be evaluated, proceed to Step 15.

In Step 5 the roadway network within the study limits has been divided into a number of individual homogenous sites (intersections and roadway segments).

The outcome of the HSM Predictive Method is the expected average crash frequency of the entire study network, which is the sum of the all of the individual sites, for each year in the study. Note that this value will be the total number of crashes expected to occur over all sites during the period of interest. If a crash frequency is desired, the total can be divided by the number of years in the period of interest.

The estimation for each site (roadway segments or intersection) is conducted one at a time. Steps 8 through 14, described below, are repeated for each site.

Step 8 – For the selected site, select the first or next year in the period of interest. If there are no more years to be evaluated for that site, proceed to Step 14

Steps 8 through 14 are repeated for each site in the study and for each year in the study period.

The individual years of the evaluation period may have to be analyzed one year at a time for any particular roadway segment or intersection because SPFs and some AMFs (e.g., lane and shoulder widths) are dependent on AADT, which may change from year to year.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type, and traffic control features.

Steps 9 through 13, described below, are repeated for each year of the evaluation period as part of the evaluation of any particular roadway segment or intersection. The predictive models in Chapter 12 follow the general form shown in Equation 12-1. Each predictive model consists of a SPF, which is adjusted to site specific conditions using AMFs (in Step 10) and adjusted to local jurisdiction conditions (in Step 11) using a calibration factor (C). The SPFs, AMFs and calibration factor obtained in Steps 9, 10, and 11 are applied to calculate the predicted average crash frequency for the selected year of the selected site. The SPFs available for urban and suburban arterials are presented in Section 12.6

The SPF (which is a regression model based on observed crash data for a set of similar sites) determines the predicted average crash frequency for a site with the same base conditions (i.e., a specific set of geometric design and traffic control features). The base conditions for each SPF are specified in Section 12.6. A detailed explanation and overview of the SPFs in *Part C* is provided in Section C.6.3 of the *Part C Introduction and Applications Guidance*.

A detailed explanation and overview of the SPFs in Part C is provided in Section C.6.3 of the Part C Introduction and Applications Guide.

The SPFs developed for Chapter 12 are summarized in Exhibit 12-4 in Section 12.6. For the selected site, determine the appropriate SPF for the site type (intersection or roadway segment) and the geometric and traffic control features (undivided roadway, divided roadway, stop-controlled intersection, signalized intersection). The SPF for the selected site is calculated using the AADT determined in Step 3 ($AADT_{maj}$ and $AADT_{min}$ for intersections) for the selected year.

Each SPF determined in Step 9 is provided with default distributions of crash severity and collision type (presented in section 12.6). These default distributions can benefit from being updated based on local data as part of the calibration process presented in Appendix A.1.1.

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust base conditions to site specific geometric design and traffic control features

In order to account for differences between the base conditions (Section 12.6) and the specific conditions of the site, AMFs are used to adjust the SPF estimate. An overview of AMFs and guidance for their use is provided in Section C.6.4 of the *Part C Introduction and Applications Guidance*, including the limitations of current knowledge related to the effects of simultaneous application of multiple AMFs. In using multiple AMFs, engineering judgment is required to assess the interrelationships and/or independence of individual elements or treatments being considered for implementation within the same project.

All AMFs used in Chapter 12 have the same base conditions as the SPFs used in Chapter 12 (i.e., when the specific site has the same condition as the SPF base condition, the AMF value for that condition is 1.00). Only the AMFs presented in Section 12.7 may be used as part of the Chapter 12 predictive method. Exhibit 12-35 indicates which AMFs are applicable to the SPFs in Section 12.6.

The AMFs for roadway segments are those described in Section 12.7.1. These AMFs are applied as shown in Equation 12-3.

The AMFs for intersections are those described in Section 12.7.2, which apply to both signalized and STOP-controlled intersections, and in Section 12.7.3, which apply to signalized intersections only. These AMFs are applied as shown in Equations 12-6 and 12-28.

In Chapter 12, the multiple- and single-vehicle base crashes determined in Step 9 and the AMFs values calculated in Step 10 are then used to estimate the vehicle-pedestrian and vehicle-bicycle base crashes for roadway segments and intersections (present in Section 12.6.1 and 12.6.2 respectively).

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

The SPFs used in the predictive method have each been developed with data from specific jurisdictions and time periods. Calibration to local conditions will

Detailed guidance for the development of calibration factors is included in Part C Appendix A.1.1.

account for these differences. A calibration factor (C_r for roadway segments or C_i for intersections) is applied to each SPF in the predictive method. An overview of the use of calibration factors is provided in the *Part C Introduction and Applications Guidance* Section C.6.5. Detailed guidance for the development of calibration factors is included in *Part C* Appendix A.1.1.

Steps 9, 10, and 11 together implement the predictive models in Equations 12-2 through 12-7 to determine predicted average crash frequency.

Step 12 – If there is another year to be evaluated in the study period for the selected site, return to Step 8. Otherwise, proceed to Step 14.

This step creates a loop through Steps 8 to 12 that is repeated for each year of the evaluation period for the selected site.

Step 13 – Apply site-specific EB Method (if applicable).

Whether the site-specific EB Method is applicable is determined in Step 3. The site-specific EB Method combines the Chapter 12 predictive model estimate of predicted average crash frequency, $N_{predicted}$ with the observed crash frequency of the specific site, $N_{observed}$. This provides a more statistically reliable estimate of the expected average crash frequency of the selected site.

In order to apply the site-specific EB Method, in addition to the material in *Part C* Appendix A.2.4 the overdispersion parameter, k, for the is also used. The overdispersion parameter provides an indication of the statistical reliability of the SPF. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. This parameter is used in the site-specific EB Method to provide a weighting to $N_{predicted}$ and $N_{observed}$. Overdispersion parameters are provided for each SPF in Section 12.6.

Apply the site-specific EB Method to a future time period, if appropriate.

The estimated expected average crash frequency obtained above applies to the time period in the past for which the observed crash data were obtained. Section A.2.6 in the Appendix to *Part C* provides a method to convert the estimate of expected average crash frequency for a past time period to a future time period. In doing this, consideration is given to significant changes in geometric or roadway characteristics cause by the treatments considered for future time period.

Step 14 –If there is another site to be evaluated, return to 7, otherwise, proceed to Step 15.

This step creates a loop through Steps 7 to 13 that is repeated for each roadway segment or intersection within the facility.

Step 15 – Apply the project level EB Method (if the site-specific EB Method is not applicable).

This step is only applicable to existing conditions when observed crash data are available, but can not be accurately assigned to specific sites (e.g., the crash report may identify crashes as occurring between two intersections, but is not accurate to determine a precise location on the segment). Detailed description of the project level EB Method is provided in *Part C* Appendix A.2.5.

Step 16 – Sum all sites and years in the study to estimate total crash frequency.

The total estimated number of crashes within the network or facility limits during a study period of n years is calculated using Equation 12-8:

$$N_{total} = \sum_{\substack{all \\ roadway \\ segments}} N_{rs} + \sum_{\substack{all \\ intersections}} N_{int} \qquad (12\text{-}8)$$

Where,

N_{total}= total expected number of crashes within the limits of a rural two-lane two-way road facility for the period of interest. Or, the sum of the expected average crash frequency for each year for each site within the defined roadway limits within the study period;

N_{rs}= expected average crash frequency for a roadway segment using the predictive method for one specific year; and

N_{int}= expected average crash frequency for an intersection using the predictive method for one specific year.

Equation 12-8 represents the total expected number of crashes estimated to occur during the study period. Equation 12-9 is used to estimate the total expected average crash frequency within the network or facility limits during the study period.

$$N_{total\ average} = \frac{N_{total}}{n} \qquad (12\text{-}9)$$

Where,

$N_{total\ average}$= total expected average crash frequency estimated to occur within the defined network or facility limits during the study period;

n = number of years in the study period.

Step 17 – Determine if there is an alternative design, treatment or forecast AADT to be evaluated.

Steps 3 through 16 of the predictive method are repeated as appropriate for the same roadway limits but for alternative conditions, treatments, periods of interest or forecast AADTs.

Step 18 – Evaluate and compare results.

The predictive method is used to provide a statistically reliable estimate of the expected average crash frequency within defined network or facility limits over a given period of time, for given geometric design and traffic control features and known or estimated AADT. In addition to estimating total crashes, the estimate can be made for different crash severity types and different collision types. Default distributions of crash severity and collision type are provided with each SPF in Section 12.6. These default distributions can benefit from being updated based on local data as part of the calibration process presented in *Part C* Appendix A.1.1.

12.5. ROADWAY SEGMENTS AND INTERSECTIONS

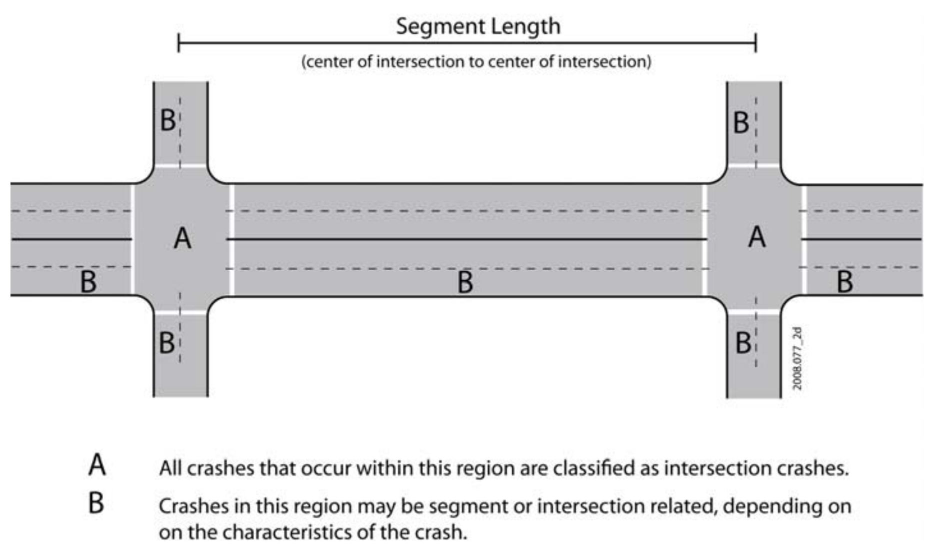
Section 12.4 provides an explanation of the predictive method. Sections 12.5 through 12.8 provide the specific detail necessary to apply the predictive method steps. Detail regarding the procedure for determining a calibration factor to apply in Step 11 is provided in the *Part C* Appendix A.1. Detail regarding the EB Method, which is applied in Steps 6, 13, and 15, is provided in the *Part C* Appendix A.2.

In Step 5 of the predictive method, the roadway within the defined limits is divided into individual sites, which are homogenous roadway segments and intersections. A facility consists of a contiguous set of individual intersections and roadway segments, referred to as "sites." A roadway network consists of a number of contiguous facilities. Predictive models have been developed to estimate crash frequencies separately for roadway segments and intersections. The definitions of roadway segments and intersections presented below are the same as those used in the FHWA Interactive Highway Safety Design Model (IHSDM) [4].

Roadway segments begin at the center of an intersection and end at either the center of the next intersection, or where there is a change from one homogeneous roadway segment to another homogenous segment. The roadway segment model estimates the frequency of roadway-segment-related crashes which occur in Region B in Exhibit 12-3. When a roadway segment begins or ends at an intersection, the length of the roadway segment is measured from the center of the intersection.

Chapter 12 provides predictive models for stop-controlled (three- and four-leg) and signalized (three- and four-leg) intersections. The intersection models estimate the predicted average frequency of crashes that occur within the limits of an intersection (Region A of Exhibit 12-3) and intersection-related crashes that occur on the intersection legs (Region B in Exhibit 12-3).

Exhibit 12-3: Definition of Roadway Segments and Intersections



The segmentation process produces a set of roadway segments of varying length, each of which is homogeneous with respect to characteristics such as traffic volumes and key roadway design characteristics and traffic control features. Exhibit 12-3 shows the segment length, L, for a single homogenous roadway segment occurring between two intersections. However, several homogenous roadway segments can occur between two intersections. A new (unique) homogeneous segment begins at the center of each intersection and where there is a change in at least one of the following characteristics of the roadway:

- Annual average daily traffic volume (AADT) (vehicles/day)
- Number of through lanes
- Presence/type of median

  The following rounded widths for medians without barriers are recommended before determining "homogeneous" segments:

Measured Median Width	Rounded Median Width
1-ft to 14-ft	10-ft
15-ft to 24-ft	20-ft
25-ft to 34-ft	30-ft
35-ft to 44-ft	40-ft
45-ft to 54-ft	50-ft
55-ft to 64-ft	60-ft
65-ft to 74-ft	70-ft
75-ft to 84-ft	80-ft
85-ft to 94-ft	90-ft
95 or more	100-ft

- Presence/type of on-street parking
- Roadside fixed object density
- Presence of lighting
- Speed category (based on actual traffic speed or posted speed limit)

In addition, each individual intersection is treated as a separate site, for which the intersection-related crashes are estimated using the predictive method.

There is no minimum roadway segment length, L, for application of the predictive models for roadway segments. When dividing roadway facilities into small homogenous roadway segments, limiting the segment length to a minimum of 0.10 miles will minimize calculation efforts and not affect results.

In order to apply the site-specific EB Method, observed crashes are assigned to the individual roadway segments and intersections. Observed crashes that occur between intersections are classified as either intersection-related or roadway segment-related. The methodology for assigning crashes to roadway segments and intersections for use in the site-specific EB Method is presented in Section A.2.3 in the Appendix to *Part C*. In applying the EB Method for urban and suburban arterials, whenever the predicted average crash frequency for a specific roadway segment during the multiyear study period is less than 1/k (the inverse of the overdispersion parameter for the relevant SPF), consideration should be given to combining adjacent roadway segments and applying the project-level EB Method. This guideline for the minimum crash frequency for a roadway segment applies only to Chapter 12 which

The methodology for assigning crashes to roadway segments and intersections for use in the site-specific EB Method is presented in Section A.2.3 in the Appendix to Part C.

uses fixed-value overdispersion parameters. It is not needed in *Chapter 10* or *Chapter 11* which use length-dependent overdispersion parameters.

12.6. SAFETY PERFORMANCE FUNCTIONS FOR BASE CONDITIONS

In Step 9 of the predictive method, the appropriate Safety Performance Functions (SPFs) are used to predict crash frequencies for specific base conditions. SPFs are regression models for estimating the predicted average crash frequency of individual roadway segments or intersections. Each SPF in the predictive method was developed with observed accident data for a set of similar sites. The SPFs, like all regression models, estimates the value of a dependent variable as a function of a set of independent variables. In the SPFs developed for the HSM, the dependent variable estimated is the predicted average crash frequency for a roadway segment or intersection under base conditions, and the independent variables are the AADTs of the roadway segment or intersection legs (and, for roadway segments, the length of the roadway segment).

The predicted crash frequencies for base conditions obtained with the SPFs are used in the predictive models in Equations 12-2 through 12-7. A detailed discussion of SPFs and their use in the HSM is presented in *Chapter 3* Section 3.5.2 and the *Part C Introduction and Applications Guidance* Section C.6.3.

Each SPF also has an associated overdispersion parameter, k. The overdispersion parameter provides an indication of the statistical reliability of the SPF. The closer the overdispersion parameter is to zero, the more statistically reliable the SPF. This parameter is used in the EB Method discussed in the *Part C* Appendix. The SPFs in *Chapter 12* are summarized in Exhibit 12-4.

Exhibit 12-4: Safety Performance Functions included in Chapter 12

Chapter 12 SPFs for Urban and Suburban Arterials	SPF Components by Collision type	SPF Equations and Exhibits
Roadway segments	multiple-vehicle nondriveway collisions	Equation 12-10, 12-11, 12-12, Exhibits 12-5, 12-6, 12-7
	single-vehicle crashes	Equations 12-13, 12-14, 12-15, Exhibits 12-8, 12-9, 12-10
	multiple-vehicle driveway-related collisions	Equations 12-16, 12-17, 12-18, Exhibits 12-11, 12-12, 12-13, 12-14, 12-15, 12-16
	vehicle-pedestrian collisions	Equation 12-19 Exhibit 12-17
	vehicle-bicycle collisions	Equation 12-20, Exhibit 12-18
Intersections	multiple-vehicle collisions	Equations 12-21, 12-22, 12-23, Exhibit 12-19, 12-20, 12-21, 12-22, 12-23, 12-24
	single-vehicle crashes	Equations 12-24, 12-25, 12-26, 12-27, Exhibit 12-25, 12-26, 12-27, 12-28, 12-29, 12-30
	vehicle-pedestrian collisions	Equations 12-28, 12-29, 12-30, Exhibits 12-31, 12-32, 12-33
	vehicle-bicycle collisions	Equation 12-31, Exhibit 12-34

Some highway agencies may have performed statistically-sound studies to develop their own jurisdiction-specific SPFs derived from local conditions and crash experience. These models may be substituted for models presented in this chapter. Criteria for the development of SPFs for use in the predictive method are addressed in the calibration procedure presented in the Appendix to *Part C*.

12.6.1. Safety Performance Functions for Urban and Suburban Arterial Roadway Segments

The predictive model for predicting average crash frequency on a particular urban or suburban arterial roadway segment was presented in Equation 12-2. The effect of traffic volume (AADT) on crash frequency is incorporated through the SPF, while the effects of geometric design and traffic control features are incorporated through the AMFs. The SPF for urban and suburban arterial roadway segments is presented in this section. Urban and suburban arterial roadway segments are defined in Section 12.3.

SPFs and adjustment factors are provided for five types of roadway segments on urban and suburban arterials:

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

Guidance on the estimation of traffic volumes for roadway segments for use in the SPFs is presented in Step 3 of the predictive method described in Section 12.4. The SPFs for roadway segments on urban and suburban arterials are applicable to the following AADT ranges:

The traffic volume boundary conditions for the chapter 12 roadway segment SPFs are presented here.

- 2U: 0 to 32,600 vehicles per day
- 3T : 0 to 32,900 vehicles per day
- 4U: 0 to 40,100 vehicles per day
- 4D: 0 to 66,000 vehicles per day
- 5T: 0 to 53,800 vehicles per day

Application to sites with AADTs substantially outside these ranges may not provide reliable results.

Other types of roadway segments may be found on urban and suburban arterials but are not addressed by the predictive model in Chapter 12.

The procedure addresses five types of collisions. The corresponding Equations and Exhibits are indicated in Exhibit 12-4 above:

- multiple-vehicle nondriveway collisions
- single-vehicle crashes

- multiple-vehicle driveway-related collisions
- vehicle-pedestrian collisions
- vehicle-bicycle collisions

The predictive model for estimating average crash frequency on roadway segments is shown in Equations 12-2 through 12-4. The effect of traffic volume on predicted crash frequency is incorporated through the SPFs, while the effects of geometric design and traffic control features are incorporated through the AMFs. SPFs are provided for multiple-vehicle nondriveway collisions and single-vehicle crashes. Adjustment factors are provided for multi-vehicle driveway-related, vehicle-pedestrian, and vehicle-bicycle collisions.

This section presents the SPFs and adjustment factors for multiple-vehicle nondriveway collisions.

Multiple-Vehicle Nondriveway Collisions

The SPF for multiple-vehicle nondriveway collisions is applied as follows:

$$N_{brmv} = exp\ (a + b \times ln(AADT) + ln(L)) \qquad (12\text{-}10)$$

Where,

AADT = average daily traffic volume (vehicles/day) on roadway segment;

L = length of roadway segment (mi); and

a, b = regression coefficients.

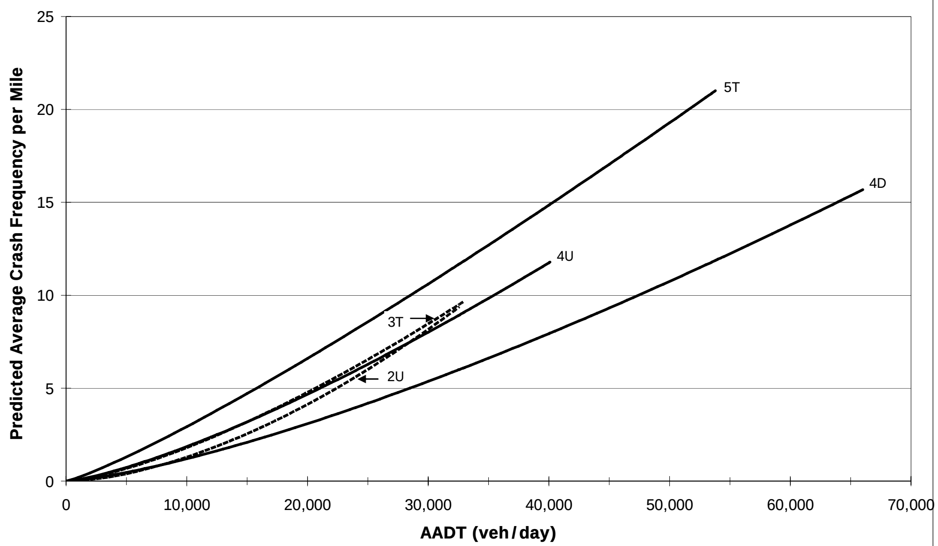
Exhibit 12-5 presents the values of the coefficients a and b used in applying Equation 12-10. The overdispersion parameter, k, is also presented in Exhibit 12-5.

Exhibit 12-5: SPF Coefficients for Multiple-Vehicle Nondriveway Collisions on Roadway Segments

Road type	Coefficients used in Equation 12-10: Intercept (a)	Coefficients used in Equation 12-10: AADT (b)	Overdispersion parameter (k)
Total crashes			
2U	-15.22	1.68	0.84
3T	-12.40	1.41	0.66
4U	-11.63	1.33	1.01
4D	-12.34	1.36	1.32
5T	-9.70	1.17	0.81
Fatal-and-injury crashes			
2U	-16.22	1.66	0.65
3T	-16.45	1.69	0.59
4U	-12.08	1.25	0.99
4D	-12.76	1.28	1.31
5T	-10.47	1.12	0.62
Property-damage-only crashes			
2U	-15.62	1.69	0.87
3T	-11.95	1.33	0.59

4U	-12.53	1.38	1.08
4D	-12.81	1.38	1.34
5T	-9.97	1.17	0.88

Exhibit 12-6: Graphical Form of the SPF for Multiple Vehicle Nondriveway collisions (from Equation 12-10 and Exhibit 12-5)



Equation 12-10 is first applied to determine N_{brmv} using the coefficients for total crashes in Exhibit 12-5. N_{brmv} is then divided into components by severity level, $N_{brmv(FI)}$ for fatal-and-injury crashes and $N_{brmv(PDO)}$ for property-damage-only crashes. These preliminary values of $N_{brmv(FI)}$ and $N_{brmv(PDO)}$, designated as $N'_{brmv(FI)}$ and $N'_{brmv(PDO)}$ in Equation 12-11, are determined with Equation 12-10 using the coefficients for fatal-and-injury and property-damage-only crashes, respectively, in Exhibit 12-5. The following adjustments are then made to assure that $N_{brmv(FI)}$ and $N_{brmv(PDO)}$ sum to N_{brmv}:

$$N_{brmv(FI)} = N_{brmv\,(TOTAL)} \left(\frac{N'_{brmv(FI)}}{N'_{brmv(FI)} + N'_{brmv(PDO)}} \right) \tag{12-11}$$

$$N_{brmv(PDO)} = N_{brmv(TOTAL)} - N_{brmv(FI)} \tag{12-12}$$

The proportions in Exhibit 12-7 are used to separate $N_{brmv(FI)}$ and $N_{brmv(PDO)}$ into components by collision type.

Exhibit 12-7: Distribution of Multiple-Vehicle Nondriveway Collisions for Roadway Segments by Manner of Collision Type

Collision type	Proportion of crashes by severity level for specific road types									
	2U		3T		4U		4D		5T	
	FI	PDO	FI	PDO	FI	PDO	FI	PDO	FI	PDO
Rear-end collision	0.730	0.778	0.845	0.842	0.511	0.506	0.832	0.662	0.846	0.651
Head-on collision	0.068	0.004	0.034	0.020	0.077	0.004	0.020	0.007	0.021	0.004
Angle collision	0.085	0.079	0.069	0.020	0.181	0.130	0.040	0.036	0.050	0.059
Sideswipe, same direction	0.015	0.031	0.001	0.078	0.093	0.249	0.050	0.223	0.061	0.248
Sideswipe, opposite direction	0.073	0.055	0.017	0.020	0.082	0.031	0.010	0.001	0.004	0.009
Other multiple-vehicle collisions	0.029	0.053	0.034	0.020	0.056	0.080	0.048	0.071	0.018	0.029

Source: HSIS data for Washington (2002-2006)

Single-Vehicle Crashes

This section presents the SPFs and adjustment factors for single-vehicle crashes for roadway segments.

SPFs for single-vehicle crashes for roadway segments are applied as follows:

$$N_{brsv} = exp\ (a + b \times ln(AADT) + ln(L)) \tag{12-13}$$

Exhibit 12-8 presents the values of the coefficients and factors used in Equation 12-13 for each roadway type. Equation 12-13 is first applied to determine N_{brsv} using the coefficients for total crashes in Exhibit 12-8. N_{brsv} is then divided into components by severity level, $N_{brsv(FI)}$ for fatal-and-injury crashes and $N_{brsv(PDO)}$ for property-damage-only crashes. Preliminary values of $N_{brsv(FI)}$ and $N_{brsv(PDO)}$, designated as $N'_{brsv(FI)}$ and $N'_{brsv(PDO)}$ in Equation 12-14, are determined with Equation 12-13 using the coefficients for fatal-and-injury and property-damage-only crashes, respectively, in Exhibit 12-8. The following adjustments are then made to assure that $N_{brsv(FI)}$ and $N_{brsv(PDO)}$ sum to N_{brsv}:

$$N_{brsv(FI)} = N_{brsv(TOTAL)} \left(\frac{N'_{brsv(FI)}}{N'_{brsv(FI)} + N'_{brsv(PDO)}} \right) \tag{12-14}$$

$$N_{brsv(PDO)} = N_{brsv(TOTAL)} - N_{brsv(FI)} \tag{12-15}$$

The proportions in Exhibit 12-10 are used to separate $N_{brsv(FI)}$ and $N_{brsv(PDO)}$ into components by crash type.

Exhibit 12-8: SPF Coefficients for Single-Vehicle Crashes on Roadway Segments

Road type	Coefficients used in Equation 12-11 Intercept (a)	Coefficients used in Equation 12-11 AADT (b)	Overdispersion parameter (k)
Total crashes			
2U	-5.47	0.56	0.81
3T	-5.74	0.54	1.37
4U	-7.99	0.81	0.91
4D	-5.05	0.47	0.86
5T	-4.82	0.54	0.52
Fatal-and-injury crashes			
2U	-3.96	0.23	0.50
3T	-6.37	0.47	1.06
4U	-7.37	0.61	0.54
4D	-8.71	0.66	0.28
5T	-4.43	0.35	0.36
Property-damage-only crashes			
2U	-6.51	0.64	0.87
3T	-6.29	0.56	1.93
4U	-8.50	0.84	0.97
4D	-5.04	0.45	1.06
5T	-5.83	0.61	0.55

Exhibit 12-9: Graphical Form of the SPF for Single-Vehicle Crashes (from Equation 12-13 and Exhibit 12-8)

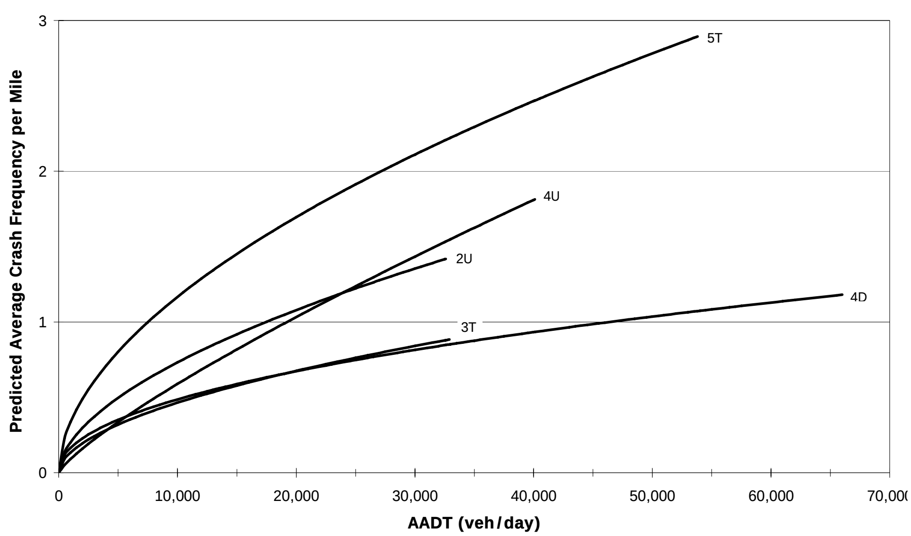


Exhibit 12-10: Distribution of Single-Vehicle Crashes for Roadway Segments by Collision Type

Collision type	Proportion of crashes by severity level for specific road types									
	2U		3T		4U		4D		5T	
	FI	PDO	FI	PDO	FI	PDO	FI	PDO	FI	PDO
Collision with animal	0.026	0.066	0.001	0.001	0.001	0.001	0.001	0.063	0.016	0.049
Collision with fixed object	0.723	0.759	0.688	0.963	0.612	0.809	0.500	0.813	0.398	0.768
Collision with other object	0.010	0.013	0.001	0.001	0.020	0.029	0.028	0.016	0.005	0.061
Other single-vehicle collision	0.241	0.162	0.310	0.035	0.367	0.161	0.471	0.108	0.581	0.122

Source: HSIS data for Washington (2002-2006)

Multiple-Vehicle Driveway-Related Collisions

This section presents the SPFs and adjustment factors for multiple-vehicle driveway-related collisions within a roadway segment.

The model presented above for multiple-vehicle collisions addressed only collisions that are not related to driveways. Driveway-related collisions also generally involve multiple vehicles, but are addressed separately because the frequency of driveway-related collisions on a roadway segment depends on the number and type of driveways. Only unsignalized driveways are considered; signalized driveways are analyzed as signalized intersections.

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

$$N_{brdwy} = \sum_{\substack{all \\ driveway \\ types}} n_j \times N_j \times \left(\frac{AADT}{15{,}000} \right)^{(t)} \qquad (12\text{-}16)$$

Where,

N_j = Number of driveway-related collisions per driveway per year for driveway type j from Exhibit 12-11;

n_j = number of driveways within roadway segment of driveway type j including all driveways on both sides of the road; and

t = coefficient for traffic volume adjustment from Exhibit 12-11.

The number of driveways of a specific type, n_j, is the sum of the number of driveways of that type for both sides of the road combined. The number of driveways is determined separately for each side of the road and then added together.

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

- Major commercial driveways
- Minor commercial driveways

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

Major driveways are those that serve sites with 50 or more parking spaces. Minor driveways are those that serve sites with less than 50 parking spaces. It is not intended that an exact count of the number of parking spaces be made for each site. Driveways can be readily classified as major or minor from a quick review of aerial photographs that show parking areas or through user judgment based on the character of the establishment served by the driveway. Commercial driveways provide access to establishments that serve retail customers. Residential driveways serve single- and multiple-family dwellings. Industrial/institutional driveways serve factories, warehouses, schools, hospitals, churches, offices, public facilities, and other places of employment. Commercial sites with no restriction on access along an entire property frontage are generally counted as two driveways.

Exhibit 12-11: SPF Coefficients for Multiple-Vehicle Driveway Related Collisions

Driveway type (j)	Coefficients for specific roadway types				
	2U	3T	4U	4D	5T
Number of driveway-related collisions per driveway per year (N_j)					
Major commercial	0.158	0.102	0.182	0.033	0.165
Minor commercial	0.050	0.032	0.058	0.011	0.053
Major industrial/institutional	0.172	0.110	0.198	0.036	0.181
Minor industrial/institutional	0.023	0.015	0.026	0.005	0.024
Major residential	0.083	0.053	0.096	0.018	0.087
Minor residential	0.016	0.010	0.018	0.003	0.016
Other	0.025	0.016	0.029	0.005	0.027
Regression coefficient for AADT (t)					
All driveways	1.000	1.000	1.172	1.106	1.172
Overdispersion parameter (k)					
All driveways	0.81	1.10	0.81	1.39	0.10
Proportion of fatal-and-injury crashes (f_{dwy})					
All driveways	0.323	0.243	0.342	0.284	0.269
Proportion of property-damage-only crashes					
All driveways	0.677	0.757	0.658	0.716	0.731

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

Exhibit 12-12: Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Two-Lane Undivided Arterials (2U) (from Equation 12-16 and Exhibit 12-11)

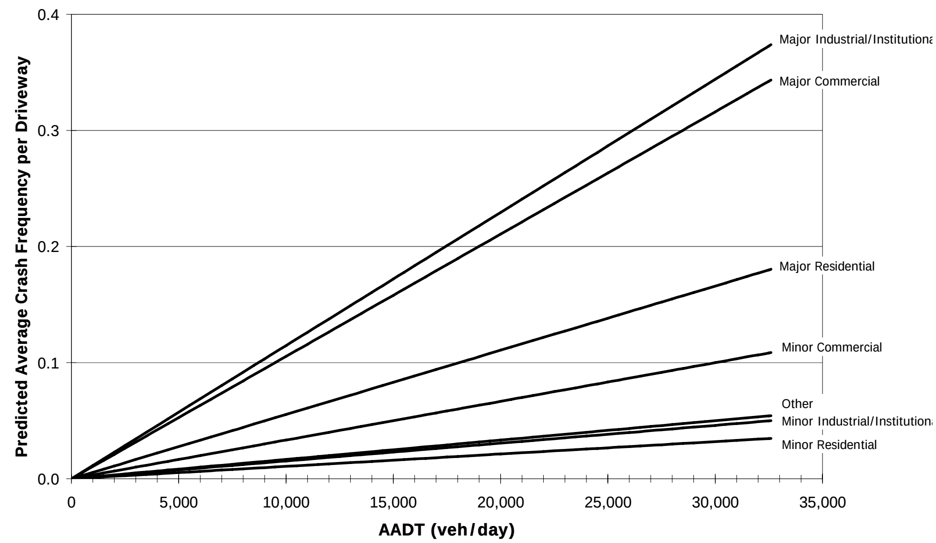


Exhibit 12-13: Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Three-Lane Undivided Arterials (3T) (from Equation 12-16 and Exhibit 12-11)

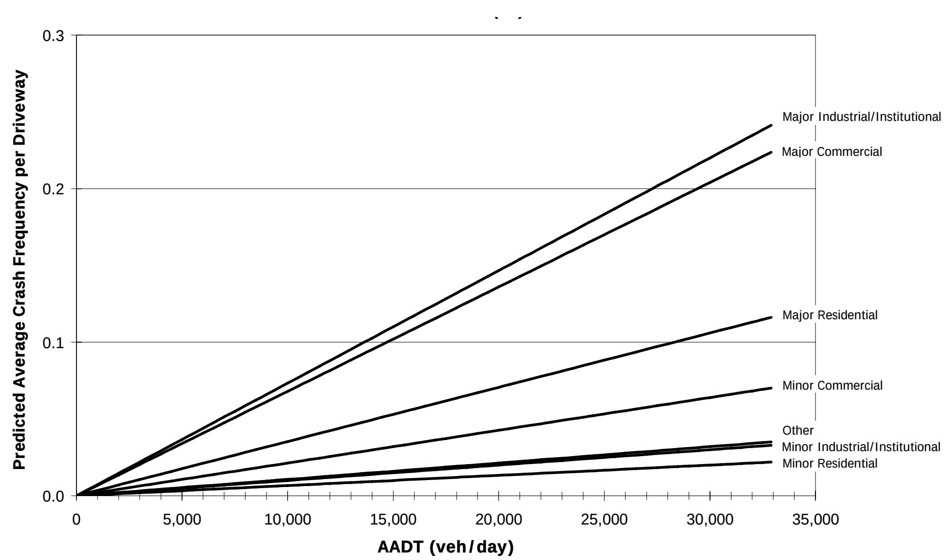


Exhibit 12-14: Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on on Four-Lane Undivided Arterials (4U)(from Equation 12-16 and Exhibit 12-11)

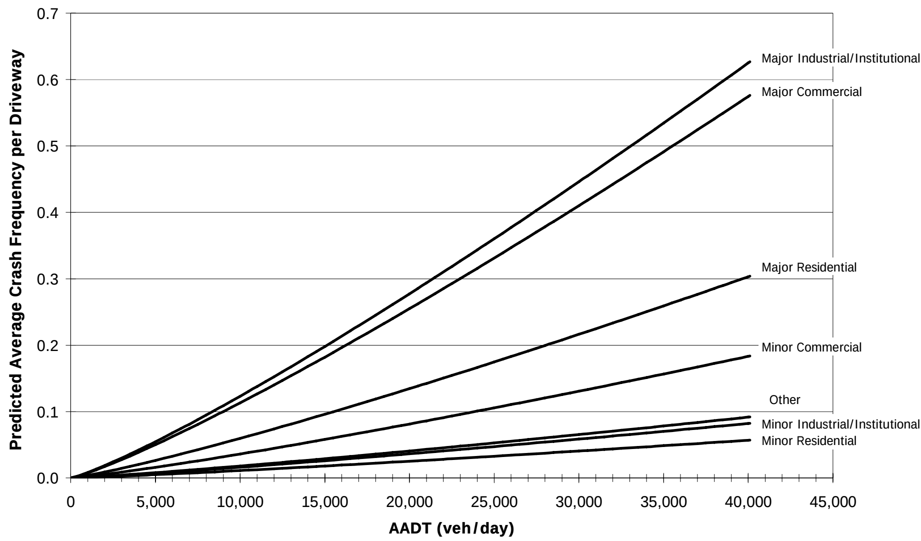


Exhibit 12-15: Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Four-Lane Divided Arterials(4D)(from Equation 12-16 and Exhibit 12-11)

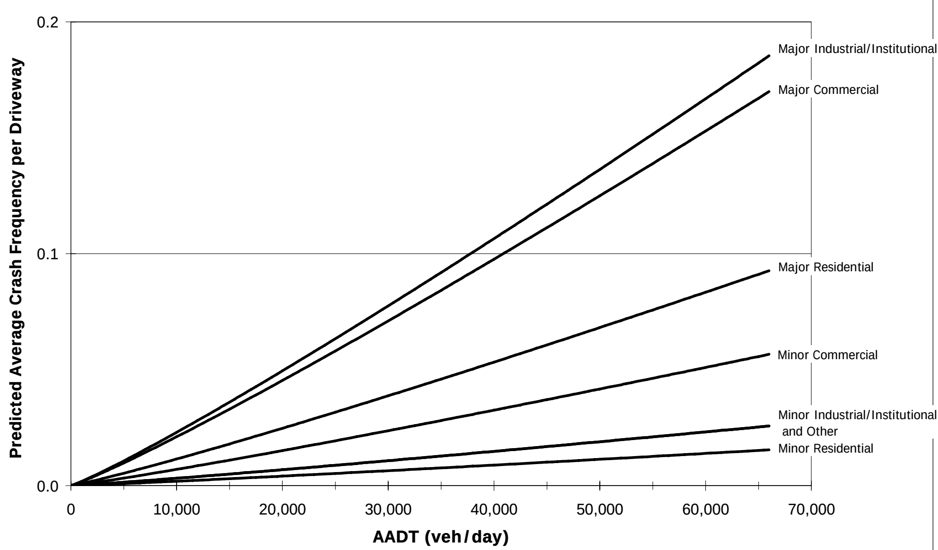
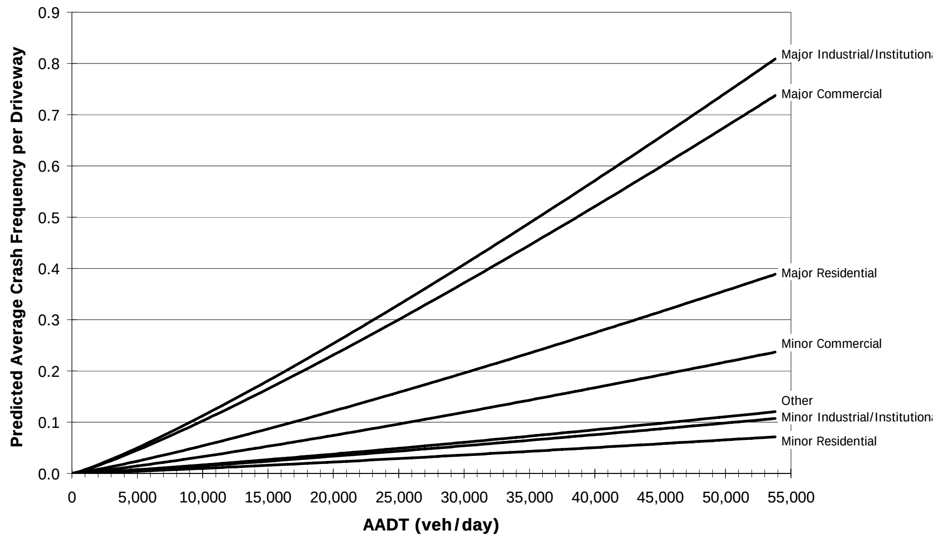


Exhibit 12-16: Graphical Form of the SPF for Multiple Vehicle Driveway Related Collisions on Five-Lane Arterials Including a Center Two-Way Left-Turn Lane)(from Equation 12-16 and Exhibit 12-11)



Driveway-related collisions can be separated into components by severity level as follows:

$$N_{brdwy(FI)} = N_{brdwy(TOTAL)} \times f_{dwy} \qquad (12\text{-}17)$$

$$N_{brdwy(PDO)} = N_{brdwy(TOTAL)} - N_{brdwy(FI)} \qquad (12\text{-}18)$$

Where,

f_{dwy} = proportion of driveway-related collisions that involve fatalities or injuries

The values of N_j and f_{dwy} are shown in Exhibit 12-11.

Vehicle-Pedestrian Collisions

This section presents the method to calculate the number of vehicle-pedestrian collisions per year for a roadway segment.

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

$$N_{pedr} = N_{br} \times f_{pedr} \qquad (12\text{-}19)$$

Where,

f_{pedr} = pedestrian accident adjustment factor.

The value N_{br} used in Equation 12-19 is that determined with Equation 12-3.

Exhibit 12-17 presents the values of f_{pedr} for use in Equation 12-19. All vehicle-pedestrian collisions are considered to be fatal-and-injury crashes. The values of f_{pedr} are likely to depend on the climate and the walking environment in particular states or communities. HSM users are encouraged to replace the values in Exhibit 12-17 with suitable values for their own state or community through the calibration process (see the Appendix to *Part C*).

Exhibit 12-17: Pedestrian Accident Adjustment Factor for Roadway Segments

Road type	Pedestrian Accident Adjustment Factor (f_{pedr})	
	Posted Speed 30 mph or Lower	Posted Speed Greater than 30 mph
2U	0.036	0.005
3T	0.041	0.013
4U	0.022	0.009
4D	0.067	0.019
5T	0.030	0.023

Note: These factors apply to the methodology for predicting total crashes (all severity levels combined). All pedestrian collisions resulting from this adjustment factor is treated as fatal-and-injury crashes and none as property-damage-only crashes. Source: HSIS data for Washington (2002-2006)

Vehicle-Bicycle Collisions

This section presents the method to calculate vehicle-bicycle collisions per year for a roadway segment.

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

$$N_{biker} = N_{br} \times f_{biker} \qquad (12\text{-}20)$$

Where,

f_{biker} = bicycle accident adjustment factor.

The value of N_{br} used in Equation 12-20 is determined with Equation 12-3.

Exhibit 12-18 presents the values of f_{biker} for use in Equation 12-18. All vehicle-bicycle collisions are considered to be fatal-and-injury crashes. The values of f_{biker} are likely to depend on the climate and bicycling environment in particular states or communities. HSM users are encouraged to replace the values in Exhibit 12-18 with suitable values for their own state or community through the calibration process (see the Appendix to *Part C*).

Exhibit 12-18: Bicycle Accident Adjustment Factors for Roadway Segments

Road type	Bicycle Accident Adjustment Factor (f_{biker})	
	Posted Speed 30 mph or Lower	Posted Speed Greater than 30 mph
2U	0.018	0.004
3T	0.027	0.007
4U	0.011	0.002
4D	0.013	0.005
5T	0.050	0.012

Note: These factors apply to the methodology for predicting total crashes (all severity levels combined). All bicycle collisions resulting from this adjustment factor are treated as fatal-and-injury crashes and none as property-damage-only crashes. Source: HSIS data for Washington (2002-2006)

This section introduces the SPFs for intersections on urban and suburban arterials.

12.6.2. Safety Performance Functions for Urban and Suburban Arterial Intersections

The predictive models for predicting the frequency of crashes related to an intersection is presented in Equations 12-5 through 12-7. The structure of the predictive models for intersections is similar to the predictive models for roadway segments.

The effect of traffic volume on predicted crash frequency for intersections is incorporated through SPFs, while the effect of geometric and traffic control features are incorporated through AMFs. Each of the SPFs for intersections incorporates separate effects for the AADTs on the major- and minor-road legs, respectively.

SPFs and adjustment factors have been developed for four types of intersections on urban and suburban arterials. These are:

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

Other types of intersections may be found on urban and suburban arterials but are not addressed by the Chapter 12 SPFs.

The SPFs for each of the four intersection types identified above predict total crash frequency per year for crashes that occur within the limits of the intersection. The SPFs and adjustment factors address the following four types of collisions, (the corresponding Equations and Exhibits are indicated in Exhibit 12-4):

- multiple-vehicle collisions
- single-vehicle crashes
- vehicle-pedestrian collisions
- vehicle-bicycle collisions

Guidance on the estimation of traffic volumes for the major and minor road legs for use in the SPFs is presented in Step 3. The AADT(s) used in the SPF are the AADT(s) for the selected year of the evaluation period. The SPFs for intersections are applicable to the following AADT ranges:

The traffic volume boundary conditions for the chapter 12 intersection SPFs are presented here.

- 3ST Intersections $AADT_{maj}$: 0 to 45,700 vehicles per day and $AADT_{min}$: 0 to 9,300 vehicles per day
- 4ST Intersections $AADT_{maj}$: 0 to 46,800 vehicles per day and $AADT_{min}$: 0 to 5,900 vehicles per day
- 3SG Intersections $AADT_{maj}$: 0 to 58,100 vehicles per day and $AADT_{min}$: 0 to 16,400 vehicles per day
- 4SG Intersections $AADT_{maj}$: 0 to 67,700 vehicles per day and $AADT_{min}$: 0 to 33,400 vehicles per day

- 4SG Intersections Pedestrian Models:
  - AADTmah: 80,200 vehicles per day
  - AADTmin: 49,100 vehicles per day
  - PedVol: 34,200 pedestrians per day crossing all four legs combined

Application to sites with AADTs substantially outside this range may not provide reliable results.

Multiple-Vehicle Collisions

SPFs for multiple-vehicle intersection-related collisions are presented here.

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

$$N_{bimv} = exp\ (a + b \times ln(AADT_{maj}\ ) + c \times ln(AADT_{min}\ )) \qquad (12\text{-}21)$$

Where,

$AADT_{maj}$ = average daily traffic volume (vehicles/day) for major road (both directions of travel combined);

$AADT_{min}$ = average daily traffic volume (vehicles/day) for minor road (both directions of travel combined); and

a, b, c = regression coefficients.

Exhibit 12-19 presents the values of the coefficients a, b, and c used in applying Equation 12-21. The SPF overdispersion parameter, k, is also presented in Exhibit 12-19.

Equation 12-21 is first applied to determine N_{bimv} using the coefficients for total crashes in Exhibit 12-19. N_{bimv} is then divided into components by crash severity level, $N_{bimv(FI)}$ for fatal-and-injury crashes and $N_{bimv(PDO)}$ for property-damage-only crashes. Preliminary values of $N_{bimv(FI)}$ and $N_{bimv(PDO)}$, designated as $N'_{bimv(FI)}$ and $N'_{bimv(PDO)}$ in Equation 12-22, are determined with Equation 12-21 using the coefficients for fatal-and-injury and property-damage-only crashes, respectively, in Exhibit 12-19. The following adjustments are then made to assure that $N_{bimv(FI)}$ and $N_{bimv(PDO)}$ sum to N_{bimv}:

$$N_{bimv(FI)} = N_{bimv(TOTAL)} \times \left(\frac{N'_{bimv(FI)}}{N'_{bimv(FI)} + N'_{bimv(PDO)}} \right) \qquad (12\text{-}22)$$

$$N_{bimv(PDO)} = N_{bimv(TOTAL)} - N_{bimv(FI)} \qquad (12\text{-}23)$$

The proportions in Exhibit 12-24 are used to separate $N_{bimv(FI)}$ and $N_{bimv(PDO)}$ into components by manner of collision.

Exhibit 12-19: SPF Coefficients for Multiple-Vehicle Collisions at Intersections

Intersection type	Coefficients used in Equation 12-21: Intercept (a)	Coefficients used in Equation 12-21: $AADT_{maj}$ (b)	Coefficients used in Equation 12-21: $AADT_{min}$ (c)	Over-dispersion parameter (k)
Total crashes				
3ST	-13.36	1.11	0.41	0.80
3SG	-12.13	1.11	0.26	0.33
4ST	-8.90	0.82	0.25	0.40
4SG	-10.99	1.07	0.23	0.39
Fatal-and-injury crashes				
3ST	-14.01	1.16	0.30	0.69
3SG	-11.58	1.02	0.17	0.30
4ST	-11.13	0.93	0.28	0.48
4SG	-13.14	1.18	0.22	0.33
Property-damage-only crashes				
3ST	-15.38	1.20	0.51	0.77
3SG	-13.24	1.14	0.30	0.36
4ST	-8.74	0.77	0.23	0.40
4SG	-11.02	1.02	0.24	0.44

Exhibit 12-20: Graphical Form of the Intersection SPF for Multiple Vehicle collisons on Three-Leg Intersections with Minor-Road Stop Control (3ST) (from Equation 12-21 and Exhibit 12-19)

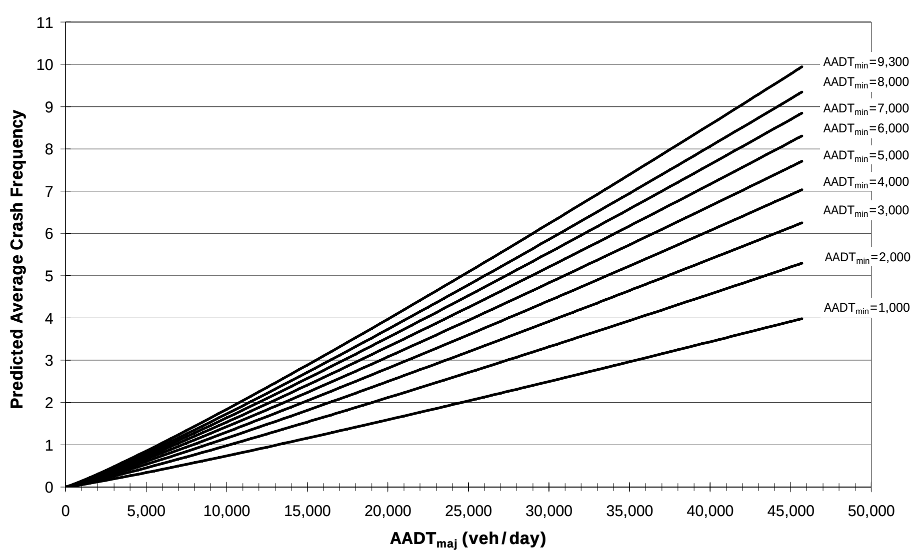


Exhibit 12-21: Graphical Form of the Intersection SPF for Multiple Vehicle collisons on Three-Leg Signalized Intersections (3SG) (from Equation 12-21 and Exhibit 12-19)

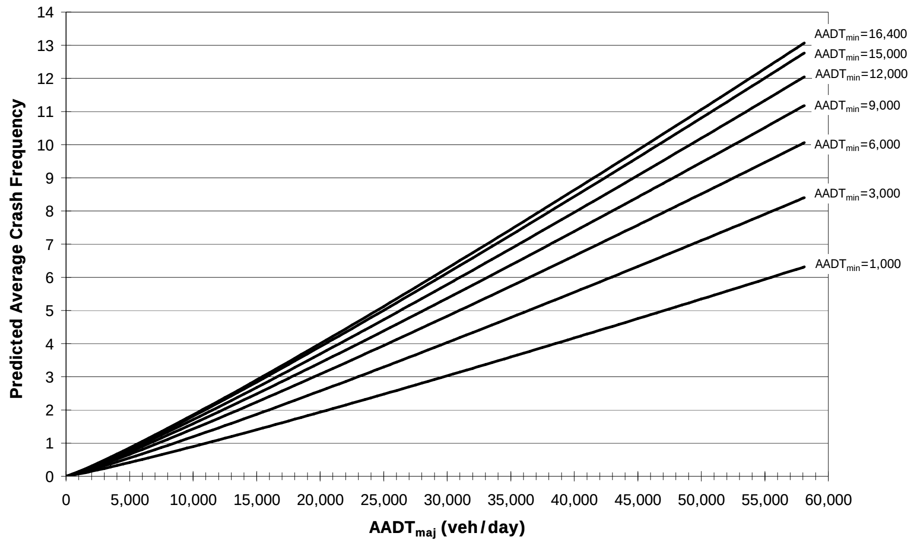


Exhibit 12-22: Graphical Form of the Intersection SPF for Multiple Vehicle collisons on Four-Leg Intersections with Minor-Road Stop Control (4ST) (from Equation 12-21 and Exhibit 12-19)

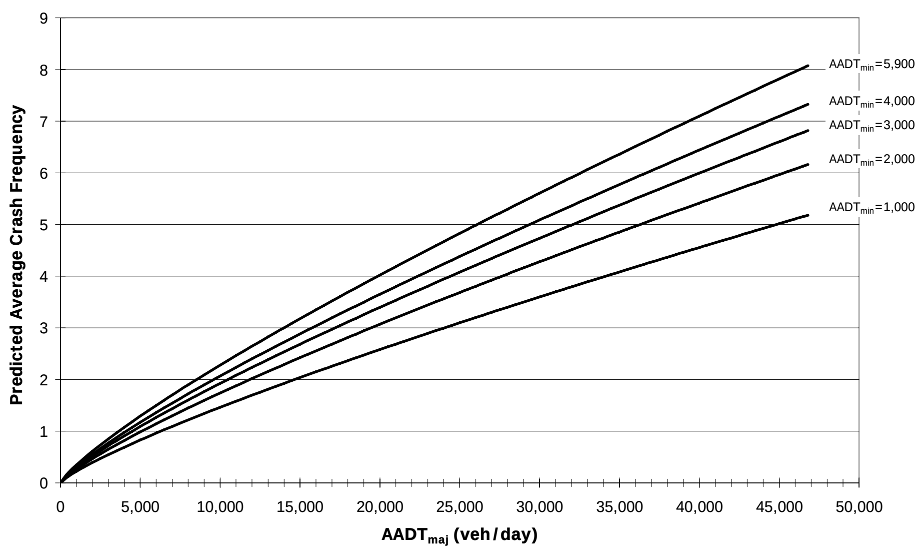


Exhibit 12-23: Graphical Form of the Intersection SPF for Multiple Vehicle collisons on Four-Leg Signalized Intersections (4SG) (from Equation 12-21 and Exhibit 12-19)

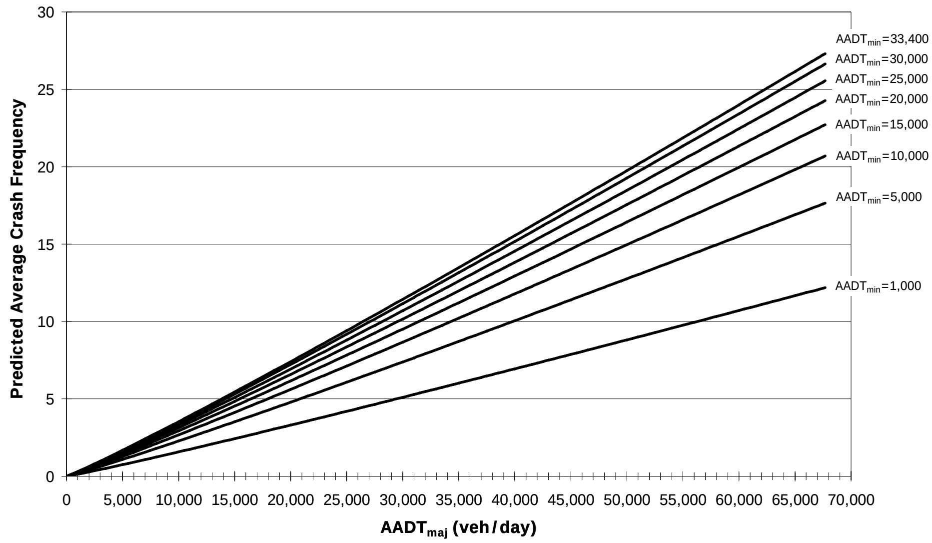


Exhibit 12-24: Distribution of Multiple-Vehicle Collisions for Intersections by Collision Type

Manner of collision	Proportion of crashes by severity level for specific intersections types							
	3ST		3SG		4ST		4SG	
	FI	PDO	FI	PDO	FI	PDO	FI	PDO
Rear-end collision	0.421	0.440	0.549	0.546	0.338	0.374	0.450	0.483
Head-on collision	0.045	0.023	0.038	0.020	0.041	0.030	0.049	0.030
Angle collision	0.343	0.262	0.280	0.204	0.440	0.335	0.347	0.244
Sideswipe	0.126	0.040	0.076	0.032	0.121	0.044	0.099	0.032
Other multiple-vehicle collisions	0.065	0.235	0.057	0.198	0.060	0.217	0.055	0.211

Source: HSIS data for California (2002-2006)

Single-Vehicle Crashes

This section presents SPFs for single-vehicle crashes at intersections.

SPFs for single-vehicle crashes are applied as follows:

$$N_{bisv} = exp\ (a + b \times ln(AADT_{maj}) + c \times ln(AADT_{min})) \qquad (12\text{-}24)$$

Exhibit 12-25 presents the values of the coefficients and factors used in Equation 12-24 for each roadway type. Equation 12-24 is first applied to determine N_{bisv} using the coefficients for total crashes in Exhibit 12-25. N_{bisv} is then divided into components by severity level, $N_{bisv(FI)}$ for fatal-and-injury crashes and $N_{bisv(PDO)}$ for property-damage-only crashes. Preliminary values of $N_{bisv(FI)}$ and $N_{bisv(PDO)}$, designated as $N'_{bisv(FI)}$ and $N'_{bisv(PDO)}$ in Equation 12-25, are determined with Equation 12-24 using the coefficients for fatal-and-injury and property-damage-only crashes, respectively, in Exhibit 12-25. The following adjustments are then made to assure that $N_{bisv(FI)}$ and $N_{bisv(PDO)}$ sum to N_{bisv}.

$$N_{bisv(FI)} = N_{bisv(TOTAL)} \times \left(\frac{N'_{bisv(FI)}}{N'_{bisv(FI)} + N'_{bisv(PDO)}} \right) \qquad (12\text{-}25)$$

$$N_{bisv(PDO)} = N_{bisv(TOTAL)} - N_{bisv(FI)} \qquad (12\text{-}26)$$

Exhibit 12-25: SPF Coefficients for Single-Vehicle Crashes at Intersections

Intersection type	Coefficients used in Equation 12-24 Intercept (a)	$AADT_{maj}$ (b)	$AADT_{min}$ (c)	Over-dispersion parameter (k)
Total crashes				
3ST	-6.81	0.16	0.51	1.14
3SG	-9.02	0.42	0.40	0.36
4ST	-5.33	0.33	0.12	0.65
4SG	-10.21	0.68	0.27	0.36
Fatal-and-injury crashes				
3ST				
3SG	-9.75	0.27	0.51	0.24
4ST				
4SG	-9.25	0.43	0.29	0.09
Property-damage-only crashes				
3ST	-8.36	0.25	0.55	1.29
3SG	-9.08	0.45	0.33	0.53
4ST	-7.04	0.36	0.25	0.54
4SG	-11.34	0.78	0.25	0.44

Note: Where no models are available, Equation 12-27 is used.

Exhibit 12-26: Graphical Form of the Intersection SPF for Single-Vehicle Crashes on Three-leg Intersections with Minor-Road Stop Control (3ST) (from Equation 12-24 and Exhibit 12-25)

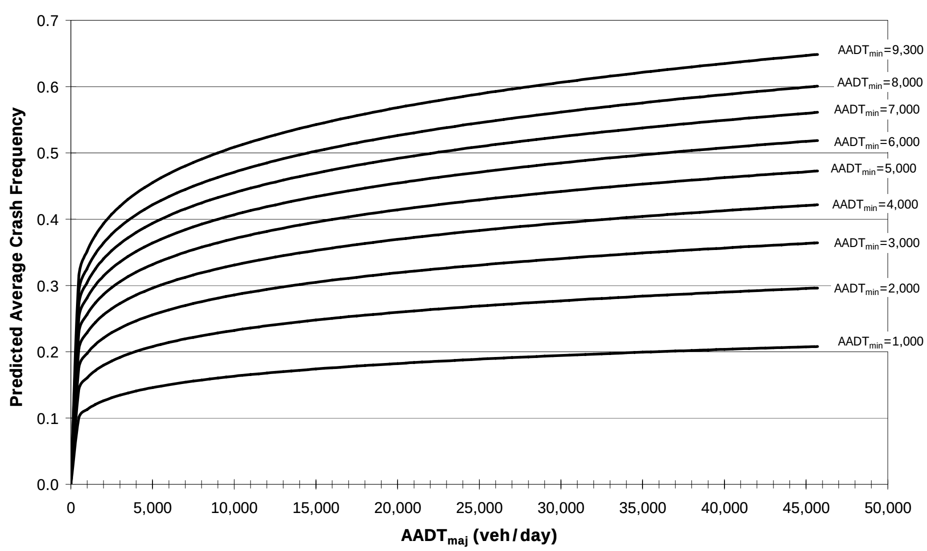


Exhibit 12-27: Graphical Form of the Intersection SPF for Single-Vehicle Crashes on Three-Leg Signalized Intersections (3SG) (from Equation 12-24 and Exhibit 12-25)

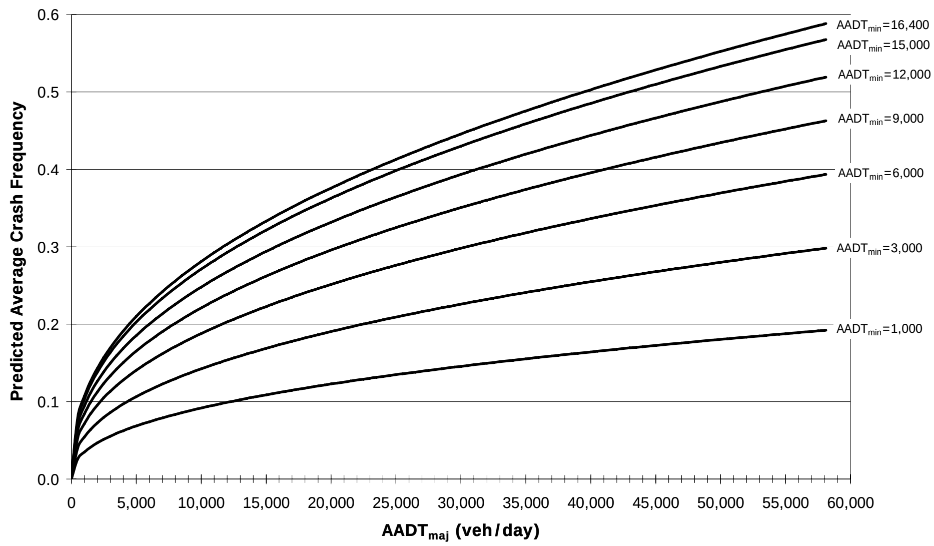


Exhibit 12-28: Graphical Form of the Intersection SPF for Single-Vehicle Crashes on Four-leg Stop Controlled Intersections (4ST) (from Equation 12-24 and Exhibit 12-25)

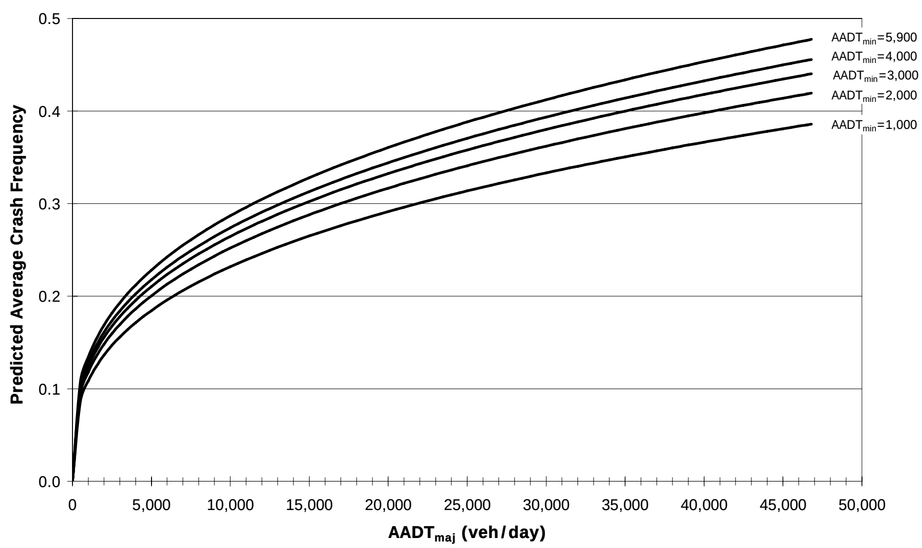
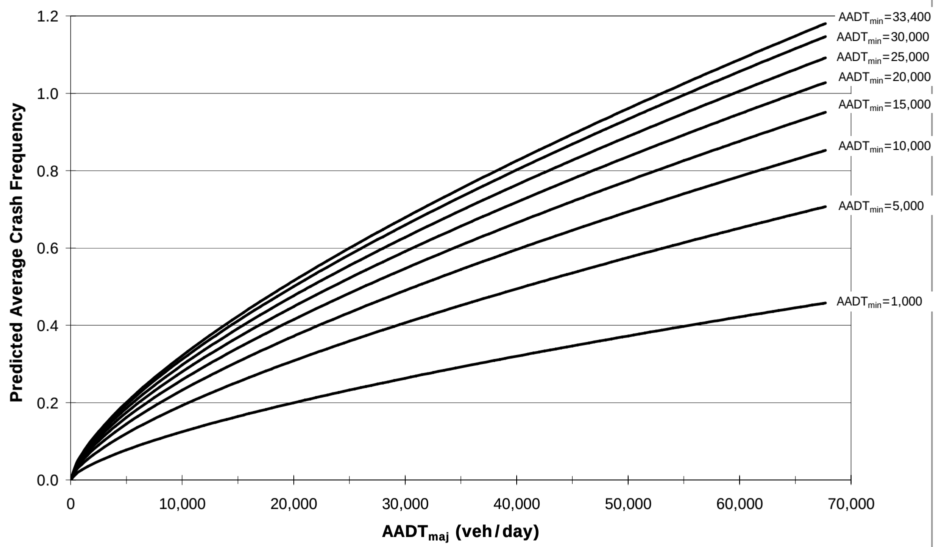


Exhibit 12-29: Graphical Form of the Intersection SPF for Single-Vehicle Crashes on Four-Leg Signalized Intersections (4SG) (from Equation 12-24 and Exhibit 12-25)



The proportions in Exhibit 12-30 are used to separate $N_{bisv(FI)}$ and $N_{bisv(PDO)}$ into components by crash type.

Exhibit 12-30: Distribution of Single-Vehicle Crashes for Intersection by Collision Type

Crash type	Proportion of crashes by severity level for specific road types							
	3ST		3SG		4ST		4SG	
	FI	PDO	FI	PDO	FI	PDO	FI	PDO
Collision with parked vehicle	0.001	0.003	0.001	0.001	0.001	0.001	0.001	0.001
Collision with animal	0.003	0.018	0.001	0.003	0.001	0.026	0.002	0.002
Collision with fixed object	0.762	0.834	0.653	0.895	0.679	0.847	0.744	0.870
Collision with other object	0.090	0.092	0.091	0.069	0.089	0.070	0.072	0.070
Other single-vehicle collision	0.039	0.023	0.045	0.018	0.051	0.007	0.040	0.023
Noncollision	0.105	0.030	0.209	0.014	0.179	0.049	0.141	0.034

Source: HSIS data for California (2002-2006)

Since there are no models for fatal-and-injury crashes at three- and four-leg STOP-controlled intersections in Exhibit 12-25, Equation 12-25 is replaced with the following equation in these cases:

$$N_{bisv(FI)} = N_{bisv(TOTAL)} \times f_{bisv} \qquad (12\text{-}27)$$

Where,

f_{bisv} = proportion of fatal-and-injury crashes for combined sites.

The default value of f_{bisv} in Equation 12-27 is 0.31 for 3ST and 0.28 for 4ST intersections. It is recommended that these default values be updated based on locally available data.

SPFs for Vehicle-Pedestrian Collisions

This section presents SPFs for estimating the number of vehicle-pedestrian collisions at signalized and unsignalized intersections.

Separate SPFs are provided for estimation of the number of vehicle-pedestrian collisions at signalized and unsignalized intersections.

SPFs for Signalized Intersections

The number of vehicle-pedestrian collisions per year at a signalized intersection is estimated with a SPF and a set of AMFs that apply specifically to vehicle-pedestrian collisions. The model for estimating vehicle-pedestrian collisions at signalized intersections is:

$$N_{pedi} = N_{pedbase} \times AMF_{1p} \times AMF_{2p} \times AMF_{3p} \tag{12-28}$$

Where,

$N_{pedbase}$ = predicted number of vehicle-pedestrian collisions per year for base conditions at signalized intersections.

$AMF_{1p}...AMF_{3p}$ = accident modification factors for vehicle-pedestrian collisions at signalized intersections.

The SPF for vehicle-pedestrian collisions at signalized intersections is:

$$N_{pedbase} = exp\ (a + b \times ln(AADT_{tot}) + c \times ln(\frac{AADT_{min}}{AADT_{maj}}) + d \times ln(PedVol) + e \times n_{lanesx}) \tag{12-29}$$

Where,

$AADT_{tot}$ = sum of the average daily traffic volumes (vehicles per day) for the major and minor roads (= $AADT_{maj}$ + $AADT_{min}$);

PedVol = sum of daily pedestrian volumes (pedestrians/day) crossing all intersection legs;

n_{lanesx} = maximum number of traffic lanes crossed by a pedestrian in any crossing maneuver at the intersection considering the presence of refuge islands;

a,b,c,d,e = regression coefficients.

Determination of values for $AADT_{maj}$ and $AADT_{min}$ is addressed in the discussion of Step 3. Only pedestrian crossing maneuvers immediately adjacent to the intersection (e.g., at a marked crosswalk or along the extended path of any sidewalk present) are considered in determining the pedestrian volumes. Exhibit 12-31 presents the values of the coefficients a,b,c,d, and e used in applying Equation 12-29.

The coefficient values in Exhibit 12-31 are intended for estimating total vehicle-pedestrian collisions. All vehicle-pedestrian collisions are considered to be fatal-and-injury crashes.

The application of Equation 12-29 requires data on the total pedestrian volumes crossing the intersection legs. Reliable estimates will be obtained when the value of PedVol in Equation 12-29 is based on actual pedestrian volume counts. Where pedestrian volume counts are not available, they may be estimated using Exhibit 12-32. Replacing the values in Exhibit 12-32 with locally derived values is encouraged.

The value of n_{lanesx} in Equation 12-29 represents the maximum number of traffic lanes that a pedestrian must cross in any crossing maneuver at the intersection. Both through and turning lanes that are crossed by a pedestrian along the crossing path are considered. If the crossing path is broken by an island that provides a suitable refuge for the pedestrian so that the crossing may be accomplished in two (or more) stages, then the number of lanes crossed in each stage is considered separately. To be considered as a suitable refuge, an island must be raised or depressed; a flush or painted island is not treated as a refuge for purposes of determining the value of n_{lanesx}.

Exhibit 12-31: SPFs for Vehicle-Pedestrian Collisions at Signalized Intersections

Intersection type	Coefficients used in Equation 12-29					Over-dispersion parameter (k)
	Intercept (a)	$AADT_{tot}$ (b)	$AADT_{min}/AADT_{maj}$ (c)	PedVol (d)	n_{lanesx} (e)	
Total crashes						
3SG	–6.60	0.05	0.24	0.41	0.09	0.52
4SG	–9.53	0.40	0.26	0.45	0.04	0.24

Exhibit 12-32: Estimates of Pedestrian Crossing Volumes Based on General Level of Pedestrian Activity

General level of pedestrian activity	Estimate of PedVol (pedestrians/day) for use in Equation 12-29	
	3SG intersections	4SG intersections
High	1,700	3,200
Medium-high	750	1,500
Medium	400	700
Medium-low	120	240
Low	20	50

SPFs for STOP-Controlled Intersections

The number of vehicle-pedestrian collisions per year for a STOP-controlled intersection is estimated as:

$$N_{pedi} = N_{bi} \times f_{pedi} \tag{12-30}$$

Where,

f_{pedi} = pedestrian accident adjustment factor.

The value of N_{bi} used in Equation 12-30 is that determined with Equation 12-6.

Exhibit 12-33 presents the values of f_{pedi} for use in Equation 12-30. All vehicle-pedestrian collisions are considered to be fatal-and-injury crashes. The values of f_{pedi} are likely to depend on the climate and walking environment in particular states or communities. HSM users are encouraged to replace the values in Exhibit 12-33 with suitable values for their own state or community through the calibration process (see the Appendix to *Part C*).

Exhibit 12-33: Pedestrian Accident Adjustment Factors for STOP-controlled Intersections

Intersection type	Pedestrian Accident Adjustment Factor (f_{pedi})
3ST	0.021
4ST	0.022

Note: These factors apply to the methodology for predicting total crashes (all severity levels combined). All pedestrian collisions resulting from this adjustment factor are treated as fatal-and-injury crashes and none as property-damage-only crashes. Source: HSIS data for California (2002-2006)

Vehicle-Bicycle Collisions

This section presents calculations for estimating the number of vehicle-bicycle collisions per year for an intersection.

The number of vehicle-bicycle collisions per year for an intersection is estimated as:

$$N_{bikei} = N_{bi} \times f_{bikei} \qquad (12\text{-}31)$$

Where,

f_{bikei} = bicycle accident adjustment factor.

The value of N_{bi} used in Equation 12-31 is determined with Equation 12-6.

Exhibit 12-34 presents the values of f_{bikei} for use in Equation 12-31. All vehicle-bicycle collisions are considered to be fatal-and-injury crashes. The values of f_{bikei} are likely to depend on the climate and bicycling environment in particular states or communities. HSM users are encouraged to replace the values in Exhibit 12-34 with suitable values for their own state or community through the calibration process (see the Appendix to *Part C*).

Exhibit 12-34: Bicycle Accident Adjustment Factors for Intersections

Intersection type	Bicycle Accident Adjustment Factor (f_{bikei})
3ST	0.016
3SG	0.011
4ST	0.018
4SG	0.015

NOTE: These factors apply to the methodology for predicting total crashes (all severity levels combined). All bicycle collisions resulting from this adjustment factor are treated as fatal-and-injury crashes and none as property-damage-only crashes. Source: HSIS data for California (2002-2006)

12.7. ACCIDENT MODIFICATION FACTORS

This section presents the AMFs for the SPFs in Chapter 12.

In Step 10 of the predictive method shown in Section 12.4, Accident Modification Factors are applied to the selected Safety Performance Function (SPF), which was selected in Step 9. SPFs provided in Chapter 12 are presented in Section 12.6. A general overview of Accident Modification Factors (AMFs) is presented in *Chapter 3* Section 3.5.3. The *Part C Introduction and Applications Guidance* provides further discussion on the relationship of AMFs to the predictive method. This section provides details of the specific AMFs applicable to the SPFs presented in Section 12.6.

Accident Modification Factors (AMFs) are used to adjust the SPF estimate of predicted average crash frequency for the effect of individual geometric design and traffic control features, as shown in the general predictive model for Chapter 12 shown in Equation 12-1. The AMF for the SPF base condition of each geometric design or traffic control feature has a value of 1.00. Any feature associated with higher crash frequency than the base condition has an AMF with a value greater than 1.00; any feature associated with lower crash frequency than the base condition has an AMF with a value less than 1.00.

The AMFs used in Chapter 12 are consistent with the AMFs in the *Part D*, although they have, in some cases, been expressed in a different form to be applicable to the base conditions of the SPFs. The AMFs presented in Chapter 12 and the specific SPFs which they apply to are summarized in Exhibit 12-35.

Exhibit 12-35: Summary of AMFs in Chapter 12 and the Corresponding SPFs

Applicable SPF	AMF	AMF Description	AMF Equations and Exhibits
Roadway Segments	AMF_{1r}	On-Street Parking	Equation 12-32 and Exhibit 12-36
	AMF_{2r}	Roadside Fixed Objects	Equation 12-33 and Exhibit 12-37 and 12-38
	AMF_{3r}	Median Width	Exhibit 12-39
	AMF_{4r}	Lighting	Equation 12-34 and Exhibit 12-40
	AMF_{5r}	Automated Speed Enforcement	See text
Multiple-vehicle collisions and single-vehicle crashes at intersections	AMF_{1i}	Intersection Left-Turn Lanes	Exhibit 12-41
	AMF_{2i}	Intersection Left-Turn Signal Phasing	Exhibit 12-42
	AMF_{3i}	Intersection Right-Turn Lanes	Exhibit 12-43
	AMF_{4i}	Right Turn on Red	Equation 12-35
	AMF_{5i}	Lighting	Equation 12-36 and Exhibit 12-44
	AMF_{6i}	Red Light Cameras	Equation 12-37, 12-38, 12-39
Vehicle-Pedestrian Collisions at Signalized Intersections	AMF_{1p}	Bus Stops	Exhibit 12-45
	AMF_{2p}	Schools	Exhibit 12-46
	AMF_{3p}	Alcohol Sales Establishments	Exhibit 12-47

12.7.1. Accident Modification Factors for Roadway Segments

The AMFs for geometric design and traffic control features of urban and suburban arterial roadway segments are presented below. These AMFs are determined in Step 10 of the predictive method and used in Equation 12-3 to adjust the SPF for urban and suburban arterial roadway segments, to account for differences between the base conditions and the local site conditions.

AMF_{1r} - On-Street Parking

The AMF for on-street parking, where present, is based on research by Bonneson[1]. The base condition is the absence of on-street parking on a roadway segment. The AMF is determined as:

$$AMF_{1r} = 1 + p_{pk} \times (f_{pk} - 1.0) \tag{12-32}$$

Where,

AMF_{1r} = accident modification factor for the effect of on-street parking on total accidents;

f_{pk} = factor from Exhibit 12-36;

p_{pk} = proportion of curb length with on-street parking = (0.5 L_{pk}/L); and

L_{pk} = sum of curb length with on-street parking for both sides of the road combined (miles);

L = length of roadway segment (miles).

This AMF applies to total roadway segment crashes.

The sum of curb length with on-street parking (L_{pk}) can be determined from field measurements or video log review to verify parking regulations. Estimates can be made by deducting from twice the roadway segment length allowances for intersection widths, crosswalks, and driveway widths.

Exhibit 12-36: Values of f_{pk} Used in Determining the Accident Modification Factor for On-street Parking

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

AMF_{2r} - Roadside Fixed Objects

The base condition is the absence of roadside fixed objects on a roadway segment. The AMF for roadside fixed objects, where present, has been adapted from the work of Zegeer and Cynecki[15] on predicting utility pole crashes. The AMF is determined with the following equation:

$$AMF_{2r} = f_{offset} \times D_{fo} \times p_{fo} + (1.0 - p_{fo}) \qquad (12\text{-}33)$$

Where,

AMF_{2r} = accident modification factor for the effect of roadside fixed objects on total crashes;

f_{offset} = fixed-object offset factor from Exhibit 12-37;

D_{fo} = fixed-object density (fixed objects/mi) for both sides of the road combined;

p_{fo} = fixed-object collisions as a proportion of total crashes from Exhibit 12-38.

This AMF applies to total roadway segment crashes. If the computed value of AMF_{2r} is less than 1.00, it is set equal to 1.00. This can only occur for very low fixed object densities.

In estimating the density of fixed objects (D_{fo}), only point objects that are 4 inches or more in diameter and do not have breakaway design are considered. Point objects that are within 70-ft of one another longitudinally along the road are counted as a single object. Continuous objects that are not behind point objects are counted as one point object for each 70-ft of length. The offset distance (O_{fo}) shown in Exhibit 12-37 is an estimate of the average distance from the edge of the traveled way to roadside objects over an extended roadway segment. If the average offset to fixed objects exceeds 30-ft, use the value of f_{offset} for 30-ft. Only fixed objects on the roadside on the right side of the roadway in each direction of travel are considered; fixed objects in the roadway median on divided arterials are not considered.

Exhibit 12-37: Fixed-Object Offset Factor

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

Exhibit 12-38: Proportion of Fixed-Object Collisions

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

AMF_{3r} - Median Width

An AMF for median widths on divided roadway segments of urban and suburban arterials is presented in Exhibit 12-39 based on the work of Harkey et al.[6]. The base condition for this AMF is a median width of 15-ft. The AMF applies to total crashes and represents the effect of median width in reducing cross-median collisions; the AMF assumes that nonintersection collision types other than cross-median collusions are not affected by median width. The AMF in Exhibit 12-39 has been adapted from the AMF in Exhibit 13-18 based on the estimate by Harkey et al.[6] that cross-median collisions represent 12.0% of crashes on divided arterials.

This AMF applies only to traversable medians without traffic barriers. The effect of traffic barriers on safety would be expected to be a function of barrier type and offset, rather than the median width; however, the effects of these factors on safety have not been quantified. Until better information is available, an AMF value of 1.00 is used for medians with traffic barriers. The value of this AMF is 1.00 for undivided facilities.

Exhibit 12-39: AMFs for Median Widths on Divided Roadway Segments without a Median Barrier (AMF_{3r})

Median width (ft)	AMF
10	1.01
15	1.00
20	0.99
30	0.98
40	0.97
50	0.96
60	0.95
70	0.94
80	0.93
90	0.93
100	0.92

AMF_{4r} - Lighting

The base condition for lighting is the absence of roadway segment lighting (AMF_{4r} = 1.00). The AMF for lighted roadway segments is determined, based on the work of Elvik and Vaa[(3)], as:

$$AMF_{4r} = 1.0 - (p_{nr} \times (1.0 - 0.72 \times p_{inr} - 0.83 \times p_{pnr})) \tag{12-34}$$

Where,

AMF_{4r} = accident modification factor for the effect of roadway segment lighting on total crashes;

p_{inr} = proportion of total nighttime crashes for unlighted roadway segments that involve a fatality or injury;

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

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

AMF_{4r} applies to total roadway segment crashes. Exhibit 12-40 presents default values for the nighttime crash proportions p_{inr}, p_{pnr}, and p_{nr}. Replacement of the estimates in Exhibit 12-40 with locally derived values is encouraged. If lighting installation increases the density of roadside fixed objects, the value of AMF_{2r} is adjusted accordingly.

Exhibit 12-40: Nighttime Crash Proportions for Unlighted Roadway Segments

Roadway Segment type	Proportion of total nighttime crashes by severity level		Proportion of crashes that occur at night
	Fatal and Injury p_{inr}	PDO p_{pnr}	p_{nr}
2U	0.424	0.576	0.316
3T	0.429	0.571	0.304
4U	0.517	0.483	0.365
4D	0.364	0.636	0.410
5T	0.432	0.568	0.274

AMF_{5r} - Automated Speed Enforcement

Automated speed enforcement systems use video or photographic identification in conjunction with radar or lasers to detect speeding drivers. These systems automatically record vehicle identification information without the need for police officers at the scene. The base condition for automated speed enforcement is that it is absent. *Chapter 17* presents an AMF of 0.83 for the reduction of all types of fatal and injury crashes from implementation of automated speed enforcement. This AMF is assumed to apply to roadway segments between intersections with fixed camera sites where the camera is always present or where drivers have no way of knowing whether the camera is present or not. No information is available on the effect of automated speed enforcement on noninjury crashes. With the conservative assumption that automated speed enforcement has no effect on noninjury crashes, the value of the AMF for automated speed enforcement would be 0.95.

12.7.2. Accident Modification Factors for Intersections

The effects of individual geometric design and traffic control features of intersections are represented in the predictive models by AMFs. AMF_{1i} through AMF_{4i} are applied to multiple-vehicle collisions and single-vehicle crashes at intersections, but not to vehicle-pedestrian and vehicle-bicycle collisions. AMF_{1p} through AMF_{3p} are applied to vehicle-pedestrian collisions at four-leg signalized intersections (4SG), but not to multiple-vehicle collisions and single-vehicle crashes and not to other intersection types.

AMF_{1i} - Intersection Left-Turn Lanes

The base condition for intersection left-turn lanes is the absence of left-turn lanes on the intersection approaches. The AMFs for presence of left-turn lanes are presented in Exhibit 12-41. These AMFs apply to installation of left-turn lanes on any approach to a signalized intersection, but only on uncontrolled major-road approaches to STOP-controlled intersections. The AMFs for installation of left-turn lanes on multiple approaches to an intersection are equal to the corresponding AMF for installation of a left-turn lane on one approach raised to a power equal to the number of approaches with left-turn lanes. There is no indication of any change in crash frequency for providing a left-turn lane on an approach controlled by a STOP sign, so the presence of a left-turn lane on a STOP-controlled approach is not considered in applying Exhibit 12-41. The AMFs in the exhibit apply to total intersection crashes (not including vehicle-pedestrian and vehicle-bicycle collisions). The AMFs for installation of left-turn lanes are based on research by Harwood et al. [(7)]. An AMF of 1.00 is always used when no left-turn lanes are present.

Exhibit 12-41: Accident Modification Factor (AMF_{1i}) for Installation of Left-Turn Lanes on Intersection Approaches

Intersection type	Intersection traffic control	Number of approaches with left-turn lanes[a]			
		One approach	Two approaches	Three approaches	Four approaches
Three-leg intersection	Minor-road STOP control[b]	0.67	0.45	–	–
	Traffic signal	0.93	0.86	0.80	–
Four-leg intersection	Minor-road STOP control[b]	0.73	0.53	–	–
	Traffic signal	0.90	0.81	0.73	0.66

[a] STOP-controlled approaches are not considered in determining the number of approaches with left-turn lanes.

[b] Stop signs present on minor-road approaches only.

AMF_{2i} - Intersection Left-Turn Signal Phasing

The AMF for left-turn signal phasing is based on the results of work by Hauer [(10)], as modified in a study by Lyon et al[(11)]. Types of left-turn signal phasing considered include permissive, protected, protected/permissive, and permissive/protected. Protected/permissive operation is also referred to as a leading left-turn signal phase; permissive/protected operation is also referred to as a lagging left-turn signal phase. The AMF values are presented in Exhibit 12-42. The base condition for this AMF is permissive left-turn signal phasing. This AMF applies to total intersection crashes (not including vehicle-pedestrian and vehicle-bicycle collisions) and is applicable only to signalized intersections. An AMF value of 1.00 is always used for unsignalized intersections.

If several approaches to a signalized intersection have left-turn phasing, the values of AMF_{2i} for each approach are multiplied together.

Exhibit 12-42: Accident Modification Factor (AMF_{2i}) for Type of Left-Turn Signal Phasing

Type of left-turn signal phasing	AMF_{2i}
Permissive	1.00
Protected/permissive or permissive/protected	0.99
Protected	0.94

Note: Use AMF_{2i} = 1.00 for all unsignalized intersections. If several approaches to a signalized intersection have left-turn phasing, the values of AMF_{2i} for each approach are multiplied together.

AMF_{3i} - Intersection Right-Turn Lanes

The base condition for intersection right-turn lanes is the absence of right-turn lanes on the intersection approaches. The AMFs for presence of right-turn lanes based on research by Harwood et al.[7] are presented in Exhibit 12-43. These AMFs apply to installation of right-turn lanes on any approach to a signalized intersection, but only on uncontrolled major-road approaches to STOP-controlled intersections. The AMFs for installation of right-turn lanes on multiple approaches to an intersection are equal to the corresponding AMF for installation of a right-turn lane on one approach raised to a power equal to the number of approaches with right-turn lanes. There is no indication of any change in crash frequency for providing a right-turn lane on an approach controlled by a STOP sign, so the presence of a right-turn lane on a STOP-controlled approach is not considered in applying Exhibit 12-43.

The AMFs in Exhibit 12-43 apply to total intersection crashes (not including vehicle-pedestrian and vehicle-bicycle collisions). An AMF value of 1.00 is always used when no right-turn lanes are present. This AMF applies only to right-turn lanes that are identified by marking or signing. The AMF is not applicable to long tapers, flares, or paved shoulders that may be used informally by right-turn traffic.

Exhibit 12-43: Accident Modification Factor (AMF_{3i}) for Installation of Right-Turn Lanes on Intersection Approaches

Intersection type	Type of traffic control	Number of approaches with right-turn lanes[a]			
		One approach	Two approaches	Three approaches	Four approaches
Three-leg intersection	Minor-road STOP control[b]	0.86	0.74	–	–
	Traffic signal	0.96	0.92	–	
Four-leg intersection	Minor-road STOP control[b]	0.86	0.74	–	–
	Traffic signal	0.96	0.92	0.88	0.85

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

AMF_{4i} - Right Turn on Red

The AMF for prohibiting right turn on red on one or more approaches to a signalized intersection has been derived from a study by Clark[2] and from the AMFs for right-turn-on-red operation shown in *Chapter 14*. The base condition for AMF_{4i} is permitting a right turn on red at all approaches to a signalized intersection. The AMF is determined as:

$$AMF_{4i} = 0.98^{(n_{prohib})} \tag{12-35}$$

Where,

AMF_{4i} = accident modification factor for the effect of prohibiting right turns on red on total crashes; and

n_{prohib} = number of signalized intersection approaches for which right turn on red is prohibited.

This AMF applies to total intersection crashes (not including vehicle-pedestrian and vehicle-bicycle collisions) and is applicable only to signalized intersections. An AMF value of 1.00 is used for unsignalized intersections.

AMF$_{5i}$ - Lighting

The base condition for lighting is the absence of intersection lighting. The AMF for lighted intersections is adapted from the work of Elvik and Vaa[3], as:

$$AMF_{5i} = 1 - 0.38 \times p_{ni} \tag{12-36}$$

Where,

AMF_{5i} = accident modification factor for the effect of intersection lighting on total crashes;

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

This AMF applies to total intersection crashes (not including vehicle-pedestrian and vehicle-bicycle collisions). Exhibit 12-44 presents default values for the nighttime crash proportion pni. HSM users are encouraged to replace the estimates in Exhibit 12-44 with locally derived values.

Exhibit 12-44: Nighttime Crash Proportions for Unlighted Intersections

Intersection Type	Proportion of crashes that occur at night p_{ni}
3ST	0.238
4ST	0.229
3SG and 4SG	0.235

AMF$_{6i}$ - Red Light Cameras

The base condition for red light cameras is their absence. The AMF for installation of a red light camera for enforcement of red signal violations at a signalized intersection is based on an evaluation by Persaud et. al.[(12)]. As shown in *Chapter 14*, this study indicates an AMF for red light camera installation of 0.74 for right-angle collisions and an AMF of 1.18 for rear-end collisions. In other words, red light cameras would typically be expected to reduce right-angle collisions and increase rear-end collisions. There is no evidence that red light camera installation affects other collision types. Therefore, an AMF for the effect of red light camera installation on total crashes can be computed with the following equations:

$$AMF_{6i} = 1 - p_{ra} \times (1 - 0.74) - p_{re} \times (1 - 1.18) \qquad (12\text{-}37)$$

$$p_{ra} = \frac{p_{ramv(FI)} \times N_{bimv(FI)} + p_{ramv(PDO)} \times N_{bimv(PDO)}}{(N_{bimv(FI)} + N_{bimv(PDO)} + N_{bisv})} \qquad (12\text{-}38)$$

$$p_{re} = \frac{p_{remv(FI)} \times N_{bimv(FI)} + p_{remv(PDO)} \times N_{bimv(PDO})}{(N_{bimv(FI)} + N_{bimv(PDO)} + N_{bisv})} \qquad (12\text{-}39)$$

Where,

AMF_{6i} = accident modification factor for installation of red light cameras at signalized intersections;

p_{ra} = proportion of crashes that are multiple-vehicle, right-angle collisions;

p_{re} = proportion of crashes that are multiple-vehicle, rear-end collisions;

$p_{ramv(FI)}$ = proportion of multiple-vehicle fatal-and-injury crashes represented by right-angle collisions;

$p_{ramv(PDO)}$ = proportion of multiple-vehicle property-damage-only crashes represented by right-angle collisions;

$p_{remv(FI)}$ = proportion of multiple-vehicle fatal-and-injury crashes represented by rear-end collisions;

$p_{remv(PDO)}$ = proportion of multiple-vehicle property-damage-only crashes represented by rear-end collisions.

The values of $N_{bimv(FI)}$ is available from Equation 12-22, the value of $N_{bimv(PDO)}$ is available from Equation 12-23, and the value of N_{bisv} is available from Equation 12-24. The values of $p_{ramv(FI)}$, $p_{ramv(PDO)}$, $p_{remv(FI)}$, and $p_{remv(PDO)}$ can be determined from data for the applicable intersection type in Exhibit 12-24. The values in Exhibit 12-24 may be updated with data for a particular jurisdiction as part of the calibration process presented in the Appendix to *Part C*. The data in Exhibit 12-24, by definition, represent average values for a broad range of signalized intersections. Because jurisdictions are likely to implement red-light cameras at intersections with higher than average proportions of right-angle collisions, it is acceptable to replace the values in Exhibit 12-24 with estimate based on data for a specific intersection when determining the value of the red light camera AMF.

This section presents the AMFs for vehicle-pedestrian collisions at signalized intersections.

Accident Modification Factors for Vehicle-Pedestrian Collisions at Signalized Intersections

The AMFs for vehicle-pedestrian collisions at signalized intersections are presented below.

AMF_{1p} - Bus Stops

The AMFs for the number of bus stops within 1,000-ft of the center of the intersection are presented in Exhibit 12-45. The base condition for bus stops is the absence of bus stops near the intersection. These AMFs apply to total vehicle-pedestrian collisions and are based on research by Harwood et al.[8].

Exhibit 12-45: Accident Modification Factor (AMF_{1p}) for the Presence of Bus Stops Near the Intersection

Number of bus stops within 1,000 ft of the intersection	AMF_{1p}
0	1.00
1 or 2	2.78
3 or more	4.15

In applying Exhibit 12-45, multiple bus stops at the same intersection (i.e., bus stops in different intersection quadrants or located some distance apart along the same intersection leg) are counted separately. Bus stops located at adjacent intersections would also be counted as long as any portion of the bus stop is located within 1,000-ft of the intersection being evaluated.

AMF_{2p} - Schools

The base condition for schools is the absence of a school near the intersection. The AMF for schools within 1,000-ft of the center of the intersection is presented in Exhibit 12-46. A school may be counted if any portion of the school grounds is within 1,000-ft of the intersection. Where one or more schools are located near the intersection, the value of the AMF is independent of the number of schools present. This AMF applies to total vehicle-pedestrian collisions and is based on research by Harwood et al.[8].

This AMF indicates that an intersection with a school nearby is likely to experience more vehicle-pedestrian collisions than an intersection without schools, even if the traffic and pedestrian volumes at the two intersections are identical. Such increased crash frequencies indicate that school children are at higher risk than other pedestrians.

Exhibit 12-46: Accident Modification Factor (AMF_{2p}) for the Presence of Schools near the Intersection

Presence of schools within 1,000 ft of the intersection	AMF_{2p}
No school present	1.00
School present	1.35

AMF_{3p} - Alcohol Sales Establishments

The base condition for alcohol sales establishments is the absence of alcohol sales establishments near the intersection. The AMF for the number of alcohol sales establishments within 1,000-ft of the center of an intersection is presented in Exhibit 12-47. Any alcohol sales establishment wholly or partly within 1,000-ft of the intersection may be counted. The AMF applies to total vehicle-pedestrian collisions and is based on research by Harwood et al.[8].

This AMF indicates that an intersection with alcohol sales establishments nearby is likely to experience more vehicle-pedestrian collisions than an intersection without alcohol sales establishments even if the traffic and pedestrian volumes at the two intersections are identical. This indicates the likelihood of higher risk behavior on the part of either pedestrians or drivers near alcohol sales establishments. The AMF includes any alcohol sales establishment which may include liquor stores, bars, restaurants, convenience stores, or grocery stores. Alcohol sales establishments are counted if they are on any intersection leg, or even on another street, as long as they are within 1,000-ft of the intersection being evaluated.

Exhibit 12-47: Accident Modification Factor (AMF_{3p}) for the Number of Alcohol Sales Establishments Near the Intersection

Number of alcohol sales establishments within 1,000-ft of the intersection	AMF_{3p}
0	1.00
1-8	1.12
9 or more	1.56

12.8. CALIBRATION OF THE SPFS TO LOCAL CONDITIONS

In Step 10 of the predictive method, presented in Section 12.4, the predictive model is calibrated to local state or geographic conditions. Crash frequencies, even for nominally similar roadway segments or intersections, can vary widely from one jurisdiction to another. Geographic regions differ markedly in climate, animal population, driver populations, crash reporting threshold, and crash reporting practices. These variations may result in some jurisdictions experiencing a different number of reported traffic crashes on urban and suburban arterial highways than others. Calibration factors are included in the methodology to allow highway agencies to adjust the SPFs to match actual local conditions.

The calibration procedures are presented in the Appendix to Part C.

The calibration factors for roadway segments and intersections (defined below as C_r and C_i, respectively) will have values greater than 1.0 for roadways that, on average, experience more crashes than the roadways used in the development of the SPFs. The calibration factors for roadways that experience fewer crashes on average than the roadways used in the development of the SPFs will have values less than 1.0. The calibration procedures are presented in the Appendix to *Part C*.

Calibration factors provide one method of incorporating local data to improve estimated crash frequencies for individual agencies or locations. Several other default values used in the methodology, such as collision type distribution, can also be replaced with locally derived values. The derivation of values for these parameters is addressed in the calibration procedure in the Appendix to *Part C*.

12.9. INTERIM PREDICTIVE METHOD FOR ROUNDABOUTS

Sufficient research has not yet been conducted to form the basis for development of a predictive method for roundabouts. Since many jurisdictions are planning projects to convert existing intersections into modern roundabouts, an interim predictive method is presented here. This interim procedure is applicable to a location at which a modern roundabout has been constructed or is being planned to replace an existing intersection with minor-road STOP control or an existing signalized intersection. The interim procedure is:

1. Apply the predictive method from Chapter 12 to estimate the crash frequency, N_{int}, for the existing intersection.
2. Multiply N_{int} by the appropriate AMF from Chapter 12 for conversion on an existing intersection to a modern roundabout. The applicable AMFs are:
    - 0.56 for conversion of a two-way STOP-controlled intersection to a modern roundabout.
    - 0.52 for conversion of a signalized intersection to a modern roundabout.

These AMFs are applicable to all crash severities and collision types for both one and two-lane roundabouts in all settings.

At present, there are no available SPFs to determine predicted average crash frequency of an existing or newly constructed roundabout where no intersection currently exists.

12.10. LIMITATIONS OF PREDICTIVE METHOD IN CHAPTER 12

The limitations of the predictive method are presented in the Part C Introduction and Applications Guide.

The limitations of the predictive method which apply generally across all of the *Part C* chapters are discussed in Section C.14 of the *Part C Introduction and Applications Guidance* chapter. This section discusses limitations of the specific predictive models and the application of the predictive method in Chapter 12.

Where urban and suburban arterials intersect access-controlled facilities (i.e., freeways), the grade-separated interchange facility, including the arterial facility within the interchange area, cannot be addressed with the predictive method for urban and suburban arterials.

12.11. APPLICATION OF CHAPTER 12 PREDICTIVE METHOD

The predictive method presented in Chapter 12 applies to urban and suburban arterials. The predictive method is applied to by following the 18 steps presented in Section 12.4. Appendix A provides a series of worksheets for applying the predictive method and the predictive models detailed in this chapter. All computations within these worksheets are conducted with values expressed to three decimal places. This level of precision is needed for consistency in computations. In the last stage of computation, rounding the final estimate expected average crash frequency to one decimal place.

12.12. SUMMARY

The predictive method is used to estimate the expected average crash frequency for a series of contiguous sites (entire urban or suburban arterial facility), or a single individual site. A urban or suburban facility is defined in Section 12.3.

The predictive method for urban and suburban arterial highways is applied by following the 18 steps of the predictive method presented in Section 12.4. Predictive models, developed for urban and suburban arterial facilities, are applied in Steps 9, 10, and 11 of the method. These models have been developed to estimate the predicted average crash frequency of an individual intersection or homogenous roadway segment. The facility is divided into these individual sites in Step 5 of the predictive method.

Where observed data are available, the EB Method may be applied in Step 13 or 15 of the predictive method, to improve the reliability of the estimate. The EB Method can be applied at the site-specific level or at the project specific level. It may also be applied to a future time period if site conditions will not change in the future period. The EB Method is described in the *Part C* Appendix A.2.

Each predictive model in Chapter 12 consists of a Safety Performance Function (SPF), Accident Modification Factors, a calibration factor and pedestrian and bicyclist factors. The SPF is selected in Step 9, and is used to estimate the predicted average crash frequency for a site with base conditions. The estimate can be for total crashes, or by crash severity or collision type distribution. In order to account for differences between the base conditions of the SPF and the actual conditions of the local site, AMFs are applied in Step 10, which adjust the predicted number of crashes according the geometric conditions of the site.

In order to account for the differences in state or regional crash frequencies, the SPF is calibrated to the specific state and or geographic region to which they apply. The process for determining calibration factors for the predictive models is described in the *Part C* Appendix A.1.

Section 12.13 presents 6 sample problems which detail the application of the predictive method. A series of template worksheets have been developed to assist with applying the predictive method in Chapter 12. These worksheets are utilized to solve the sample problems in Section 12.13 and Appendix A contains blank versions of the worksheets.

12.13. SAMPLE PROBLEMS

In this section, six sample problems are presented using the predictive method steps for urban and suburban arterials. Sample Problems 1 and 2 illustrate how to calculate the predicted average crash frequency for urban and suburban arterial roadway segments. Sample Problem 3 illustrates how to calculate the predicted average crash frequency for a STOP-controlled intersection. Sample Problem 4 illustrates a similar calculation for a signalized intersection. Sample Problem 5 illustrates how to combine the results from Sample Problems 1 through 4 in a case where site-specific observed crash data are available (i.e. using the site-specific EB Method). Sample Problem 6 illustrates how to combine the results from sample Problems 1 through 4 in a case where site-specific observed crash data are not available (i.e. using the project-level EB Method).

1550 **Exhibit 12-48: List of Sample Problems in Chapter 12**

Problem No.	Page No.	Description
1	12-61	Predicted average crash frequency for a three-lane TWLTL arterial roadway segment
2	12-79	Predicted average crash frequency for a four-lane divided arterial roadway segment
3	12-95	Predicted average crash frequency for a three-leg STOP-controlled intersection
4	12-109	Predicted average crash frequency for a four-leg signalized intersection
5	12-123	Expected average crash frequency for a facility when site-specific observed crash data are available
6	12-130	Expected average crash frequency for a facility when site-specific observed crash data are not available

1551

12.13.1. Sample Problem 1

The Site/Facility

A three-lane urban arterial roadway segment with a center two-way left-turn lane (TWLTL).

The Question

What is the predicted average crash frequency of the roadway segment for a particular year?

The Facts

- 1.5-mi length
- 11,000 veh/day
- 1.0 mile of parallel on-street commercial parking on each side of street
- 30 driveways (10 minor commercial, 2 major residential, 15 minor residential, 3 minor industrial/institutional)
- 10 roadside fixed objects per mile
- 6-ft offset to roadside fixed objects
- Lighting present
- 35-mph posted speed

Assumptions

- Collision type distributions used are the default values presented in Exhibits 12-7 and 12-10 and Equations 12-19 and 12-20.
- The calibration factor is assumed to be 1.00.

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the roadway segment in Sample Problem 1 is determined to be 7.0 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8

To determine the predicted average crash frequency of the roadway segment in Sample Problem 1, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year, and the EB Method is not applied.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type, and traffic control features.

For a three-lane urban arterial roadway segment with TWLTL, SPF values for multiple-vehicle nondriveway, single-vehicle, multiple-vehicle driveway-related, vehicle-pedestrian and vehicle-bicycle collisions are determined. The calculations for vehicle-pedestrian and vehicle-bicycle collisions are shown in Step 10 since the AMF values are needed for these models.

Multiple-Vehicle Nondriveway Collisions

The SPF for multiple-vehicle nondriveway collisions for the roadway segment is calculated from Equation 12-10 and Exhibit 12-5 as follows:

$$N_{brmv} = exp\ (a + b \times ln(AADT) + ln(L))$$

$$N_{brmv(TOTAL)} = exp\ (-12.40 + 1.41 \times ln(11{,}000) + ln(1.5))$$

$$= 3.085 \text{ crashes/year}$$

$$N_{brmv(FI)} = exp\ (-16.45 + 1.69 \times ln(11{,}000) + ln(1.5))$$

$$= 0.728 \text{ crashes/year}$$

$$N_{brmv(PDO)} = exp\ (-11.95 + 1.33 \times ln(11{,}000) + ln(1.5))$$

$$= 2.298 \text{ crashes/year}$$

These initial values for fatal and injury (FI) and property damage only (PDO) crashes are then adjusted using Equations 12-11 and 12-12 to assure that they sum to the value for total crashes as follows:

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

$$= 3.085 \left(\frac{0.728}{0.728 + 2.298} \right)$$

$$= 0.742 \text{ crashes/year}$$

$$N_{brmv(PDO)} = N_{brmv(TOTAL)} - N_{brmv(FI)}$$

$$= 3.085 - 0.742$$

$$= 2.343 \text{ crashes/year}$$

Single-Vehicle Crashes

The SFP for single-vehicle crashes for the roadway segments is calculated from Equation 12-13 and Exhibit 12-8 as follows:

$$N_{brsv} = exp\ (a + b \times ln(AADT) + ln(L))$$

$$N_{brsv(TOTAL)} = exp\ (-5.74 + 0.54 \times ln(11{,}000) + ln(1.5))$$

$$= 0.734 \text{ crashes/year}$$

$$N_{brsv(FI)} = exp\ (-6.37 + 0.47 \times ln(11{,}000) + ln(1.5))$$

$$= 0.204 \text{ crashes/year}$$

$$N_{brsv(PDO)} = exp\ (-6.29 + 0.56 \times ln(11{,}000) + ln(1.5))$$

$$= 0.510 \text{ crashes/year}$$

These initial values for fatal and injury (FI) and property damage only (PDO) crashes are then adjusted using Equations 12-14 and 12-15 to assure that they sum to the value for total crashes as follows:

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

$$= 0.734 \times \left(\frac{0.204}{0.204 + 0.510} \right)$$

$$= 0.210 \text{ crashes/year}$$

$$N_{brsv(PDO)} = N_{brsv(TOTAL)} - N_{brsv(FI)}$$

$$= 0.734 - 0.210$$

$$= 0.524 \text{ crashes/year}$$

Multiple-Vehicle Driveway-Related Collisions

The SPF for multiple-vehicle driveway-related collisions for the roadway segment is calculated from Equation 12-16 as follows:

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

The number of driveways within the roadway segment, n_j, for Sample Problem 1 is 10 minor commercial, 2 major residential, 15 minor residential, and 3 minor industrial/institutional.

The number of driveway-related collisions, N_j, and the regression coefficient for AADT, t, for a three-lane arterial, are provided in Exhibit 12-11.

$$N_{brdwy(TOTAL)} = 10 \times 0.032 \times \left(\frac{11{,}000}{15{,}000} \right)^{(1.0)} + 2 \times 0.053 \times \left(\frac{11{,}000}{15{,}000} \right)^{(1.0)}$$

$$+ 15 \times 0.010 \times \left(\frac{11{,}000}{15{,}000} \right)^{(1.0)} + 3 \times 0.015 \times \left(\frac{11{,}000}{15{,}000} \right)^{(1.0)}$$

$$= 0.455 \text{ crashes/year}$$

Driveway-related collisions can be separated into components by severity level using Equations 12-17 and 12-18 as follows:

From Exhibit 12-11, for a three-lane arterial the proportion of driveway-related collisions that involve fatalities and injuries, f_{dwy} = 0.243

$$N_{brdwy(FI)} = N_{brdwy(TOTAL)} \times f_{dwy}$$
$$= 0.455 \times 0.243$$
$$= 0.111 \text{ crashes/year}$$
$$N_{brdwy(PDO)} = N_{brdwy(TOTAL)} - N_{brdwy(FI)}$$
$$= 0.455 - 0.111$$
$$= 0.344 \text{ crashes/year}$$

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust base conditions to site specific geometric design and traffic control features

Each AMF used in the calculation of the predicted average crash frequency of the roadway segment is calculated below:

On-Street Parking (AMF_{1r})

AMF_{1r} is calculated from Equation 12-32 as follows:

$$AMF_{1r} = 1 + p_{pk} \times (f_{pk} - 1.0)$$

The proportion of curb length with on-street parking, p_{pk}, is determined as follows:

$$p_{pk} = 0.5 \times \frac{L_{pk}}{L}$$

Since 1.0 mile of on-street parking on each side of the road is provided, the sum of curb length with on-street parking for both sides of the road combined, L_{pk} = 2.

$$p_{pk} = 0.5 \times \frac{2}{1.5}$$
$$= 0.66$$

From Exhibit 12-36, f_{pk} = 2.074.

$$AMF_{1r} = 1 + 0.66 \times (2.074 - 1.0)$$
$$= 1.71$$

Roadside Fixed Objects (AMF_{2r})

AMF_{2r} is calculated from Equation 12-33 as follows:

$$AMF_{2r} = f_{offset} \times D_{fo} \times p_{fo} + (1.0 - p_{fo})$$

From Exhibit 12-37, for a roadside fixed object with a 6-ft offset, the fixed-object offset factor, foffset, is interpolated as 0.124.

From Exhibit 12-38, for a three-lane arterial the proportion of total crashes, p_{fo} = 0.034.

$$AMF_{2r} = 0.124 \times 10 \times 0.034 + (1.0 - 0.034)$$
$$= 1.01$$

Median Width (AMF_{3r})

The value of AMF_{3r} is 1.00 for undivided facilities (see Section 12.7.1). It is assumed that a roadway with TWLTL is undivided.

Lighting (AMF_{4r})

AMF_{4r} is calculated from Equation 12-34 as follows:

$$AMF_{4r} = 1.0 - (p_{nr} \times (1.0 - 0.72 \times p_{inr} - 0.83 \times p_{pnr}))$$

For a three-lane arterial, p_{inr} = 0.429, p_{pnr} = 0.571 and p_{nr} = 0.304 (see Exhibit 12-40).

$$AMF_{4r} = 1.0 - (0.304 \times (1.0 - 0.72 \times 0.429 - 0.83 \times 0.571))$$

$$= 0.93$$

Automated Speed Enforcement (AMF_{5r})

Since there is no automated speed enforcement in Sample Problem 1, AMF_{5r} = 1.00 (i.e. the base condition for AMF_{5r} is the absent of automated speed enforcement).

The combined AMF value for Sample Problem 1 is calculated below.

$$AMF_{COMB} = 1.71 \times 1.01 \times 0.93$$

$$= 1.61$$

Vehicle-Pedestrian and Vehicle-Bicycle Collisions

The predicted average crash frequency of an individual roadway segment (excluding vehicle-pedestrian and vehicle-bicycle collisions) for SPF base conditions, N_{br}, is calculated first in order to determine vehicle-pedestrian and vehicle-bicycle crashes. N_{br} is determined from Equation 12-3 as follows:

$$N_{br} = N_{spf\ rs} \times (AMF_{1r} \times AMF_{2r} \times \ldots \times AMF_{nr})$$

From Equation 12-4, $N_{spf\ rs}$ can be calculated as follows:

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

$$= 3.085 + 0.734 + 0.455$$

$$= 4.274 \text{ crashes/year}$$

The combined AMF value for Sample Problem 1 is 1.61.

$$N_{br} = 4.274 \times (1.61)$$

$$= 6.881 \text{ crashes/year}$$

The SPF for vehicle-pedestrian collisions for the roadway segment is calculated from Equation 12-19 as follows:

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

From Exhibit 12-17, for a posted speed greater than 30 mph on three-lane arterials the pedestrian accident adjustment factor, f_{pedr}= 0.013.

$$N_{pedr} = 6.881 \times 0.013$$

$$= 0.089 \text{ crashes/year}$$

The SPF for vehicle-bicycle collisions is calculated from Equation 12-20 as follows:

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

From Exhibit 12-18, for a posted speed greater than 30 mph on three-lane arterials the bicycle accident adjustment factor, f_{biker}=0.007.

$$N_{biker} = 6.881 \times 0.007$$

$$= 0.048 \text{ crashes/year}$$

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed in that a calibration factor, C_r, of 1.00 has been determined for local conditions. See *Part C* Appendix A.1 for further discussion on calibration of the predicted models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated using Equation 12-2 based on the results obtained in Steps 9 through 11 as follows:

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

$$= 1.00 \times (6.881 + 0.089 + 0.048)$$

$$= 7.018 \text{ crashes/year}$$

Worksheets

The step-by-step instructions above are provided to illustrate the predictive method for calculating the predicted average crash frequency for a roadway segment. To apply the predictive method steps to multiple segments, a series of twelve worksheets are provided for determining the predicted average crash frequency. The twelve worksheets include:

- Worksheet 1A – General Information and Input Data for Urban and Suburban Arterial Roadway Segments
- Worksheet 1B – Accident Modification Factors for Urban and Suburban Arterial Roadway Segments
- Worksheet 1C – Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Arterial Roadway Segments
- Worksheet 1D – Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Arterial Roadway Segments
- Worksheet 1E – Single-Vehicle Crashes by Severity Level for Urban and Suburban Arterial Roadway Segments

- Worksheet 1F – Single-Vehicle Crashes by Collision Type for Urban and Suburban Arterial Roadway Segments
- Worksheet 1G – Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Arterial Roadway Segments
- Worksheet 1H – Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Arterial Roadway Segments
- Worksheet 1I – Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Roadway Segments
- Worksheet 1J – Vehicle-Bicycle Collisions for Urban and Suburban Arterial Roadway Segments
- Worksheet 1K – Crash Severity Distribution for Urban and Suburban Arterial Roadway Segments
- Worksheet 1L – Summary Results for Urban and Suburban Arterial Roadway Segments

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 12 Appendix A.

Worksheet 1A – General Information and Input Data for Urban and Suburban Roadway Segments

Worksheet 1A is a summary of general information about the roadway segment, analysis, input data (i.e., “The Facts”) and assumptions for Sample Problem 1.

Worksheet 1A – General Information and Input Data for Urban and Suburban Roadway Segments

General Information		**Location Information**	
Analyst		Roadway	
Agency or Company		Roadway Section	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	**Base Conditions**	**Site Conditions**
Road type (2U, 3T, 4U, 4D, 5T)	-	3T
Length of segment, L (mi)	-	1.5
AADT (veh/day)	-	11,000
Type of on-street parking (none/parallel/angle)	none	parallel - commercial
Proportion of curb length with on-street parking	-	0.66
Median width (ft)	15	not present
Lighting (present / not present)	not present	present
Auto speed enforcement (present/not present)	not present	not present
Major commercial driveways (number)	-	0
Minor commercial driveways (number)	-	10
Major industrial/institutional driveways (number)	-	0
Minor industrial/institutional driveways (number)	-	3
Major residential driveways (number)	-	2
Minor residential driveways (number)	-	15
Other driveways (number)	-	0
Speed Category	-	intermediate or high speed (>30 mph)
Roadside fixed object density (fixed objects/mi)	not present	10
Offset to roadside fixed objects (ft)	not present	6
Calibration Factor, C_r	1.0	1.0

1764

Worksheet 1B – Accident Modification Factors for Urban and Suburban Roadway Segments

In Step 10 of the predictive method, Accident Modification Factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 12.7 presents the tables and equations necessary for determining the AMF values. Once the value for each AMF has been determined, all of the AMFs are multiplied together in Column 6 of Worksheet 1B which indicates the combined AMF value.

Worksheet 1B – Accident Modification Factors for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
AMF for On-Street Parking	AMF for Roadside Fixed Objects	AMF for Median Width	AMF for Lighting	AMF for Auto Speed Enforcement	Combined AMF
AMF_{1r}	AMF_{2r}	AMF_{3r}	AMF_{4r}	AMF_{5r}	AMF_{COMB}
from Equation 12-32	from Equation 12-33	from Exhibit 12-39	from Equation 12-34	from Section 12.7.1	(1)*(2)*(3)*(4)*(5)
1.71	1.01	1.00	0.93	1.00	1.61

Worksheet 1C – Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Roadway Segments

The SPF for multiple-vehicle nondriveway collisions along the roadway segment in Sample Problem 1 is calculated using Equation 12-10 and entered into Column 4 of Worksheet 1C. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 1 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values(from Column 4) to assure that fatal and injury (FI) and property damage only (PDO) crashes sum to the total crashes, as illustrated in Column 6. Column 7 represents the combined AMF (from Column 6 in Worksheet 1B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of multiple-vehicle nondriveway crashes using the values in Column 6, the combined AMF in Column 7, and the calibration factor in Column 8.

Worksheet 1C – Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)
Crash severity level	**SPF Coefficients**		**Overdispersion Parameter, k**	**Initial N_{brmv}**	**Proportion of total crashes**	**Adjusted N_{brmv}**	**Combined AMFs**	**Calibration factor**	**Predicted N_{brmv}**
	from Exhibit 12-5		from Exhibit 12-5	from Equation 12-10		$(4)_{TOTAL}*(5)$	(6) from Worksheet 1B	C_r	(6)*(7)*(8)
	a	b							
Total	-12.40	1.41	0.66	3.085	1.000	3.085	1.61	1.00	4.967
Fatal and injury (FI)	-16.45	1.69	0.59	0.728	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ 0.241	0.743	1.61	1.00	1.196
Property damage only (PDO)	-11.95	1.33	0.59	2.298	$(5)_{TOTAL}-(5)_{FI}$ 0.759	2.342	1.61	1.00	3.771

Worksheet 1D –Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Roadway Segments

Worksheet 1D presents the default proportions for collision type (from Exhibit 12-7) by crash severity level as follows:

- Fatal and injury crashes (Column 2)
- Property damage only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for multiple-vehicle nondriveway crashes by collision type is presented in Columns 3 (Fatal and Injury, FI), 5 (Property Damage Only, PDO), and 6 (Total).

These proportions may be used to separate the predicted average crash frequency for multiple-vehicle nondriveway crashes (from Column 9, Worksheet 1C) into components by crash severity and collision type.

Worksheet 1D – Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
Collision type	**Proportion of Collision Type $_{(FI)}$**	**Predicted $N_{brmv\ (FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**Predicted $N_{brmv\ (PDO)}$ (crashes/year)**	**Predicted $N_{brmv\ (TOTAL)}$ (crashes/year)**
	from Exhibit 12-7	$(9)_{FI}$ from Worksheet 1C	from Exhibit 12-7	$(9)_{PDO}$ from Worksheet 1C	$(9)_{TOTAL}$ from Worksheet 1C
Total	1.000	1.196	1.000	3.771	4.967
		$(2)*(3)_{FI}$		$(4)*(5)_{PDO}$	(3)+(5)
Rear-end collision	0.845	1.011	0.842	3.175	4.186
Head-on collision	0.034	0.041	0.020	0.075	0.116
Angle collision	0.069	0.083	0.020	0.075	0.158
Sideswipe, same direction	0.001	0.001	0.078	0.294	0.295
Sideswipe, opposite direction	0.017	0.020	0.020	0.075	0.095
Other multiple-vehicle collision	0.034	0.041	0.020	0.075	0.116

Worksheet 1E – Single-Vehicle Crashes by Severity Level for Urban and Suburban Roadway Segments

The SPF for single-vehicle crashes along the roadway segment in Sample Problem 1 is calculated using Equation 12-13 and entered into Column 4 of Worksheet 1E. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 1 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values (from Column 4) to assure that fatal and injury (FI) and property damage only (PDO) crashes sum to the total crashes, as illustrated in Column 6. Column 7 represents the combined AMF (from Column 6 in Worksheet 1B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of multiple-vehicle nondriveway crashes using the values in Column 6, the combined AMF in Column 7, and the calibration factor in Column 8.

Worksheet 1E – Single-Vehicle Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)
Crash severity level	**SPF Coefficients**		**Overdispersion Parameter, k**	**Initial N_{brsv}**	**Proportion of total crashes**	**Adjusted N_{brsv}**	**Combined AMFs**	**Calibration factor**	**Predicted N_{brsv}**
	from Exhibit 12-8		from Exhibit 12-8	from Equation 12-13		$(4)_{TOTAL}$*(5)	(6) from Worksheet 1B	C_r	(6)*(7)*(8)
	a	b							
Total	-5.74	0.54	1.37	0.734	1.000	0.734	1.61	1.00	1.182
Fatal and injury (FI)	-6.37	0.47	1.06	0.204	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ 0.286	0.210	1.61	1.00	0.338
Property damage only (PDO)	-6.29	0.56	1.93	0.510	$(5)_{TOTAL}-(5)_{FI}$ 0.714	0.524	1.61	1.00	0.844

Worksheet 1F –Single-Vehicle Crashes by Collision Type for Urban and Suburban Roadway Segments

Worksheet 1F presents the default proportions for collision type (from Exhibit 12-8) by crash severity level as follows:

- Fatal and injury crashes (Column 2)
- Property damage only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for single-vehicle crashes by collision type is presented in 3 (Fatal and Injury, FI), 5 (Property Damage Only, PDO), and Columns 6 (Total).

These proportions may be used to separate the predicted average crash frequency for single-vehicle crashes (from Column 9, Worksheet 1E) into components by crash severity and collision type.

Worksheet 1F – Single-Vehicle Collisions by Collision Type for Urban and Suburban Roadway Segments					
(1)	(2)	(3)	(4)	(5)	(6)
Collision type	**Proportion of Collision Type $_{(FI)}$**	**Predicted $N_{brsv\,(FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**Predicted $N_{brsv\,(PDO)}$ (crashes/year)**	**Predicted $N_{brsv\,(TOTAL)}$ (crashes/year)**
	from Exhibit 12-10	$(9)_{FI}$ from Worksheet 1E	from Exhibit 12-10	$(9)_{PDO}$ from Worksheet 1E	$(9)_{TOTAL}$ from Worksheet 1E
Total	1.000	0.338	1.000	0.844	1.182
		$(2)*(3)_{FI}$		$(4)*(5)_{PDO}$	(3)+(5)
Collision with animal	0.001	0.000	0.001	0.001	0.001
Collision with fixed object	0.688	0.233	0.963	0.813	1.046
Collision with other object	0.001	0.000	0.001	0.001	0.001
Other single-vehicle collision	0.310	0.105	0.035	0.030	0.135

Worksheet 1G – Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Roadway Segments

Worksheet 1G determines and presents the number of driveway-related multiple-vehicle collisions. The number of driveways along both sides of the road is entered in Column 2 by driveway type (Column 1). The associated number of crashes per driveway per year by driveway type as found in Exhibit 12-11 is entered in Column 3. Column 4 contains the regression coefficient for AADT also found in Exhibit 12-11. The initial average crash frequency of multiple-vehicle driveway-related crashes is calculated from Equation 12-16 and entered into Column 5. The overdispersion parameter from Exhibit 12-11 is entered into Column 6; however, the overdispersion parameter is not needed for Sample Problem 1 (as the EB Method is not utilized).

Worksheet 1G – Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Roadway Segments					
(1)	(2)	(3)	(4)	(5)	(6)
Driveway type	**Number of driveways, n_j**	**Crashes per driveway per year, N_j**	**Coefficient for traffic adjustment, t**	**Initial N_{brdwy}**	**Overdispersion parameter, k**
		from Exhibit 12-11	from Exhibit 12-11	Equation 12-16 $n_j*N_j*(AADT/15,000)^t$	from Exhibit 12-11
Major commercial	0	0.102	1.000	0.000	
Minor commercial	10	0.032	1.000	0.235	
Major industrial/institutional	0	0.110	1.000	0.000	
Minor industrial/institutional	3	0.015	1.000	0.033	-
Major residential	2	0.053	1.000	0.078	
Minor residential	15	0.010	1.000	0.110	
Other	0	0.016	1.000	0.000	
Total	-	-	-	0.456	1.10

Worksheet 1H – Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Roadway Segments

The initial average crash frequency of multiple-vehicle driveway-related crashes from Column 5 of Worksheet 1G is entered in Column 2. This value is multiplied by the proportion of crashes by severity (Column 3) found in Exhibit 12-11 and the adjusted value is entered into Column 4. Column 5 represents the combined AMF (from Column 6 in Worksheet 1B), and Column 6 represents the calibration factor. Column 7 calculates the predicted average crash frequency of multiple-vehicle driveway-related crashes using the values in Column 4, the combined AMF in Column 5, and the calibration factor in Column 6.

Worksheet 1H – Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Roadway Segments						
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Crash severity level	**Initial N_{brdwy}**	**Proportion of total accidents (f_{dwy})**	**Adjusted N_{brdwy}**	**Combined AMFs**	**Calibration factor, C_r**	**Predicted N_{brdwy}**
	$(5)_{TOTAL}$ from Worksheet 1G	from Exhibit 12-11	$(2)_{TOTAL}$ *(3)	(6) from Worksheet 1B		(4)*(5)*(6)
Total	0.456	1.000	0.456	1.61	1.00	0.734
Fatal and injury (FI)	-	0.243	0.111	1.61	1.00	0.179
Property damage only (PDO)	-	0.757	0.345	1.61	1.00	0.555

Worksheet 1I– Vehicle-Pedestrian Collisions for Urban and Suburban Roadway Segments

The predicted average crash frequency of multiple-vehicle nondriveway, single-vehicle and multiple-vehicle driveway-related predicted crashes from Worksheets 1C, 1E, and 1H are entered into Columns 2, 3, and 4, respectively. These values are summed in Column 5. Column 6 contains the pedestrian accident adjustment factor (see Exhibit 12-17). Column 7 represents the calibration factor. The predicted average crash frequency of vehicle-pedestrian collisions (Column 8) is the product of Columns 5, 6 and 7. Since all vehicle-pedestrian crashes are assumed to involve some level of injury, there are no property damage only crashes.

Worksheet 1I – Vehicle-Pedestrian Collisions for Urban and Suburban Roadway Segments							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crash severity level	**Predicted N_{brmv}**	**Predicted N_{brsv}**	**Predicted N_{brdwy}**	**Predicted N_{br}**	**f_{pedr}**	**Calibration factor, C_r**	**Predicted N_{pedr}**
	(9) from Worksheet 1C	(9) from Worksheet 1E	(7) from Worksheet 1H	(2)+(3)+(4)	from Exhibit 12-17		(5)*(6)*(7)
Total	4.967	1.182	0.734	6.883	0.013	1.00	0.089
Fatal and injury (FI)	-	-	-	-	-	1.00	0.089

Worksheet 1J – Vehicle-Bicycle Collisions for Urban and Suburban Roadway Segments

The predicted average crash frequency of multiple-vehicle nondriveway, single-vehicle and multiple-vehicle driveway-related predicted crashes from Worksheets 1C, 1E, and 1H are entered into Columns 2, 3, and 4, respectively. These values are summed in Column 5. Column 6 contains the bicycle accident adjustment factor (see Exhibit 12-18). Column 7 represents the calibration factor. The predicted average crash frequency of vehicle-bicycle collisions (Column 8) is the product of Columns 5, 6 and 7. Since all vehicle-bicycle collisions are assumed to involve some level of injury, there are no property damage only crashes.

Worksheet 1J – Vehicle-Bicycle Collisions for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crash severity level	**Predicted N_{brmv}**	**Predicted N_{brsv}**	**Predicted N_{brdwy}**	**Predicted N_{br}**	**f_{biker}**	**Calibration factor, C_r**	**Predicted N_{biker}**
	(9) from Worksheet 1C	(9) from Worksheet 1E	(7) from Worksheet 1H	(2)+(3)+(4)	from Exhibit 12-18		(5)*(6)*(7)
Total	4.967	1.182	0.734	6.883	0.007	1.00	0.048
Fatal and injury	-	-	-	-	-	1.00	0.048

Worksheet 1K – Crash Severity Distribution for Urban and Suburban Roadway Segments

Worksheet 1K provides a summary of all collision types by severity level. Values from Worksheets 1C, 1E, 1H, 1I, and 1J are presented and summed to provide the predicted average crash frequency for each severity level as follows:

- Fatal and injury crashes (Column 2)
- Property damage only crashes (Column 3)
- Total crashes (Column 4)

Worksheet 1K – Crash Severity Distribution for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)
Collision type	**Fatal and injury (FI)**	**Property damage only (PDO)**	**Total**
	(3) from Worksheet 1D and 1F; (7) from Worksheet 1H; and (8) from Worksheet 1I and 1J	(5) from Worksheet 1D and 1F; and (7) from Worksheet 1H;	(6) from Worksheet 1D and 1F; (7) from Worksheet 1H; and (8) from Worksheet 1I and 1J
MULTIPLE-VEHICLE			
Rear-end collisions (from Worksheet 1D)	1.011	3.175	4.186
Head-on collisions (from Worksheet 1D)	0.041	0.075	0.116
Angle collisions (from Worksheet 1D)	0.083	0.075	0.158
Sideswipe, same direction (from Worksheet 1D)	0.001	0.294	0.295
Sideswipe, opposite direction (from Worksheet 1D)	0.020	0.075	0.095
Driveway-related collisions (from Worksheet 1H)	0.179	0.555	0.734
Other multiple-vehicle collision (from Worksheet 1D)	0.041	0.075	0.116
Subtotal	1.376	4.324	5.700
SINGLE-VEHICLE			
Collision with animal (from Worksheet 1F)	0.000	0.001	0.001
Collision with fixed object (from Worksheet 1F)	0.233	0.813	1.046
Collision with other object (from Worksheet 1F)	0.000	0.001	0.001
Other single-vehicle collision (from Worksheet 1F)	0.105	0.030	0.135
Collision with pedestrian (from Worksheet 1I)	0.089	0.000	0.089
Collision with bicycle (from Worksheet 1J)	0.048	0.000	0.048
Subtotal	0.475	0.845	1.320
Total	1.851	5.169	7.020

1844

Worksheet 1L– Summary Results for Urban and Suburban Roadway Segments

Worksheet 1L presents a summary of the results. Using the roadway segment length and the AADT, the worksheet presents the crash rate in miles per year (Column 4) and in million vehicle miles (Column 6).

Worksheet 1L – Summary Results for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)
Crash severity level	**Predicted average crash frequency, $N_{predicted\ rs}$ (crashes/year)**	**Roadway segment length, L (mi)**	**Crash rate (crashes/mi/year)**
	(Total) from Worksheet 1K		(2)/(3)
Total	7.020	1.5	4.7
Fatal and injury (FI)	1.851	1.5	1.2
Property damage only (PDO)	5.169	1.5	3.4

12.13.2. Sample Problem 2

The Highway

A four-lane divided urban arterial roadway segment.

The Question

What is the predicted average crash frequency of the roadway segment for a particular year?

The Facts

- 0.75-mi length
- 23,000 veh/day
- On-street parking not permitted
- 8 driveways (1 major commercial, 4 minor commercial, 1 major residential, 1 minor residential, 1 minor industrial/institutional)
- 20 roadside fixed objects per mile
- 12-ft offset to roadside fixed objects
- 40-ft median
- Lighting present
- 30-mph posted speed

Assumptions

- Collision type distributions used are the default values presented in Exhibits 12-7 and 12-10 and Equations 12-19 and 12-20.
- The calibration factor is assumed to be 1.00.

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the roadway segment in Sample Problem 2 is determined to be 3.4 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8

To determine the predicted average crash frequency of the roadway segment in Sample Problem 2, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year, and the EB Method is not applied.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type, and traffic control features.

For a four-lane divided urban arterial roadway segment, SPF values for multiple-vehicle nondriveway, single-vehicle, multiple-vehicle driveway-related, vehicle-pedestrian and vehicle-bicycle collisions are determined. The calculations for total multiple-vehicle nondriveway, single-vehicle and multiple-vehicle driveway-related collisions are presented below. Detailed steps for calculating SPFs for fatal and injury (FI) and property damage only (PDO) crashes are presented in Sample Problem 1. The calculations for vehicle-pedestrian and vehicle-bicycle collisions are shown in Step 10 since the AMF values are needed for these two models.

Multiple-Vehicle Nondriveway Collisions

The SPF for multiple-vehicle nondriveway collisions for the roadway segment is calculated from Equation 12-10 and Exhibit 12-5 as follows:

$$N_{brmv} = exp\ (a + b \times ln(AADT) + ln(L))$$

$$N_{brmv(TOTAL)} = exp\ (-12.34 + 1.36 \times ln(23{,}000) + ln(0.75))$$

$$= 2.804 \text{ crashes/year}$$

Single-Vehicle Crashes

The SFP for single-vehicle crashes for the roadway segments is calculated from Equation 12-13 and Exhibit 12-8 as follows:

$$N_{brsv} = exp\ (a + b \times ln(AADT) + ln(L))$$

$$N_{brsv(TOTAL)} = exp\ (-5.05 + 0.47 \times ln(23{,}000) + ln(0.75))$$

$$= 0.539 \text{ crashes/year}$$

Multiple-Vehicle Driveway-Related Collisions

The SPF for multiple-vehicle driveway-related collisions for the roadway segment is calculated from Equation 12-16 as follows:

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

The number of driveways within the roadway segment, n_j, for Sample Problem 1 is 1 major commercial, 4 minor commercial, 1 major residential, 1 minor residential, and 1 minor industrial/institutional.

The number of driveway-related collisions, N_j, and the regression coefficient for AADT, t, for a four-lane divided arterial, are provided in Exhibit 12-11.

$$N_{brdwy(TOTAL)} = 1 \times 0.033 \times \left(\frac{23{,}000}{15{,}000}\right)^{(1.106)} + 4 \times 0.011 \times \left(\frac{23{,}000}{15{,}000}\right)^{(1.106)} + 1 \times 0.018 \times \left(\frac{23{,}000}{15{,}000}\right)^{(1.106)}$$

$$+ 1 \times 0.003 \times \left(\frac{23{,}000}{15{,}000}\right)^{(1.106)} + 1 \times 0.005 \times \left(\frac{23{,}000}{15{,}000}\right)^{(1.106)}$$

$$= 0.165 \text{ crashes/year}$$

The fatal and injury (FI) and property damage only (PDO) SPF values for multiple-vehicle nondriveway collisions, single-vehicle crashes and multiple-vehicle driveway-related collisions can be determined by using the same procedure presented in Sample Problem 1.

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust base conditions to site specific geometric design and traffic control features

Each AMF used in the calculation of the predicted average crash frequency of the roadway segment is calculated below:

On-Street Parking (AMF_{1r})

Since on-street parking is not permitted, AMF_{1r}=1.00 (i.e. the base condition for AMF_{1r} is the absence of on-street parking).

Roadside Fixed Objects (AMF_{2r})

AMF_{2r} is calculated from Equation 12-33 as follows:

$$AMF_{2r} = f_{offset} \times D_{fo} \times p_{fo} + (1.0 - p_{fo})$$

From Exhibit 12-37, for a roadside fixed object with a 12-ft offset, the fixed-object offset factor, f_{offset}, is interpolated as 0.079.

From Exhibit 12-38, for a four-lane divided arterial the proportion of total crashes, p_{fo} = 0.036.

$$AMF_{2r} = 0.079 \times 20 \times 0.036 + (1.0 - 0.036)$$

$$= 1.02$$

Median Width (AMF_{3r})

From Exhibit 12-39, for a four-lane divided arterial with a 40-ft median, AMF_{3r} = 0.97.

Lighting (AMF_{4r})

AMF_{4r} can be calculated from Equation 12-34 as follows:

$$AMF_{4r} = 1.0 - (p_{nr} \times (1.0 - 0.72 \times p_{inr} - 0.83 \times p_{pnr}))$$

For a for-lane divided arterial, p_{inr} = 0.364, p_{pnr} = 0.636 and p_{nr} = 0.410 (see Exhibit 12-40).

$$AMF_{4r} = 1.0 - (0.410 \times (1.0 - 0.72 \times 0.364 - 0.83 \times 0.636))$$

$$= 0.91$$

Automated Speed Enforcement (AMF_{5r})

Since there is no automated speed enforcement in Sample Problem 2, AMF_{5r} = 1.00 (i.e. the base condition for AMF_{5r} is the absent of automated speed enforcement).

The combined AMF value for Sample Problem 2 is calculated below.

$$AMF_{COMB} = 1.02 \times 0.97 \times 0.91$$

$$= 0.90$$

Vehicle-Pedestrian and Vehicle-Bicycle Collisions

The predicted average crash frequency of an individual roadway segment (excluding vehicle-pedestrian and vehicle-bicycle collisions) for SPF base conditions, N_{br}, is calculated first in order to determine vehicle-pedestrian and vehicle-bicycle crashes. N_{br} is determined from Equation 12-3 as follows:

$$N_{br} = N_{spf\ rs} \times (AMF_{1r} \times AMF_{2r} \times \ldots \times AMF_{nr})$$

From Equation 12-4, $N_{spf\ rs}$ can be calculated as follows:

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

$$= 2.804 + 0.539 + 0.165$$

$$= 3.508 \text{ crashes/year}$$

The combined AMF value for Sample Problem 2 is 0.90.

$$N_{br} = 3.508 \times (0.90)$$

$$= 3.157 \text{ crashes/year}$$

The SPF for vehicle-pedestrian collisions for the roadway segment is calculated from Equation 12-19 as follows:

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

From Exhibit 12-17, for a posted speed of 30 mph on four-lane divided arterials the pedestrian accident adjustment factor, f_{pedr}= 0.067.

$$N_{pedr} = 3.157 \times 0.067$$

$$= 0.212 \text{ crashes/year}$$

The SPF for vehicle-bicycle collisions is calculated from Equation 12-20 as follows:

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

From Exhibit 12-18, for a posted speed of 30 mph on four-lane divided arterials the bicycle accident adjustment factor, f_{biker}=0.013.

$$N_{biker} = 3.157 \times 0.013$$

$$= 0.041 \text{ crashes/year}$$

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed in that a calibration factor, C_r, of 1.00 has been determined for local conditions. See *Part C* Appendix A.1 for further discussion on calibration of the predicted models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated using Equation 12-2 based on the results obtained in Steps 9 through 11 as follows:

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

$$= 1.00 \times (3.157 + 0.212 + 0.041)$$

$$= 3.410$$

Worksheets

The step-by-step instructions above are provided to illustrate the predictive method for calculating the predicted average crash frequency for a roadway segment. To apply the predictive method steps to multiple segments, a series of twelve worksheets are provided for determining the predicted average crash frequency. The twelve worksheets include:

- Worksheet 1A – General Information and Input Data for Urban and Suburban Arterial Roadway Segments
- Worksheet 1B – Accident Modification Factors for Urban and Suburban Arterial Roadway Segments
- Worksheet 1C – Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Arterial Roadway Segments
- Worksheet 1D – Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Arterial Roadway Segments
- Worksheet 1E – Single-Vehicle Crashes by Severity Level for Urban and Suburban Arterial Roadway Segments
- Worksheet 1F – Single-Vehicle Crashes by Collision Type for Urban and Suburban Arterial Roadway Segments
- Worksheet 1G – Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Arterial Roadway Segments
- Worksheet 1H – Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Arterial Roadway Segments
- Worksheet 1I – Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Roadway Segments
- Worksheet 1J – Vehicle-Bicycle Collisions for Urban and Suburban Arterial Roadway Segments
- Worksheet 1K – Crash Severity Distribution for Urban and Suburban Arterial Roadway Segments
- Worksheet 1L – Summary Results for Urban and Suburban Arterial Roadway Segments

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 12 Appendix A.

Worksheet 1A – General Information and Input Data for Urban and Suburban Roadway Segments

Worksheet 1A is a summary of general information about the roadway segment, analysis, input data (i.e., "The Facts") and assumptions for Sample Problem 2.

Worksheet 1A – General Information and Input Data for Urban and Suburban Roadway Segments

General Information		**Location Information**	
Analyst		Roadway	
Agency or Company		Roadway Section	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	**Base Conditions**	**Site Conditions**
Road type (2U, 3T, 4U, 4D, 5T)	-	4D
Length of segment, L (mi)	-	0.75
AADT (veh/day)	-	23,000
Type of on-street parking (none/parallel/angle)	none	None
Proportion of curb length with on-street parking	-	N/A
Median width (ft)	15	40
Lighting (present / not present)	not present	present
Auto speed enforcement (present/not present)	not present	not present
Major commercial driveways (number)	-	1
Minor commercial driveways (number)	-	4
Major industrial/institutional driveways (number)	-	-
Minor industrial/institutional driveways (number)	-	1
Major residential driveways (number)	-	1
Minor residential driveways (number)	-	1
Other driveways (number)	-	-
Speed Category	-	Low (30mph)
Roadside fixed object density (fixed objects/mi)	not present	20
Offset to roadside fixed objects (ft)	not present	12
Calibration Factor, C_r	1.0	1.0

Worksheet 1B – Accident Modification Factors for Urban and Suburban Roadway Segments

In Step 10 of the predictive method, Accident Modification Factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 12.7 presents the tables and equations necessary for determining the AMF values. Once the value for each AMF has been determined, all of the AMFs are multiplied together in Column 6 of Worksheet 1B which indicates the combined AMF value.

Worksheet 1B – Accident Modification Factors for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
AMF for On-Street Parking	AMF for Roadside Fixed Objects	AMF for Median Width	AMF for Lighting	AMF for Auto Speed Enforcement	Combined AMF
AMF_{1r}	AMF_{2r}	AMF_{3r}	AMF_{4r}	AMF_{5r}	AMF_{COMB}
from Equation 12-32	from Equation 12-33	from Exhibit 12-39	from Equation 12-34	from Section 12.7.1	(1)*(2)*(3)*(4)*(5)
1.00	1.02	0.97	0.91	1.00	0.90

Worksheet 1C – Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Roadway Segments

The SPF for multiple-vehicle nondriveway collisions along the roadway segment in Sample Problem 1 is calculated using Equation 12-10 and entered into Column 4 of Worksheet 1C. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 2 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values(from Column 4) to assure that fatal and injury (FI) and property damage only (PDO) crashes sum to the total crashes, as illustrated in Column 6. Column 7 represents the combined AMF (from Column 6 in Worksheet 1B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of multiple-vehicle nondriveway crashes using the values in Column 6, the combined AMF in Column 7, and the calibration factor in Column 8.

Worksheet 1C – Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)
Crash severity level	**SPF Coefficients**		**Overdispersion Parameter, k**	**Initial N_{brmv}**	**Proportion of total crashes**	**Adjusted N_{brmv}**	**Combined AMFs**	**Calibration factor**	**Predicted N_{brmv}**
	from Exhibit 12-5		from Exhibit 12-5	from Equation 12-10		$(4)_{TOTAL}*(5)$	(6) from Worksheet 1B	C_r	(6)*(7)*(8)
	a	b							
Total	-12.34	1.36	1.32	2.804	1.000	2.804	0.90	1.00	2.524
Fatal and injury (FI)	-12.76	1.28	1.31	0.825	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ 0.278	0.780	0.90	1.00	0.702
Property damage only (PDO)	-12.81	1.38	1.34	2.143	$(5)_{TOTAL}-(5)_{FI}$ 0.722	2.024	0.90	1.00	1.822

Worksheet 1D –Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Roadway Segments

Worksheet 1D presents the default proportions for collision type (from Exhibit 12-7) by crash severity level as follows:

- Fatal and injury crashes (Column 2)
- Property damage only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for multiple-vehicle nondriveway crashes by collision type is presented in Columns 3 (Fatal and Injury, FI), 5 (Property Damage Only, PDO), and 6 (Total).

These proportions may be used to separate the predicted average crash frequency for multiple-vehicle nondriveway crashes (from Column 9, Worksheet 1C) into components by crash severity and collision type.

Worksheet 1D – Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
Collision type	**Proportion of Collision Type $_{(FI)}$**	**Predicted $N_{brmv\ (FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**Predicted $N_{brmv\ (PDO)}$ (crashes/year)**	**Predicted $N_{brmv\ (TOTAL)}$ (crashes/year)**
	from Exhibit 12-7	$(9)_{FI}$ from Worksheet 1C	from Exhibit 12-7	$(9)_{PDO}$ from Worksheet 1C	$(9)_{TOTAL}$ from Worksheet 1C
Total	1.000	0.702	1.000	1.822	2.524
		$(2)*(3)_{FI}$		$(4)*(5)_{PDO}$	(3)+(5)
Rear-end collision	0.832	0.584	0.662	1.206	1.790
Head-on collision	0.020	0.014	0.007	0.013	0.027
Angle collision	0.040	0.028	0.036	0.066	0.094
Sideswipe, same direction	0.050	0.035	0.223	0.406	0.441
Sideswipe, opposite direction	0.010	0.007	0.001	0.002	0.009
Other multiple-vehicle collision	0.048	0.034	0.071	0.129	0.163

Worksheet 1E – Single-Vehicle Crashes by Severity Level for Urban and Suburban Roadway Segments

The SPF for single-vehicle crashes along the roadway segment in Sample Problem 1 is calculated using Equation 12-13 and entered into Column 4 of Worksheet 1E. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 2 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values (from Column 4) to assure that fatal and injury (FI) and property damage only (PDO) crashes sum to the total crashes, as illustrated in Column 6. Column 7 represents the combined AMF (from Column 6 in Worksheet 1B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of multiple-vehicle nondriveway crashes using the values in Column 6, the combined AMF in Column 7, and the calibration factor in Column 8.

Worksheet 1E – Single-Vehicle Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)
Crash severity level	**SPF Coefficients**		**Overdispersion Parameter, k**	**Initial N_{brsv}**	**Proportion of total crashes**	**Adjusted N_{brsv}**	**Combined AMFs**	**Calibration factor**	**Predicted N_{brsv}**
	from Exhibit 12-8		from Exhibit 12-8	from Equation 12-13		$(4)_{TOTAL}$*(5)	(6) from Worksheet 1B	C_r	(6)*(7)*(8)
	a	b							
Total	-5.05	0.47	0.86	0.539	1.000	0.539	0.90	1.00	0.485
Fatal and injury (FI)	-8.71	0.66	0.28	0.094	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ 0.174	0.094	0.90	1.00	0.085
Property damage only (PDO)	-5.04	0.45	1.06	0.446	$(5)_{TOTAL}-(5)_{FI}$ 0.826	0.445	0.90	1.00	0.401

Worksheet 1F –Single-Vehicle Crashes by Collision Type for Urban and Suburban Roadway Segments

Worksheet 1F presents the default proportions for collision type (from Exhibit 12-8) by crash severity level as follows:

- Fatal and injury crashes (Column 2)
- Property damage only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for single-vehicle crashes by collision type is presented in 3 (Fatal and Injury, FI), 5 (Property Damage Only, PDO), and Columns 6 (Total).

These proportions may be used to separate the predicted average crash frequency for single-vehicle crashes (from Column 9, Worksheet 1E) into components by crash severity and collision type.

Worksheet 1F – Single-Vehicle Collisions by Collision Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
Collision type	**Proportion of Collision Type $_{(FI)}$**	**Predicted $N_{brsv\ (FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**Predicted $N_{brsv\ (PDO)}$ (crashes/year)**	**Predicted $N_{brsv\ (TOTAL)}$ (crashes/year)**
	from Exhibit 12-10	$(9)_{FI}$ from Worksheet 1E	from Exhibit 12-10	$(9)_{PDO}$ from Worksheet 1E	$(9)_{TOTAL}$ from Worksheet 1E
Total	1.000	0.085	1.000	0.401	0.485
		$(2)*(3)_{FI}$		$(4)*(5)_{PDO}$	(3)+(5)
Collision with animal	0.001	0.000	0.063	0.025	0.025
Collision with fixed object	0.500	0.043	0.813	0.326	0.369
Collision with other object	0.028	0.002	0.016	0.006	0.008
Other single-vehicle collision	0.471	0.040	0.108	0.043	0.083

Worksheet 1G – Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Roadway Segments

Worksheet 1G determines and presents the number of driveway-related multiple-vehicle collisions. The number of driveways along both sides of the road is entered in Column 2 by driveway type (Column 1). The associated number of crashes per driveway per year by driveway type as found in Exhibit 12-11 is entered in Column 3. Column 4 contains the regression coefficient for AADT also found in Exhibit 12-11. The initial average crash frequency of multiple-vehicle driveway-related crashes is calculated from Equation 12-16 and entered into Column 5. The overdispersion parameter from Exhibit 12-11 is entered into Column 6; however, the overdispersion parameter is not needed for Sample Problem 2 (as the EB Method is not utilized).

Worksheet 1G – Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
Driveway type	**Number of driveways, n_j**	**Crashes per driveway per year, N_j**	**Coefficient for traffic adjustment, t**	**Initial N_{brdwy}**	**Overdispersion parameter, k**
		from Exhibit 12-11	from Exhibit 12-11	Equation 12-16 $n_j * N_j * (AADT/15,000)^t$	from Exhibit 12-11
Major commercial	1	0.033	1.106	0.053	
Minor commercial	4	0.011	1.106	0.071	
Major industrial/institutional	0	0.036	1.106	0.000	
Minor industrial/institutional	1	0.005	1.106	0.008	-
Major residential	1	0.018	1.106	0.029	
Minor residential	1	0.003	1.106	0.005	
Other	0	0.005	1.106	0.000	
Total	-	-	-	0.166	1.39

Worksheet 1H – Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Roadway Segments

The initial average crash frequency of multiple-vehicle driveway-related crashes from Column 5 of Worksheet 1G is entered in Column 2. This value is multiplied by the proportion of crashes by severity (Column 3) found in Exhibit 12-11 and the adjusted value is entered into Column 4. Column 5 represents the combined AMF (from Column 6 in Worksheet 1B), and Column 6 represents the calibration factor. Column 7 calculates the predicted average crash frequency of multiple-vehicle driveway-related crashes using the values in Column 4, the combined AMF in Column 5, and the calibration factor in Column 6.

Worksheet 1H – Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Crash severity level	**Initial N_{brdwy}**	**Proportion of total accidents (f_{dwy})**	**Adjusted N_{brdwy}**	**Combined AMFs**	**Calibration factor, C_r**	**Predicted N_{brdwy}**
	$(5)_{TOTAL}$ from Worksheet 1G	from Exhibit 12-11	$(2)_{TOTAL}$ *(3)	(6) from Worksheet 1B		(4)*(5)*(6)
Total	0.166	1.000	0.166	0.90	1.00	0.149
Fatal and injury (FI)	-	0.284	0.047	0.90	1.00	0.042
Property damage only (PDO)	-	0.716	0.119	0.90	1.00	0.107

Worksheet 1I– Vehicle-Pedestrian Collisions for Urban and Suburban Roadway Segments

The predicted average crash frequency of multiple-vehicle nondriveway, single-vehicle and multiple-vehicle driveway-related predicted crashes from Worksheets 1C, 1E, and 1H are entered into Columns 2, 3, and 4, respectively. These values are summed in Column 5. Column 6 contains the pedestrian accident adjustment factor (see Exhibit 12-17). Column 7 represents the calibration factor. The predicted average crash frequency of vehicle-pedestrian collisions (Column 8) is the product of Columns 5, 6 and 7. Since all vehicle-pedestrian crashes are assumed to involve some level of injury, there are no property damage only crashes.

Worksheet 1I – Vehicle-Pedestrian Collisions

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crash severity level	**Predicted N_{brmv}**	**Predicted N_{brsv}**	**Predicted N_{brdwy}**	**Predicted N_{br}**	**f_{pedr}**	**Calibration factor, C_r**	**Predicted N_{pedr}**
	(9) from Worksheet 1C	(9) from Worksheet 1E	(7) from Worksheet 1H	(2)+(3)+(4)	from Exhibit 12-17		(5)*(6)*(7)
Total	2.524	0.485	0.149	3.158	0.067	1.000	0.212
Fatal and injury (FI)	-	-	-	-	-	1.00	0.212

Worksheet 1J – Vehicle-Bicycle Collisions for Urban and Suburban Roadway Segments

The predicted average crash frequency of multiple-vehicle nondriveway, single-vehicle and multiple-vehicle driveway-related predicted crashes from Worksheets 1C, 1E, and 1H are entered into Columns 2, 3, and 4, respectively. These values are summed in Column 5. Column 6 contains the bicycle accident adjustment factor (see Exhibit 12-18). Column 7 represents the calibration factor. The predicted average crash frequency of vehicle-bicycle collisions (Column 8) is the product of Columns 5, 6 and 7. Since all vehicle-bicycle collisions are assumed to involve some level of injury, there are no property damage only crashes.

Worksheet 1J – Vehicle-Bicycle Collisions for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crash severity level	**Predicted N_{brmv}**	**Predicted N_{brsv}**	**Predicted N_{brdwy}**	**Predicted N_{br}**	**f_{biker}**	**Calibration factor, C_r**	**Predicted N_{biker}**
	(9) from Worksheet 1C	(9) from Worksheet 1E	(7) from Worksheet 1H	(2)+(3)+(4)	from Exhibit 12-18		(5)*(6)*(7)
Total	2.524	0.485	0.149	3.158	0.013	1.00	0.041
Fatal and injury	-	-	-	-	-	1.00	0.041

Worksheet 1K – Crash Severity Distribution for Urban and Suburban Roadway Segments

Worksheet 1K provides a summary of all collision types by severity level. Values from Worksheets 1C, 1E, 1H, 1I, and 1J are presented and summed to provide the predicted average crash frequency for each severity level as follows:

- Fatal and injury crashes (Column 2)
- Property damage only crashes (Column 3)
- Total crashes (Column 4)

Worksheet 1K – Crash Severity Distribution for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)
Collision type	**Fatal and injury (FI)**	**Property damage only (PDO)**	**Total**
	(3) from Worksheet 1D and 1F; (7) from Worksheet 1H; and (8) from Worksheet 1I and 1J	(5) from Worksheet 1D and 1F; and (7) from Worksheet 1H;	(6) from Worksheet 1D and 1F; (7) from Worksheet 1H; and (8) from Worksheet 1I and 1J
MULTIPLE-VEHICLE			
Rear-end collisions (from Worksheet 1D)	0.584	1.206	1.790
Head-on collisions (from Worksheet 1D)	0.014	0.013	0.027
Angle collisions (from Worksheet 1D)	0.028	0.066	0.094
Sideswipe, same direction (from Worksheet 1D)	0.035	0.406	0.441
Sideswipe, opposite direction (from Worksheet 1D)	0.007	0.002	0.009
Driveway-related collisions (from Worksheet 1H)	0.042	0.107	0.149
Other multiple-vehicle collision (from Worksheet 1D)	0.034	0.129	0.163
Subtotal	0.744	1.929	2.673
SINGLE-VEHICLE			
Collision with animal (from Worksheet 1F)	0.000	0.025	0.025
Collision with fixed object (from Worksheet 1F)	0.043	0.326	0.369
Collision with other object (from Worksheet 1F)	0.002	0.006	0.008
Other single-vehicle collision (from Worksheet 1F)	0.040	0.043	0.083
Collision with pedestrian (from Worksheet 1I)	0.212	0.000	0.212
Collision with bicycle (from Worksheet 1J)	0.041	0.000	0.041
Subtotal	0.338	0.400	0.738
Total	1.082	2.329	3.411

Worksheet 1L– Summary Results for Urban and Suburban Roadway Segments

Worksheet 1L presents a summary of the results. Using the roadway segment length and the AADT, the worksheet presents the crash rate in miles per year (Column 4) and in million vehicle miles (Column 6).

Worksheet 1L – Summary Results for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)
Crash severity level	**Predicted average crash frequency, $N_{predicted\ rs}$ (crashes/year)**	**Roadway segment length, L (mi)**	**Crash rate (crashes/mi/year)**
	(Total) from Worksheet 1K		(2)/(3)
Total	3.411	0.75	4.5
Fatal and injury (FI)	1.082	0.75	1.4
Property damage only (PDO)	2.329	0.75	3.1

12.13.3. Sample Problem 3

The Site/Facility

A three-leg stop-controlled intersection located on an urban arterial.

The Question

What is the predicted accident frequency of the unsignalized intersection for a particular year?

The Facts

- 1 left-turn lane on one major road approach
- No right-turn lanes on any approach
- AADT of major road is 14,000 veh/day
- AADT of minor road is 4,000 veh/day

Assumptions

- Collision type distributions used are the default values from Exhibits 12-24 and 12-30 and Equations 12-30 and 12-31.
- The calibration factor is assumed to be 1.00.

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the unsignalized intersection in Sample Problem 3 is determined to be 1.6 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8

To determine the predicted average crash frequency of the roadway segment in Sample Problem 3, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year, and the EB Method is not applied.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type, and traffic control features.

For a three-leg stop-controlled intersection, SPF values for multiple-vehicle, single-vehicle, vehicle-pedestrian and vehicle-bicycle collisions are determined. The calculations for vehicle-pedestrian and vehicle-bicycle collisions are shown in Step 10 since the AMF values are needed for these two models.

Multiple-Vehicle Crashes

The SPF for multiple-vehicle collisions for a single three-leg stop-controlled intersection is calculated from Equation 12-21 and Exhibit 12-19 as follows:

$$N_{bimv} = exp\ (a + b \times ln(AADT_{maj}\ ) + c \times ln(AADT_{min}\ ))$$

$$N_{bimv(TOTAL)} = exp\ (-13.63 + 1.11 \times ln(14{,}000) + 0.41 \times ln(4{,}000))$$

$$= 1.892 \text{ crashes/year}$$

$$N_{bimv\ (FI)} = exp\ (-14.01 + 1.16 \times ln(14{,}000) + 0.30 \times ln(4{,}000))$$

$$= 0.639 \text{ crashes/year}$$

$$N_{bimv\ (PDO)} = exp\ (-15.38 + 1.20 \times ln(14{,}000) + 0.51 \times ln(4{,}000))$$

$$= 1.358 \text{ crashes/year}$$

These initial values for fatal and injury (FI) and property damage only (PDO) crashes are then adjusted using Equations 12-22 and 12-23 to assure that they sum to the value for total crashes as follows:

$$N_{bimv(FI)} = N_{bimv(TOTAL)} \times \left(\frac{N'_{bimv(FI)}}{N'_{bimv(FI)} + N'_{bimv(PDO)}} \right)$$

$$= 1.892 \times \left(\frac{0.639}{0.639 + 1.358} \right)$$

$$= 0.605 \text{ crashes/year}$$

$$N_{bimv(PDO)} = N_{bimv(TOTAL)} - N_{bimv(FI)}$$

$$= 1.892 - 0.605$$

$$= 1.287 \text{ crashes/year}$$

Single-Vehicle Crashes

The SPF for single-vehicle crashes for a single three-leg stop-controlled intersection is calculated from Equation 12-24 and Exhibit 12-25 as follows:

$$N_{bisv} = exp\ (a + b \times ln(AADT_{maj}\ ) + c \times ln(AADT_{min}\ ))$$

$$N_{bisv(TOTAL)} = exp\ (-6.81 + 0.16 \times ln(14{,}000) + 0.51 \times ln(4{,}000))$$

$$= 0.349 \text{ crashes/year}$$

$$N_{bisv(PDO)} = exp\ (-8.36 + 0.25 \times ln(14{,}000) + 0.55 \times ln(4{,}000))$$

$$= 0.244 \text{ crashes/year}$$

Since there are no models for fatal and injury crashes at a three-leg stop-controlled intersections, $N_{bisv(FI)}$ is calculated using Equation 12-27 (in place of Equation 12-25), and the initial value for $N_{bisv(PDO)}$ calculated above is then adjusted using Equation 12-26 to assure that fatal-and-injury and property-damage-only crashes sum to the value for total crashes as follows:

$$N_{bisv(FI)} = N_{bisv(TOTAL)} \times f_{bisv}$$

For a three-leg stop-controlled intersection, the default proportion of fatal-and-injury crashes, f_{bisv}= 0.31 (see Section 12.6.2, Single-Vehicle Crashes)

$$N_{bisv(FI)} = 0.349 \times 0.31$$

$$= 0.108 \text{ crashes/year}$$

$$N_{bisv(PDO)} = N_{bisv(TOTAL)} - N_{bisv(FI)}$$

$$= 0.349 - 0.108$$

$$= 0.241 \text{ crashes/year}$$

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust base conditions to site specific geometric design and traffic control features

Each AMF used in the calculation of the predicted average crash frequency of the intersection is calculated below:

Intersection Left-Turn Lanes (AMF_{1i})

From Exhibit 12-41, for a three-leg stop-controlled intersection with one left-turn lane on the major road, AMF_{1i} = 0.67.

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

For unsignalized intersections, AMF_{2i} = 1.00.

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

Since no right-turn lanes are present, AMF_{3i} is 1.00 (i.e. the base condition for AMF_{3i} is the absent of right-turn lanes on the intersection approaches).

Right Turn on Red (AMF_{4i})

For unsignalized intersections, AMF_{4i} = 1.00.

Lighting (AMF_{5i})

Since there is no lighting at this intersection, AMF_{5i} is 1.00 (i.e. the base condition for AMF_{5i} is the absence of intersection lighting).

Red Light Cameras (AMF_{6i})

For unsignalized intersections, AMF_{6i} is always 1.00.

The combined AMF value for Sample Problem 3 is 0.67.

Vehicle-Pedestrian and Vehicle-Bicycle Collisions

The predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle collisions) for SPF base conditions, N_{bi}, must be calculated in order to determine vehicle-pedestrian and vehicle-bicycle crashes. N_{bi} is determined from Equation 12-6 as follows:

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

From Equation 12-7, $N_{spf\ int}$ can be calculated as follows:

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

$$= 1.892 + 0.349$$

$$= 2.241 \text{ crashes/year}$$

The combined AMF value for Sample Problem 3 is 0.67.

$$N_{bi} = 2.241 \times (0.67)$$

$$= 1.501 \text{ crashes/year}$$

The SPF for vehicle-pedestrian collisions for a three-leg stop-controlled intersection is calculated from Equation 12-30 as follows:

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

From Exhibit 12-33, for a three-leg stop-controlled intersection the pedestrian accident adjustment factor, f_{pedi} = 0.211.

$$N_{pedi} = 1.501 \times 0.021$$

$$= 0.032 \text{ crashes/year}$$

The SPF for vehicle-bicycle collisions is calculated from Equation 12-31 as follows:

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

From Exhibit 12-34, for a three-leg stop-controlled intersection the bicycle accident adjustment factor, f_{bikei} = 0.016.

$$N_{bikei} = 1.501 \times 0.016$$

$$= 0.024 \text{ crashes/year}$$

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed in Sample Problem 3 that a calibration factor, C_i, of 1.00 has been determined for local conditions. See *Part C* Appendix A.1 for further discussion on calibration of the predicted models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated using Equation 12-5 based on results obtained in Steps 9 through 11 as follows:

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

$$= 1.00 \times (1.501 + 0.032 + 0.024)$$

$$= 1.557 \text{ crashes/year}$$

Worksheets

The step-by-step instructions above are provided to illustrate the predictive method for calculating the predicted average crash frequency for an intersection. To apply the predictive method steps to multiple intersections, a series of twelve worksheets are provided for determining the predicted average crash frequency at intersections. The twelve worksheets include:

- Worksheet 2A – General Information and Input Data for Urban and Suburban Arterial Intersections
- Worksheet 2B – Accident Modification Factors for Urban and Suburban Arterial Intersections
- Worksheet 2C – Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections
- Worksheet 2D – Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections
- Worksheet 2E – Single-Vehicle Crashes by Severity Level for Urban and Suburban Arterial Intersections
- Worksheet 2F – Single-Vehicle Crashes by Collision Type for Urban and Suburban Arterial Intersections
- Worksheet 2G – Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Stop-Controlled Intersections
- Worksheet 2H – Accident Modification Factors for Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections
- Worksheet 2I – Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections
- Worksheet 2J – Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections
- Worksheet 2K – Crash Severity Distribution for Urban and Suburban Arterial Intersections
- Worksheet 2L – Summary Results for Urban and Suburban Arterial Intersections

Details of these worksheets are provided below, except for Worksheets 2H and 2I which are only used for signalized intersections. Blank versions of worksheets used in the Sample Problems are provided in Chapter 12 Appendix A.

Worksheet 2A – General Information and Input Data for Urban and Suburban Arterial Intersections

Worksheet 2A is a summary of general information about the intersection, analysis, input data (i.e., "The Facts") and assumptions for Sample Problem 3.

Worksheet 2A – General Information and Input Data for Urban and Suburban Arterial Intersections

General Information		**Location Information**	
Analyst		Roadway	
Agency or Company		Intersection	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	**Base Conditions**	**Site Conditions**
Intersection type (3ST, 3SG, 4ST, 4SG)	-	3ST
$AADT_{major}$ (veh/day)	-	14,000
$AADT_{minor}$ (veh/day)	-	4,000
Intersection lighting (present/not present)	not present	not present
Calibration factor, C_i	1.00	1.00
Data for unsignalized intersections only:	-	-
Number of major-road approaches with left-turn lanes (0,1,2)	0	1
Number of major-road approaches with right-turn lanes (0,1,2)	0	0
Data for signalized intersections only:	-	-
Number of approaches with left-turn lanes (0,1,2,3,4)	0	N/A
Number of approaches with right-turn lanes (0,1,2,3,4)	0	N/A
Number of approaches with left-turn signal phasing	-	N/A
Type of left-turn signal phasing	permissive	N/A
Intersection red light cameras (present/not present)	not present	N/A
Sum of all pedestrian crossing volumes (PedVol)	-	N/A
Maximum number of lanes crossed by a pedestrian (n_{lanesx})	-	N/A
Number of bus stops within 300 m (1,000 ft) of the intersection	0	N/A
Schools within 300 m (1,000 ft) of the intersection (present/not present)	not present	N/A
Number of alcohol sales establishments within 300 m (1,000 ft) of the intersection	0	N/A

Worksheet 2B – Accident Modification Factors for Urban and Suburban Arterial Intersections

In Step 10 of the predictive method, Accident Modification Factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 12.7 presents the tables and equations necessary for determining the AMF values. Once the value for each AMF has been determined, all of the AMFs are multiplied together in Column 7 of Worksheet 2B which indicates the combined AMF value.

Worksheet 2B – Accident Modification Factors for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
AMF for Left-Turn Lanes	AMF for Left-Turn Signal Phasing	AMF for Right-Turn Lanes	AMF for Right Turn on Red	AMF for Lighting	AMF for Red Light Cameras	Combined AMF
AMF_{1i}	AMF_{2i}	AMF_{3i}	AMF_{4i}	AMF_{5i}	AMF_{6i}	AMF_{COMB}
from Exhibit 12-41	from Exhibit 12-42	from Exhibit 12-43	from Equation 12-35	from Equation 12-36	from Equation 12-37	(1)*(2)*(3)*(4)*(5)*(6)
0.67	1.00	1.00	1.00	1.00	1.00	0.67

Worksheet 2C – Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

The SPF for multiple-vehicle collisions at the intersection in Sample Problem 3 is calculated using Equation 12-22 and entered into Column 4 of Worksheet 2C. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 3 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values(from Column 4) to assure that fatal and injury (FI) and property damage only (PDO) crashes sum to the total crashes, as illustrated in Column 6. Column 7 represents the combined AMF (from Column 7 in Worksheet 2B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of multiple-vehicle crashes using the values in Column 6, the combined AMF in Column 7, and the calibration factor in Column 8.

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)
Crash severity Level	**SPF Coefficients**			**Overdispersion Parameter, k**	**Initial N_{bimv}**	**Proportion of total crashes**	**Adjusted N_{bimv}**	**Combined AMFs**	**Calibration Factor, C_i**	**Predicted N_{bimv}**
	from Exhibit 12-19			from Exhibit 12-19	from Equation 12-22		$(4)_{TOTAL}*(5)$	(7) from Worksheet 2B		(6)*(7)*(8)
	a	b	c							
Total	-13.36	1.11	0.41	0.80	1.892	1.000	1.892	0.67	1.00	1.268
Fatal and injury (FI)	-14.01	1.16	0.30	0.69	0.639	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ 0.320	0.605	0.67	1.00	0.405
Property damage only (PDO)	-15.38	1.20	0.51	0.77	1.358	$(5)_{TOTAL}-(5)_{FI}$ 0.680	1.287	0.67	1.00	0.862

Table: Worksheet 2C – Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

Worksheet 2D –Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

Worksheet 2D presents the default proportions for collision type (from Exhibit 12-24) by crash severity level as follows:

- Fatal and injury crashes (Column 2)
- Property damage only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for multiple-vehicle crashes by collision type is presented in Columns 3 (Fatal and Injury, FI), 5 (Property Damage Only PDO), and 6 (Total).

These proportions may be used to separate the predicted average crash frequency for multiple-vehicle crashes (from Column 9, Worksheet 2C) into components by crash severity and collision type.

Worksheet 2D – Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
Collision Type	**Proportion of Collision Type $_{(FI)}$**	**Predicted $N_{bimv\,(FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**Predicted $N_{bimv\,(PDO)}$ (crashes/year)**	**Predicted $N_{bimv\,(TOTAL)}$ (crashes/year)**
	from Exhibit 12-24	$(9)_{FI}$ from Worksheet 2C	from Exhibit 12-24	$(9)_{PDO}$ from Worksheet 2C	$(9)_{PDO}$ from Worksheet 2C
Total	1.000	0.405	1.000	0.862	1.268
		$(2)*(3)_{FI}$		$(4)*(5)_{PDO}$	(3)+(5)
Rear-end collision	0.421	0.171	0.440	0.379	0.550
Head-on collision	0.045	0.018	0.023	0.020	0.038
Angle collision	0.343	0.139	0.262	0.226	0.365
Sideswipe	0.126	0.051	0.040	0.034	0.085
Other multiple-vehicle collision	0.065	0.026	0.235	0.203	0.229

Worksheet 2E– Single-Vehicle Crashes by Severity Level for Urban and Suburban Arterial Intersections

The SPF for single-vehicle crashes at the intersection in Sample Problem 3 is calculated using Equation 12-25 for total and property damage only (PDO) crashes and entered into Column 4 of Worksheet 2E. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2, and 3; however, the overdispersion parameter is not needed for Sample Problem 3 (as the EB Method is not utilized). Since there are no models for fatal and injury crashes at a three-leg stop-controlled intersections, $N_{bisv(FI)}$ is calculated using Equation 12-27 (in place of Equation 12-25), and the value is entered into Column 4 and 6 since no further adjustment is required. Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values (from Column 4) to assure that fatal and injury (FI) and property damage only (PDO) crashes sum to the total crashes, as illustrated in Column 6. Column 7 represents the combined AMF (from Column 7 in Worksheet 2B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of single-vehicle crashes using the values in Column 6, the combined AMF in Column 7, and the calibration factor in Column 8.

Worksheet 2E – Single-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)
Crash severity Level	**SPF Coefficients**			**Overdispersion Parameter, k**	**Initial N_{bisv}**	**Proportion of total crashes**	**Adjusted N_{bisv}**	**Combined AMFs**	**Calibration Factor, C_i**	**Predicted N_{bisv}**
	from Exhibit 12-25			from Exhibit 12-25	from Equation 12-25; (FI) from Equation 12-25 or 12-27		$(4)_{TOTAL}$*(5)	(7) from Worksheet 2B		(6)*(7)*(8)
	a	b	c							
Total	-6.81	0.16	0.51	1.14	0.349	1.000	0.349	0.67	1.00	0.234
Fatal and injury (FI)	N/A	N/A	N/A	N/A	0.108	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ N/A	0.108	0.67	1.00	0.072
Property damage only (PDO)	-8.36	0.25	0.55	1.29	0.244	$(5)_{TOTAL}-(5)_{FI}$ 0.693	0.242	0.67	1.00	0.162

Worksheet 2F –Single-Vehicle Crashes by Collision Type for Urban and Suburban Arterial Intersections

Worksheet 2F presents the default proportions for collision type (from Exhibit 12-30) by crash severity level as follows:

- Fatal and injury crashes (Column 2)
- Property damage only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for single-vehicle crashes by collision type is presented in Columns 3 (Fatal and Injury, FI), 5 (Property Damage Only PDO), and 6 (Total).

These proportions may be used to separate the predicted average crash frequency for single-vehicle crashes (from Column 9, Worksheet 2E) into components by crash severity and collision type.

Worksheet 2F – Single-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
Collision Type	**Proportion of Collision Type $_{(FI)}$**	**Predicted $N_{bisv\,(FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**Predicted $N_{bisv\,(PDO)}$ (crashes/year)**	**Predicted $N_{bisv\,(TOTAL)}$ (crashes/year)**
	Exhibit 12-30	$(9)_{FI}$ from Worksheet 2E	Exhibit 12-30	$(9)_{PDO}$ from Worksheet 2E	$(9)_{PDO}$ from Worksheet 2E
Total	1.000	0.072	1.000	0.162	0.234
		$(2)*(3)_{FI}$		$(4)*(5)_{PDO}$	(3)+(5)
Collision with parked vehicle	0.001	0.000	0.003	0.000	0.000
Collision with animal	0.003	0.000	0.018	0.003	0.003
Collision with fixed object	0.762	0.055	0.834	0.135	0.190
Collision with other object	0.090	0.006	0.092	0.015	0.021
Other single-vehicle collision	0.039	0.003	0.023	0.004	0.007
Single-vehicle noncollision	0.105	0.008	0.030	0.005	0.013

Worksheet 2G– Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Stop-Controlled Intersections

The predicted average crash frequency of multiple-vehicle predicted crashes and single-vehicle predicted crashes from Worksheets 2C and 2E are entered into Columns 2 and 3 respectively. These values are summed in Column 4. Column 5 contains the pedestrian accident adjustment factor (see Exhibit 12-33). Column 6 presents the calibration factor. The predicted average crash frequency of vehicle-pedestrian collision (Column 7) is the product of Columns 4, 5 and 6. Since all vehicle-pedestrian crashes are assumed to involve some level of injury, there are no property damage only crashes.

Worksheet 2G – Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Stop-Controlled Intersections

(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**
Crash severity level	**Predicted N_{bimv}**	**Predicted N_{bisv}**	**Predicted N_{bi}**	**f_{pedi}**	**Calibration factor, C_i**	**Predicted N_{pedi}**
	(9) from Worksheet 2C	(9) from Worksheet 2E	(2)+(3)	from Exhibit 12-33		(4)*(5)*(6)
Total	1.268	0.234	1.502	0.021	1.00	0.032
Fatal and injury (FI)	-	-	-	-	1.00	0.032

Worksheet 2J– Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections

The predicted average crash frequency of multiple-vehicle predicted crashes and single-vehicle predicted crashes from Worksheets 2C and 2E are entered into Columns 2 and 3 respectively. These values are summed in Column 4. Column 5 contains the bicycle accident adjustment factor (see Exhibit 12-34). Column 6 presents the calibration factor. The predicted average crash frequency of vehicle-bicycle collision (Column 7) is the product of Columns 4, 5 and 6. Since all vehicle-bicycle crashes are assumed to involve some level of injury, there are no property damage only crashes.

Worksheet 2J – Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Crash severity level	**Predicted N_{bimv}**	**Predicted N_{bisv}**	**Predicted N_{bi}**	**f_{bikei}**	**Calibration factor, C_i**	**Predicted N_{pedi}**
	(9) from Worksheet 2C	(9) from Worksheet 2E	(2)+(3)	from Exhibit 10-34		(4)*(5)*(6)
Total	1.268	0.234	1.502	0.016	1.000	0.024
Fatal and injury (FI)	-	-	-	-	1.000	0.024

Worksheet 2K– Crash Severity Distribution for Urban and Suburban Arterial Intersections

Worksheet 2K provides a summary of all collision types by severity level. Values from Worksheets 2D, 2F, 2G and 2J are presented and summed to provide the predicted average crash frequency for each severity level as follows:

- Fatal and injury crashes (Column 2)
- Property damage only crashes (Column 3)
- Total crashes (Column 4)

Worksheet 2K– Crash Severity Distribution for Urban and Suburban Arterial Intersections			
(1)	(2)	(3)	(4)
Collision type	**Fatal and injury (FI)**	**Property damage only (PDO)**	**Total**
	(3) from Worksheet 2D and 2F; (7) from 2G or 2I and 2J	(5) from Worksheet 2D and 2F	(6) from Worksheet 2D and 2F; (7) from 2G or 2I and 2J
MULTIPLE-VEHICLE COLLISIONS			
Rear-end collisions (from Worksheet 2D)	0.171	0.379	0.550
Head-on collisions (from Worksheet 2D)	0.018	0.020	0.038
Angle collisions (from Worksheet 2D)	0.139	0.226	0.365
Sideswipe (from Worksheet 2D)	0.051	0.034	0.085
Other multiple-vehicle collision (from Worksheet 2D)	0.026	0.203	0.229
Subtotal	0.405	0.862	1.267
SINGLE-VEHICLE COLLISIONS			
Collision with parked vehicle (from Worksheet 2F)	0.000	0.000	0.000
Collision with animal (from Worksheet 2F)	0.000	0.003	0.003
Collision with fixed object (from Worksheet 2F)	0.055	0.135	0.190
Collision with other object (from Worksheet 2F)	0.006	0.015	0.021
Other single-vehicle collision (from Worksheet 2F)	0.003	0.004	0.007
Single-vehicle noncollision (from Worksheet 2F)	0.008	0.005	0.013
Collision with pedestrian (from Worksheet 2G or 2I)	0.032	0.000	0.032
Collision with bicycle (from Worksheet 2J)	0.024	0.000	0.024
Subtotal	0.128	0.162	0.290
Total	0.533	1.024	1.557

2336

Worksheet 2L– Summary Results for Urban and Suburban Arterial Intersections

Worksheet 2L presents a summary of the results.

Worksheet 2L – Summary Results for Urban and Suburban Arterial Intersections	
(1)	(2)
Crash severity level	**Predicted average crash frequency, $N_{predicted\ int}$ (crashes/year)**
	(Total) from Worksheet 2K
Total	1.557
Fatal and injury (FI)	0.533
Property damage only (PDO)	1.024

12.13.4. Sample Problem 4

The Intersection

A four-leg signalized intersection located on an urban arterial.

The Question

What is the predicted accident frequency of the signalized intersection for a particular year?

The Facts

- 1 left-turn lane on each of the two major road approaches
- 1 right-turn lane on each of the two major road approaches
- Protected/permissive left-turn signal phasing on major road
- AADT of major road is 15,000 veh/day
- AADT of minor road is 9,000 veh/day
- Lighting is present
- No approaches with prohibited RTOR
- Four-lane divided major road
- Two-lane undivided minor road
- Pedestrian volume is 1,500 peds/day
- The number of bus stops within 1,000 ft of intersection is 2
- A school is present within 1,000 ft of intersection
- The number of alcohol establishments within 1,000 ft of intersection is 6

Assumptions

- Collision type distributions used are the default values from Exhibits 12-24 and 12-30 and Equations 12-28 and 12-31.
- The calibration factor is assumed to be 1.00.
- The maximum number of lanes crossed by a pedestrian is assumed to be 4 (crossing two through lanes, one left turn lane, and one right turn lane across one side of the divided major road).

Results

Using the predictive method steps as outlined below, the predicted average crash frequency for the unsignalized intersection in Sample Problem 4 is determined to be 3.4 crashes per year (rounded to one decimal place).

Steps

Step 1 through 8

To determine the predicted average crash frequency of the roadway segment in Sample Problem 4, only Steps 9 through 11 are conducted. No other steps are necessary because only one roadway segment is analyzed for one year, and the EB Method is not applied.

Step 9 – For the selected site, determine and apply the appropriate Safety Performance Function (SPF) for the site's facility type, and traffic control features.

For a four-leg signalized intersection, SPF values for multiple-vehicle, single-vehicle, vehicle-pedestrian and vehicle-bicycle collisions are determined. The calculations for total multiple- and single-vehicle collisions are presented below. Detailed steps for calculating SPFs for fatal and injury (FI) and property damage only (PDO) crashes are presented in Sample Problem 3 (for fatal and injury base crashes at a four-leg signalized intersection Equation 12-25 in place of Equation 12-27 is used). The calculations for vehicle-pedestrian and vehicle-bicycle collisions are shown in Step 10 since the AMF values are needed for these two models.

Multiple-Vehicle Collisions

The SPF for multiple-vehicle collisions for a single four-leg signalized intersection is calculated from Equation 12-21 and Exhibit 12-19 as follows:

$$N_{bimv} = exp\ (a + b \times ln(AADT_{maj}\ ) + c \times ln(AADT_{min}))$$

$$N_{bimv\ (TOTAL)} = exp\ (-10.99 + 1.07 \times ln(15{,}000) + 0.23 \times ln(9{,}000))$$

$$= 4.027 \text{ crashes/year}$$

Single-Vehicle Crashes

The SPF for single-vehicle crashes for a single four-leg signalized intersection is calculated from Equation 12-24 and Exhibit 12-25 as follows:

$$N_{bisv} = exp\ (a + b \times ln(AADT_{maj}\ ) + c \times ln(AADT_{min}))$$

$$N_{bisv\ Total} = exp\ (-10.21 + 0.68 \times ln(15{,}000) + 0.27 \times ln(9{,}000))$$

$$= 0.297 \text{ crashes/year}$$

Step 10 – Multiply the result obtained in Step 9 by the appropriate AMFs to adjust base conditions to site specific geometric design and traffic control features

Each AMF used in the calculation of the predicted average crash frequency of the intersection is calculated below. AMF_{1i} through AMF_{2i} are applied to multiple-vehicle collisions and single-vehicle crashes, while AMF_{1p} through AMF_{3p} are applied to vehicle-pedestrian collisions.

Intersection Left-Turn Lanes (AMF_{1i})

From Exhibit 12-41, for a four-leg signalized intersection with one left-turn lane on each of two approaches, AMF_{1i} = 0.81.

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

From Exhibit 12-42, for a four-leg signalized intersection with protected/permissive left-turn signal phasing for two approaches, AMF_{2i} = 0.98 (0.99*0.99).

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

Form Exhibit 12-43, for a four-leg signalized intersection with one right-turn lane on each of two approaches, AMF_{3i} = 0.92.

Right Turn on Red (AMF_{4i})

Since RTOR is not prohibited on any of the intersection legs, AMF_{4i} = 1.00 (i.e. the base condition for AMF_{4i} is permitting a RTOR at all approaches to a signalized intersection).

Lighting (AMF_{5i})

AMF_{5i} is calculated from Equation 12-36.

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

From Exhibit 12-44, the proportion of crashes that occur at night, p_{ni} = 0.235.

$$AMF_{5i} = 1 - 0.38 \times 0.235$$

$$= 0.91$$

Red Light Cameras (AMF_{6i})

Since no red light cameras are present at this intersection, AMF_{6i} = 1.00 (i.e. the base condition for AMF_{6i} is the absence of red light cameras).

The combined AMF value applied to multiple- and single-vehicle crashes in Sample Problem 4 is calculated below.

$$AMF_{COMB} = 0.81 \times 0.98 \times 0.92 \times 0.91$$

$$= 0.66$$

Bus Stop (AMF_{1p})

From Exhibit 12-45, for two bus stops within 1,000-ft of the center of the intersection, AMF_{1p} = 2.78.

Schools (AMF_{2p})

From Exhibit 12-46, for one school within 1,000-ft of the center of the intersection, AMF_{2p} = 1.35.

Alcohol Sales Establishments (AMF_{3p})

From Exhibit 12-47, for six alcohol establishments within 1,000-ft of the center of the intersection, AMF_{3p} = 1.12.

Vehicle-Pedestrian and Vehicle-Bicycle Collisions

The SPF for vehicle-pedestrian collisions for a four-leg signalized intersection is calculated from Equation 12-28 as follows:

$$N_{pedi} = N_{pedbase} \times AMF_{1p} \times AMF_{2p} \times AMF_{3p}$$

$N_{pedbase}$ is calculated from Equation 12-29 using the coefficients from Exhibit 12-31.

$$N_{pedbase} = exp\ (a + b \times ln(AADT_{tot}) + c \times ln(\frac{AADT_{min}}{AADT_{maj}}) + d \times ln(PedVol) + e \times n_{lanesx})$$

$$= exp\ (-9.53 + 0.40 \times ln(24{,}000) + 0.26 \times ln(\frac{9{,}000}{15{,}000}) + 0.45 \times ln(1{,}500) + 0.04 \times 4)$$

$$= 0.113 \text{ crashes/year}$$

The AMF vehicle-pedestrian collision values calculated above, are AMF_{1p} =2.78, AMF_{2p} 1.35 and AMF_{3p} = 1.12.

$$N_{pedi} = 0.113 \times 2.78 \times 1.35 \times 1.12$$

$$= 0.475 \text{ crashes/year}$$

The predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle collisions) for SPF base conditions, N_{bi}, must be calculated in order to determine vehicle-bicycle crashes. N_{bi} is determined from Equation 12-6 as follows:

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

From Equation 12-7, $N_{spf\ int}$ can be calculated as follows:

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

$$= 4.027 + 0.297$$

$$= 4.324 \text{ crashes/year}$$

The combined AMF value for Sample Problem 4 is 0.66.

$$N_{bi} = 4.324 \times (0.66)$$

$$= 2.854 \text{ crashes/year}$$

The SPF for vehicle-bicycle collisions is calculated from Equation 12-31 as follows:

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

From Exhibit 12-34, for a four-leg signalized intersection the bicycle accident adjustment factor, f_{bikei} = 0.015.

$$N_{bikei} = 2.854 \times 0.015$$

$$= 0.043 \text{ crashes/year}$$

Step 11 – Multiply the result obtained in Step 10 by the appropriate calibration factor.

It is assumed in Sample Problem 4 that a calibration factor, C_i, of 1.00 has been determined for local conditions. See *Part C* Appendix A.1 for further discussion on calibration of the predicted models.

Calculation of Predicted Average Crash Frequency

The predicted average crash frequency is calculated from Equation 12-5 based on the results obtained in Steps 9 through 11 as follows:

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

$$= 1.00 \times (2.854 + 0.475 + 0.043)$$

$$= 3.372 \text{ crashes/year}$$

Worksheets

The step-by-step instructions above are provided to illustrate the predictive method for calculating the predicted average crash frequency for an intersection. To apply the predictive method steps to multiple intersections, a series of twelve worksheets are provided for determining the predicted average crash frequency at intersections. The twelve worksheets include:

- Worksheet 2A – General Information and Input Data for Urban and Suburban Arterial Intersections
- Worksheet 2B – Accident Modification Factors for Urban and Suburban Arterial Intersections
- Worksheet 2C – Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections
- Worksheet 2D – Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections
- Worksheet 2E – Single-Vehicle Crashes by Severity Level for Urban and Suburban Arterial Intersections
- Worksheet 2F – Single-Vehicle Crashes by Collision Type for Urban and Suburban Arterial Intersections
- Worksheet 2G – Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Stop-Controlled Intersections
- Worksheet 2H – Accident Modification Factors for Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections
- Worksheet 2I – Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections
- Worksheet 2J – Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections
- Worksheet 2K – Crash Severity Distribution for Urban and Suburban Arterial Intersections
- Worksheet 2L – Summary Results for Urban and Suburban Arterial Intersections

Details of these worksheets are provided below, except for Worksheets 2G which is only used for stop-controlled intersections. Blank versions of worksheets used in the Sample Problems are provided in Chapter 12 Appendix A.

Worksheet 2A – General Information and Input Data for Urban and Suburban Arterial Intersections

Worksheet 2A is a summary of general information about the intersection, analysis, input data (i.e., "The Facts") and assumptions for Sample Problem 4.

Worksheet 2A – General Information and Input Data for Urban and Suburban Arterial Intersections

General Information		Location Information	
Analyst		Roadway	
Agency or Company		Intersection	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	Base Conditions	Site Conditions
Intersection type (3ST, 3SG, 4ST, 4SG)	-	4SG
$AADT_{major}$ (veh/day)	-	15,000
$AADT_{minor}$ (veh/day)	-	9,000
Intersection lighting (present/not present)	not present	present
Calibration factor, C_i	1.00	1.00
Data for unsignalized intersections only:	-	-
Number of major-road approaches with left-turn lanes (0,1,2)	0	N/A
Number of major-road approaches with right-turn lanes (0,1,2)	0	N/A
Data for signalized intersections only:	-	-
Number of approaches with left-turn lanes (0,1,2,3,4)	0	2
Number of approaches with right-turn lanes (0,1,2,3,4)	0	2
Number of approaches with left-turn signal phasing	-	2
Type of left-turn signal phasing	permissive	protected/permissive
Intersection red light cameras (present/not present)	not present	not present
Sum of all pedestrian crossing volumes (PedVol)	-	1,500
Maximum number of lanes crossed by a pedestrian (n_{lanesx})	-	4
Number of bus stops within 300 m (1,000 ft) of the intersection	0	2
Schools within 300 m (1,000 ft) of the intersection (present/not present)	not present	present
Number of alcohol sales establishments within 300 m (1,000 ft) of the intersection	0	6

Worksheet 2B – Accident Modification Factors for Urban and Suburban Arterial Intersections

In Step 10 of the predictive method, Accident Modification Factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 12.7 presents the tables and equations necessary for determining the AMF values. Once the value for each AMF has been determined, all of the AMFs are multiplied together in Column 7 of Worksheet 2B which indicates the combined AMF value.

Worksheet 2B – Accident Modification Factors for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
AMF for Left-Turn Lanes	AMF for Left-Turn Signal Phasing	AMF for Right-Turn Lanes	AMF for Right-Turn-on-Red	AMF for Lighting	AMF for Red Light Cameras	Combined AMF
AMF_{1i}	AMF_{2i}	AMF_{3i}	AMF_{4i}	AMF_{5i}	AMF_{6i}	AMF_{COMB}
from Exhibit 12-41	from Exhibit 12-42	from Exhibit 12-43	from Equation 12-35	from Equation 12-36	from Equation 12-37	(1)*(2)*(3)*(4)*(5)*(6)
0.81	0.98	0.92	1.00	0.91	1.00	0.66

Worksheet 2C – Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

The SPF for multiple-vehicle collisions at the intersection in Sample Problem 4 is calculated using Equation 12-22 and entered into Column 4 of Worksheet 2C. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2 and 3; however, the overdispersion parameter is not needed for Sample Problem 4 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values(from Column 4) to assure that fatal and injury (FI) and property damage only (PDO) crashes sum to the total crashes, as illustrated in Column 6. Column 7 represents the combined AMF (from Column 7 in Worksheet 2B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of multiple-vehicle crashes using the values in Column 6, the combined AMF in Column 7, and the calibration factor in Column 8.

Worksheet 2C – Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)
Crash severity Level	**SPF Coefficients**			**Overdispersion Parameter, k**	**Initial N_{bimv}**	**Proportion of total crashes**	**Adjusted N_{bimv}**	**Combined AMFs**	**Calibration Factor, C_i**	**Predicted N_{bimv}**
	from Exhibit 12-19			from Exhibit 12-19	from Equation 12-22		$(4)_{TOTAL}*(5)$	(7) from Worksheet 2B		(6)*(7)*(8)
	a	b	c							
Total	-10.99	1.07	0.23	0.39	4.027	1.000	4.027	0.66	1.00	2.658
Fatal and injury (FI)	-13.14	1.18	0.22	0.33	1.233	$(4)_{FI}/((4)_{FI}+(4)_{PDO})$ 0.318	1.281	0.66	1.00	0.845
Property damage only (PDO)	-11.02	1.02	0.24	0.44	2.647	$(5)_{TOTAL}-(5)_{FI}$ 0.682	2.746	0.66	1.00	1.812

Worksheet 2D –Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

Worksheet 2D presents the default proportions for collision type (from Exhibit 12-24) by crash severity level as follows:

- Fatal and injury crashes (Column 2)
- Property damage only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for multiple-vehicle crashes by collision type is presented in Columns 3 (Fatal and Injury, FI), 5 (Property Damage Only PDO), and 6 (Total).

These proportions may be used to separate the predicted average crash frequency for multiple-vehicle crashes (from Column 9, Worksheet 2C) into components by crash severity and collision type.

Worksheet 2D – Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
Collision Type	**Proportion of Collision Type** $_{(FI)}$	**Predicted** $N_{bimv\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type** $_{(PDO)}$	**Predicted** $N_{bimv\ (PDO)}$ **(crashes/year)**	**Predicted** $N_{bimv\ (TOTAL)}$ **(crashes/year)**
	from Exhibit 12-24	$(9)_{FI}$ from Worksheet 2C	from Exhibit 12-24	$(9)_{PDO}$ from Worksheet 2C	$(9)_{PDO}$ from Worksheet 2C
Total	1.000	0.845	1.000	1.812	2.658
		$(2)*(3)_{FI}$		$(4)*(5)_{PDO}$	(3)+(5)
Rear-end collision	0.450	0.380	0.483	0.875	1.255
Head-on collision	0.049	0.041	0.030	0.054	0.095
Angle collision	0.347	0.293	0.244	0.442	0.735
Sideswipe	0.099	0.084	0.032	0.058	0.142
Other multiple-vehicle collision	0.55	0.046	0.211	0.382	0.428

Worksheet 2E– Single-Vehicle Crashes by Severity Level for Urban and Suburban Arterial Intersections

The SPF for single-vehicle crashes at the intersection in Sample Problem 4 is calculated using Equation 12-25 for total and property damage only (PDO) crashes and entered into Column 4 of Worksheet 2E. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2, and 3; however, the overdispersion parameter is not needed for Sample Problem 4 (as the EB Method is not utilized). Column 5 of the worksheet presents the proportions for crash severity levels calculated from the results in Column 4. These proportions are used to adjust the initial SPF values (from Column 4) to assure that fatal and injury (FI) and property damage only (PDO) crashes sum to the total crashes, as illustrated in Column 6. Column 7 represents the combined AMF (from Column 7 in Worksheet 2B), and Column 8 represents the calibration factor. Column 9 calculates the predicted average crash frequency of single-vehicle crashes using the values in Column 6, the combined AMF in Column 7, and the calibration factor in Column 8.

Worksheet 2E – Single-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)
Crash severity Level	**SPF Coefficients**			**Overdispersion Parameter, k**	**Initial N_{bisv}**	**Proportion of total crashes**	**Adjusted N_{bisv}**	**Combined AMFs**	**Calibration Factor, C_i**	**Predicted N_{bisv}**
	from Exhibit 12-25			from Exhibit 12-25	from Equation 12-25; (FI) from Equation 12-25 or 12-27		$(4)_{TOTAL}$*(5)	(7) from Worksheet 2B		(6)*(7)*(8)
	a	b	c							
Total	-10.21	0.68	0.27	0.36	0.297	1.000	0.297	0.66	1.000	0.196
Fatal and injury (FI)	-9.25	0.43	0.29	0.09	0.084	(4)FI/((4)FI+(4)PDO) 0.287	0.085	0.66	1.000	0.056
Property damage only (PDO)	-11.34	0.78	0.25	0.44	0.209	(5)TOTAL-(5)FI 0.713	0.212	0.66	1.000	0.140

Worksheet 2F –Single-Vehicle Crashes by Collision Type for Urban and Suburban Arterial Intersections

Worksheet 2F presents the default proportions for collision type (from Exhibit 12-30) by crash severity level as follows:

- Fatal and injury crashes (Column 2)
- Property damage only crashes (Column 4)

Using the default proportions, the predicted average crash frequency for single-vehicle crashes by collision type is presented in Columns 3 (Fatal and Injury, FI), 5 (Property Damage Only PDO), and 6 (Total).

These proportions may be used to separate the predicted average crash frequency for single-vehicle crashes (from Column 9, Worksheet 2E) into components by crash severity and collision type.

Worksheet 2F – Single-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections					
(1)	(2)	(3)	(4)	(5)	(6)
Collision Type	**Proportion of Collision Type (FI)**	**Predicted $N_{bisv\ (FI)}$ (crashes/year)**	**Proportion of Collision Type (PDO)**	**Predicted $N_{bisv\ (PDO)}$ (crashes/year)**	**Predicted $N_{bisv\ (TOTAL)}$ (crashes/year)**
	Exhibit 12-30	$(9)_{FI}$ from Worksheet 2E	Exhibit 12-30	$(9)_{PDO}$ from Worksheet 2E	$(9)_{PDO}$ from Worksheet 2E
Total	1.000	0.056	1.000	0.140	0.196
		$(2)*(3)_{FI}$		$(4)*(5)_{PDO}$	(3)+(5)
Collision with parked vehicle	0.001	0.000	0.001	0.000	0.000
Collision with animal	0.002	0.000	0.002	0.000	0.000
Collision with fixed object	0.744	0.042	0.870	0.122	0.164
Collision with other object	0.072	0.004	0.070	0.010	0.014
Other single-vehicle collision	0.040	0.002	0.023	0.003	0.005
Single-vehicle noncollision	0.141	0.008	0.034	0.005	0.013

Worksheet 2H– Accident Modification Factors for Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections

In Step 10 of the predictive method, Accident Modification Factors are applied to account for the effects of site specific geometric design and traffic control devices. Section 12.7 presents the tables and equations necessary for determining the AMF values for vehicle-pedestrian collision. Once the value for each AMF has been determined, all of the AMFs are multiplied together in Column 4 of Worksheet 2H which indicates the combined AMF value for vehicle-pedestrian collisions.

Worksheet 2H – Accident Modification Factors for Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections			
(1)	(2)	(3)	(4)
AMF for Bus Stops	AMF for Schools	AMF for Alcohol Sales Establishments	Combined AMF
AMF_{1p}	AMF_{2p}	AMF_{3p}	
from Exhibit 12-45	from Exhibit 12-46	from Exhibit 12-47	(1)*(2)*(3)
2.78	1.35	1.12	4.20

Worksheet 2I– Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections

The predicted number of vehicle-pedestrian collisions per year for base conditions at a signalized intersection, $N_{pedbase}$, is calculated using Equation 12-30 and entered into Column 4 of Worksheet 2I. The coefficients for the SPF and the overdispersion parameter associated with the SPF are entered into Columns 2, and 3; however, the overdispersion parameter is not needed for Sample Problem 4 (as the EB Method is not utilized). Column 5 represents the combined AMF for vehicle-pedestrian collisions (from Column 4 in Worksheet 2H), and Column 6 represents the calibration factor. Column 7 calculates the predicted average crash frequency of vehicle-pedestrian collisions using the values in Column 4, the combined AMF in Column 5, and the calibration factor in Column 6. Since all vehicle-pedestrian crashes are assumed to involve some level of injury, there are no property damage only crashes.

Worksheet 2I – Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections

(1)	(2)					(3)	(4)	(5)	(6)	(7)
Crash severity level	**SPF Coefficients**					**Overdispersion Parameter, k**	**$N_{pedbase}$**	**Combined AMF**	**Calibration factor, C_i**	**Predicted N_{pedi}**
	from Exhibit 12-31						from Equation 12-30	(4) from Worksheet 2H		(8)*(9)*(10)
	a	b	c	d	e					
Total	-9.53	0.40	0.26	0.45	0.04	0.24	0.113	4.20	1.00	0.475
Fatal and injury (FI)	-	-	-	-	-	-	-	-	1.00	0.475

Worksheet 2J– Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections

The predicted average crash frequency of multiple-vehicle predicted crashes and single-vehicle predicted crashes from Worksheets 2C and 2E are entered into Columns 2 and 3 respectively. These values are summed in Column 4. Column 5 contains the bicycle accident adjustment factor (see Exhibit 12-34). Column 6 presents the calibration factor. The predicted average crash frequency of vehicle-bicycle collision (Column 7) is the product of Columns 4, 5 and 6. Since all vehicle-bicycle crashes are assumed to involve some level of injury, there are no property damage only crashes.

Worksheet 2J – Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Crash severity level	**Predicted N_{bimv}**	**Predicted N_{bisv}**	**Predicted N_{bi}**	**f_{bikei}**	**Calibration factor, C_i**	**Predicted N_{pedi}**
	(9) from Worksheet 2C	(9) from Worksheet 2E	(2)+(3)	from Exhibit 10-34		(4)*(5)*(6)
Total	2.658	0.196	2.854	0.015	1.00	0.043
Fatal and injury (FI)	-	-	-	-	1.00	0.043

Worksheet 2K– Crash Severity Distribution for Urban and Suburban Arterial Intersections

Worksheet 2K provides a summary of all collision types by severity level. Values from Worksheets 2D, 2F, 2I and 2J are presented and summed to provide the predicted average crash frequency for each severity level as follows:

- Fatal and injury crashes (Column 2)
- Property damage only crashes (Column 3)
- Total crashes (Column 4)

Worksheet 2K– Crash Severity Distribution for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)
Collision type	**Fatal and injury (FI)**	**Property damage only (PDO)**	**Total**
	(3) from Worksheet 2D and 2F; (7) from 2G or 2I and 2J	(5) from Worksheet 2D and 2F;	(6) from Worksheet 2D and 2F; (7) from 2G or 2I and 2J
MULTIPLE-VEHICLE COLLISIONS			
Rear-end collisions (from Worksheet 2D)	0.380	0.875	1.255
Head-on collisions (from Worksheet 2D)	0.041	0.054	0.095
Angle collisions (from Worksheet 2D)	0.293	0.442	0.735
Sideswipe (from Worksheet 2D)	0.084	0.058	0.142
Other multiple-vehicle collision (from Worksheet 2D)	0.046	0.382	0.428
Subtotal	0.844	1.811	2.655
SINGLE-VEHICLE COLLISIONS			
Collision with parked vehicle (from Worksheet 2F)	0.000	0.000	0.000
Collision with animal (from Worksheet 2F)	0.000	0.000	0.000
Collision with fixed object (from Worksheet 2F)	0.042	0.122	0.164
Collision with other object (from Worksheet 2F)	0.004	0.010	0.014
Other single-vehicle collision (from Worksheet 2F)	0.002	0.003	0.005
Single-vehicle noncollision (from Worksheet 2F)	0.008	0.005	0.013
Collision with pedestrian (from Worksheet 2G or 2I)	0.475	0.000	0.475
Collision with bicycle (from Worksheet 2J)	0.043	0.000	0.043
Subtotal	0.574	0.140	0.714
Total	1.418	1.951	3.369

Worksheet 2L– Summary Results for Urban and Suburban Arterial Intersections

Worksheet 2L presents a summary of the results.

Worksheet 2L – Summary Results for Urban and Suburban Arterial Intersections	
(1)	(2)
Crash severity level	**Predicted average crash frequency, $N_{predicted\ int}$ (crashes/year)**
	(Total) from Worksheet 2K
Total	3.369
Fatal and injury (FI)	1.418
Property damage only (PDO)	1.951

12.13.5. Sample Problem 5

The Project

A project of interest consists of four sites located on an urban arterial: a three-lane TWLTL segment; a four-lane divided segment; a three-leg intersection with minor-road stop control; and a four-leg signalized intersection. (This project is a compilation of roadway segments and intersections from Sample Problems 1 through 4.)

The Question

What is the expected accident frequency of the project for a particular year incorporating both the predicted crash frequencies from Sample Problems 1 through 4 and the observed crash frequencies using the **site-specific EB Method**?

The Facts

- 2 roadway segments (3T segment, 4D segment)
- 2 intersections (3ST intersection, 4SG intersection)
- 34 observed crashes (3T segment: 7 multiple-vehicle nondriveway, 4 single-vehicle, 2 multiple-vehicle driveway related; 4D: 6 multiple-vehicle nondriveway, 3 single-vehicle, 1 multiple-vehicle driveway related; 3ST: 2 multiple-vehicle, 3 single-vehicle; 4SG 6 multiple-vehicle, 0 single-vehicle)

Outline of Solution

To calculate the expected average crash frequency, site-specific observed crash frequencies are combined with predicted crash frequencies for the project using the site-specific EB Method (i.e. observed crashes are assigned to specific intersections or roadway segments) presented in Section A.2.4 of *Part C* Appendix.

Results

The expected average crash frequency for the project is 25.4 crashes per year (rounded to one decimal place).

Worksheets

To apply the site-specific EB Method to multiple roadway segments and intersections on an urban or suburban arterial combined, three worksheets are provided for determining the expected average crash frequency. The three worksheets include:

- Worksheet 3A – Predicted Crashes by Collision and Site Type and Observed Crashes Using the Site-Specific EB Method for Urban and Suburban Arterials.
- Worksheet 3B – Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials.

- Worksheet 3C – Site-Specific EB Method Summary Results for Urban and Suburban Arterials

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 12 Appendix A.

Worksheets 3A – Predicted Crashes by Collision and Site Type and Observed Crashes Using the Site-Specific EB Method for Urban and Suburban Arterials.

The predicted average crash frequencies by severity level and collision type determined in Sample Problems 1 through 4 are entered into Columns 2 through 4 of Worksheet 3A. Column 5 presents the observed crash frequencies by site and collision type, and Column 6 presents the overdispersion parameters. The expected average crash frequency is calculated by applying the site-specific EB Method which considers both the predicted model estimate and observed crash frequencies for each roadway segment and intersection. Equation A-5 from *Part C* Appendix is used to calculate the weighted adjustment and entered into Column 7. The expected average crash frequency is calculated using Equation A-4 and entered into Column 8. Detailed calculation of Columns 7 and 8 are provided below.

Worksheet 3A – Predicted Crashes by Collision and Site Type and Observed Crashes Using the Site-Specific EB Method for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Collision type / site type	**Predicted average crash frequency (crashes/year)**			**Observed crashes, $N_{observed}$ (crashes/year)**	**Overdispersion parameter, k**	**Weighted adjustment, w**	**Expected average crash frequency, $N_{expected\ (VEHICLE)}$**
	$N_{predicted\ (TOTAL)}$	$N_{predicted\ (FI)}$	$N_{predicted\ (PDO)}$			Equation A-5 from Part C Appendix	Equation A-4 from Part C Appendix
ROADWAY SEGMENTS							
Multiple-vehicle nondriveway							
Segment 1	4.967	1.196	3.771	7	0.66	0.234	6.524
Segment 2	2.524	0.702	1.822	6	1.32	0.231	5.197
Single-vehicle							
Segment 1	1.182	0.338	0.844	4	1.37	0.382	2.924
Segment 2	0.485	0.085	0.401	3	0.86	0.706	1.224
Multiple-vehicle driveway-related							
Segment 1	0.734	0.179	0.555	2	1.10	0.553	1.300
Segment 2	0.149	0.042	0.107	1	1.39	0.828	0.295
INTERSECTIONS							
Multiple-vehicle							
Intersection 1	1.268	0.405	0.862	2	0.80	0.496	1.637
Intersection 2	2.658	0.845	1.812	6	0.39	0.491	4.359
Single-vehicle							
Intersection 1	0.234	0.072	0.162	3	1.14	0.789	0.818
Intersection 2	0.196	0.056	0.140	0	0.36	0.934	0.183
COMBINED (sum of column)	14.397	3.920	10.476	34	-	-	24.461

2632

Column 7 - Weighted Adjustment

The weighted adjustment, w, to be placed on the predictive model estimate is calculated using Equation A-5 from *Part C* Appendix as follows:

$$w = \frac{1}{1 + k \times \left(\sum_{\substack{\text{all study} \\ \text{years}}} N_{predicted} \right)}$$

Multiple-Vehicle Nondriveway Collisions

Segment 1 $$w = \frac{1}{1 + 0.66 \times (4.967)}$$

$$= 0.234$$

Segment 2 $$w = \frac{1}{1 + 1.32 \times (2.524)}$$

$$= 0.231$$

Single-Vehicle Crashes

Segment 1 $$w = \frac{1}{1 + 1.37 \times (1.182)}$$

$$= 0.382$$

Segment 2 $$w = \frac{1}{1 + 0.86 \times (0.485)}$$

$$= 0.706$$

Multiple-Vehicle Driveway Related Collisions

Segment 1 $$w = \frac{1}{1 + 1.10 \times (0.734)}$$

$$= 0.553$$

Segment 2 $$w = \frac{1}{1 + 1.39 \times (0.149)}$$

$$= 0.828$$

Multiple-Vehicle Collisions

Intersection 1 $$w = \frac{1}{1 + 0.80 \times (1.268)}$$

$$= 0.496$$

Intersection 2 $$w = \frac{1}{1 + 0.39 \times (2.658)}$$

$$= 0.491$$

Single-Vehicle Crashes

Intersection 1 $$w = \frac{1}{1 + 1.14 \times (0.234)}$$

$$= 0.789$$

Intersection 2 $$w = \frac{1}{1 + 0.36 \times (0.196)}$$

$$= 0.934$$

Column 8 - Expected Average Crash Frequency

The estimate of expected average crash frequency, $N_{expected}$, is calculated using Equation A-4 from *Part C* Appendix as follows:

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

Multiple-Vehicle Nondriveway Collisions

Segment 1 $$N_{expected} = 0.234 \times 4.967 + (1 - 0.234) \times 7$$

$$= 6.524$$

Segment 2 $$N_{expected} = 0.231 \times 2.524 + (1 - 0.231) \times 6$$

$$= 5.197$$

Single-Vehicle Crashes

Segment 1 $$N_{expected} = 0.382 \times 1.182 + (1 - 0.382) \times 4$$

$$= 2.924$$

Segment 2 $$N_{expected} = 0.706 \times 0.485 + (1 - 0.706) \times 3$$

$$= 1.224$$

Multiple-Vehicle Driveway Related Collisions

Segment 1 $$N_{expected} = 0.553 \times 0.734 + (1 - 0.553) \times 2$$

$$= 1.300$$

Segment 2 $$N_{expected} = 0.828 \times 0.149 + (1 - 0.828) \times 1$$

$$= 0.295$$

Multiple-Vehicle Collisions

Intersection 1 $$N_{expected} = 0.496 \times 1.268 + (1 - 0.496) \times 2$$

$$= 1.637$$

Intersection 2 $$N_{expected} = 0.491 \times 2.658 + (1 - 0.491) \times 6$$

$$= 4.359$$

Single-Vehicle Crashes

Intersection 1 $$N_{expected} = 0.789 \times 0.234 + (1 - 0.789) \times 3$$

$$= 0.818$$

Intersection 2 $$N_{expected} = 0.934 \times 0.196 + (1 - 0.934) \times 0$$

$$= 0.183$$

Worksheets 3B – Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials

Worksheet 3B provides a summary of the vehicle-pedestrian and vehicle-bicycle crashes determined in Sample Problems 1 through 4.

Worksheet 3B – Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials

(1)	(2)	(3)
Site Type	N_{ped}	N_{bike}
ROADWAY SEGMENTS		
Segment 1	0.089	0.048
Segment 2	0.212	0.041
INTERSECTIONS		
Intersection 1	0.032	0.024
Intersection 2	0.475	0.043
COMBINED (sum of column)	0.808	0.156

Worksheets 3C – Site-Specific EB Method Summary Results for Urban and Suburban Arterials

Worksheet 3C presents a summary of the results. Column 5 calculates the expected average crash frequency by severity level for vehicle crashes only by applying the proportion of predicted average crash frequency by severity level (Column 2) to the expected average crash frequency calculated using the site-specific EB Method. Column 6 calculates the total expected average crash frequency by severity level using the values in Column 3, 4 and 5.

Worksheet 3C – Site-Specific EB Method Summary Results for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)
Crash severity level	$N_{predicted}$	N_{ped}	N_{bike}	$N_{expected\ (VEHICLE)}$	$N_{expected}$
Total	$(2)_{COMB}$ Worksheet 3A	$(2)_{COMB}$ Worksheet 3B	$(3)_{COMB}$ Worksheet 3B	$(13)_{COMB}$ Worksheet 3A	(3)+(4)+(5)
	14.397	0.808	0.156	24.461	25.4
Fatal and injury (FI)	$(3)_{COMB}$ Worksheet 3A	$(2)_{COMB}$ Worksheet 3B	$(3)_{COMB}$ Worksheet 3B	$(5)_{TOTAL}*(2)_{FI}/(2)_{TOTAL}$	(3)+(4)+(5)
	3.920	0.808	0.156	6.660	7.6
Property damage only (PDO)	$(4)_{COMB}$ Worksheet 3A	-	-	$(5)_{TOTAL}*(2)_{PDO}/(2)_{TOTAL}$	(3)+(4)+(5)
	10.476	0.000	0.000	17.800	17.8

12.13.6. Sample Problem 6

The Project

A project of interest consists of four sites located on an urban arterial: a three-lane TWLTL segment; a four-lane divided segment; a three-leg intersection with minor-road stop control; and a four-leg signalized intersection. (This project is a compilation of roadway segments and intersections from Sample Problems 1 through 4.)

The Question

What is the expected average crash frequency of the project for a particular year incorporating both the predicted average crash frequencies from Sample Problems 1 through 4 and the observed crash frequencies using the **project-level EB Method**?

The Facts

- 2 roadway segments (3T segment, 4D segment)
- 2 intersection (3ST intersection, 4SG intersection)
- 34 observed crashes (but no information is available to attribute specific crashes to specific sites)

Outline of Solution

Observed crash frequencies for the project as a whole are combined with predicted average crash frequencies for the project as a whole using the project-level EB Method (i.e. observed crash data for individual roadway segments and intersections are not available, but observed crashes are assigned to a facility as a whole) presented in Section A.2.5 of *Part C* Appendix.

Results

The expected average crash frequency for the project is 26.0 crashes per year (rounded to one decimal place).

Worksheets

To apply the project-level EB Method to multiple roadway segments and intersections on an urban or suburban arterial combined, three worksheets are provided for determining the expected average crash frequency. The three worksheets include:

- Worksheet 4A – Predicted Crashes by Collision and Site Type and Observed Crashes Using the Project-Level EB Method for Urban and Suburban Arterials
- Worksheet 4B – Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials
- Worksheet 4C – Project-EB Method Summary Results for Urban and Suburban Arterials

Details of these worksheets are provided below. Blank versions of worksheets used in the Sample Problems are provided in Chapter 12 Appendix A.

Worksheets 4A – Predicted Crashes by Collision and Site Type and Observed Crashes Using the Project-Level EB Method for Urban and Suburban Arterials

The predicted average crash frequencies by severity level and collision type, excluding vehicle-pedestrian and vehicle-bicycle collisions, determined in Sample Problems 1 through 4 are entered in Columns 2 through 4 of Worksheet 4A. Column 5 presents the total observed crash frequencies combined for all sites, and Column 6 presents the overdispersion parameters. The expected average crash frequency is calculated by applying the project-level EB Method which considers both the predicted model estimate for each roadway segment and intersection and the project observed crashes. Column 7 calculates N_{w0} and Column 8 N_{w1}. Equations A-10 through A-14 from *Part C* Appendix are used to calculate the expected average crash frequency of combined sites. The results obtained from each equation are presented in Columns 9 through 14. Section A.2.5 in *Part C* Appendix defines all the variables used in this worksheet. Detailed calculations of Columns 9 through 13 are provided below.

Worksheet 4A – Predicted Crashes by Collision and Site Type and Observed Crashes Using the Project-Level EB Method for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Collision type / site type	**Predicted crashes**			**Observed crashes, $N_{observed}$ (crashes/year)**	**Overdispersion parameter, k**	**$N_{predicted\ w0}$**	**$N_{predicted\ w1}$**	**w_0**	**N_0**	**w_1**	**N_1**	**$N_{expected/comb}$ (VEHICLE)**
	$N_{predicted}$ (TOTAL)	$N_{predicted}$ (FI)	$N_{predicted}$ (PDO)			Equation A-8 $(6)*(2)^2$	Equation A-9 (sqrt((6)*(2))	Equation A-10	Equation A-11	Equation A-12	Equation A-13	Equation A-14
ROADWAY SEGMENTS												
Multiple-vehicle nondriveway												
Segment 1	4.967	1.196	3.771	-	0.66	16.283	1.811	-	-	-	-	-
Segment 2	2.524	0.702	1.822	-	1.32	8.409	1.825	-	-	-	-	-
Single-vehicle												
Segment 1	1.182	0.338	0.844	-	1.37	1.914	1.273	-	-	-	-	-
Segment 2	0.485	0.085	0.401	-	0.86	0.202	0.646	-	-	-	-	-
Multiple-vehicle driveway-related												
Segment 1	0.734	0.179	0.555	-	1.10	0.593	0.899	-	-	-	-	-
Segment 2	0.149	0.042	0.107	-	1.39	0.031	0.455	-	-	-	-	-
INTERSECTIONS												
Multiple-vehicle												
Intersection 1	1.268	0.405	0.862	-	0.80	1.286	1.007	-	-	-	-	-
Intersection 2	2.658	0.845	1.812	-	0.39	2.755	1.018	-	-	-	-	-
Single-vehicle												
Intersection 1	0.234	0.072	0.162	-	1.14	0.062	0.516	-	-	-	-	-
Intersection 2	0.196	0.056	0.140	-	0.36	0.014	0.266	-	-	-	-	-
COMBINED (sum of column)	14.397	3.920	10.476	34	-	31.549	9.716	0.313	27.864	0.597	22.297	25.080

NOTE: $N_{predicted\ w0}$ = Predicted number of total accidents assuming that accidents frequencies are statistically independent

$$N_{predicted\ w0} = \sum_{j=1}^{5} k_{rmj}\ N_{rmj}^2 + \sum_{j=1}^{5} k_{rsj}\ N_{rsj}^2 + \sum_{j=1}^{5} k_{rdj}\ N_{rdj}^2 + \sum_{j=1}^{4} k_{imj}\ N_{imj}^2 + \sum_{j=1}^{4} k_{isj}\ N_{isj}^2 \quad \text{(A-8)}$$

$N_{predicted\ w1}$ = Predicted number of total accidents assuming that accidents frequencies are perfectly correlated

$$N_{predicted\ w1} = \sum_{j=1}^{5} \sqrt{k_{rmj}\ N_{rmj}} + \sum_{j=1}^{5} \sqrt{k_{rsj}\ N_{isj}} + \sum_{j=1}^{5} \sqrt{k_{rdj}\ N_{rdj}} + \sum_{j=1}^{4} \sqrt{k_{imj}\ N_{imj}} + \sum_{j=1}^{4} \sqrt{k_{isj}\ N_{isj}} \quad \text{(A-9)}$$

Column 9 – w_0

The weight placed on predicted crash frequency under the assumption that accidents frequencies for different roadway elements are statistically independent, w_0, is calculated using Equation A-10 from *Part C* Appendix as follows:

$$w_0 = \frac{1}{1 + \frac{N_{predicted\ w0}}{N_{predicted\ (TOTAL)}}}$$

$$= \frac{1}{1 + \frac{31.549}{14.397}}$$

$$= 0.313$$

Column 10 – N_0

The expected crash frequency based on the assumption that different roadway elements are statistically independent, N_0, is calculated using Equation A-11 from *Part C* Appendix as follows:

$$N_0 = w_0\ N_{predicted\ (TOTAL)} + (1 - w_0)\ N_{observed\ (TOTAL)}$$

$$= 0.313 \times 14.397 + (1 - 0.313) \times 34$$

$$= 27.864$$

Column 11 – w_1

The weight placed on predicted crash frequency under the assumption that accidents frequencies for different roadway elements are perfectly correlated, w_1, is calculated using Equation A-12 from *Part C* Appendix as follows:

$$w_1 = \frac{1}{1 + \frac{N_{predicted\ w1}}{N_{predicted\ (TOTAL)}}}$$

$$= \frac{1}{1 + \frac{9.716}{14.397}}$$

$$= 0.597$$

Column 12 – N_1

The expected crash frequency based on the assumption that different roadway elements are perfectly correlated, N_1, is calculated using Equation A-13 from *Part C* Appendix as follows:

$$N_1 = w_1\ N_{predicted\ (TOTAL)} + (1 - w_1)\ N_{observed\ (TOTAL)}$$

$$= 0.597 \times 14.397 + (1 - 0.597) \times 34$$

$$= 22.297$$

Column 13 – $N_{expected/comb}$

The expected average crash frequency based of combined sites, $N_{expected/comb}$, is calculated using Equation A-14 from *Part C* Appendix as follows:

$$N_{expected/comb} = \frac{N_0 + N_1}{2}$$

$$= \frac{27.864 + 22.297}{2}$$

$$= 25.080$$

Worksheets 4B – Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials

Worksheet 4B provides a summary of the vehicle-pedestrian and vehicle-bicycle crashes determined in Sample Problems 1 through 4.

Worksheet 4B – Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials

(1)	(2)	(3)
Site Type	N_{ped}	N_{bike}
ROADWAY SEGMENTS		
Segment 1	0.089	0.048
Segment 2	0.212	0.041
INTERSECTIONS		
Intersection 1	0.032	0.024
Intersection 2	0.475	0.043
COMBINED (sum of column)	0.808	0.156

Worksheets 4C – Project-Level EB Method Summary Results for Urban and Suburban Arterials

Worksheet 4C presents a summary of the results. Column 5 calculates the expected average crash frequency by severity level for vehicle crashes only by applying the proportion of predicted average crash frequency by severity level (Column 2) to the expected average crash frequency calculated using the project-level EB Method. Column 6 calculates the total expected average crash frequency by severity level using the values in Column 3, 4 and 5.

Worksheet 4C – Project-Level EB Method Summary Results for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)
Crash severity level	$N_{predicted}$	N_{ped}	N_{bike}	$N_{expected/comb}$ (VEHICLE)	$N_{expected}$
Total	$(2)_{COMB}$ Worksheet 4A	$(2)_{COMB}$ Worksheet 4B	$(3)_{COMB}$ Worksheet 4B	$(13)_{COMB}$ Worksheet 4A	(3)+(4)+(5)
	14.397	0.808	0.156	25.080	26.0
Fatal and injury (FI)	$(3)_{COMB}$ Worksheet 4A	$(2)_{COMB}$ Worksheet 4B	$(3)_{COMB}$ Worksheet 4B	$(5)_{TOTAL}*(2)_{FI}/(2)_{TOTAL}$	(3)+(4)+(5)
	3.920	0.808	0.156	6.829	7.8
Property damage only (PDO)	$(4)_{COMB}$ Worksheet 4A	-	-	$(5)_{TOTAL}*(2)_{PDO}/(2)_{TOTAL}$	(3)+(4)+(5)
	10.476	0.000	0.000	18.250	18.3

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APPENDIX A - WORKSHEETS FOR PREDICTIVE METHOD FOR URBAN AND SUBURBAN ARTERIALS

Worksheet 1A – General Information and Input Data for Urban and Suburban Roadway Segments

General Information		**Location Information**	
Analyst		Roadway	
Agency or Company		Roadway Section	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	**Base Conditions**	**Site Conditions**
Road type (2U, 3T, 4U, 4D, 5T)	-	
Length of segment, L (mi)	-	
AADT (veh/day)	-	
Type of on-street parking (none/parallel/angle)	none	
Proportion of curb length with on-street parking	-	
Median width (ft)	15	
Lighting (present / not present)	not present	
Auto speed enforcement (present/not present)	not present	
Major commercial driveways (number)	-	
Minor commercial driveways (number)	-	
Major industrial/institutional driveways (number)	-	
Minor industrial/institutional driveways (number)	-	
Major residential driveways (number)	-	
Minor residential driveways (number)	-	
Other driveways (number)	-	
Speed Category	-	
Roadside fixed object density (fixed objects/mi)	not present	
Offset to roadside fixed objects (ft)	not present	
Calibration Factor, C_r	1.0	

Worksheet 1B – Accident Modification Factors for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
AMF for On-Street Parking	AMF for Roadside Fixed Objects	AMF for Median Width	AMF for Lighting	AMF for Auto Speed Enforcement	Combined AMF
AMF_{1r}	AMF_{2r}	AMF_{3r}	AMF_{4r}	AMF_{5r}	AMF_{COMB}
from Equation 12-32	from Equation 12-33	from Exhibit 12-39	from Equation 12-34	from Section 12.7.1	(1)*(2)*(3)*(4)*(5)

Worksheet 1C – Multiple-Vehicle Nondriveway Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)
Crash severity level	**SPF Coefficients**		**Overdispersion Parameter, k**	**Initial N_{brmv}**	**Proportion of total crashes**	**Adjusted N_{brmv}**	**Combined AMFs**	**Calibration factor**	**Predicted N_{brmv}**
	from Exhibit 12-5		from Exhibit 12-5	from Equation 12-10		$(4)_{TOTAL}$*(5)	(6) from Worksheet 1B	C_r	(6)*(7)*(8)
	a	b							
Total									
Fatal and injury (FI)					$(4)_{FI}/((4)_{FI}+(4)_{PDO})$				
Property damage only (PDO)					$(5)_{TOTAL}-(5)_{FI}$				

Worksheet 1D – Multiple-Vehicle Nondriveway Collisions by Collision Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
Collision type	**Proportion of Collision Type $_{(FI)}$**	**Predicted $N_{brmv\ (FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**Predicted $N_{brmv\ (PDO)}$ (crashes/year)**	**Predicted $N_{brmv\ (TOTAL)}$ (crashes/year)**
	from Exhibit 12-7	$(9)_{FI}$ from Worksheet 1C	from Exhibit 12-7	$(9)_{PDO}$ from Worksheet 1C	$(9)_{TOTAL}$ from Worksheet 1C
Total	1.000		1.000		
		$(2)*(3)_{FI}$		$(4)*(5)_{PDO}$	(3)+(5)
Rear-end collision					
Head-on collision					
Angle collision					
Sideswipe, same direction					
Sideswipe, opposite direction					
Other multiple-vehicle collision					

2868

Worksheet 1E – Single-Vehicle Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)
Crash severity level	**SPF Coefficients**		**Overdispersion Parameter, k**	**Initial N_{brsv}**	**Proportion of total crashes**	**Adjusted N_{brsv}**	**Combined AMFs**	**Calibration factor**	**Predicted N_{brsv}**
	from Exhibit 12-8		from Exhibit 12-8	from Equation 12-13		$(4)_{TOTAL}*(5)$	(6) from Worksheet 1B	C_r	(6)*(7)*(8)
	a	b							
Total									
Fatal and injury (FI)					$(4)_{FI}/((4)_{FI}+(4)_{PDO})$				
Property damage only (PDO)					$(5)_{TOTAL}-(5)_{FI}$				

2869

Worksheet 1F – Single-Vehicle Collisions by Collision Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
Collision type	**Proportion of Collision Type $_{(FI)}$**	**Predicted $N_{brsv\,(FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**Predicted $N_{brsv\,(PDO)}$ (crashes/year)**	**Predicted $N_{brsv\,(TOTAL)}$ (crashes/year)**
	from Exhibit 12-10	$(9)_{FI}$ from Worksheet 1E	from Exhibit 12-10	$(9)_{PDO}$ from Worksheet 1E	$(9)_{TOTAL}$ from Worksheet 1E
Total	1.000		1.000		
		$(2)*(3)_{FI}$		$(4)*(5)_{PDO}$	(3)+(5)
Collision with animal					
Collision with fixed object					
Collision with other object					
Other single-vehicle collision					

2870

Worksheet 1G – Multiple-Vehicle Driveway-Related Collisions by Driveway Type for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)
Driveway type	**Number of driveways, n_j**	**Crashes per driveway per year, N_j**	**Coefficient for traffic adjustment, t**	**Initial N_{brdwy}**	**Overdispersion parameter, k**
		from Exhibit 12-11	from Exhibit 12-11	Equation 12-16 $n_j * N_j * (AADT/15{,}000)^t$	from Exhibit 12-11
Major commercial					
Minor commercial					
Major industrial/institutional					
Minor industrial/institutional					-
Major residential					
Minor residential					
Other					
Total	-	-	-		

2871

Worksheet 1H – Multiple-Vehicle Driveway-Related Collisions by Severity Level for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Crash severity level	**Initial N_{brdwy}**	**Proportion of total accidents (f_{dwy})**	**Adjusted N_{brdwy}**	**Combined AMFs**	**Calibration factor, C_r**	**Predicted N_{brdwy}**
	$(5)_{TOTAL}$ from Worksheet 1G	from Exhibit 12-11	$(2)_{TOTAL}$ *(3)	(6) from Worksheet 1B		(4)*(5)*(6)
Total						
Fatal and injury (FI)	-					
Property damage only (PDO)	-					

Worksheet 1I – Vehicle-Pedestrian Collisions for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crash severity level	**Predicted N_{brmv}**	**Predicted N_{brsv}**	**Predicted N_{brdwy}**	**Predicted N_{br}**	**f_{pedr}**	**Calibration factor, C_r**	**Predicted N_{pedr}**
	(9) from Worksheet 1C	(9) from Worksheet 1E	(7) from Worksheet 1H	(2)+(3)+(4)	from Exhibit 12-17		(5)*(6)*(7)
Total							
Fatal and injury (FI)	-	-	-	-	-		

Worksheet 1J – Vehicle-Bicycle Collisions for Urban and Suburban Roadway Segments

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Crash severity level	**Predicted N_{brmv}**	**Predicted N_{brsv}**	**Predicted N_{brdwy}**	**Predicted N_{br}**	**f_{biker}**	**Calibration factor, C_r**	**Predicted N_{biker}**
	(9) from Worksheet 1C	(9) from Worksheet 1E	(7) from Worksheet 1H	(2)+(3)+(4)	from Exhibit 12-18		(5)*(6)*(7)
Total							
Fatal and injury	-	-	-	-	-		

Worksheet 1K – Crash Severity Distribution for Urban and Suburban Roadway Segments			
(1)	(2)	(3)	(4)
Collision type	**Fatal and injury (FI)**	**Property damage only (PDO)**	**Total**
	(3) from Worksheet 1D and 1F; (7) from Worksheet 1H; and (8) from Worksheet 1I and 1J	(5) from Worksheet 1D and 1F; and (7) from Worksheet 1H;	(6) from Worksheet 1D and 1F; (7) from Worksheet 1H; and (8) from Worksheet 1I and 1J
MULTIPLE-VEHICLE			
Rear-end collisions (from Worksheet 1D)			
Head-on collisions (from Worksheet 1D)			
Angle collisions (from Worksheet 1D)			
Sideswipe, same direction (from Worksheet 1D)			
Sideswipe, opposite direction (from Worksheet 1D)			
Driveway-related collisions (from Worksheet 1H)			
Other multiple-vehicle collision (from Worksheet 1D)			
Subtotal			
SINGLE-VEHICLE			
Collision with animal (from Worksheet 1F)			
Collision with fixed object (from Worksheet 1F)			
Collision with other object (from Worksheet 1F)			
Other single-vehicle collision (from Worksheet 1F)			
Collision with pedestrian (from Worksheet 1I)			
Collision with bicycle (from Worksheet 1J)			
Subtotal			
Total			

Worksheet 1L – Summary Results for Urban and Suburban Roadway Segments			
(1)	(2)	(3)	(4)
Crash severity level	**Predicted average crash frequency, $N_{predicted\ rs}$ (crashes/year)**	**Roadway segment length, L (mi)**	**Crash rate (crashes/mi/year)**
	(Total) from Worksheet 1K		(2)/(3)
Total			
Fatal and injury (FI)			
Property damage only (PDO)			

2876

Worksheet 2A – General Information and Input Data for Urban and Suburban Arterial Intersections

General Information		Location Information	
Analyst		Roadway	
Agency or Company		Intersection	
Date Performed		Jurisdiction	
		Analysis Year	

Input Data	Base Conditions	Site Conditions
Intersection type (3ST, 3SG, 4ST, 4SG)	-	
$AADT_{major}$ (veh/day)	-	
$AADT_{minor}$ (veh/day)	-	
Intersection lighting (present/not present)	not present	
Calibration factor, C_i	1.00	
Data for unsignalized intersections only:	-	
Number of major-road approaches with left-turn lanes (0,1,2)	0	
Number of major-road approaches with right-turn lanes (0,1,2)	0	
Data for signalized intersections only:	-	
Number of approaches with left-turn lanes (0,1,2,3,4)	0	
Number of approaches with right-turn lanes (0,1,2,3,4)	0	
Number of approaches with left-turn signal phasing	-	
Type of left-turn signal phasing	permissive	
Intersection red light cameras (present/not present)	not present	
Sum of all pedestrian crossing volumes (PedVol)	-	
Maximum number of lanes crossed by a pedestrian (n_{lanesx})	-	
Number of bus stops within 300 m (1,000 ft) of the intersection	0	
Schools within 300 m (1,000 ft) of the intersection (present/not present)	not present	
Number of alcohol sales establishments within 300 m (1,000 ft) of the intersection	0	

2877

Worksheet 2B – Accident Modification Factors for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
AMF for Left-Turn Lanes	AMF for Left-Turn Signal Phasing	AMF for Right-Turn Lanes	AMF for Right Turn on Red	AMF for Lighting	AMF for Red Light Cameras	Combined AMF
AMF_{1i}	AMF_{2i}	AMF_{3i}	AMF_{4i}	AMF_{5i}	AMF_{6i}	AMF_{COMB}
from Exhibit 12-41	from Exhibit 12-42	from Exhibit 12-43	from Equation 12-35	from Equation 12-36	from Equation 12-37	(1)*(2)*(3)*(4)*(5)*(6)

Worksheet 2C – Multiple-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)
Crash severity Level	**SPF Coefficients**			**Overdispersion Parameter, k**	**Initial N_{bimv}**	**Proportion of total crashes**	**Adjusted N_{bimv}**	**Combined AMFs**	**Calibration Factor, C_i**	**Predicted N_{bimv}**
	from Exhibit 12-19			from Exhibit 12-19	from Equation 12-22		$(4)_{TOTAL}$*(5)	(7) from Worksheet 2B		(6)*(7)*(8)
	a	b	c							
Total										
Fatal and injury (FI)						$(4)_{FI}/((4)_{FI}+(4)_{PDO})$				
Property damage only (PDO)						$(5)_{TOTAL}-(5)_{FI}$				

Worksheet 2D – Multiple-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
Collision Type	**Proportion of Collision Type** $_{(FI)}$	**Predicted** $N_{bimv\ (FI)}$ **(crashes/year)**	**Proportion of Collision Type** $_{(PDO)}$	**Predicted** $N_{bimv\ (PDO)}$ **(crashes/year)**	**Predicted** $N_{bimv\ (TOTAL)}$ **(crashes/year)**
	from Exhibit 12-24	$(9)_{FI}$ from Worksheet 2C	from Exhibit 12-24	$(9)_{PDO}$ from Worksheet 2C	$(9)_{PDO}$ from Worksheet 2C
Total	1.000		1.000		
		$(2)*(3)_{FI}$		$(4)*(5)_{PDO}$	(3)+(5)
Rear-end collision					
Head-on collision					
Angle collision					
Sideswipe					
Other multiple-vehicle collision					

2880

Worksheet 2E – Single-Vehicle Collisions by Severity Level for Urban and Suburban Arterial Intersections

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)
Crash severity Level	**SPF Coefficients**			**Overdispersion Parameter, k**	**Initial** N_{bisv}	**Proportion of total crashes**	**Adjusted** N_{bisv}	**Combined AMFs**	**Calibration Factor,** C_i	**Predicted** N_{bisv}
	from Exhibit 12-25			from Exhibit 12-25	from Equation 12-25; (FI) from Equation 12-25 or 12-27		$(4)_{TOTAL}*(5)$	(7) from Worksheet 2B		(6)*(7)*(8)
	a	b	c							
Total										
Fatal and injury (FI)						$(4)_{FI}/((4)_{FI}+(4)_{PDO})$				
Property damage only (PDO)						$(5)_{TOTAL}-(5)_{FI}$				

2881

Worksheet 2F – Single-Vehicle Collisions by Collision Type for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)	(5)	(6)
Collision Type	**Proportion of Collision Type $_{(FI)}$**	**Predicted $N_{bisv\,(FI)}$ (crashes/year)**	**Proportion of Collision Type $_{(PDO)}$**	**Predicted $N_{bisv\,(PDO)}$ (crashes/year)**	**Predicted $N_{bisv\,(TOTAL)}$ (crashes/year)**
	Exhibit 12-30	$(9)_{FI}$ from Worksheet 2E	Exhibit 12-30	$(9)_{PDO}$ from Worksheet 2E	$(9)_{PDO}$ from Worksheet 2E
Total	1.000		1.000		
		$(2)*(3)_{FI}$		$(4)*(5)_{PDO}$	(3)+(5)
Collision with parked vehicle					
Collision with animal					
Collision with fixed object					
Collision with other object					
Other single-vehicle collision					
Single-vehicle noncollision					

Worksheet 2G – Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Stop-Controlled Intersections

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Crash severity level	**Predicted N_{bimv}**	**Predicted N_{bisv}**	**Predicted N_{bi}**	**f_{pedi}**	**Calibration factor, C_i**	**Predicted N_{pedi}**
	(9) from Worksheet 2C	(9) from Worksheet 2E	(2)+(3)	from Exhibit 12-33		(4)*(5)*(6)
Total						
Fatal and injury (FI)	-	-	-	-		

Worksheet 2H – Accident Modification Factors for Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections

(1)	(2)	(3)	(4)
AMF for Bus Stops	AMF for Schools	AMF for Alcohol Sales Establishments	Combined AMF
AMF_{1p}	AMF_{2p}	AMF_{3p}	
from Exhibit 12-45	from Exhibit 12-46	from Exhibit 12-47	(1)*(2)*(3)

Worksheet 2I – Vehicle-Pedestrian Collisions for Urban and Suburban Arterial Signalized Intersections

(1)	(2)					(3)	(4)	(5)	(6)	(7)
Crash severity level	**SPF Coefficients**					**Overdispersion Parameter, k**	**$N_{pedbase}$**	**Combined AMF**	**Calibration factor, C_i**	**Predicted N_{pedi}**
	from Exhibit 12-31						from Equation 12-30	(4) from Worksheet 2H		(8)*(9)*(10)
	a	b	c	d	e					
Total										
Fatal and injury (FI)	-	-	-	-	-	-	-	-		

Worksheet 2J – Vehicle-Bicycle Collisions for Urban and Suburban Arterial Intersections

(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**
Crash severity level	**Predicted N_{bimv}**	**Predicted N_{bisv}**	**Predicted N_{bi}**	**f_{bikei}**	**Calibration factor, C_i**	**Predicted N_{pedi}**
	(9) from Worksheet 2C	(9) from Worksheet 2E	(2)+(3)	from Exhibit 10-34		(4)*(5)*(6)
Total						
Fatal and injury (FI)	-	-	-	-		

Worksheet 2K– Crash Severity Distribution for Urban and Suburban Arterial Intersections

(1)	(2)	(3)	(4)
Collision type	**Fatal and injury (FI)**	**Property damage only (PDO)**	**Total**
	(3) from Worksheet 2D and 2F; (7) from 2G or 2I and 2J	(5) from Worksheet 2D and 2F;	(6) from Worksheet 2D and 2F; (7) from 2G or 2I and 2J
MULTIPLE-VEHICLE COLLISIONS			
Rear-end collisions (from Worksheet 2D)			
Head-on collisions (from Worksheet 2D)			
Angle collisions (from Worksheet 2D)			
Sideswipe (from Worksheet 2D)			
Other multiple-vehicle collision (from Worksheet 2D)			
Subtotal			
SINGLE-VEHICLE COLLISIONS			
Collision with parked vehicle (from Worksheet 2F)			
Collision with animal (from Worksheet 2F)			
Collision with fixed object (from Worksheet 2F)			
Collision with other object (from Worksheet 2F)			
Other single-vehicle collision (from Worksheet 2F)			
Single-vehicle noncollision (from Worksheet 2F)			
Collision with pedestrian (from Worksheet 2G or 2I)			
Collision with bicycle (from Worksheet 2J)			
Subtotal			
Total			

2887

Worksheet 2L – Summary Results for Urban and Suburban Arterial Intersections

(1)	(2)
Crash severity level	**Predicted average crash frequency, $N_{predicted\ int}$ (crashes/year)**
	(Total) from Worksheet 2K
Total	
Fatal and injury (FI)	
Property damage only (PDO)	

Worksheet 3A – Predicted Crashes by Collision and Site Type and Observed Crashes Using the Site-Specific EB Method for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Collision type / site type	**Predicted average crash frequency (crashes/year)**			**Observed crashes, $N_{observed}$ (crashes/year)**	**Overdispersion parameter, k**	**Weighted adjustment, w**	**Expected average crash frequency, $N_{expected\ (VEHICLE)}$**
	$N_{predicted\ (TOTAL)}$	$N_{predicted\ (FI)}$	$N_{predicted\ (PDO)}$			Equation A-5 from Part C Appendix	Equation A-4 from Part C Appendix
ROADWAY SEGMENTS							
Multiple-vehicle nondriveway							
Segment 1							
Segment 2							
Segment 3							
Segment 4							
Single-vehicle							
Segment 1							
Segment 2							
Segment 3							
Segment 4							
Multiple-vehicle driveway-related							
Segment 1							
Segment 2							
Segment 3							
Segment 4							
INTERSECTIONS							
Multiple-vehicle							
Intersection 1							
Intersection 2							
Intersection 3							
Intersection 4							
Single-vehicle							
Intersection 1							
Intersection 2							
Intersection 3							
Intersection 4							
COMBINED (sum of column)					-	-	

Worksheet 3B – Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials		
(1)	(2)	(3)
Site Type	**N_{ped}**	**N_{bike}**
ROADWAY SEGMENTS		
Segment 1		
Segment 2		
Segment 3		
Segment 4		
INTERSECTIONS		
Intersection 1		
Intersection 2		
Intersection 3		
Intersection 4		
COMBINED (sum of column)		

Worksheet 3C – Site-Specific EB Method Summary Results for Urban and Suburban Arterials					
(1)	(2)	(3)	(4)	(5)	(6)
Crash severity level	**$N_{predicted}$**	**N_{ped}**	**N_{bike}**	**$N_{expected\ (VEHICLE)}$**	**$N_{expected}$**
Total	$(2)_{COMB}$ Worksheet 3A	$(2)_{COMB}$ Worksheet 3B	$(3)_{COMB}$ Worksheet 3B	$(13)_{COMB}$ Worksheet 3A	(3)+(4)+(5)
Fatal and injury (FI)	$(3)_{COMB}$ Worksheet 3A	$(2)_{COMB}$ Worksheet 3B	$(3)_{COMB}$ Worksheet 3B	$(5)_{TOTAL}*(2)_{FI}/(2)_{TOTAL}$	(3)+(4)+(5)
Property damage only (PDO)	$(4)_{COMB}$ Worksheet 3A	-	-	$(5)_{TOTAL}*(2)_{PDO}/(2)_{TOTAL}$	(3)+(4)+(5)
		0.000	0.000		

Worksheet 4A – Predicted Crashes by Collision and Site Type and Observed Crashes Using the Project-Level EB Method for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
Collision type / site type	**Predicted crashes**			**Observed crashes, $N_{observed}$ (crashes/year)**	**Overdispersion parameter, k**	$N_{predicted\ wo}$	$N_{predicted\ w1}$	w_o	N_o	w_1	N_1	$N_{expected/comb}$ (VEHICLE)
	$N_{predicted}$ (TOTAL)	$N_{predicted}$ (FI)	$N_{predicted}$ (PDO)			Equation A-8 (6)*(2)^2	Equation A-9 (sqrt((6)*(2)))	Equation A-10	Equation A-11	Equation A-12	Equation A-13	Equation A-14
ROADWAY SEGMENTS												
Multiple-vehicle nondriveway												
Segment 1				-				-	-	-	-	-
Segment 2				-				-	-	-	-	-
Segment 3				-				-	-	-	-	-
Segment 4				-				-	-	-	-	-
Single-vehicle												
Segment 1				-				-	-	-	-	-
Segment 2				-				-	-	-	-	-
Segment 3				-				-	-	-	-	-
Segment 4				-				-	-	-	-	-
Multiple-vehicle driveway-related												
Segment 1				-				-	-	-	-	-
Segment 2				-				-	-	-	-	-
Segment 3				-				-	-	-	-	-
Segment 4				-				-	-	-	-	-
INTERSECTIONS												
Multiple-vehicle												
Intersection 1				-				-	-	-	-	-
Intersection 2				-				-	-	-	-	-
Intersection 3				-				-	-	-	-	-
Intersection 4				-				-	-	-	-	-
Single-vehicle												
Intersection 1				-				-	-	-	-	-
Intersection 2				-				-	-	-	-	-
Intersection 3				-				-	-	-	-	-
Intersection 4				-				-	-	-	-	-
COMBINED (sum of column)					-							

PART C —PREDICTIVE METHOD

APPENDIX

APPENDIX TO HSM PART C– SPECIALIZED PROCEDURES COMMON TO ALL PART C CHAPTERS

This appendix presents two specialized procedures intended for use with the predictive method presented in Chapters 10, 11, and 12. These include the procedure for calibrating the predictive models presented in the Part C chapters to local conditions and the Empirical Bayes (EB) Method for combining observed crash frequencies with the estimate provided by the predictive models in Part C. Both of these procedures are an integral part of the predictive method in Chapters 10, 11, and 12, and are presented in an Appendix only to avoid repetition across the chapters.

A.1 Calibration of the Part C Predictive Models

The Part C predictive method in Chapters 10, 11, and 12 include predictive models which consist of Safety Performance Functions (SPFs), Accident Modification Factors (AMFs) and Calibration factors, and have been developed for specific roadway segment and intersection types. The SPF functions are the basis of the predictive models and were developed in HSM-related research from the most complete and consistent available data sets. However, the general level of accident frequencies may vary substantially from one jurisdiction to another for a variety of reasons including climate, driver populations, animal populations, accident reporting thresholds, and accident reporting system procedures. Therefore, for the Part C predictive models to provide results that are meaningful and accurate for each jurisdiction, it is important that the SPFs be calibrated for application in each jurisdiction. A procedure for determining the calibration factors for the Part C predictive models is presented below in Section A.1.1.

Some HSM users may prefer to develop SPFs with data from their own jurisdiction for use in the Part C predictive models rather than calibrating the Part C SPFs. Calibration of the Part C SPFs will provide satisfactory results. However, SPFs developed directly with data for a specific jurisdiction may provide more reliable estimates for that jurisdiction than calibration of Part C SPFs. Therefore, jurisdictions that have the capability, and wish to develop their own models are encouraged to do so. Guidance on development of jurisdiction-specific SPFs that are suitable for use in the Part C predictive method is presented in Section A.1.2.

Most of the regression coefficients and distribution values used in the Part C predictive models in Chapters 10, 11, and 12 have been determined through research and modification by users is not recommended. However, a few specific quantities, such as the distribution of crashes by collision type or the proportion of crashes occurring during night-time conditions, are known to vary substantially from jurisdiction to jurisdiction. Where appropriate local data are available, users are encouraged to replace these default values with locally derived values. The values in the predictive models that may be updated by users to fit local conditions are explicitly identified in Chapters 10, 11, and 12. Unless explicitly identified, values in the predictive models should not be modified by the user. A procedure for deriving jurisdiction-specific values to replace these selected parameters is presented below in Section A.1.3.

A.1.1 Calibration of Predictive Models

The purpose of the Part C calibration procedure is to adjust the predictive models which were developed with data from one jurisdiction for application in another jurisdiction. Calibration provides a method to account for differences between jurisdictions in factors such as climate, driver populations, animal populations, accident reporting thresholds, and accident reporting system procedures.

The calibration procedure is used to derive the values of the calibration factors for roadway segments and for intersections that are used in the Part C predictive models. The calibration factor for roadway segments, C_r, is used in Equations 10-2, 11-2, 11-3, and 12-2. The calibration factor for intersections, C_i, is used in Equations 10-3, 11-4, and 12-5. The calibration factors, C_r and C_i, are based on the ratio of the total observed accident frequencies for a selected set of sites to the total expected average crash frequency estimated for the same sites, during the same time period, using the applicable Part C predictive method. Thus, the nominal value of the calibration factor, when the observed and predicted crash frequencies happen to be equal, is 1.00. When there are more accidents observed than are predicted by the Part C predictive method, the computed calibration factor will be greater than 1.00. When there are fewer accidents observed than are predicted by the Part C predictive method, the computed calibration factor will be less than 1.00.

It is recommended that new values of the calibration factors be derived at least every two to three years, and some HSM users may prefer to develop calibration factors on an annual basis. The calibration factor for the most recent available period is to be used for all assessment of proposed future projects. If available, calibration factors for the specific time periods included in the evaluation periods before and after a project or treatment implementation are to be used in effectiveness evaluations that use the procedures presented in Chapter 9.

If the procedures in Section A.1.3 are used to calibrate any default values in the Part C predictive models to local conditions, the locally-calibrated values should be used in the calibration process described below.

The calibration procedure involves five steps:

- Step 1 – Identify facility types for which the applicable Part C predictive model is to be calibrated
- Step 2 – Select sites for calibration of the predictive model for each facility type
- Step 3 – Obtain data for each facility type applicable to a specific calibration period
- Step 4 – Apply the applicable Part C predictive model to predict total crash frequency for each site during the calibration period as a whole
- Step 5 – Compute calibration factors for use in Part C predictive model

Each of these steps is described below.

A.1.1.1 Step 1 – Identify facility types for which the applicable Part C SPFs are to be calibrated

Calibration is performed separately for each facility type addressed in each Part C chapter. Exhibit A-1 identifies all of the facility types included in the Part C chapters for which calibration factors need to be derived. The Part C SPFs for each of these facility types are to be calibrated before use, but HSM users may choose not to calibrate the SPFs for particular facility types if they do not plan to apply the Part C SPFs for those facility types.

Exhibit A-1. SPFs in the Part C Predictive Models that Need Calibration

Facility, Segment, or Intersection Type	Calibration Factor to be Derived	
	Symbol	Equation Number(s)
ROADWAY SEGMENTS		
Rural two-lane roads		
Two-lane undivided segments	C_r	10-2
Rural multilane highways		
Undivided segments	C_r	11-2
Divided segments	C_r	11-3
Urban and suburban arterials		
Two-lane undivided segments	C_r	12-2
Three-lane segments with center TWLTL	C_r	12-2
Four-lane undivided segments	C_r	12-2
Four-lane divided segments	C_r	12-2
Five-lane segments with center TWLTL	C_r	12-2
INTERSECTIONS		
Rural two-lane roads		
Three-leg intersections with minor-road STOP control	C_i	10-3
Four-leg intersections with minor-road STOP control	C_i	10-3
Four-leg signalized intersections	C_i	10-3
Rural multilane highways		
Three-leg intersections with minor-road STOP control	C_i	11-4
Four-leg intersections with minor-road STOP control	C_i	11-4
Four-leg signalized intersections	C_i	11-4
Urban and suburban arterials		
Three-leg intersections with minor-road STOP control	C_i	12-5
Three-leg signalized intersections	C_i	12-5
Four-leg intersections with minor-road STOP control	C_i	12-5
Four-leg signalized intersections	C_i	12-5

A.1.1.2 Step 2 – Select sites for calibration of the SPF for each facility type

For each facility type, the desirable minimum sample size for the calibration data set is 30 to 50 sites, with each site long enough to adequately represent physical and safety conditions for the facility. Calibration sites should be selected without regard to the number of crashes on individual sites; in other words, calibration sites should not be selected to intentionally limit the calibration data set to include only sites with either high or low accident frequencies. Where practical, this may be accomplished by selecting calibration sites randomly from a larger set of candidate sites. Following site selection, the entire group of calibration sites should represent a total of at least 100 accidents per year. These calibration sites will be either roadway segments or intersections, as appropriate to the facility type being addressed. If the required data discussed in Step 3 are readily available for a larger number of sites, that larger number of sites should be used for calibration. If a jurisdiction has fewer than 30 sites for a particular facility type, then it is desirable to use all of those available sites for calibration. For large jurisdictions, such as entire states, with a variety of topographical and climate conditions, it may be desirable to assemble a separate set of sites and develop separate calibration factors for each specific terrain type or geographical region. For example, a state with distinct plains and mountains regions, or with distinct dry and wet regions, might choose to develop separate calibration factors for those regions. On the other hand, a state that is relatively uniform in terrain and climate, might choose to perform a single calibration for the entire state. Where separate calibration factors are developed by terrain type or region, this needs to be done consistently for all applicable facility types in those regions.

It is desirable that the calibration sites for each facility type be reasonably representative of the range of site characteristics to which the predictive model will be applied. However, no formal stratification by traffic volume or other site characteristics is needed in selecting the calibration sites, so the sites can be selected in a manner to make the data collection needed for Step 3 as efficient as practical. There is no need to develop a new data set, if an existing data set with sites suitable for calibration is already available. If no existing data set is available so that a calibration data set consisting entirely of new data needs to be developed, or if some new sites need to be chosen to supplement an existing data set, it is desirable to choose the new calibration sites by random selection from among all sites of the applicable facility type.

Step 2 needs only be performed the first time that calibration is performed for a given facility type. For calibration in subsequent years, the same sites may be used again.

A.1.1.3 Step 3 – Obtain data for each facility type applicable to a specific calibration period

Once the calibration sites have been selection, the next step is to assemble the calibration data set if a suitable data set is not already available. For each site in the calibration data set, the calibration data set should include:

- Total observed crash frequency for a period of one or more years in duration.
- All site characteristics data needed to apply the applicable Part C predictive model.

Observed crashes for all severity levels should be included in calibration. The duration of crash frequency data should correspond to the period for which the resulting calibration factor, C_r or C_i, will be applied in the Part C predictive models. Thus, if an annual calibration factor is being developed, the duration of the calibration period should include just that one year. If the resulting calibration factor will be employed for two or three years, the duration of the calibration period should include only those years. Since crash frequency is likely to change over time, calibration periods longer than three years are not recommended. All calibration periods should have durations that are multiples of 12 months to avoid seasonal effects. For ease of application, it is recommended that the calibration periods consist of one, two, or three full calendar years. It is recommended to use the same calibration period for all sites, but exceptions may be made where necessary.

The observed crash data used for calibration should include all crashes related to each roadway segment or intersection selected for the calibration data set. Crashes should be assigned to specific roadway segments or intersections based on the guidelines presented below in Section A.2.3.

Exhibit A-2 identifies the site characteristics data that are needed to apply the Part C predictive models for each facility type. The exhibit classifies each data element as either required or desirable for the calibration procedure. Data for each of the required elements are needed for calibration. If data for some required elements are not readily available, it may be possible to select sites in Step 2 for which these data are available. For example, in calibrating the predictive models for roadway segments on rural two-lane highways, if data on the radii of horizontal curves are not readily available, the calibration data set could be limited to tangent roadways. Decisions of this type should be made, as needed, to keep the effort required to assemble the calibration data set within reasonable bounds. For the data elements identified in Exhibit A-2 as desirable, but not required, it is recommended that actual data be used if available, but assumptions are suggested in the exhibit for application where data are not available.

Exhibit A-2: Data Needs for Calibration of Part C Predictive Models by Facility Type

Chapter	Data Element	Data Need		Default Assumption
		Required	Desirable	
ROADWAY SEGMENTS				
10 - Rural two-lane roads	Segment length	X		Need actual data
	Average annual daily traffic (AADT)	X		Need actual data
	Lengths of horizontal curves and tangents	X		Need actual data
	Radii of horizontal curves	X		Need actual data
	Presence of spiral transition for horizontal curves		X	Base default on agency design policy
	Superelevation variance for horizontal curves		X	No superelevation variance
	Percent grade		X	Base default on terrain[a]
	Lane width	X		Need actual data
	Shoulder type	X		Need actual data
	Shoulder width	X		Need actual data
	Presence of lighting		X	Assume no lighting
	Driveway density		X	Assume 5 driveways per mile
	Presence of passing lane		X	Assume not present
	Presence of short four-lane section		X	Assume not present
	Presence of center two-way left-turn lane	X		Need actual data
	Presence of centerline rumble strip		X	Base default on agency design policy
	Roadside hazard rating		X	Assume roadside hazard rating = 3
	Use of automated speed enforcement		X	Base default on current practice
11 - Rural multilane highways	*For all rural multilane highways:*			
	Segment length	X		Need actual data
	Average annual daily traffic (AADT)	X		Need actual data
	Lane width	X		Need actual data
	Shoulder width	X		Need actual data
	Presence of lighting	X		Assume no lighting
	Use of automated speed enforcement		X	Base default on current practice

Chapter	Data Element	Data Need		Default Assumption
		Required	Desirable	
	For undivided highways only:			
	Side slope	X		Need actual data
	For divided highways only:			
	Median width	X		Need actual data
12 - Urban and suburban arterials	Segment length	X		Need actual data
	Number of through traffic lanes	X		Need actual data
	Presence of median	X		Need actual data
	Presence of center two-way left-turn lane	X		Need actual data
	Average annual daily traffic (AADT)	X		Need actual data
	Number of driveways by land-use type	X		Need actual data[b]
	Low-speed vs. intermediate or high speed	X		Need actual data
	Presence of on-street parking	X		Need actual data
	Type of on-street parking	X		Need actual data
	Roadside fixed object density		X	database default on fixed-object offset and density categories[c]
	Presence of lighting		X	Base default on agency practice
	Presence of automated speed enforcement		X	Base default on agency practice
INTERSECTIONS				
10 - Rural two-lane roads	Number of intersection legs	X		Need actual data
	Type of traffic control	X		Need actual data
	Average annual daily traffic (AADT) for major road	X		Need actual data
	Average daily traffic (AADT) for minor road	X		Need actual data or best estimate
	Intersection skew angle		X	Assume no skew[d]
	Number of approaches with left-turn lanes	X		Need actual data
	Number of approaches with right-turn lanes	X		Need actual data
	Presence of lighting	X		Need actual data
11 - Rural multilane highways	*For all rural multilane highways:*			
	Number of intersection legs	X		Need actual data
	Type of traffic control	X		Need actual data
	Average annual daily traffic (AADT) for major road	X		Need actual data
	Average annual daily traffic (AADT) for minor road	X		Need actual data or best estimate
	Presence of lighting	X		Need actual data[d]
	Intersection skew angle		X	Assume no skew
	Number of approaches with left-turn lanes	X		Need actual data
	Number of approaches with right-turn lanes	X		Need actual data
12 - Urban and suburban arterials	*For all intersections on arterials:*			
	Number of intersection legs	X		Need actual data
	Type of traffic control	X		Need actual data
	Average annual daily traffic (AADT) for major road	X		Need actual data

Chapter	Data Element	Data Need		Default Assumption
		Required	Desirable	
	Average annual daily traffic (AADT) for minor road	X		Need actual data or best estimate
	Number of approaches with left-turn lanes	X		Need actual data
	Number of approaches with right-turn lanes	X		Need actual data
	Presence of lighting	X		Need actual data
	For signalized intersections only:			
	Presence of left-turn phasing	X		Need actual data
	Type of left-turn phasing	X		Prefer actual data, but agency practice may be used as a default
	Use of right-turn-on-red signal operation	X		Need actual data
	Use of red-light cameras	X		Need actual data
	Pedestrian volume		X	Estimate with Table 12-21
	Maximum number of lanes crossed by pedestrians on any approach		X	Estimate from number of lanes and presence of median on major road
	Presence of bus stops within 1,000 ft		X	Assume not present
	Presence of schools within 1,000 ft		X	Assume not present
	Presence of alcohol sales establishments within 1,000 ft		X	Assume not present

[a] Suggested default values for calibration purposes: AMF = 1.00 for level terrain; AMF = 1.06 for rolling terrain; AMF=1.14 for mountainous terrain

[b] Use actual data for number of driveways, but simplified land-use categories may be used (e.g., commercial and residential only)

[c] AMFs may be estimated based on two categories of fixed-object offset (O_{fo}) – either 5 or 20 ft – and three categories of fixed-object density (D_{fo}) – 0, 50, or 100 objects per mile

[d] If measurements of intersection skew angles are not available, the calibration should preferably be performed for intersections with no skew.

A.1.1.4 Step 4 – Apply the applicable Part C predictive method to predict total crash frequency for each site during the calibration period as a whole

The site characteristics data assembled in Step 3 should be used to apply the applicable predictive method from Chapter 10, 11, or 12 to each site in the calibration data set. For this application, the predictive method should be applied without using the EB Method and, of course, without employing a calibration factor (i.e., a calibration factor of 1.00 is assumed). Using the predictive models, the expected average crash frequency is obtained for either one, two, or three years, depending on the duration of the calibration period selected.

A.1.1.5 Step 5 – Compute calibration factors for use in Part C predictive models

The final step is to compute the calibration factor as:

$$C_r\ (or\ C_i) = \frac{\sum_{all\ sites} observed\ crashes}{\sum_{all\ sites} predicted\ crashes} \qquad \text{(A-1)}$$

The computation is performed separately for each facility type. The computed calibration factor is rounded to two decimal places for application in the appropriate Part C predictive model.

Example Calibration Factor Calculation

The SPF for four-leg signalized intersections on rural two-lane roads from Equation 10-18 is:

$$N_{spf\ int} = \exp[-5.73 + 0.60 \times \ln(AADT_{maj}) + 0.20 \times \ln(AADT_{min})]$$

Where,

$N_{spf\ int}$ = predicted number of total intersection-related accidents per year for base conditions
$AADT_{maj}$ = average annual daily entering traffic volumes (vehicles/day) on the major road
$AADT_{min}$ = average annual daily entering traffic volumes (vehicles/day) on the minor road

The base conditions are:

- No Left turn lanes on any approach
- No Right turn lanes on any approach

The AMF values from Chapter 10 are:

- AMF for one approach with a left-turn lane = 0.82
- AMF for one approach with a right-turn lane = 0.96
- AMF for two approaches with right-turn lanes = 0.92
- No lighting present (so lighting AMF = 1.00 for all cases)

Typical data for eight intersections is shown in an example calculation shown below. Note that for an actual calibration, the recommended minimum sample size would be 30 to 50 sites that experience at least 100 accidents per year. Thus, the number of sites used here is smaller than recommended, and is intended solely to illustrate the calculations.

For the first intersection in the example the predicted crash frequency for base conditions is:

$$N_{bibase} = \exp(-5.73 + 0.60 \times \ln(4000) + 0.20 \times \ln(2000) = 2.152 \text{ accidents/year}$$

The intersection has a left-turn lane on the major road, for which AMF_{1i} is 0.67, and a right-turn lane on one approach, a feature for which AMF_{2i} is 0.98. There are three years of data, during which four accidents were observed (shown in Column 10 of Table 1). The predicted average crash frequency from the Chapter 10 for this intersection without calibration is, from Equation 10-2:

N_{bi} = (N_{bibase}) x (AMF_{1i}) x (AMF_{2i}) x (number of years of data)

= 2.152 x 0.67 x 0.98 x 3 = 4.240 accidents in three years, shown in Column 9.

Similar calculations were done for each intersection in the table shown below. The sum of the observed accident frequencies in Column 10 (43) is divided by the sum of the predicted average crash frequencies in Column 9 (45.594) to obtain the calibration factor, C_i, equal to 0.943. It is recommended that calibration factors be rounded to two decimal places, so calibration factor equal to 0.94 should be used in the Chapter 10 predictive model for four-leg signalized intersections.

Example of calibration factor computation

1	2	3	4	5	6	7	8	9	10
ADT_{maj}	ADT_{min}	SPF Prediction	Intersection Approaches with Left-Turn Lanes	AMF_1	Intersection Approaches With Right-Turn Lane	AMF_2	Years of Data	Predicted Average Crash Frequency	Observed Crash Frequency
4000	2000	2.152	1	0.67	1	0.98	3	4.240	4
3000	1500	1.710	0	1.00	2	0.95	2	3.249	5
5000	3400	2.736	0	1.00	2	0.95	3	7.799	10
6500	3000	3.124	0	1.00	2	0.95	3	8.902	5
3600	2300	2.078	1	0.67	1	0.98	3	4.093	2
4600	4500	2.753	0	1.00	2	0.95	3	7.846	8
5700	3300	2.943	1	0.67	1	0.98	3	5.796	5
6800	1500	2.794	1	0.67	1	0.98	2	3.669	4
						Sum		45.594	43
						Calibration Factor (C_i)			**0.943**

A.1.2 Development of Jurisdiction-Specific Safety Performance Functions for Use in the Part C Predictive Method

Satisfactory results from the Part C predictive method can be obtained by calibrating the predictive model for each facility type, as explained in Section A.1.1. However, some users may prefer to develop jurisdiction-specific SPFs using their agency's own data and this is likely to enhance the reliability of the Part C predictive method. While there is no requirement that this be done, HSM users are welcome to use local data to develop their own SPFs, or if they wish, replace some SPFs with jurisdiction-specific models and retain other SPFs from the Part C chapters. Within the first two to three years after a jurisdiction-specific SPF is developed, calibration of the jurisdiction-specific SPF using the procedure presented in Section A.1.1 may not be necessary, particularly if other default values in the Part C models are replaced with locally-derived values, as explained in Section A.1.3.

If jurisdiction-specific SPFs are used in the Part C predictive method, they need to be developed with methods that are statistically valid and developed in such a manner that they fit into the applicable Part C predictive method. The following guidelines for development of jurisdiction-specific SPFs that are acceptable for use in HSM Part C include:

- In preparing the accident data to be used for development of jurisdiction-specific SPFs, crashes are assigned to roadway segments and intersections following the definitions explained in Section A.2.3. and illustrated in Exhibit A-4.

- The jurisdiction-specific SPF should be developed with a statistical technique such as negative binomial regression that accounts for the overdispersion typically found in accident data and quantifies an overdispersion parameter so that the model's predictions can be combined with observed crash frequency data using the EB Method.

- The jurisdiction-specific SPF should use the same base conditions as the corresponding SPF in Part C or should be capable of being converted to those base conditions.

- The jurisdiction-specific SPF should include the effects of the following traffic volumes: average annual daily traffic volume for roadway segment and major- and minor-road average annual daily traffic volumes for intersections.

- The jurisdiction-specific SPF for any roadway segment facility type should have a functional form in which predicted average crash frequency is directly proportional to segment length.

These guidelines are not intended to stifle creativity and innovation in model development. However, a model that does not account for overdispersed data or that cannot be integrated with the rest of the Part C predictive method will not be useful.

Two types of data sets may be used for SPF development. First, SPFs may be developed using only data that represent the base conditions, which are defined for each SPF in Chapters 10, 11, and 12. Second, it is also acceptable to develop models using data for a broader set of conditions than the base conditions. In this approach, all variables that are part of the applicable base-condition definition, but have non-base-condition values, should be included in an initial model. Then, the initial model should be made applicable to the base conditions by substituting values that correspond to those base conditions into the model. Several examples of this process are presented in Appendix A to Chapter 10.

A.1.3 Replacement of Selected Default Values in the Part C Predictive Models to Local Conditions

The Part C predictive models use many default values that have been derived from accident data in HSM-related research. For example, the urban intersection predictive model in Chapter 12 uses pedestrian factors that are based on the proportion of pedestrian crashes compared to total crashes. Replacing these default values with locally derived values will improve the reliability of the Part C predictive models. Exhibit A-3 identifies the specific exhibits in Part C that may be replaced with locally derived values. In addition to exhibits, there is one equation – Equation 10-18 – which uses constant values given in the accompanying text in Chapter 10. These constant values may be replaced with locally derived values.

Providing locally-derived values for the data elements identified in Exhibit A-3 is optional. Satisfactory results can be obtained with the Part C predictive models, as they stand, when the predictive model for each facility type is calibrated with the procedure given in Section A.1.1. But, more reliable results may be obtained by updating the data elements listed in Exhibit A-3. It is acceptable to replace some, but not all of these data elements, if data to replace all of them are not available. Each element that is updated with locally-derived values should provide a small improvement in the reliability of that specific predictive model. To preserve the integrity of the Part C predictive method, the quantitative values in the predictive models, (other than those listed in Exhibit A-3 and those discussed in Sections A.1.1 and A.2.2), should not be modified. Any replacement values derived with the procedures presented in this section should be incorporated in the predictive models before the calibration described in Section A.1.1 is performed.

Exhibit A-3: Default Accident Distributions Used in Part C Predictive Models Which May Be Calibrated by Users to Local Conditions

Chapter	Exhibit or Equation Number	Type of Roadway Element		Data Element or Distribution That May Be Calibrated to Local Conditions
		Roadway Segments	Intersections	
10 - Rural two-lane roads	Exhibit 10-6	X		Crash severity by facility type for roadway segments
	Exhibit 10-7	X		Collision type by facility type for roadway segments
	Exhibit 10-11		X	Crash severity by facility type for intersections
	Exhibit 10-12		X	Collision type by facility type for intersections
	Equation 10-18	X		Driveway-related accidents as a proportion of total accidents (P_D)
	Exhibit 10-20	X		Nighttime accidents as a proportion of total accidents by severity level
	Exhibit 10-23		X	Nighttime accidents as a proportion of total accidents by severity level and by intersection type
11 - Rural multilane highways	Exhibit 11-7	X		Crash severity and collision type for undivided segments
	Exhibit 11-10	X		Crash severity and collision type for divided segments
	Exhibit 11-16		X	Crash severity and collision type by intersection type
	Exhibit 11-24	X		Nighttime accidents as a proportion of total accidents by severity level and by roadway segment type for undivided roadway segments
	Exhibit 11-29		X	Nighttime accidents as a proportion of total accidents by severity level and by roadway segment type for divided roadway segments
	Exhibit 11-34		X	Nighttime accidents as a proportion of total accidents by severity level and by intersection type
12 - Urban and suburban arterials	Exhibit 12-7	X		Crash severity and collision type for multiple-vehicle nondriveway collisions by roadway segment type
	Exhibit 12-10	X		Crash severity and collision type for single-vehicle accidents by roadway segment type
	Exhibit 12-11	X		Crash severity for driveway-related collisions by roadway segment type (see Footnote a)
	Exhibit 12-17	X		Pedestrian accident adjustment factor by roadway segment type
	Exhibit 12-18	X		Bicycle accident adjustment factor by roadway segment type
	Exhibit 12-24		X	Crash severity and collision type for multiple-vehicle collisions by intersection type
	Exhibit 12-30		X	Crash severity and collision type for single-vehicle accidents by intersection type
	Exhibit 12-33		X	Pedestrian accident adjustment factor by intersection type for STOP-controlled intersections
	Exhibit 12-34		X	Bicycle accident adjustment factor by intersection type
	Exhibit 12-40	X		Nighttime accidents as a proportion of total accidents by severity level and by roadway segment type
	Exhibit 12-44		X	Nighttime crashes as a proportion of total crashes by severity level and by intersection type

NOTE: No quantitative values in the Part C predictive models, other than those listed here and those discussed in Sections A.1.1 and A.1.2, should be modified by HSM users.

Footnote a: The only portion of Exhibit 12-10 that should be modified by the user are the crash severity proportions.

Procedures for developing replacement values for each data element identified in Exhibit A-3 are presented below. Most of the data elements to be replaced are proportions of crash severity levels and/or crash types that are part of a specific distribution. Each replacement value for a given facility type should be derived from data for a set of sites that, as a group, includes at least 100 accidents and preferably more. The duration of the study period for a given set of sites may be as long as necessary to include at least 100 accidents. In the following discussion, the term "sufficient data" refers to a data set including a sufficient number of sites to meet this criterion for total accidents. In a few cases, explicitly identified below, the definition of sufficient data will be expressed in terms of an accident category other than total accidents. In assembling data for developing replacements for default values, accidents are to be assigned to specific roadway segments or intersections following the definitions explained in Section A.2.3. and illustrated in Exhibit A-4.

A.1.3.1 Replacement of Default Values for Rural Two-Lane Highways

Five specific sets of default values for rural two-lane highways may be updated with locally-derived replacement values by HSM users. Procedures to develop each of these replacement values are presented below.

Crash severity by Facility Type

Exhibits 10-6 and 10-11 present the distribution of accidents by five crash severity levels for roadway segments and intersections, respectively, on rural two-lane highways. If sufficient data including these five severity levels (fatal, incapacitating injury, nonincapacitating injury, possible injury, and property damage only) are available for a given facility type, the values in Exhibits 10-6 and 10-11 for that facility type may be updated. If sufficient data are available only for the three standard crash severity levels (fatal, injury, and property damage only), the existing values in Exhibits 10-6 and 10-11 may be used to allocate the injury accidents to specific injury severity levels (incapacitating injury, nonincapacitating injury, and possible injury).

Collision Type by Facility Type

Exhibit 10-7 presents the distribution of accidents by collision type for seven specific types of single-vehicle accidents and six specific types of multiple-vehicle accidents for roadway segments and Exhibit 10-12 presents the distribution of accidents by collision type for three intersection types on rural two-lane highways. If sufficient data are available for a given facility type, the values in Exhibits 10-7 and 10-12 for that facility type may be updated.

Driveway-Related Accidents as a Proportion of Total Accidents for Roadway Segments

Equation 10-18 includes a factor, P_D, which represents the proportion of total accidents represented by driveway-related accidents. A value for P_D based on research is presented in the accompanying text. This value may be replaced with a locally-derived value, if data are available for a set for sites that, as a group, have expereinced at least 100 driveway-related accidents.

Nighttime Accidents as a Proportion of Total Accidents for Roadway Segments

Exhibit 10-20 presents the proportions of total night-time accidents by severity level and the proportion of total accidents that occur at night for roadway segments on rural two-lane highways. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 nighttime accidents.

Nighttime Accidents as a Proportion of Total Accidents for Intersections

Exhibit 10-23 presents the proportion of total accidents that occur at night for intersections on rural two-lane highways. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 nighttime accidents.

A.1.3.2 Replacement of Default Values for Rural Multilane Highways

Five specific sets of default values for rural multilane highways may be updated with locally-derived replacement values by HSM users. Procedures to develop each of these replacement values are presented below.

Crash severity and Collision Type for Undivided Roadway Segments

Exhibit 11-7 presents the combined distribution of accidents for four crash severity levels and six collision types. If sufficient data are available for undivided roadway segments, the values in Exhibit 11-7 for this facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 accidents in the time period for which data are available.

Crash severity and Collision Type for Divided Roadway Segments

Exhibit 11-10 presents the combined distribution of accidents for four crash severity levels and six collision types. If sufficient data are available for divided roadway segments, the values in Exhibit 11-10 for this facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires sites that have experienced at least 200 accidents in the time period for which data are available.

Crash severity and Collision Type by Intersection Type

Exhibit 11-16 presents the combined distribution of accidents at intersections for four crash severity levels and six collision types. If sufficient data are available for a given intersection type, the values in Exhibit 11-16 for that intersection type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 accidents in the time period for which data are available.

Night-time Accidents as a Proportion of Total Accidents for Roadway Segments

Exhibits 11-24 and 11-29 present the proportions of total nighttime accidents by severity level and the proportion of total accidents that occur at night for undivided and divided roadway segments, respectively, on rural multilane highways. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites sites that, as a group, have experienced at least 100 nighttime accidents.

Nighttime Accidents as a Proportion of Total Accidents for Intersections

Exhibit 11-34 presents the proportion of total accidents that occur at night for intersections on rural multilane highways. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 night-time accidents.

A.1.3.3 Replacement of Default Values for Urban and Suburban Arterials

Eleven specific sets of default values for urban and suburban arterial highways may be updated with locally-derived replacement values by HSM users. Procedures to develop each of these replacement values are presented below.

Crash severity and Collision Type for Multiple-Vehicle Nondriveway Accidents by Roadway Segment Type

Exhibit 12-7 presents the combined distribution of accidents for two crash severity levels and six collision types. If sufficient data are available for a given facility type, the values in Exhibit 12-4 for that facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 accidents in the time period for which data are available.

Crash severity and Collision Type for Single-Vehicle Accidents by Roadway Segment Type

Exhibit 12-10 presents the combined distribution of accidents for two crash severity levels and six collision types. If sufficient data are available for a given facility type, the values in Exhibit 12-10 for that facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 accidents in the time period for which data are available.

Crash severity for Driveway-Related Collision by Roadway Segment Type

Exhibit 12-11 includes data on the proportions of driveway-related accidents for two crash severity levels (fatal-and-injury and property-damage-only accidents) by facility type for roadway segments. If sufficient data are available for a given facility type, these specific severity-related values in Exhibit 12-11 for that facility type may be updated. The rest of Exhibit 12-11, other than the last two rows of data which are related to crash severity, should not be modified.

Pedestrian Accident Adjustment Factor by Roadway Segment Type

Exhibit 12-17 presents a pedestrian accident adjustment factor for specific roadway segment facility types and for two speed categories: low speed (traffic speeds or posted speed limits of 30 mph or less) and intermediate or high speed (traffic speeds or posted speed limits greater than 30 mph). For a given facility type and speed category, the pedestrian accident adjustment factor is computed as:

$$f_{pedr} = \frac{K_{ped}}{K_{non}} \qquad \text{(A-2)}$$

Where,

f_{pedr} = pedestrian accident adjustment factor

K_{ped} = observed vehicle-pedestrian crash frequency

K_{non} = observed frequency for all accidents not including vehicle-pedestrian and vehicle-bicycle crash

The pedestrian accident adjustment factor for a given facility type should be determined with a set of sites of that speed type that, as a group, includes at least 20 vehicle-pedestrian collisions.

Bicycle Accident Adjustment Factor by Roadway Segment Type

Exhibit 12-18 presents a bicycle accident adjustment factor for specific roadway segment facility types and for two speed categories: low speed (traffic speeds or posted speed limits of 30 mph or less) and intermediate or high speed (traffic speeds or posted speed limits greater than 30 mph). For a given facility type and speed category, the bicycle accident adjustment factor is computed as:

$$f_{biker} = \frac{K_{bike}}{K_{non}} \tag{A-3}$$

Where,

f_{biker} = bicycle accident adjustment factor

K_{bike} = observed vehicle-bicycle crash frequency

K_{non} = observed frequency for all accidents not including vehicle-pedestrian and vehicle-bicycle crashes

The bicycle accident adjustment factor for a given facility type should be determined with a set of sites of that speed type that, as a group, includes at least 20 vehicle-bicycle collisions.

Crash severity and Collision Type for Multiple-Vehicle Accidents by Intersection Type

Exhibit 12-24 presents the combined distribution of accidents for two crash severity levels and six collision types. If sufficient data are available for a given facility type, the values in Exhibit 12-24 for that facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 accidents in the time period for which data are available.

Crash severity and Collision Type for Single-Vehicle Accidents by Intersection Type

Exhibit 12-30 presents the combined distribution of accidents for two crash severity levels and six collision types. If sufficient data are available for a given facility type, the values in Exhibit 12-30 for that facility type may be updated. Given that this is a joint distribution of two variables, sufficient data for this application requires a set of sites of a given type that, as a group, have experienced at least 200 accidents in the time period for which data are available.

Pedestrian Accident Adjustment Factor by Intersection Type

Exhibit 12-33 presents a pedestrian accident adjustment factor for two specific types of intersections with STOP control on the minor road. For a given facility type and speed category, the pedestrian accident adjustment factor is computed using Equation A-2. The pedestrian accident adjustment factor for a given facility type is determined with a set of sites that, as a group, have experienced at least 20 vehicle-pedestrian collisions.

Bicycle Accident Adjustment Factor by Intersection Type

Exhibit 12-34 presents a pedestrian accident adjustment factor for four specific intersection facility types. For a given facility type and speed category, the bicycle accident adjustment factor is computed using Equation A-3. The bicycle accident adjustment factor for a given facility type is determined with a set of sites that, as a group, have experienced at least 20 vehicle-bicycle collisions.

Nighttime Accidents as a Proportion of Total Accidents for Roadway Segments

Exhibit 12-40 presents the proportions of total nighttime accidents by severity level for specific facility types for roadway segments and the proportion of total accidents that occur at night. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 night-time accidents.

Nighttime Accidents as a Proportion of Total Accidents for Intersections

Exhibit 12-44 presents the proportions of total nighttime accidents by severity level for specific facility types for intersections and the proportion of total accidents that occur at night. These values may be replaced with locally-derived values for a given facility type, if data are available for a set of sites that, as a group, have experienced at least 100 nighttime accidents.

A.2 Use of the Empirical Bayes Method to Combine Predicted Average Crash Frequency and Observed Crash Frequency

Application of the EB Method provides a method to combined the estimate using a Part C predictive model and observed crash frequencies to obtain a more reliable estimate of expected average crash frequency. The EB Method is a key tool to compensate for the potential bias due to regression-to-the-mean. Accident frequencies vary naturally from one time period to the next. When a site has a higher than average frequency for a particular time period, the site is likely to have lower crash frequency in subsequent time periods. Statistical methods can help to assure that this natural decrease in crash frequency following a high observed value is not mistaken for the effect of a project or for a true shift in the long-term expected crash frequency.

There are several statistical methods that can be employed to compensate for regression-to-the-mean. The EB Method is used in the HSM because it is best suited to the context of the HSM. The Part C predictive models include negative binomial regression models that were developed before the publication of the HSM by researchers who had no data on the specific sites to which HSM users would later apply those predictive models. The HSM users are generally engineers and planners, without formal statistical training, who would not generally be capable of developing custom models for each set of the sites they wish to apply the HSM to and, even if there were, would have no wish to spend the time and effort needed for model development each time they apply the HSM. The EB Method provides the most suitable tool for compensating for regression-to-the-mean that works in this context.

Each of the Part C chapters presents a four-step process for applying the EB Method. The EB Method assumes that the appropriate Part C predictive model (see Section 10.3.1 for rural two-lane highways, Section 11.3.1 for rural multilane highways, or Section 12.3.1 for urban and suburban arterials) has been applied to determine the predicted crash frequency for the sites that make up a particular project or facility for a particular past time period of interest. The steps in applying the EB Method are:

- Determine whether the EB Method is applicable, as explained in Section A.2.1
- Determine whether observed crash frequency data are available for the project or facility for the time period for which the predictive model was applied and, if so, obtain those crash frequency data, as explained in Section A.2.2. Assign each accident instance to individual roadway segments and intersections, as explained in Section A.2.3.
- Apply the EB Method to estimate the expected crash frequency by combining the predicted and observed accident frequencies for the time period of interest. The site-specific EB Method, applicable when observed crash frequency data are available for the individual roadway segments and intersections that make up a project or facility, is presented in Section A.2.4. The project-level EB Method, applicable when observed crash frequency data are available only for the project or facility as a whole, is presented in Section A.2.5.
- Adjust the estimated value of expected crash frequency to a future time period, if appropriate, as explained in Section A.2.6

Consideration of observed accident history data in the Part C predictive method increases the reliability of the estimate of the expected accident frequencies. When at least two years of observed accident history data are available for the facility or project being evaluated, and when the facility or project meets certain criteria discussed below, the observed crash data should be used. When considering observed accident history data, the procedure must consider both the existing geometric design and traffic control for the facility or project (i.e., the conditions that existed during the before period while the observed accident history was accumulated) and the proposed geometric design and traffic control for the project (i.e., the conditions that will exist during the after period, the period for which accident predictions are being made). In estimating the expected crash frequency for an existing arterial facility in a future time period where no improvement project is planned, only the traffic volumes should differ between the before and after periods. For an arterial on which an improvement project is planned, traffic volumes, geometric design features, and traffic control features may all change between the before and after periods. The EB Method presented below provides a method to combine predicted and observed accident frequencies.

A.2.1 Determine Whether the EB Method is Applicable

The applicability of the EB Method to a particular project or facility depends on the type of analysis being performed and the type of future project work that is anticipated. If the analysis is being performed to assess the expected average crash frequency of a specific highway facility, but is not part of the analysis of a planned future project, then the EB Method should be applied. If a future project is being planned, then the nature of that future project should be considered in deciding whether to apply the EB Method.

The EB Method should be applied for the analyses involving the following future project types:

- Sites at which the roadway geometrics and traffic control are not being changed (e.g., the "do-nothing" alternative);
- Projects in which the roadway cross section is modified but the basic number of through lanes remains the same (This would include, for example, projects for which lanes or shoulders were widened or the roadside was improved, but the roadway remained a rural two-lane highway);
- Projects in which minor changes in alignment are made, such as flattening individual horizontal curves while leaving most of the alignment intact;
- Projects in which a passing lane or a short four-lane section is added to a rural two-lane highway to increase passing opportunities; and,
- Any combination of the above improvements.

The EB Method is not applicable to the following types of improvements:

- Projects in which a new alignment is developed for a substantial proportion of the project length.
- Intersections at which the basic number of intersection legs or type of traffic control is changed as part of a project.

The reason that the EB Method is not used for these project types is that the observed accident data for a previous time period is not necessarily indicative of the accident experience that is likely to occur in the future, after such a major geometric improvement. Since, for these project types, the observed crash frequency for the existing design is not relevant to estimation of the future crash frequencies for the site, the EB Method is not needed and should not be applied. If the EB Method is applied to individual roadway segments and intersections, and some roadway segments and intersections within the project limits will not be affected by the major geometric improvement, it is acceptable to apply the EB Method to those unaffected segments and intersections.

If the EB Method is not applicable, do not proceed to the remaining steps. Instead, follow the procedure described in the Applications section of the applicable Part C Chapter.

A.2.2 Determine Whether Observed Crash frequency Data are Available for the Project or Facility and, If So, Obtain Those Data

If the EB Method is applicable, it should be determined whether observed crash frequency data are available for then project or facility of interest directly from the jurisdiction's accident record system or indirectly from another source. At least two years of observed crash frequency data are desirable to apply the EB Method. The best results in applying the EB Method will be obtained if observed crash frequency data are available for each individual roadway segment and intersection that makes up the project of interest. The EB Method applicable to this situation is presented in Section A.2.4. Criteria for assigning accidents to individual roadway segments and intersections are presented in Section A.2.3. If observed crash frequency data are not available for individual roadway segments and intersections, the EB Method can still be applied if observed crash frequency data are available for the project or facility as a whole. The EB Method applicable to this situation is presented in Section A.2.5.

If appropriate crash frequency data are not available, do not proceed to the remaining steps. Instead, follow the procedure described in the Applications section of the applicable Part C Chapter.

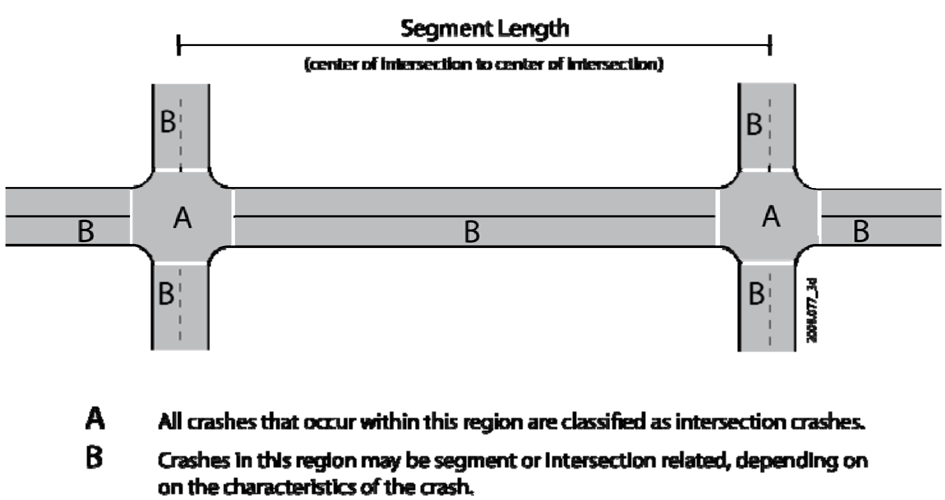
A.2.3 Assign accidents to individual roadway segments and intersections for use in the EB Method

The Part C predictive method has been developed to estimate crash frequencies separately for intersections and roadways segments. In the site-specific EB Method presented in section A.2.4, observed crashes are combined with the predictive model estimate of crash frequency to provide a more reliable estimate of the expected average crash frequency of a particular site. In Step 6 of the predictive method, if the site-specific EB Method is applicable, observed crashes are assigned to each individual site identified within the facility of interest. Because the predictive models estimate crashes separately for intersections and roadway segments, which may physically overall in some cases, observed crashes are differentiated and assigned as either intersection related crashes or roadway segment related crashes.

Intersection crashes include crashes that occur at an intersection (i.e., within the curb limits) and crashes that occur on the intersection legs and are intersection-related. All crashes that are not classified as intersection or intersection-related crashes are considered to be roadway segment crashes. Exhibit A-4 illustrates the method used to assign crashes to roadway segments or intersections. As shown:

- All crashes that occur within the curbline limits of an intersection (Region A in the exhibit) are assigned to that intersection.
- Crashes that occur outside the curbline limits of an intersection (Region B in the exhibit) are assigned to either the roadway segment on which they occur or an intersection, depending on their characteristics. Crashes that are classified on the crash report as intersection-related or have characteristics consistent with an intersection-related crash are assigned to the intersection to which they are related; such crashes would include rear-end collisions related to queues on an intersection approach. Crashes that occur between intersections and are not related to an intersection, such as collisions related to turning maneuvers at driveways, are assigned to the roadway segment on which they occur.

Exhibit A-4: Definition of Roadway Segments and Intersections



In some jurisdictions, crash reports include a field that allows the reporting officer to designate the crash as intersection-related. When this field is available on the crash reports, crashes should be assigned to the intersection or the segment based on the way the officer marked the field on the report. In jurisdictions where there is not a field on the crash report that allows the officer to designate crashes as intersection-related, the characteristics of the crash may be considered to make a judgment as to whether the crash should be assigned to the intersection or the segment. Other fields on the report, such as collision type, number of vehicles involved, contributing circumstances, weather condition, pavement condition, traffic control malfunction, and sequence of events can provide helpful information in making this determination.

If the officer's narrative and crash diagram are available to the user, they can also assist in making the determination. The following crash characteristics may indicate that the crash was related to the intersection:

- Rear-end collision in which both vehicles were going straight approaching an intersection or in which one vehicle was going straight and struck a stopped vehicle
- Collision in which the report indicates a signal malfunction or improper traffic control at the intersection

The following crash characteristics may indicate that the crash was not related to the intersection and should be assigned to the segment on which it occurred:

- Collision related to a driveway or involving a turning movement not at an intersection
- Single-vehicle run-off-road or fixed object collision in which pavement surface condition was marked as wet or icy and identified as a contributing factor

These examples are provided as guidance when an "intersection-related" field is not available on the crash report; they are not strict rules for assigning crashes. Information on the crash report should be considered to help make the determination, which will rely on judgment. The information needed for classifying crashes is whether each crash is, or is not, related to an intersection. The consideration of crash type data is presented here only as an example of one approach to making this determination.

Using these guidelines, the roadway segment predictive models estimate the average frequency of crashes that would occur on the roadway if no intersection were present. The intersection predictive models estimate the average frequency of additional crashes that occur because of the presence of an intersection.

A.2.4 Apply the Site-Specific EB Method

Equations A-4 and A-5 are used directly to estimate the expected crash frequency for a specific site by combining the predictive model estimate with observed crash frequency. The value of $N_{expected}$ from Equation A-4 represents the expected crash frequency for the same time period represented by the predicted and observed accident frequencies. $N_{predicted}$, $N_{observed}$, and $N_{expected}$ all represent either total crashes or a specific severity level or collision type of interest. The expected average crash frequency considering both the predictive model estimate and observed accident frequencies for an individual roadway segment or intersection is computed as:

$$N_{expected} = w \times N_{predicted} + (1 - w) \times N_{observed} \tag{A-4}$$

$$w = \frac{1}{1 + k \times (\sum_{\substack{\text{all study}\\ \text{years}}} N_{predicted})} \tag{A-5}$$

Where,

$N_{expected}$ = estimate of expected average crashes frequency for the study period.

$N_{predicted}$ = predictive model estimate of average crash frequency predicted for the study period under the given conditions.

$N_{observed}$ = observed crash frequency at the site over the study period.

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

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

When observed crash data by severity level is not available, the estimate of expected average crash frequency for fatal-and-injury and property-damage-only crashes is calculated by applying the proportion of predicted average crash frequency by severity level ($N_{predicted(FI)}/N_{predicted(TOTAL)}$ and $N_{predicted(PDO)}/N_{predicted(TOTAL)}$) to the total expected average crash frequency from Equation A-4.

Equation A-5 shows an inverse relationship between the overdispersion parameter k, and the weight, w. This implies that when a model with little overdispersion is available, more reliance will be placed on the predictive model estimate, $N_{predicted}$, and less reliance on the observed crash frequency, $N_{observed}$. The opposite is also the case; when a model with substantial overdispersion is available, less reliance will be placed on the predictive model estimate, $N_{predicted}$,, and more reliance on the observed crash frequency, $N_{observed}$.

It is important to note in Equation A-5 that, as $N_{predicted}$ increases, there is less weight placed on $N_{predicted}$ and more on $N_{observed}$. This might seem counterintuitive at first. However, this implies that for longer sites and for longer study periods, there are more opportunities for crashes to occur. Thus, the observed crash history is likely to be more meaningful and the model prediction less important. So, as $N_{predicted}$ increases, the EB Method places more weight on the number of crashes that actually occur, $N_{observed}$. When few crashes are predicted, the observed crash frequency, $N_{observed}$, is not likely to be meaningful, in statistical terms, so greater reliance is placed on the predicted crash frequency, $N_{predicted}$.

The values of the overdispersion parameters, k, for the Safety Performance Functions used in the predictive models are presented with each SPF in sections 10.6, 11.6 and 12.6.

Since application of the EB Method requires use of an overdispersion parameter, it cannot be applied to portions of the prediction method where no overdispersion parameter is available. For example, vehicle-pedestrian and vehicle-bicycle collisions are estimated in portions of Chapter 12 from adjustment factors rather than from models and should, therefore, be excluded from the computations with the EB Method. Chapter 12 uses multiple models with different overdispersion parameters in safety predictions for any specific roadway segment or intersection. Where observed crash data are aggregated so that the corresponding value of predicted crash frequency is determined as the sum of the results from multiple predictive models with differing overdispersion parameters, the project-level EB Method presented in Section A.2.5 should be applied rather than the site-specific method presented here.

Chapters 10, 11, and 12 each present worksheets that can be used to apply the site-specific EB Method as presented in this section.

Section A.2.6 explains how to update $N_{expected}$ to a future time period, such as the time period when a proposed future project will be implemented. This procedure is only applicable if the conditions of the proposed project will not be substantially different from the roadway conditions during which the observed crash data was collected.

A.2.5 Apply the Project-Level EB Method

HSM users may not always have location specific information for observed accident data for the individual roadway segments and intersections that make up a facility or project of interest. Alternative procedures are available where observed crash frequency data are aggregated across several sites (e.g., for an entire facility or project). This requires a more complex EB Method for two reasons. First, the overdispersion parameter, k, in the denominator of Equation A-5 is not uniquely defined, because estimate of crash frequency from two or more predictive models with different overdispersion parameters are combined. Second, it cannot be assumed, as is normally done, that the expected average crash frequency for different site types are statistically correlated with one another. Rather, an estimate of expected average crash frequency should be computed based on the assumption that the various roadway segments and intersections are statistically independent (r=0) and on the alternative assumption that they are perfectly correlated (r=1). The expected average crash frequency is then estimated as the average of the estimates for r=0 and r=1.

The following equations implement this approach, summing the first three terms, which represent the three roadway-segment-related accident types, over the five types of roadway segments considered in the (2U, 3T, 4U, 4D, 5T) and the last two terms, which represent the two intersection-related accident types, over the four types of intersections (3ST, 3SG, 4ST, 4SG):

$$N_{predicted\ (TOTAL)} = \sum_{j=1}^{5} N_{predicted\ rmj} + \sum_{j=1}^{5} N_{predicted\ rsj} + \sum_{j=1}^{5} N_{predicted\ rdj} + \sum_{j=1}^{4} N_{predicted\ imj} + \sum_{j=1}^{4} N_{predicted\ isj} \quad \text{(A-6)}$$

$$N_{observed\ (TOTAL)} = \sum_{j=1}^{5} N_{observed\ rmj} + \sum_{j=1}^{5} N_{observed\ rsj} + \sum_{j=1}^{5} N_{observed\ rdj} + \sum_{j=1}^{4} N_{observed\ imj} + \sum_{j=1}^{4} N_{observed\ isj} \quad \text{(A-7)}$$

$$N_{predicted\ w0} = \sum_{j=1}^{5} k_{rmj} N_{rmj}^{2} + \sum_{j=1}^{5} k_{rsj} N_{rsj}^{2} + \sum_{j=1}^{5} k_{rdj} N_{rdj}^{2} + \sum_{j=1}^{4} k_{imj} N_{imj}^{2} + \sum_{j=1}^{4} k_{isj} N_{isj}^{2} \quad \text{(A-8)}$$

$$N_{predicted\ w1} = \sum_{j=1}^{5} \sqrt{k_{rmj} N_{rmj}} + \sum_{j=1}^{5} \sqrt{k_{rsj} N_{isj}} + \sum_{j=1}^{5} \sqrt{k_{rdj} N_{rdj}} + \sum_{j=1}^{4} \sqrt{k_{imj} N_{imj}} + \sum_{j=1}^{4} \sqrt{k_{isj} N_{isj}} \quad \text{(A-9)}$$

$$w_0 = \frac{1}{1 + \frac{N_{predicted\ w0}}{N_{predicted\ (TOTAL)}}} \tag{A-10}$$

$$N_0 = w_0\ N_{predicted\ (TOTAL)} + (1 - w_0)\ N_{observed\ (TOTAL)} \tag{A-11}$$

$$w_1 = \frac{1}{1 + \frac{N_{predicted\ w1}}{N_{predicted\ (TOTAL)}}} \tag{A-12}$$

$$N_1 = w_1\ N_{predicted\ (TOTAL)} + (1 - w_1)\ N_{Observed\ (TOTAL)} \tag{A-13}$$

$$N_{expected/comb} = \frac{N_0 + N_1}{2} \tag{A-14}$$

Where:

$N_{predicted\ (TOTAL)}$ = predicted number of total accidents for the facility or project of interest during the same period for which accidents were observed;

$N_{predicted\ rmj}$ = Predicted number of multiple-vehicle nondriveway collisions for roadway segments of type j, j = 1..., 5, during the same period for which accidents were observed;

$N_{predicted\ rsj}$ = Predicted number of single-vehicle collisions for roadway segments of type j, during the same period for which accidents were observed;

$N_{predicted\ rdj}$ = Predicted number of multiple-vehicle driveway-related collisions for roadway segments of type j, during the same period for which accidents were observed;

$N_{predicted\ imj}$ = Predicted number of multiple-vehicle collisions for intersections of type j, j = 1..., 4, during the same period for which accidents were observed;

$N_{predicted\ isj}$ = Predicted number of single-vehicle collisions for intersections of type j, during the same period for which accidents were observed;

$N_{observed\ (TOTAL)}$ = Observed number of total accidents for the facility or project of interest;

$N_{observed\ rmj}$ = Observed number of multiple-vehicle nondriveway collisions for roadway segments of type j;

$N_{observed\ rsj}$ = Observed number of single-vehicle collisions for roadway segments of type j;

$N_{observed\ rdj}$ = Observed number of driveway-related collisions for roadway segments of type j;

$N_{observed\ imj}$ = Observed number of multiple-vehicle collisions for intersections of type j;

$N_{observed\ isj}$ = Observed number of single-vehicle collisions for intersections of type j;

$N_{predicted\ w0}$ = Predicted number of total accidents during the same period for which accidents were observed under the assumption that accident frequencies for different roadway elements are statistically independent ($\rho = 0$);

k_{rmj} = Overdispersion parameter for multiple-vehicle nondriveway collisions for roadway segments of type j;

k_{rsj} = Overdispersion parameter for single-vehicle collisions for roadway segments of type j;

k_{rdj} = Overdispersion parameter for driveway-related collisions for roadway segments of type j;

k_{imj} = Overdispersion parameter for multiple-vehicle collisions for intersections of type j;

k_{isj} = Overdispersion parameter for single-vehicle collisions for intersections of type j;

$N_{predicted\ w1}$ = Predicted number of total accidents under the assumption that accident frequencies for different roadway elements are perfectly correlated ($\rho = 1$);

w_0 = weight placed on predicted crash frequency under the assumption that accident frequencies for different roadway elements are statistically independent (r=0);

w_1 = weight placed on predicted crash frequency under the assumption that accident frequencies for different roadway elements are perfectly correlated (r=1);

N_0 = expected crash frequency based on the assumption that different roadway elements are statistically independent (r=0);

N_1 = expected crash frequency based on the assumption that different roadway elements are perfectly correlated (r=1); and

$N_{expected/comb}$ = expected average crash frequency of combined sites including two or more roadway segments or intersections.

All of the accident terms for roadway segments and intersections presented in Equations A-6 through A-9 are used for analysis of urban and suburban arterials (*Chapter 12*). The predictive models for rural two-lane roads and multilane highways (*Chapters 10* and *11*) are based on the site type and not on the collision type; therefore, only one of the predicted accident terms for roadway segments ($N_{predicted\ rmj}$, $N_{predicted\ rsj}$, $N_{predicted\ rdj}$), one of the predicted accident terms for intersections ($N_{predicted\ imj}$, $N_{predicted\ isj}$), one of the observed accident terms for roadway segments ($N_{observed\ rmj}$, $N_{observed\ rsj}$, $N_{observed\ rdj}$), and one of the observed accident terms for intersections ($N_{observed\ imj}$, $N_{observed\ isj}$) is used. For rural two-lane roads and multilane highways, it is recommended that the multiple-vehicle collision terms (with subscripts rmj and imj) be used to represent total accidents; the remaining unneeded terms can be set to zero.

Chapters 10, 11, and 12 each present worksheets that can be used to apply the project-level EB Method as presented in this section.

The value of $N_{expected/comb}$ from Equation A-14 represents the expected average crash frequency for the same time period represented by the predicted and observed accident frequencies. The estimate of expected average crash frequency of combined sites for fatal-and-injury and property-damage-only crashes is calculated by multiplying the proportion of predicted average crash frequency by severity level ($N_{predicted(FI)}/N_{predicted(TOTAL)}$ and $N_{predicted(PDO)}/N_{predicted(TOTAL)}$) to the total expected average crash frequency of combined sites from Equation A-14. Section A.2.6 explains how to update $N_{p/comb}$ to a future time period, such as the time period when a proposed future project will be implemented.

A.2.6 Adjust the Estimated Value of Expected Average Crash frequency to a Future Time Period, If Appropriate

The value of the expected average crash frequency ($N_{expected}$) from Equation A-4 or $N_{expected/comb}$ from Equation A-14 represents the expected average crash frequency for a given roadway segment or intersection (or project, for $N_{expected/comb}$)during the before period. To obtain an estimate of expected average crash frequency in a future period (the after period), the estimate is corrected for (1) any difference in the duration of the before and after periods; (2) any growth or decline in AADTs between the before and after periods; and (3) any changes in geometric design or traffic control features between the before and after periods that affect the values of the AMFs for the roadway segment or intersection. The expected average crash frequency for a roadway segment or intersection in the after period can be estimated as:

$$N_f = N_p \left(\frac{N_{bf}}{N_{bp}}\right)\left(\frac{AMF_{1f}}{AMF_{1p}}\right)\left(\frac{AMF_{2f}}{AMF_{2p}}\right)\cdots\left(\frac{AMF_{nf}}{AMF_{np}}\right) \tag{A-15}$$

Where,

N_f = expected average crash frequency during the future time period for which accidents are being forecast for the segment or intersection in question (i.e., the after period);

N_p = expected average crash frequency for the past time period for which observed accident history data were available (i.e., the before period);

N_{bf} = number of accidents forecast by the SPF using the future AADT data, the specified nominal values for geometric parameters, and—in the case of a roadway segment—the actual length of the segment;

N_{bp} = number of accidents forecast by the SPF using the past AADT data, the specified nominal values for geometric parameters, and—in the case of a roadway segment—the actual length of the segment;

AMF_{nf} = value of the nth AMF for the geometric conditions planned for the future (i.e., proposed) design; and

AMF_{np} = value of the nth AMF for the geometric conditions for the past (i.e., existing) design.

Because of the form of the SPFs for roadway segments, if the length of the roadway segments are not changed, the ratio N_{bf} / N_{bp} is the same as the ratio of the traffic volumes, $AADT_f$ / $AADT_p$. However, for intersections, the ratio N_{bf} / N_{bp} is evaluated explicitly with the SPFs because the intersection SPFs incorporate separate major- and minor-road AADT terms with differing coefficients. In applying Equation A-15, the values of N_{bp}, N_{bf}, AMF_{np}, and AMF_{nf} should be based on the average AADTs during the entire before or after period, respectively.

In projects that involve roadway realignment, if only a small portion of the roadway is realigned, the ratio N_{bf} / N_{bp} should be determined so that its value reflects the change in roadway length. In projects that involve extensive roadway realignment, the EB Method may not be applicable (see discussion in Section A.2.1).

Equation A-15 is applied to total average crash frequency. The expected future average crash frequencies by severity level should also be determined by multiplying the expected average crash frequency from the before period for each severity level by the ratio N_f / N_p.

In the case of minor changes in roadway alignment (i.e., flattening a horizontal curve), the length of an analysis segment may change from the past to the future time period, and this would be reflected in the values of N_{bp} and N_{bf}.

Equation A-15 can also be applied in cases for which only facility- or project-level data are available for observed crash frequencies. In this situation, $N_{expected/comb}$ should be used instead of $N_{expected}$ in the equation.

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PART D— ACCIDENT MODIFICATION FACTORS

INTRODUCTION AND APPLICATIONS GUIDANCE

D.1. Purpose of Part D D-1
D.2. Relationship to the Project Development Process D-1
D.3. Relationship to Parts A, B, and C of the Highway Safety Manual D-2
D.4. Guide to Applying Part D D-3
D.4.1. Categories of Information D-3
D.4.2. Standard Error and Notation Accompanying AMFs D-4
D.4.3. Terminology D-6
D.4.4. Application of AMFs to Estimate Crash Frequency D-6
D.4.5. Considerations when Applying AMFs to Estimate Crash Frequency D-7
D.5. Development of AMFs in Part D D-8
D.5.1. Literature Review Procedure D-8
D.5.2. Inclusion Process D-9
D.5.3. Expert Panel Review D-9
D.6. Conclusion D-9

EXHIBITS

Exhibit D-1: Part D Relation to the Project Development Process D-2
Exhibit D-2: Categories of Information in Part D D-4
Exhibit D-3: Precision and Accuracy D-5

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PART D INTRODUCTION AND APPLICATIONS GUIDANCE

D.1. PURPOSE OF PART D

Part D presents information regarding the effects of various safety treatments (i.e. countermeasures). This information is used to estimate how effective a countermeasure or set of countermeasures will be in reducing crashes at a specific location. The effects of treatments, geometric characteristics, and operational characteristics of a location can be quantified as an accident modification factor (AMF) or described by trends (e.g. appears to cause a decrease in total crashes). The level of information (e.g. an AMF, a known trend, unknown effect) depends on the quality and quantity of research completed regarding the treatment's effect on crash frequency. The research that developed the HSM established a screening process and convened a series of expert panels to determine which safety evaluation results are considered sufficiently reliable for inclusion in the HSM (see Section D.5 for more information). *Part D* presents the information that passed the screening test and/or met expert panel approval; this information is organized in the following chapters:

Part D presents treatments (i.e. countermeasures) with known Accident Modification Factors, safety trends, or unknown effects. The AMFs can be used to estimate the change in number or severity of crashes as a result of implementing a countermeasure

- Chapter 13 - Roadway Segments
- Chapter 14 - Intersections
- Chapter 15 - Interchanges
- Chapter 16 - Special Facilities and Geometric Situations
- Chapter 17 - Road Networks

Accident modification factors presented in *Part D* can also be used in the methods and calculations shown in *Chapter 6 Select Countermeasures*, and *Chapter 7 Economic Appraisal*. These methods are used to calculate the potential crash reduction due to a treatment, convert the crash reduction to a monetary value and compare the monetary benefits of reduced crashes to the monetary cost of implementing the countermeasure(s), as well as to the cost of other associated impacts (e.g., delay, right-of-way). Some accident modification factors may also be used in the predictive method presented in *Part C.*

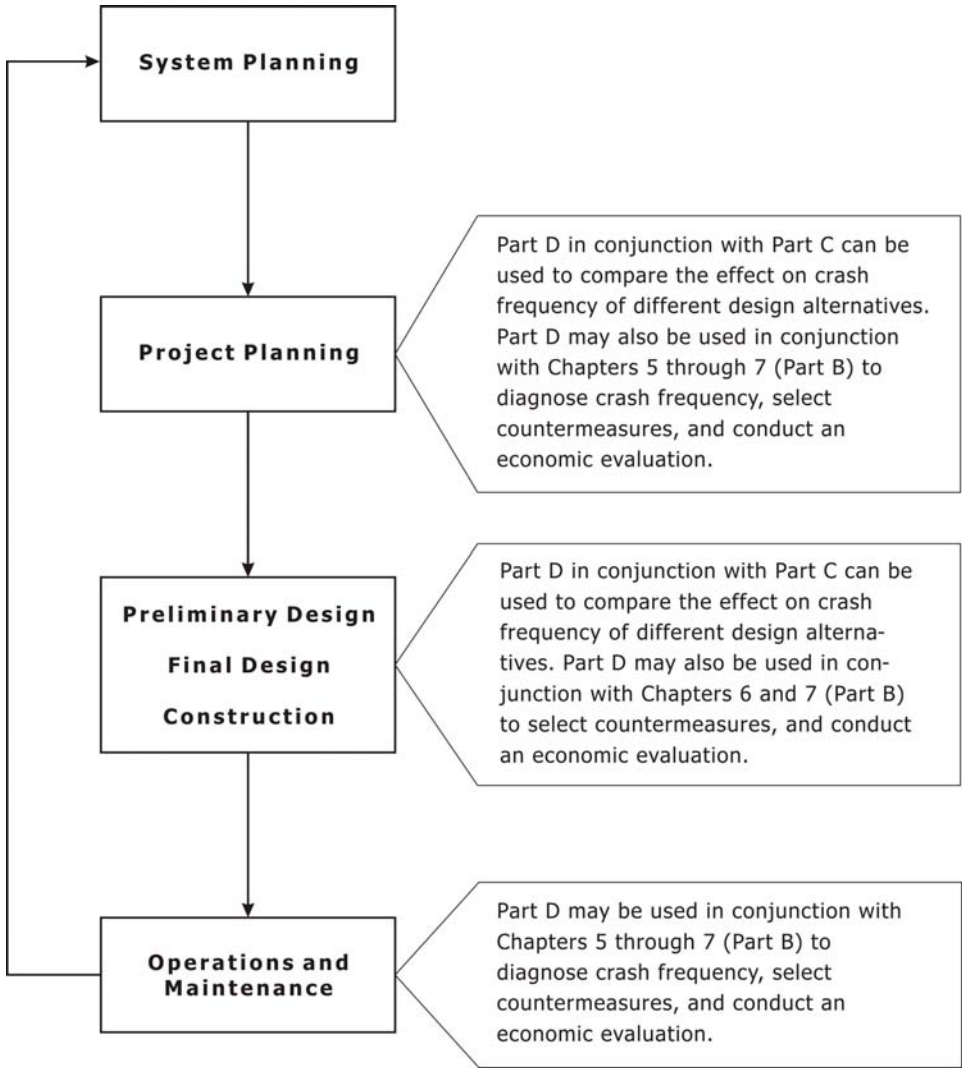
D.2. RELATIONSHIP TO THE PROJECT DEVELOPMENT PROCESS

The accident modification factors in *Part D* are used to estimate the change in crashes as a result of implementing a countermeasure(s). Applying the *Part D* material to estimate change in crashes often occurs within operations and maintenance activities. It can also occur in projects in which the existing roadway network is assessed and modifications are identified, designed and implemented with the intent of improving the performance of the facility from a capacity, safety, or multimodal perspective.

Chapter 1 provides an overview of the project development process considered in the HSM.

Exhibit D-1 illustrates the relationship between *Part D* and the project development process. As discussed in *Chapter 1*, the project development process is the framework being used in the HSM to relate safety analysis to activities within planning, design, construction, operations and maintenance.

Exhibit D-1: Part D Relation to the Project Development Process



D.3. RELATIONSHIP TO PARTS A, B, AND C OF THE HIGHWAY SAFETY MANUAL

Part A Chapter 3 provides fundamental information about Accident Modification Factors

Part A of the HSM provides introductory and fundamental knowledge needed for applying the HSM. It introduces concepts such as human factors, how to count crashes, data needs, regression-to–the-mean, countermeasures, and accident modification factors. The material in *Part A* provides valuable context regarding how to apply different parts of the HSM and how to use the HSM effectively in typical project activities or within established processes. Prior to using the information in *Part D*, an understanding of the material regarding AMFs presented in *Part A, Chapter 3 Fundamentals* is recommended, as well as an understanding of the information presented in the D.4 Guide to Applying *Part D* section below.

Part B presents the six basic components of a roadway safety management process as related to transportation engineering and planning. The material is useful for monitoring, improving and maintaining safety on an existing roadway network. Applying the methods and information presented in *Part B* creates an awareness of sites most likely to experience crash reductions with the implementation of improvements, the type of improvement most likely to yield benefits, an estimate of the benefit and cost of improvement(s), and an assessment of an improvement's effectiveness. The information presented in *Part D* should be used in conjunction with the information presented in *Chapter 6 Select Countermeasures* and *Chapter 7 Economic Appraisal*.

Part C introduces techniques for predicting crashes on two-lane rural highways, multilane rural highways, and urban and suburban arterials. This material is particularly useful for estimating expected average crash frequency of new facilities under design, and extensive re-design of existing facilities. It facilitates a proactive approach to considering safety before crashes occur. Some *Part D* AMFs are included in *Part C* and for use with specific Safety Performance Functions (SPFs). Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7.

D.4. GUIDE TO APPLYING PART D

The notations and terms cited and defined in the subsections below are used to indicate the level of knowledge regarding the effects on crash frequency of the various geometric and operational elements presented throughout *Part D*.

Section D.4 provides an explanation of the type of information associated with each treatment in Part D.

The following subsections explain useful information about:

- How the AMFs are categorized and organized in each chapter;
- The notation used to convey the reliability of each AMF;
- Terminology used in each chapter;
- Application of AMFs; and,
- Considerations when Applying AMFs.

To effectively use the accident modification factors in *Part D*, it is important to understand the notations and terminology, as well as the situation in which the countermeasure associated with the AMF is going to be applied. Understanding these items will increase the likelihood of success when implementing countermeasures.

D.4.1. Categories of Information

At the beginning of each section of *Part D*, treatments are summarized in tables according to the category of information available (i.e. accident modification factors, or evidence of trends). These tables serve as a quick reference of the information available related to a specific treatment. Exhibit D-2 summarizes how the information is categorized.

Exhibit D-2: Categories of Information in Part D

Symbol Used in Part D Summary Tables	Available Information
✓	• AMFs are available (i.e. sufficient quantitative information is available to determine a reliable AMF). • The AMFs and standard errors passed the screening test to be included in the HSM.
T	• There is some evidence of the effects on crash frequency, although insufficient quantitative information is available to determine a reliable AMF. • In some instances the quantitative information is sufficient to identify a known trend or apparent trend in crash frequency and/or user behavior; but not sufficient to apply in estimating changes in crash frequency. • Published documentation regarding the treatment was not sufficiently reliable to present an AMF in this edition of the HSM.
-	• Quantitative information about the effects on crash frequency is not available for this edition of the HSM. • Published documentation did not include quantitative information regarding the effects on crash frequency of the treatment. • A list of these treatments is presented in the appendices to each chapter.

For those treatments with AMFs, the AMFs and standard errors are provided in tables. When available each table supplies the specific treatment, road type or intersection type, setting (i.e., rural, urban, suburban), traffic volumes, accident type and severity to which the AMF can be applied.

The appendix to each chapter presents those treatments with known trends and unknown effects. For those treatments without AMFs, but which present a trend in crashes or user behavior, it is reasonable to apply them in situations where there are indications that they may be effective in reducing crash frequency. A treatment without an AMF indicates that there is an opportunity to apply and study the effects of the treatments; thereby adding to the current understanding of the treatment's effect on crashes. See *Chapter 9 Safety Effectiveness Evaluation* for more information regarding methods to assess the effectiveness of a treatment.

D.4.2. Standard Error and Notation Accompanying AMFs

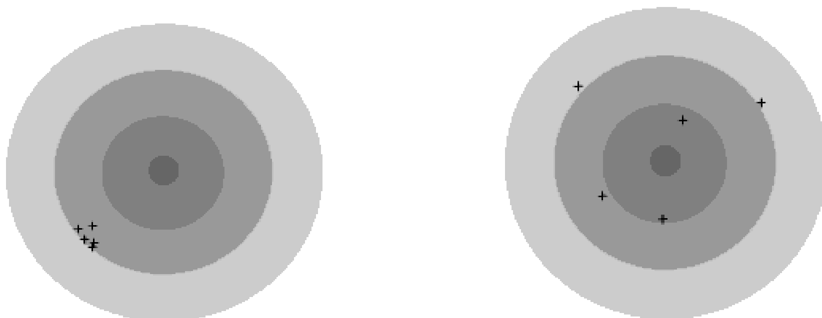
The standard deviation of the mean is the standard error. The standard error indicates the precision of an estimated AMF.

In general, the standard deviation indicates the precision of a set of repeated measurements, in other words, precision is the degree to which repeated measurements are close to each other. When calculating for example the mean of a set of measurements, then the mean itself has a standard deviation; the standard deviation of the mean is called the *standard error*. In *Part D*, the standard error indicates the precision of an estimated AMF. *Accuracy* is a measure of the proximity of an estimate to its actual or true value. The difference between the average of repeated measurements and its true value is an estimate of its bias. The true value of an AMF is seldom known but steps can be taken to minimize the bias associated with its estimate (e.g. by using an appropriate statistical approach, applying an EB adjustment for regression-to-the-mean bias). Accuracy and precision estimates are generally difficult to separate mathematically because precision is to some degree built into accuracy. Standard error in *Part D* is important because more accurate and precise AMFs lead to more cost effective decisions.

Exhibit D-3 illustrates the concepts of precision and accuracy. If the estimates (the + signs) form a tight cluster, the estimates are precise. However, if the center of that cluster is not the bull's-eye, then the estimates are precise but not accurate. If the estimates are scattered and do not form a tight cluster they are neither precise nor accurate.

Standard error reflects accuracy and precision. The lower the standard error, the more effective the treatment.

Exhibit D-3: Precision and Accuracy



Precise but not Accurate **Neither Precise nor Accurate**

Some AMFs in *Part D* have a standard error associated with them. Standard errors in *Part D* with values less than 0.1 are presented to two decimal places, standard errors greater than 0.1 have been rounded to the nearest 0.1 and are presented to one decimal place. The most reliable (i.e., valid) AMFs have a standard error of 0.1 or less, and are indicated with **bold** font. Reliability indicates that the AMF is unlikely to change substantially with new research. Less reliable AMFs have standard errors of 0.2 or 0.3 and are indicated with *italic* font. All quantitative standard errors presented with AMFs in *Part D* are less than or equal to 0.3.

To emphasize the meaning and awareness of each standard error, some AMFs in *Part D* are accompanied by a superscript. These superscripts have specific meanings:

The AMFs are summarized in Part D with additional notation about standard error. This information emphasizes the reliability of the AMF and the stability of the treatment.

- *: The asterisk indicates that the AMF value itself is within the range 0.90 to 1.10, but that the confidence interval defined by the AMF ± two times the standard error may contain the value 1.0. This is important to note since a treatment with such an AMF could potentially result in (a) a reduction in crashes (safety benefit), (b) no change, or (c) an increase in crashes (safety disbenefit). These AMFs should be used with caution.

- ^: The carat indicates that the AMF value itself is within the range 0.90 to 1.10 but that the lower or upper end of the confidence interval (defined by the AMF ± two times the standard error) may be exactly at 1.0. This is important to note since a treatment with such an AMF may result in no change in safety. These AMFs should be used with caution.

- o: The degree symbol "o" indicates that the standard error has not been quantified for the AMF; therefore, the potential error inherent in the value is not known. This usually occurs when the factor is included as an equation.

- +: The plus sign indicates that the AMF is the result of combining AMFs from multiple studies.

- ?: The question mark indicates AMFs that have the opposite effects on different crash types or crash severities. For example, a treatment may increase rear-end crashes but decrease angle crashes. Or a treatment may reduce fatal crashes but increase property damage only (PDO) crashes.

Understanding the meanings of the superscripts and the standard error of an AMF will build familiarity with the reliability and stability that can be expected from each treatment. An AMF with a relatively high standard error does not mean that it should not be used; it means that the AMF should be used with the awareness of the range of results that could be obtained. Applying these treatments is also an opportunity to study the effectiveness of the treatment after implementation and add to the current information available regarding the treatment's effectiveness (see *Chapter 9 Safety Effectiveness Evaluation* for more information).

D.4.3. Terminology

Described below are some of the key words used in *Part D* to describe the AMF values or information provided. Key words to understand are:

- Unspecified: In some cases, AMF tables include some characteristics that are "unspecified". This indicates that the research did not clearly state the road type or intersection type, setting, or traffic volumes of the study.
- Injury: In *Part D* of the HSM, injury accidents include fatal accidents unless otherwise noted.
- All Settings: In some instances, research presented aggregated results for multiple settings (e.g. urban and suburban signalized intersections); the same level of information is reflected in the HSM.
- Insufficient or No Quantitative Information Available: Indicates that the documentation reviewed for the HSM did not contain quantitative information that passed the screening test for inclusion in the HSM. It doesn't mean that such documentation does not exist.

Section D.4.4 provides an overview of the ways in which AMFs are applied to estimate crash frequency.

D.4.4. Application of AMFs to Estimate Crash Frequency

As discussed above, AMFs are used to estimate crash frequency or the change in crashes due to a treatment. There are multiple approaches to calculating an estimated number of crashes using an AMF. These include:

1. Applying the AMF to an expected number of crashes calculated using a calibrated safety performance function and Empirical Bayes to account for regression-to-the-mean bias; or
2. Applying the AMF to an expected number of crashes calculated using a calibrated safety performance function; or
3. Applying the AMF to historic crash count data.

Of the three ways to apply AMFs, listed above, the first approach produces the most reliable results. The second approach is the second most reliable and the third approach is the approach used if a safety performance function is not available to calculate the expected number of crashes. Additional details regarding safety performance functions, expected number of crashes, regression to the mean, and empirical Bayes methodology are discussed in *Chapter 3 Fundamentals.* The specific step-by-step process for calculating an estimated change in crashes using approach number 1 or number 2 listed above is presented in *Chapter 7 Economic Appraisal*.

AMFs may be presented in *Part D* chapters as numerical values, equations, graphs, or a combination of these. AMFs may be applied under any of the following scenarios:

1. Direct application of a numerical AMF value and standard error obtained from a table: The AMF is multiplied directly with the base crash frequency to estimate the crash frequency and standard error with the treatment in place.

2. Direct application of an AMF value obtained from a graph: The AMF value is obtained from a graph (which presents a range for a given treatment) and is subsequently multiplied directly with the base crash frequency to estimate the crash frequency with the treatment in place. No standard error is provided for graphical AMFs.

3. Direct application of an AMF value obtained from an equation: The AMF value is calculated from an equation (which is a function of a treatment range) and is subsequently multiplied with the base crash frequency to estimate the crash frequency with the treatment in place. No standard error is provided for AMFs calculated using equations.

4. Multiplication of multiple AMF values from a table, graph, or equation: Multiple AMFs are obtained or calculated from a table, graph, or equation and are subsequently multiplied. This procedure is followed when more than one treatment is being considered for implementation at the same time at a given location. See *Chapter 3* for guidance about the independence assumption when applying multiple AMFs.

5. Division of two AMF values from a table, graph, or equation: Two AMFs are obtained or calculated from a table, graph, or equation and are subsequently divided. This procedure is followed when one of the AMFs (denominator) represents an initial condition (not equal to the AMF base condition, and therefore not equal to an AMF value of 1.0) and the other AMF (numerator) represents the treatment condition.

6. Interpolation between two numerical AMF values from a table: An unknown AMF value is calculated as the interpolation of two known AMF values.

The graybox examples presented throughout *Part D* chapters illustrate the application of AMFs under these scenarios.

D.4.5. Considerations when Applying AMFs to Estimate Crash Frequency

Section D.4.5 presents considerations prior to the application of AMFs.

Standard errors have been provided for many AMFs in Part D. Where standard errors are available, these should be used to calculate the confidence interval of the projected change in crash frequency. Section 3.5.3 in *Chapter 3 Fundamentals* provides additional information regarding the application of standard errors.

AMFs are multiplicative when a treatment can be applied in multiple increments, or when multiple AMFs are applied simultaneously. When applying multiple AMFs, engineering judgment should be used to assess the interrelationship and/or independence of individual treatments being considered for implementation. Section 3.5.3 in *Chapter 3 Fundamentals* provides additional information regarding the application of multiplicative AMFs.

AMFs may be divided when the existing condition corresponds to an AMF value (other than the base value of 1.00) and the treatment condition corresponds to another AMF value. In this case a ratio of the AMFs may be calculated to account for the variation between the existing condition and the treatment condition. Section 3.5.3 in *Chapter 3 Fundamentals* provides additional information regarding the application of AMF ratios.

Section D.5 provides an overview of how the AMFs were developed for the HSM

D.5. DEVELOPMENT OF AMFS IN PART D

The following sections provide an overview of the Literature Review Procedure, Inclusion Process, and Expert Panel that were developed and applied while creating *Part D* of the HSM. This information provides background to the knowledge included in the HSM, and may also be useful to others in the field of transportation safety by:

- Providing a framework to review safety literature to determine the reliability of published results;
- Outlining the characteristics of safety studies that lead to more reliable results;
- Promoting higher quality evaluation of treatments to advance the knowledge of safety effects; and
- Encouraging improvements to the methods applied for the first edition by expanding and enhancing the knowledge for future editions of the HSM.

D.5.1. Literature Review Procedure

AMFs in Part D were developed through a literature review and inclusion process and through an Expert Panel review process.

The information presented in *Part D* is based on an extensive literature review of published transportation safety research mostly dated from the 1960s to June 2008.

A literature review procedure was developed to document available knowledge using a consistent approach. The procedure includes methods to calculate Accident Modification Factors (AMFs) based on published data, estimate the standard error of published or calculated AMFs, and adjust the AMFs and standard errors to account for study quality and method. The steps followed in the literature review procedure are:

1. Determine the estimate of the effect on crash frequency, user behavior, or Accident Modification Factor or Function (AMF) of a treatment based on one published study
2. Adjust the estimate to account for potential bias from regression-to-mean and/or changes in traffic volume
3. Determine the ideal standard error of the AMF
4. Apply a Method Correction Factor to ideal standard error, based on the study characteristics
5. Adjust the corrected standard error to account for bias from regression-to-mean and/or changes in traffic volume

In a limited number of cases, multiple studies provided results for the same treatment in similar conditions.

D.5.2. Inclusion Process

The AMFs from the literature review process were evaluated during the Inclusion Process, based on their standard errors, to determine whether or not they are sufficiently reliable and stable to be presented in the HSM. A standard error of 0.10 or less indicates an AMF value that is sufficiently accurate, precise, and stable. For treatments that have an AMF with a standard error of 0.1 or less, other related AMFs with standard errors of 0.2 to 0.3 may also be included to account for the effects of the same treatment on other facilities, other crash types or other severities.

Not all potentially relevant AMFs could be evaluated in the inclusion process. For example, AMFs that are expressed as functions, rather than as single values, typically do not have an explicitly defined standard error that can be considered in the inclusion process.

The basis for the inclusion process is providing sound support for selecting the most cost-effective road safety treatments. For any decision-making process, it is generally accepted that a more accurate and precise estimate is preferable to a less accurate or less precise one. The greater the accuracy of the information used to make a decision, the greater the chance that the decision is correct. A higher degree of precision is preferable to improve the chance that the decision is correct.

D.5.3. Expert Panel Review

In addition, several expert panels were formed and convened as part of the research projects that developed the predictive method presented in *Part C*. These expert panels reviewed and assessed the relevant research literature related to the effects on crash frequency of particular geometric design and traffic control features. The expert panels subsequently recommended which research results were appropriate for use as AMFs in the *Part C* predictive method. These AMFs are presented in both *Parts C* and *D*. Many, but not all, of the AMFs recommended by the expert panels meet the criteria for the literature review and inclusion processes presented in Sections D.5.1 and D.5.2. For example, AMFs that are expressed as functions, rather than as single values, often did not have explicitly defined standard errors and, therefore, did not lend themselves to formal assessment in the literature review process.

D.6. CONCLUSION

Part D presents the effects on crash frequency of various treatments, geometric design characteristics, and operational characteristics. The information in *Part D* was developed using a literature review process, an inclusion process, and a series of expert panels. These processes led to identification of AMFs, trends, or unknown effects for each treatment in *Part D*. The level of information presented in the HSM is dependent on the quality and quantity of previous research.

Part D includes all AMFs assessed with the literature review and inclusion process, including measures of their reliability and stability. These AMFs are applicable to a broad range of roadway segment and intersection facility types, not just those facility types addressed in the *Part C* predictive methods.

Some *Part D* AMFs are included in *Part C* and for use with specific SPFs. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7 of the *Part C Introduction and Applications Guidance*.

The information presented in *Part D* is used to estimate the effect on crash frequency of various treatments. It can be used in conjunction with the methodologies in *Chapter 6 Select Countermeasures* and *Chapter 7 Economic Appraisal*. When applying the AMFs in *Part D*, understanding the standard error and the corresponding potential range of results increases opportunities to make cost-effective choices. Implementing treatments with limited quantitative information presented in the HSM presents the opportunity to study the treatment's effectiveness and add to the current base of information.

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Worksheet 4B – Predicted Pedestrian and Bicycle Crashes for Urban and Suburban Arterials

(1)	(2)	(3)
Site Type	**N_{ped}**	**N_{bike}**
ROADWAY SEGMENTS		
Segment 1		
Segment 2		
Segment 3		
Segment 4		
INTERSECTIONS		
Intersection 1		
Intersection 2		
Intersection 3		
Intersection 4		
COMBINED (sum of column)		

Worksheet 4C – Project-Level EB Method Summary Results for Urban and Suburban Arterials

(1)	(2)	(3)	(4)	(5)	(6)
Crash severity level	**$N_{predicted}$**	**N_{ped}**	**N_{bike}**	**$N_{expected/comb}$ (VEHICLE)**	**$N_{expected}$**
Total	$(2)_{COMB}$ Worksheet 4A	$(2)_{COMB}$ Worksheet 4B	$(3)_{COMB}$ Worksheet 4B	$(13)_{COMB}$ Worksheet 4A	(3)+(4)+(5)
Fatal and injury (FI)	$(3)_{COMB}$ Worksheet 4A	$(2)_{COMB}$ Worksheet 4B	$(3)_{COMB}$ Worksheet 4B	$(5)_{TOTAL}*(2)_{FI}/(2)_{TOTAL}$	(3)+(4)+(5)
Property damage only (PDO)	$(4)_{COMB}$ Worksheet 4A	-	-	$(5)_{TOTAL}*(2)_{PDO}/(2)_{TOTAL}$	(3)+(4)+(5)
		0.000	0.000		

PART D— ACCIDENT MODIFICATION FACTORS

CHAPTER 13—ROADWAY SEGMENTS

13.1. Introduction 13-1
13.2. Definition, Application, and Organization of AMFs 13-1
13.3. Definition of a Roadway Segment 13-2
13.4. Crash Effects of Roadway Elements 13-2
13.4.1. Background and Availability of AMFs 13-2
13.4.2. Roadway Element Treatments with AMFs 13-3
13.4.2.1. Modify Lane Width 13-3
13.4.2.2. Add Lanes by Narrowing Existing Lanes and Shoulders 13-9
13.4.2.3. Remove Through Lanes or "Road Diets" 13-10
13.4.2.4. Add or Widen Paved Shoulder 13-11
13.4.2.5. Modify Shoulder Type 13-14
13.4.2.6. Provide a Raised Median 13-15
13.4.2.7. Change the Width of an Existing Median 13-16
13.4.3. Conversion Factor for Total-Crashes 13-18
13.5. Crash Effects of Roadside Elements 13-19
13.5.1. Background and Availability of AMFs 13-19
13.5.2. Roadside Element Treatments with AMFs 13-20
13.5.2.1. Flatten Sideslopes 13-20
13.5.2.2. Increase the Distance to Roadside Features 13-22
13.5.2.3. Change Roadside Barrier along Embankment to Less Rigid Type 13-23
13.5.2.4. Install Median Barrier 13-23
13.5.2.5. Install Crash Cushions at Fixed Roadside Features 13-24
13.5.2.6. Reduce Roadside Hazard Rating 13-25
13.6. Crash Effects of Alignment Elements 13-26
13.6.1. Background and Availability of AMFs 13-26
13.6.2. Alignment Treatments with AMFs 13-27
13.6.2.1. Modify Horizontal Curve Radius and Length, and Provide Spiral Transitions 13-27
13.6.2.2. Improve Superelevation of Horizontal Curves 13-28
13.6.2.3. Change Vertical Grade 13-28
13.7. Crash Effects of Roadway Signs 13-29
13.7.1. Background and Availability of AMFs 13-29
13.7.2. Roadway Sign Treatments with AMFs 13-30
13.7.2.1. Install Combination Horizontal Alignment/Advisory Speed Signs (W1-1a, W1-2a) 13-30
13.7.2.2. Install Changeable Accident Ahead Warning Signs 13-31

13.7.2.3. Install Changeable “Queue Ahead” Warning Signs 13-31
13.7.2.4. Install Changeable Speed Warning Signs 13-32
13.8. Crash Effects of Roadway Delineation 13-32
13.8.1. Background and Availability of AMFs 13-32
13.8.2. Roadway Delineation Treatments with AMFs 13-33
13.8.2.1. Install Post-Mounted Delineators (PMDs) 13-33
13.8.2.2. Place Standard Edgeline Markings (4 to 6 in) 13-34
13.8.2.3. Place Wide (8-in) Edgeline Markings 13-34
13.8.2.4. Place Centerline Markings 13-35
13.8.2.5. Place Edgeline and Centerline Markings 13-36
13.8.2.6. Install Edgelines, Centerlines and Post-Mounted Delineators 13-36
13.8.2.7. Install Snowplowable Permanent Raised Pavement Markers (RPMs) 13-36
13.9. Crash Effects of Rumble Strips 13-38
13.9.1. Background and Availability of AMFs 13-38
13.9.2. Rumble Strip Treatments with AMFs 13-39
13.9.2.1. Install Continuous Shoulder Rumble Strips 13-39
13.9.2.2. Install Centerline Rumble Strips 13-41
13.10. Crash Effects of Traffic Calming 13-42
13.10.1. Background and Availability of AMFs 13-42
13.10.2. Traffic Calming Treatments with AMFs 13-43
13.10.2.1. Install Speed Humps 13-43
13.11. Crash Effects of On-Street Parking 13-44
13.11.1. Background and Availability of AMFs 13-44
13.11.2. Parking Treatments with AMFs 13-45
13.11.2.1. Prohibit On-Street Parking 13-45
13.11.2.2. Convert Free to Regulated On-Street Parking 13-45
13.11.2.3. Implement Time-Limited On-Street Parking Restrictions 13-46
13.11.2.4. Convert Angle Parking to Parallel Parking 13-46
13.12. Crash Effects of Roadway Treatments for Pedestrians and Bicyclists 13-50
13.12.1. Background and Availability of AMFs 13-50
13.13. Crash Effects of Highway Lighting 13-52
13.13.1. Background and Availability of AMFs 13-52
13.13.2. Highway Lighting Treatments with AMFs 13-52
13.13.2.1. Provide Highway Lighting 13-52
13.14. Crash Effects of Roadway Access Management 13-53
13.14.1. Background and Availability of AMFs 13-53
13.14.2. Access Management Treatments with AMFs 13-54
13.14.2.1. Modify Access Point Density 13-54
13.15. Crash Effects of Weather Issues 13-55
13.15.1. Background and Availability of AMFs 13-55

13.15.2. Weather Related Treatments with AMFs 13-56
13.15.2.1. Implement Faster Response Times for Winter Maintenance 13-56
13.16. Conclusion 13-57
13.17. References 13-58

EXHIBITS

Exhibit 13-1: Summary of Treatments Related to Roadway Elements 13-3

Exhibit 13-2: AMF for Lane Width on Rural Two-Lane Roadway Segments[16] 13-4

Exhibit 13-3: Potential Crash Effects of Lane Width on Rural Two-Lane Roads Relative to 12-ft Lanes [3] 13-4

Exhibit 13-4: AMF for Lane Width on Undivided Rural Multilane Roadway Segments[34] 13-7

Exhibit 13-5: Potential crash Effects of Lane Width on Undivided Rural Multilane Roads Relative to 12-ft Lanes [34] 13-7

Exhibit 13-6: AMF for Lane Width on Divided Rural Multilane Roadway Segments[34] 13-8

Exhibit 13-7: Potential Crash Effects of Lane Width on Divided Rural Multilane Roads Relative to 12-ft Lanes [34] 13-8

Exhibit 13-8: Potential Crash Effects of Lane Width on Rural Frontage Roads[22] 13-9

Exhibit 13-9: Potential Crash Effects of Adding Lanes by Narrowing Existing Lanes and Shoulders [4] 13-10

Exhibit 13-10: Potential Crash Effects of Four to Three Lane Conversion or "Road Diet"[15] 13-11

Exhibit 13-11: AMF for Shoulder Width on Rural Two-Lane Roadway Segments......... 13-11

Exhibit 13-12: Potential Crash Effects of Paved Shoulder Width on Rural Two-Lane Roads Relative to 6-ft Paved Shoulders[16] 13-12

Exhibit 13-13: Potential Crash Effects of Paved Right Shoulder Width on Divided Segments[15] 13-13

Exhibit 13-14: Potential Crash Effects of Paved Shoulder Width on Rural Frontage Roads [22] 13-13

Exhibit 13-15: Potential Crash Effects of Shoulder Type on Rural Two-Lane Roads for Related Accident Types [16,33,36] 13-14

Exhibit 13-16: Potential Crash Effects of Providing a Median on Urban Two-Lane Roads [8] 13-15

Exhibit 13-17: Potential Crash Effects of Providing a Median on Multi-Lane Roads [8] . 13-15

Exhibit 13-18: Potential Crash Effects of Median Width on Rural Four-Lane Roads with Full Access Control[15] 13-16

Exhibit 13-19: Potential Crash Effects of Median Width on Rural Four-Lane Roads with Partial or No Access Control[15] 13-16

Exhibit 13-20: Potential Crash Effects of Median Width on Urban Four-Lane Roads with Full Access Control[15] 13-17

Exhibit 13-21: Potential Crash Effects of Median Width on Urban Roads with at least Five Lanes with Full Access Control[15] 13-17

Exhibit 13-22: Potential Crash Effects of Median Width on Urban Four-Lane Roads with Partial or No Access Control[15] 13-18

Exhibit 13-23: Potential Crash Effects of Lane Width on Rural Two-Lane Roads on Total Accidents[16] 13-18

Exhibit 13-24: Summary of Treatments Related to Roadside Elements 13-19

Exhibit 13-25: Potential Crash Effects on Total Accidents of Flattening Sideslopes[15] . 13-20

Exhibit 13-26: Potential Crash Effects on Single Vehicle Accidents of Flattening Sideslopes (15) 13-20
Exhibit 13-27: Potential Crash Effects of Sideslopes on Undivided Segments(15,34) 13-22
Exhibit 13-28: Potential Crash Effects of Increased Distance to Roadside Features (8) 13-22
Exhibit 13-29: Potential Crash Effects of Changing Barrier to Less Rigid Type(8) 13-23
Exhibit 13-30: Potential Crash Effects of Installing a Median Barrier (8) 13-24
Exhibit 13-31: Potential Crash Effects of Installing Crash Cushions at Fixed Roadside Features (8) 13-24
Exhibit 13-32: Quantitative Descriptors for the Seven Roadside Hazard Ratings (16) 13-25
Exhibit 13-33: Potential Crash Effects of Roadside Hazard Rating for Total Accidents on Rural Two-Lane Highways(16) 13-26
Exhibit 13-34: Summary of Treatments Related to Alignment Elements..................... 13-26
Exhibit 13-35: Potential Crash Effect of the Radius, Length, and Presence of Spiral Transition Curves in a Horizontal Curve 13-27
Exhibit 13-36: Potential Crash Effects of Improving Superelevation Variance (SV) of Horizontal Curves on Rural Two-Lane Roads(16,35) 13-28
Exhibit 13-37: Potential Crash Effects of Changing Vertical Grade on Rural Two-Lane Roads(16,24) 13-29
Exhibit 13-38: Summary of Treatments Related to Roadway Signs 13-30
Exhibit 13-39: Potential Crash Effects of Installing Combination Horizontal Alignment/ Advisory Speed Signs (W1-1a, W1-2a) (8) 13-31
Exhibit 13-40: Potential Crash Effects of Installing Changeable Accident Ahead Warning Signs (8) 13-31
Exhibit 13-41: Potential Crash Effects of Installing Changeable "Queue Ahead" Warning Signs (8) 13-32
Exhibit 13-42: Potential Crash Effects of Installing Changeable Speed Warning Signs for Individual Drivers (8) 13-32
Exhibit 13-43: Summary of Treatments Related to Delineation 13-33
Exhibit 13-44: Potential Crash Effects of Installing Post-Mounted Delineators (8) 13-34
Exhibit 13-45: Potential Crash Effects of Placing Standard Edgeline Markings(8) 13-34
Exhibit 13-46: Potential Crash Effects of Placing Wide Edgeline Markings(8) 13-35
Exhibit 13-47: Potential Crash Effects of Placing Centerline Markings(8) 13-35
Exhibit 13-48: Potential Crash Effects of Placing Edgeline and Centerline Markings (8) 13-36
Exhibit 13-49: Potential Crash Effects of Installing Edgelines, Centerlines and Post-Mounted Delineators (8) 13-36
Exhibit 13-50: Potential Crash Effects of Installing Snowplowable Permanent Raised Pavement Markers (RPMs) (2) 13-37
Exhibit 13-51: Potential Crash Effects of Installing Snowplowable Permanent Raised Pavement Markers (RPMs) (2) 13-38
Exhibit 13-52: Summary of Treatments Related to Rumble Strips 13-39
Exhibit 13-53: Potential Crash Effects of Installing Continuous Shoulder Rumble Strips on Multi-Lane Highways (6) 13-40

Exhibit 13-54: Potential Crash Effects of Installing Continuous Shoulder Rumble Strips on Freeways (25,13) 13-40
Exhibit 13-55: Potential Crash Effects of Installing Centerline Rumble Strips (14) 13-42
Exhibit 13-56: Summary of Treatments Related to Traffic Calming 13-43
Exhibit 13-57: Potential Crash Effects Of Installing Speed Humps (8) 13-43
Exhibit 13-58: Summary of Treatments Related to On-Street Parking 13-44
Exhibit 13-59: Potential Crash Effects of Prohibiting On-Street Parking(22,19) 13-45
Exhibit 13-60: Potential Crash Effects of Converting from Free to Regulated On-Street Parking (8) 13-46
Exhibit 13-61: Potential Crash Effects of Implementing Time-Limited On-Street Parking(8) 13-46
Exhibit 13-63: Type of Parking and Land Use Factor (f_{pk} in Equation 13-6) 13-48
Exhibit 13-64: Summary of Roadway Treatments for Pedestrians and Bicyclists 13-51
Exhibit 13-65: Summary of Treatments Related to Highway Lighting 13-52
Exhibit 13-66: Potential Crash Effects of Providing Highway Lighting(7,8,12,27) 13-52
Exhibit 13-67: Summary of Treatments Related to Access Management 13-53
Exhibit 13-68: Potential Crash Effects of Access Point Density on Rural Two-lane Roads 13-54
Exhibit 13-69: Potential Crash Effects of Reducing Access Point Density(8) 13-55
Exhibit 13-70: Summary of Treatments Related to Weather Issues 13-55
Exhibit 13-71: Potential Crash Effects of Raising Standards for Winter Maintenance for the Whole Winter Season(8) 13-57
Exhibit 13-72: Clear Zone Distance with Example of a Parallel Foreslope Design(3) 13-64
Exhibit 13-73: Typical Roadway with Roadside Hazard Rating of 1 13-65
Exhibit 13-74: Typical Roadway with Roadside Hazard Rating of 2 13-66
Exhibit 13-75: Typical Roadway with Roadside Hazard Rating of 3 13-66
Exhibit 13-76: Typical Roadway with Roadside Hazard Rating of 4 13-67
Exhibit 13-77: Typical Roadway with Roadside Hazard Rating of 5 13-67
Exhibit 13-78: Typical Roadway with Roadside Hazard Rating of 6 13-68
Exhibit 13-79: Typical Roadway with Roadside Hazard Rating of 7 13-68
Exhibit 13-80: Zebra Crossing 13-78
Exhibit 13-81: Pelican Crossing 13-79
Exhibit 13-82: Puffin Crossing 13-79
Exhibit 13-83: Toucan Crossing 13-80

APPENDIX A

A.1 Introduction 13-61
A.2 Roadway Elements 13-61
A.2.1 General Information 13-61
A.2.2 Roadway Element Treatments with no AMFs - Trends in Crashes or User Behavior 13-63
A.2.2.1 Increase Median Width 13-63
A.3 Roadside Elements 13-63
A.3.1 General Information 13-63
A.3.2 Roadside Element Treatments with no AMFs - Trends in Crashes or User Behavior 13-68
A.3.2.1 Install Median Barrier 13-68
A.3.2.2 Increase Clear Roadside Recovery Distance 13-69
A.3.2.3 Install Curb 13-69
A.3.2.4 Increase Distance to Utility Poles and Decrease Utility Pole Density 13-69
A.3.2.5 Install Roadside Barrier along Embankment 13-70
A.4 Alignment Elements 13-70
A.4.1 General Information 13-70
A.4.2 Alignment Treatments with no AMFs - Trends in Crashes or User Behavior 13-71
A.4.2.1 Modify Tangent Length Prior to Curve 13-71
A.4.2.2 Modify Horizontal Curve Radius 13-71
A.5 Roadway Signs 13-71
A.5.1 Roadway Sign Treatments with no AMFs - Trends in Crashes or User Behavior 13-71
A.5.1.1 Install Signs to Conform to MUTCD 13-71
A.6 Roadway Delineation 13-72
A.6.1 Roadway Delineation Treatments with no AMFs - Trends in Crashes or User Behavior 13-72
A.6.1.1 Install Chevron Signs on Horizontal Curves 13-72
A.6.1.2 Provide Distance Markers 13-72
A.6.1.3 Place Converging Chevron Pattern Markings 13-72
A.6.1.4 Place Edgeline and Directional Pavement Markings on Horizontal Curves 13-72
A.7 Rumble Strips 13-73
A.7.1 Rumble Strip Treatments with no AMFs - Trends in Crashes or User Behavior 13-73
A.7.1.1 Install Continuous Shoulder Rumble Strips and Wider Shoulders 13-73
A.7.1.2 Install Transverse Rumble Strips 13-73
A.7.1.3 Install Centerline Rumble Strips and Centerline Markings 13-73
A.8 Traffic Calming 13-73
A.8.1 General Information 13-73

A.8.2 Traffic Calming Treatments with no AMFs - Trends in Crashes or User Behavior 13-74

A.8.2.1 Install Transverse Rumble Strips on Intersection Approaches 13-74

A.8.2.2 Apply Several Traffic Calming Measures to a Road Segment 13-74

A.9 Roadway Treatments for Pedestrians and Bicyclists 13-74

A.9.1 Pedestrian and Bicycle Treatments with no AMFs - Trends in Crashes or User Behavior 13-74

A.9.1.1 Provide a Sidewalk or Shoulder 13-74

A.9.1.2 Install Raised Pedestrian Crosswalks 13-75

A.9.1.3 Install Pedestrian-Activated Flashing Yellow Beacons with Overhead Signs 13-75

A.9.1.4 Install Pedestrian-Activated Flashing Yellow Beacons with Overhead Signs and Advance Pavement Markings 13-75

A.9.1.5 Install overhead electronic signs with pedestrian-activated crosswalk flashing beacons 13-76

A.9.1.6 Reduce Posted Speed Limit through Schools Zones during School Times 13-77

A.9.1.7 Provide Pedestrian Overpass and Underpass 13-77

A.9.1.8 Mark Crosswalks at Uncontrolled Locations, Intersections, or Mid-block 13-77

A.9.1.9 Use Alternative Crosswalk Markings at Mid-block Locations 13-78

A.9.1.10 Use Alternative Crosswalk Devices at Mid-block Locations 13-78

A.9.1.11 Provide a Raised Median or Refuge Island at Marked and Unmarked Crosswalks 13-80

A.9.1.12 Provide a Raised or Flush Median or Center Two-way Left-turn Lane at Marked and Unmarked Crosswalks 13-80

A.9.1.13 Install Pedestrian Refuge Islands or Split Pedestrian Crossovers 13-81

A.9.1.14 Widen Median 13-81

A.9.1.15 Provide Dedicated Bicycle Lanes 13-81

A.9.1.16 Provide Wide Curb Lanes (WCLs) 13-82

A.9.1.17 Provide Shared Bus/Bicycle Lanes 13-82

A.9.1.18 Re-stripe Roadway to Provide Bicycle Lane 13-82

A.9.1.19 Pave Highway Shoulders for Bicyclist Use 13-82

A.9.1.20 Provide Separate Bicycle Facilities 13-83

A.10 Roadway Access Management 13-83

A.10.1 Roadway Access Management Treatments with no AMFs – Trends in Crashes or User Behavior 13-83

A.10.1.1 Reduce Number of Median Crossings and Intersections 13-83

A.11 Weather Issues 13-83

A.11.1 General Information 13-83

A.11.2 Weather Issue Treatments with No AMFs – Trends in Crashes or User Behavior 13-84

A.11.2.1 Install Changeable Fog Warnings Signs 13-84

A.11.2.2 Install Snow Fences for the Whole Winter Season 13-84

A.11.2.3 Raise the State of Preparedness for Winter Maintenance 13-84

A.11.2.4 Apply Preventive Chemical Anti-icing During Entire Winter Season 13-84

A.12 Treatments with Unknown Crash Effects 13-85
A.12.1 Treatments Related to Roadway Elements 13-85
A.12.2 Treatments Related to Roadside Elements 13-85
A.12.3 Treatments Related to Alignment Elements 13-85
A.12.4 Treatments Related to Roadway Signs 13-85
A.12.5 Treatments Related to Roadway Delineation 13-86
A.12.6 Treatments Related to Rumble Strips 13-86
A.12.7 Treatments Related to Passing Zones 13-86
A.12.8 Treatments Related to Traffic Calming 13-86
A.12.9 Treatments Related to On-Street Parking 13-87
A.12.10 Roadway Treatments for Pedestrians and Bicyclists 13-87
A.12.11 Treatments Related to Access Management 13-87
A.12.12 Treatments Related to Weather Issues 13-87
A.13 Appendix References 13-89

CHAPTER 13 ROADWAY SEGMENTS

13.1. INTRODUCTION

Chapter 13 presents the Accident Modification Factors (AMFs) for design, traffic control, and operational treatments on roadway segments. Pedestrian and bicyclist treatments, and the effects on expected average crash frequency of other treatments such as illumination, access points, and weather issues, are also discussed. The information presented in this chapter is used to identify effects on expected average crash frequency resulting from treatments applied to roadway segments.

Chapter 13 presents the Accident Modification Factors (AMFs) for design, traffic control, and operational treatments on roadway segments.

The *Part D Introduction and Applications Guidance* section provides more information about the processes used to determine the AMFs presented in this chapter.

Chapter 13 is organized into the following sections:

- Definition, Application, and Organization of AMFs (Section 13.2)
- Definition of a Roadway Segment (Section 13.3)
- Crash Effects of Roadway Elements (Section 13.4)
- Crash Effects of Roadside Elements (Section 13.5)
- Crash Effects of Alignment Elements (Section 13.6)
- Crash Effects of Roadway Signs (Section 13.7)
- Crash Effects of Roadway Delineation (Section 13.8)
- Crash Effects of Rumble Strips (Section 13.9)
- Crash Effects of Traffic Calming (Section 13.10)
- Crash Effects of On-Street Parking (Section 13.11)
- Crash Effects of Roadway Treatments for Pedestrians and Bicyclists (Section 13.12)
-
-
- Crash Effects of Highway Lighting (Section 0)
- Crash Effects of Roadway Access Management (Section 13.14)
- Crash Effects of Weather Issues (Section 13.15)
- Conclusion (Section 13.16)

Appendix A presents the crash trends for treatments for which AMFs are not currently known, and a listing of treatments for which neither AMFs nor trends are unknown.

13.2. DEFINITION, APPLICATION, AND ORGANIZATION OF AMFS

AMFs quantify the change in expected average crash frequency (crash effect) at a site caused by implementing a particular treatment (also known as a countermeasure, intervention, action, or alternative), design modification, or change in operations. AMFs are used to estimate the potential change in expected crash frequency or crash severity plus or minus a standard error due to implementing a

Chapter 3 provides a thorough definition and explanation of AMFs.

particular action. The application of AMFs involves evaluating the expected average crash frequency with or without a particular treatment, or estimating it with one treatment versus a different treatment.

Specifically, the AMFs presented in this chapter can be used in conjunction with activities in *Chapter 6 Select Countermeasures*, and *Chapter 7 Economic Appraisal*. Some *Part D* AMFs are included in *Part C* for use in the predictive method. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7 of the *Part C Introduction and Applications Guidance*. *Chapter 3 Fundamentals*, Section 3.5.3 Accident Modification Factors provides a comprehensive discussion of AMFs including: an introduction to AMFs, how to interpret and apply AMFs, and applying the standard error associated with AMFs.

In all *Part D* chapters, the treatments are organized into one of the following categories:

1. AMF is available;
2. Sufficient information is available to present a potential trend in crashes or user behavior, but not to provide an AMF;
3. Quantitative information is not available.

Treatments with AMFs (Category 1 above) are typically estimated for three accident severities: fatal, injury, and non-injury. In the HSM, fatal and injury are generally combined and noted as injury. Where distinct AMFs are available for fatal and injury severities, they are presented separately. Non-injury severity is also known as property-damage-only severity.

Treatments for which AMFs are not presented (Categories 2 and 3 above) indicate that quantitative information currently available did not meet the criteria for inclusion in the HSM. However, in Category 2 there was sufficient information to identify a trend associated with the treatments. The absence of an AMF indicates additional research is needed to reach a level of statistical reliability and stability to meet the criteria set forth within the HSM. Treatments for which AMFs are not presented are discussed in Appendix A.

13.3. DEFINITION OF A ROADWAY SEGMENT

Section 13.4 provides a summary of roadway elements with AMFs.

A roadway is defined as "the portion of a highway, including shoulders, for vehicular use; a divided highway has two or more roadways."[17] A roadway segment consists of a continuous portion of a roadway with similar geometric, operational, and vehicular characteristics. Roadways where significant changes in these characteristics are observed from one location to another should be analyzed as separate segments. [30]

13.4. CRASH EFFECTS OF ROADWAY ELEMENTS

13.4.1. Background and Availability of AMFs

Roadway elements vary depending on road type, road function, environment and terrain. Exhibit 13-1 summarizes common treatments related to roadway elements and the corresponding AMF availability.

Exhibit 13-1: Summary of Treatments Related to Roadway Elements

HSM Section	Treatment	Rural Two-Lane Road	Rural Multi-Lane Highway	Rural Frontage Road	Freeway	Expressway	Urban Arterial	Suburban Arterial
13.4.2.1	Modify lane width	✓	✓	✓	-	-	-	-
13.4.2.2	Add lanes by narrowing existing lanes and shoulders	N/A	-	N/A	✓	-	-	-
13.4.2.3	Remove through lanes or "road diets"	N/A	N/A	N/A	N/A	N/A	✓	N/A
13.4.2.4	Add or widen paved shoulder	✓	✓	✓	-	-	-	-
13.4.2.5	Modify shoulder type	✓	-	-	-	-	-	-
13.4.2.6	Provide a raised median	-	✓	N/A	-	-	✓	-
13.4.2.7	Change width of existing median	N/A	✓	N/A	-	-	✓	-
Appendix A	Increase median width	-	T	N/A	T	T	-	-

NOTE:
✓ = Indicates that an AMF is available for this treatment.
T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.
- = Indicates that an AMF is not available and a trend is not known.
N/A = Indicates that the treatment is not applicable to the corresponding setting.

13.4.2. Roadway Element Treatments with AMFs

13.4.2.1. Modify Lane Width

Rural two-lane roads

Widening lanes on rural two-lane roads reduces a specific set of related accident types, namely single-vehicle run-off-road accidents and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe collisions. The AMF for lane width is determined with the equations presented in Exhibit 13-2, which are illustrated by the graphs in Exhibit 13-3.[10,16,33] The crash effect of lane width varies with traffic volume, as shown in the exhibits.

Relative to a 12-ft lanes base condition, 9-ft wide lanes increase the frequency of related accident types identified above. [10,16]

For roads with an AADT of 2,000 or more, lane width has a greater effect on expected average crash frequency. Relative to 12-ft lanes, 9-ft wide lanes increase the frequency of related accident types identified above more than either 10-ft or 11-ft lanes.[16,33]

For lane widths other than 9, 10, 11, and 12 ft, the crash effect can be interpolated between the lines shown in Exhibit 13-3.

If lane widths for the two directions of travel on a roadway segment differ, the AMF is determined separately for the lane width in each direction of travel and then averaged.[16] The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is 12 ft lanes.

Exhibit 13-2: AMF for Lane Width on Rural Two-Lane Roadway Segments[16]

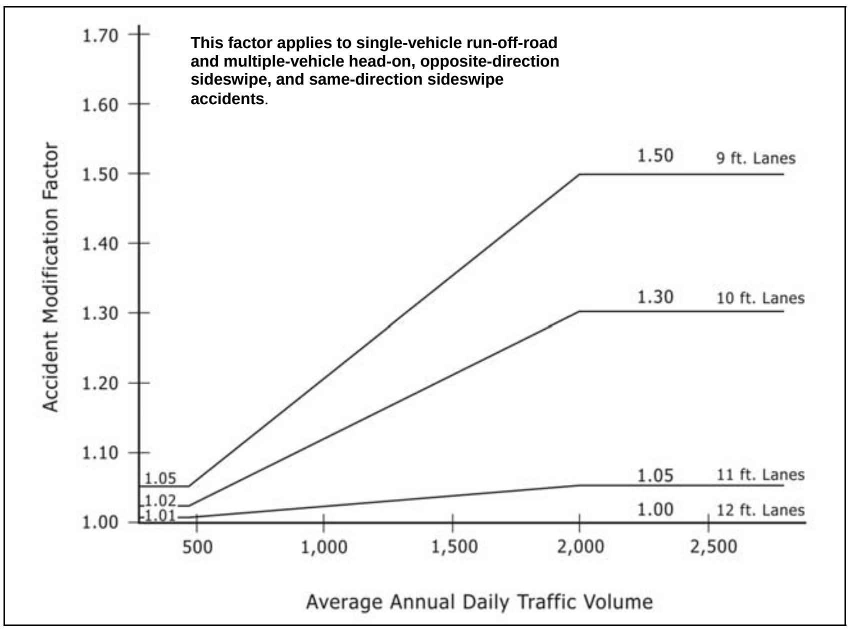
Lane Width	Average Annual Daily Traffic (AADT) (vehicles/day)		
	< 400	400 to 2000	> 2000
9 ft or less	1.05	$1.05+2.81\times10^{-4}(AADT-400)$	1.50
10 ft	1.02	$1.02+1.75\times10^{-4}(AADT-400)$	1.30
11 ft	1.01	$1.01+2.5\times10^{-5}(AADT-400)$	1.05
12 ft or more	1.00	1.00	1.00

NOTE: The collision types related to lane width to which these AMFs apply are single-vehicle run-off the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents.

Standard error of the AMF is unknown.

To determine the AMF for changing lane width and/or AADT, divide the "new" condition AMF by the "existing" condition AMF.

Exhibit 13-3: Potential Crash Effects of Lane Width on Rural Two-Lane Roads Relative to 12-ft Lanes [3]



NOTE: Standard error of the AMF is unknown.
To determine the AMF for changing lane width and/or AADT, divide the "new" condition AMF by the "existing" condition AMF.

Exhibit 13-23 and Equation 13-3 in Section 13.4.3 (Conversion Factor for Total-Crashes) may be used to express the lane width AMFs in terms of the crash effect on total crashes, rather than just the accident types identified in Exhibit 13-2 and Exhibit 13-3.[10,16,33]

The gray box below presents an example of how to apply the preceding equations and graphs to assess the total crash effects of modifying the lane width on a rural two-lane highway.

Effectiveness of Modifying Lane Width

Question:

As part of improvements to a 5-mile section of a rural two-lane road, the local jurisdiction has proposed widening the roadway from 10-ft to 11-ft lanes. What will be the likely reduction in expected average crash frequency for opposite direction sideswipe crashes, and for total crashes?

Given Information:

- Existing roadway = rural two-lane
- AADT = 2,200 vehicles per day
- Expected average crash frequency without treatment for the five mile segment (see Part C Predictive Method):

a) 9 opposite direction sideswipe crashes/year

b) 30 total crashes/year

Find:

- Expected average opposite direction sideswipe crash frequency with the implementation of 11-ft lanes
- Expected average total crash frequency with the implementation of 11-ft lanes
- Expected average opposite direction sideswipe crash frequency reduction
- Expected average total crash frequency reduction

Answer:

1) Identify the Applicable AMFs

a) Exhibit 13-3 for opposite direction sideswipe crashes

b) Equation 13-3 or Exhibit 13-23 for all crashes

Note that for a conversion from *opposite direction sideswipe* crashes to *all* crashes the information in Section 13.4.3 which contains Equation 13-3 and Exhibit 13-23 may be applied.

2) Calculate the AMF for the existing condition 10-ft lane width

a) For opposite direction sideswipe crashes

AMF_{ra} = 1.30 (Exhibit 13-3)

b) For total crashes

AMF_{total} = (1.30-1.00) x 0.30 + 1.00 = 1.09 (Equation 13-3 or Exhibit 13-23)

3) Calculate the AMF for the proposed condition 11-ft lane width

a) For opposite direction sideswipe crashes

AMF_{ra} = 1.05 From (Exhibit 13-3)

b) For total crashes

AMF_{total} = (1.05-1.00) x 0.30 + 1.00 = 1.01 (Equation 13-3 or Exhibit 13-23)

Effectiveness of Modifying Lane Width (Continued)

4) Calculate the treatment ($AMF_{Treatment}$) corresponding to the change in lane width for opposite direction sideswipe crashes and for all crashes.

   a) For opposite direction sideswipe crashes

   $AMF_{ra\ Treatment}$ = 1.05/1.30 = 0.81

   b) For total crashes

   $AMF_{total\ Treatment}$ = 1.01/1.09 = 0.93

5) Apply the treatment AMF ($AMF_{Treatment}$) to the expected number of crashes at the intersection without the treatment.

   a) For opposite direction sideswipe crashes

   = 0.81(9 crashes/year) = 7.3 crashes/year

   b) For total crashes

   = 0.93(30 crashes/year) = 27.9 crashes/year

6) Calculate the difference between the expected number of crashes without the treatment and the expected number with the treatment.

   Change in Expected Average Crash Frequency:

   a) For opposite direction sideswipe crashes

   9.0 – 7.3 = 1.7 crashes/year reduction

   b) For total crashes

   30.0 – 27.9 = 2.1 crashes/year reduction

7) **Discussion: The proposed change in lane width may potentially reduce opposite direction sideswipe crashes by 1.7 crashes/year and total crashes by 2.1 crashes per year. Note that a standard error has not been determined for this AMF, therefore a confidence interval cannot be calculated.**

Rural Multilane Highways

Widening lanes on rural multilane highways reduces the same specific set of related accident types as rural two-lane highways, namely single-vehicle run-off-road accidents and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe collisions. The AMF for lane width is determined with the equations presented in Exhibit 13-4 for undivided multilane highways and in Exhibit 13-6 for divided multilane highways. These equations are illustrated by the graphs shown in Exhibit 13-5 and Exhibit 13-7, respectively. The crash effect of lane width varies with traffic volume, as shown in the exhibits.

For roads with an AADT of 400 or less, lane width has a small crash effect. Relative to a 12-ft lanes base condition, 9-ft wide lanes increase the frequency of related accident types identified above.

For roads with an AADT of 2,000 or more, lane width has a greater effect on expected average crash frequency. Relative to 12-ft lanes, 9-ft wide lanes increase the frequency of related accident types identified above more than either 10-ft or 11-ft lanes.

For lane widths other than 9, 10, 11, and 12 ft, the crash effect can be interpolated between the lines shown in Exhibits 13-3b and 13-3d. Lanes less than 9 ft wide can be assigned an AMF equal to 9-ft lanes. Lanes greater than 12-ft wide can be assigned a crash effect equal to 12-ft lanes.

The effect of lane width on undivided rural multilane highways is equal to approximately 75% of the effect of lane width on rural two-lane roads.(34) Where the lane widths on a roadway vary, the AMF is determined separately for the lane width in each direction of travel and the resulting AMFs are then averaged. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is 12-ft lanes.

Exhibit 13-4: AMF for Lane Width on Undivided Rural Multilane Roadway Segments(34)

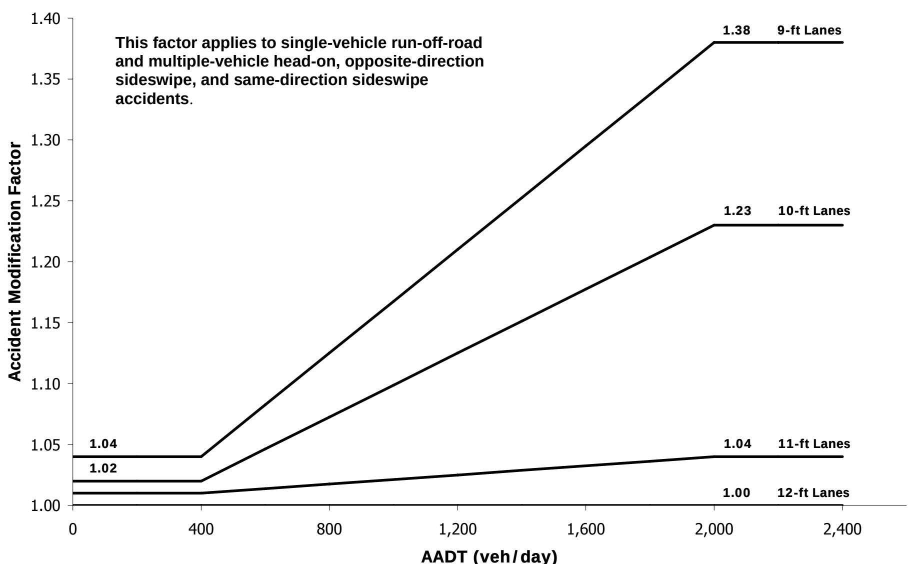
Lane Width	Average Annual Daily Traffic (AADT) (veh/day)		
	< 400	400 to 2000	> 2000
9 ft or less	1.04	$1.04+2.13\times10^{-4}$(AADT-400)	1.38
10 ft	1.02	$1.02+1.31\times10^{-4}$(AADT-400)	1.23
11 ft	1.01	$1.01+1.88\times10^{-5}$(AADT-400)	1.04
12 ft or more	1.00	1.00	1.00

NOTE: The collision types related to lane width to which these AMFs apply are single-vehicle run-off the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents.

Standard error of the AMF is unknown.

To determine the AMF for changing lane width and/or AADT, divide the "new" condition AMF by the "existing" condition AMF.

Exhibit 13-5: Potential crash Effects of Lane Width on Undivided Rural Multilane Roads Relative to 12-ft Lanes (34)



NOTE: Standard error of the AMF is unknown.

To determine the AMF for changing lane width and/or AADT, divide the "new" condition AMF by the "existing" condition AMF.

The effect of lane width on divided rural multilane highways is equal to approximately 50% of the effect of lane width on rural two-lane roads.(34) Where the lane widths on a roadway vary, the AMF should be determined separately for the lane width in each direction of travel and the resulting AMFs is then averaged. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is 12-ft lanes.

Exhibit 13-6: AMF for Lane Width on Divided Rural Multilane Roadway Segments[(34)]

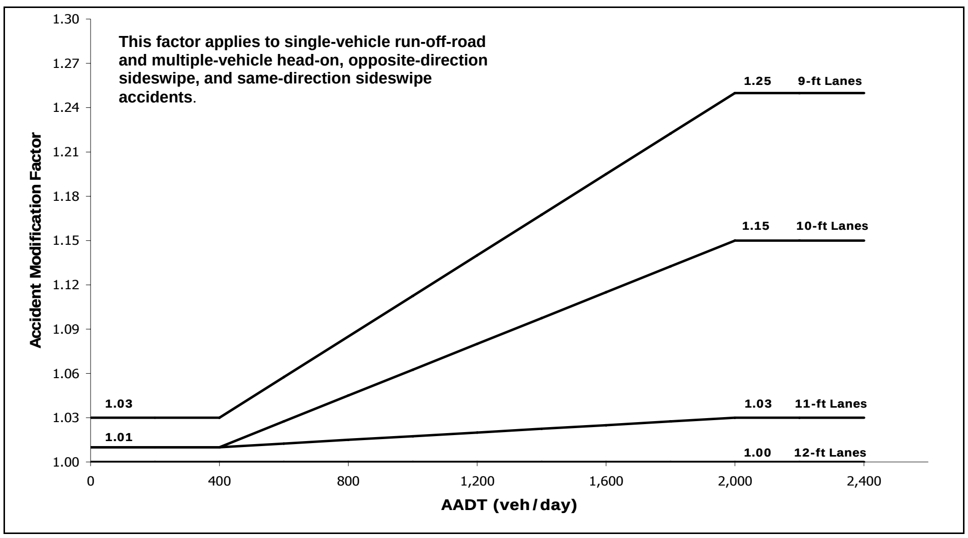
Lane Width	Average Annual Daily Traffic (AADT) (veh/day)		
	< 400	400 to 2000	> 2000
9 ft or less	1.03	$1.03+1.38\times10^{-4}$(AADT-400)	1.25
10 ft	1.01	$1.01+8.75\times10^{-5}$(AADT-400)	1.15
11 ft	1.01	$1.01+1.25\times10^{-5}$(AADT-400)	1.03
12 ft or more	1.00	1.00	1.00

NOTE: The collision types related to lane width to which these AMFs apply are single-vehicle run-off the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents.

Standard error of the AMF is unknown.

To determine the AMF for changing lane width and/or AADT, divide the "new" condition AMF by the "existing" condition AMF.

Exhibit 13-7: Potential Crash Effects of Lane Width on Divided Rural Multilane Roads Relative to 12-ft Lanes [(34)]



NOTE: Standard error of the AMF is unknown.
To determine the AMF for changing lane width and/or AADT, divide the "new" condition AMF by the "existing" condition AMF.

Equation 13-3 in Section 13.4.3 (Conversion Factor for Total Crashes) may be used to express the lane width AMFs in terms of the crash effect on total crashes, rather than just the collision types identified in in the exhibits presented above.

Rural Frontage Roads

Rural frontage roads differ from rural two-lane roads because they have restricted access along at least one side of the road, a higher percentage of turning traffic, and periodic ramp-frontage-road terminals with yield control[(22)]. AMFs for rural frontage roads are provided separately from AMFs for rural two-lane roads.

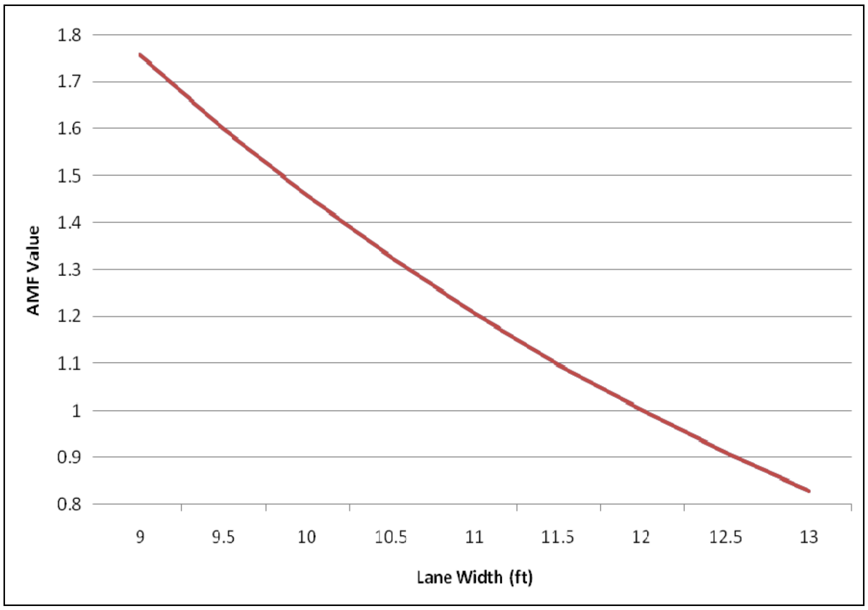
Equation 13-1 presents the AMF for lane width on rural frontage roads between successive interchanges[22]. Exhibit 13-8 is based on Equation 13-1. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is 12 ft lanes.

$$AMF_{LW} = e^{-0.188(LW-12.0)} \quad (13\text{-}1)$$

where:

LW = average lane width (ft)

Exhibit 13-8: Potential Crash Effects of Lane Width on Rural Frontage Roads[22]



NOTE: The standard error of the AMF is unknown.

To determine the AMF for changing lane width and/or AADT, divide the "new" condition AMF by the "existing" condition AMF.

The average lane width represents the total width of the traveled way divided by the number of through lanes on the frontage road. Relative to 12-ft lanes, 9-ft wide lanes increase the number of accidents more than either 10-ft or 11-ft lanes.

Both one-way and two-way frontage roads were considered in the development of this AMF. Development of this AMF was limited to lane widths ranging from 9 to 13 ft and AADT values from 100 to 6,200.

13.4.2.2. Add Lanes by Narrowing Existing Lanes and Shoulders

This treatment consists of maintaining the existing roadway right-of-way and implementing additional lanes by narrowing existing lanes and shoulders. This treatment is only applicable to roadways with multiple lanes in one direction.

Freeways

The crash effects of adding a fifth lane to a base condition four-lane urban freeway within the existing right-of-way, by narrowing existing lanes and shoulders are shown in Exhibit 13-9.[4] The crash effects of adding a sixth lane to a base condition five-lane urban freeway by accident severity are also shown in Exhibit 13-9.[4]

These AMFs apply to urban freeways with median barriers with a base condition (i.e., the condition in which the AMF = 1.00) of 12-ft lanes. The type of median barrier is undefined.

For this treatment, lanes are narrowed to 11-ft lanes and the inside shoulders are narrowed to provide the additional width for the extra lane. The new lane may be used as a general purpose lane or a High Occupancy Vehicle (HOV) lane.

Exhibit 13-9: Potential Crash Effects of Adding Lanes by Narrowing Existing Lanes and Shoulders [4]

Treatment	Setting (Road type)	Traffic Volume AADT	Accident type (Severity)	AMF	Std. Error
Four to five lane conversion	Urban (Freeway)	79,000 to 128,000, one direction	All types (All severities)	**1.11**	**0.05**
			All types (Injury and Non-injury tow-away)	**1.10***	**0.07**
			All types (Injury)	**1.11**	**0.08**
Five to six lane conversion		77,000 to 126,000, one direction	All types (All severities)	**1.03***	**0.08**
			All types (Injury and Non-injury tow-away)	**1.04***	**0.1**
			All types (Injury)	**1.07***	**0.1**
Base Condition: Four or Five 12-ft lanes depending on initial roadway geometry.					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

Accident migration is generally not found to be a statistically significant outcome of this treatment.[20]

13.4.2.3. Remove Through Lanes or "Road Diets"

A "road diet" usually refers to the conversion of a four-lane undivided road into three lanes: two through lanes plus a center two-way left-turn lane. The remaining roadway width may be converted to bicycle lanes, sidewalks, or on-street parking.[4]

Urban arterials

The effect on crash frequency of removing two through lanes on urban four-lane undivided roads and adding a center two-way left-turn lane is shown in Exhibit 13-10.[15] The base condition for this AMF (i.e., the condition in which the AMF = 1.00) is a four lane roadway cross section. Original lane width is unknown.

Exhibit 13-10: Potential Crash Effects of Four to Three Lane Conversion or "Road Diet"[15]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Four to three lane conversion	Urban (Arterials)	Unspecified	All types (All severities)	**0.71**	**0.02**
Base Condition: Four-lane roadway cross section.					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Original lane width is unknown.

13.4.2.4. Add or Widen Paved Shoulder

Rural two-lane roads

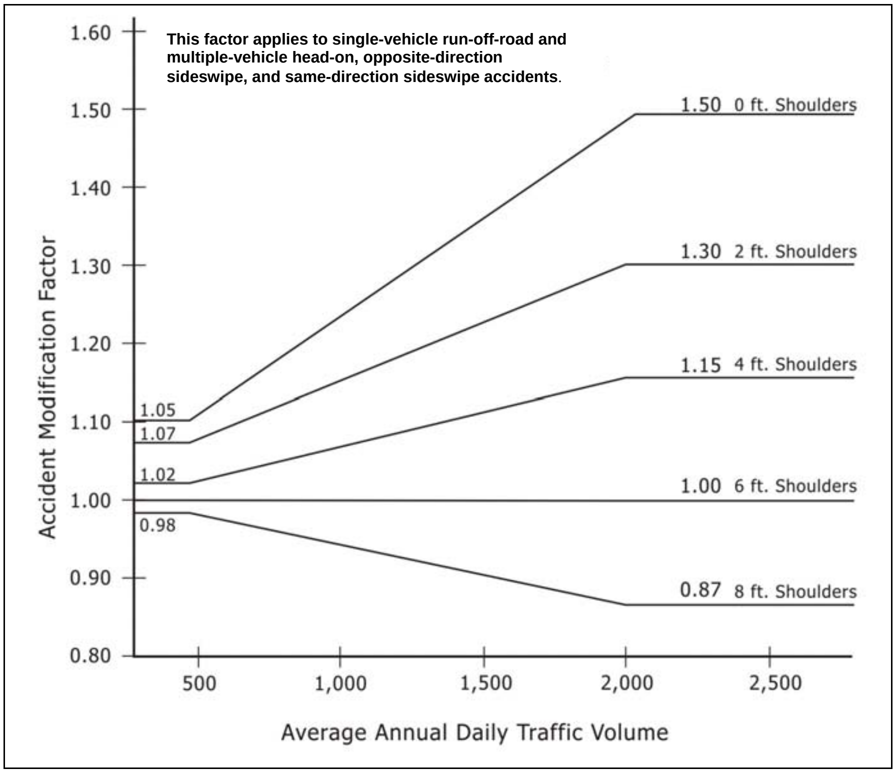
Widening paved shoulders on rural two-lane roads reduces the same related accidents types as widening lanes; single-vehicle run-off-road accidents, multi-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe collisions. The AMF for shoulder width is determined with the equations presented in Exhibit 13-11, which are illustrated by the graph in Exhibit 13-12.[16,33,36] The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is a 6 ft shoulder width.

Exhibit 13-11: AMF for Shoulder Width on Rural Two-Lane Roadway Segments

Shoulder Width	Average Annual Daily Traffic (AADT) (vehicles/day)		
	< 400	400 to 2000	> 2000
0 ft	1.10	$1.10 + 2.5 \times 10^{-4}$ (AADT - 400)	1.50
2 ft	1.07	$1.07 + 1.43 \times 10^{-4}$ (AADT - 400)	1.30
4 ft	1.02	$1.02 + 8.125 \times 10^{-5}$ (AADT - 400)	1.15
6 ft	1.00	1.00	1.00
8 ft or more	0.98	$0.98 + 6.875 \times 10^{-5}$ (AADT - 400)	0.87

NOTE: The collision types related to shoulder width to which this AMF applies include single-vehicle run-off the-road and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe accidents.
Standard error of the AMF is unknown.
To determine the AMF for changing paved shoulder width and/or AADT, divide the "new" condition AMF by the "existing" condition AMF.

Exhibit 13-12: Potential Crash Effects of Paved Shoulder Width on Rural Two-Lane Roads Relative to 6-ft Paved Shoulders(16)



NOTE: Standard error of AMF is unknown.

To determine the AMF for changing paved shoulder width and/or AADT, divide the "new" condition AMF by the "existing" condition AMF.

For roads with an Average Annual Daily Traffic Volume (AADT) of 400 or less, shoulder width has a small crash effect. Relative to 6-ft paved shoulders, no shoulders (0-ft) increase the related accident types by a small amount.(16,33,36) Relative to 6-ft paved shoulders, shoulders 8-ft wide decrease the related collision types by a small amount.(16,33,36)

For shoulder widths within the range of 0 to 8-ft, the crash effect can be interpolated between the lines shown in Exhibit 13-12. Shoulders greater than 8-ft wide can be assigned an AMF equal to 8-ft wide shoulders. (16)

If the shoulder widths for the two travel directions on a roadway segment differ, the AMF is determined separately for each travel direction and then averaged.(16)

Exhibit 13-23 and Equation 13-3 in Section 13.4.3 (Conversion Factor for Total-Crashes) may be used to express the crash effect of paved shoulder width on rural two-lane roads as an effect on total accidents, rather than just the accident types identified in Exhibit 13-12.(16)

Rural multilane highways

Research by Harkey et al. (15) concluded that the shoulder width AMF presented in Exhibit 13-11 and Exhibit 13-12 may be applied to undivided segments of rural multilane highways, as well as to rural two-lane highways.

The AMF for changing shoulder width on multilane divided highways in Exhibit 13-13 applies to the shoulder on the right side of a divided roadway. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is an 8 ft shoulder width.

Exhibit 13-13: Potential Crash Effects of Paved Right Shoulder Width on Divided Segments[15]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
8 to 6-ft conversion	Rural (Multi-lane Highways)	Unspecified	All types (Unspecified)	1.04	N/A
8 to 4-ft conversion				1.09	N/A
8 to 2-ft conversion				1.13	N/A
8 to 0-ft conversion				1.18	N/A
Base Condition: 8-ft shoulder width.					

NOTE: N/A = Standard error of AMF is unknown

Rural frontage roads

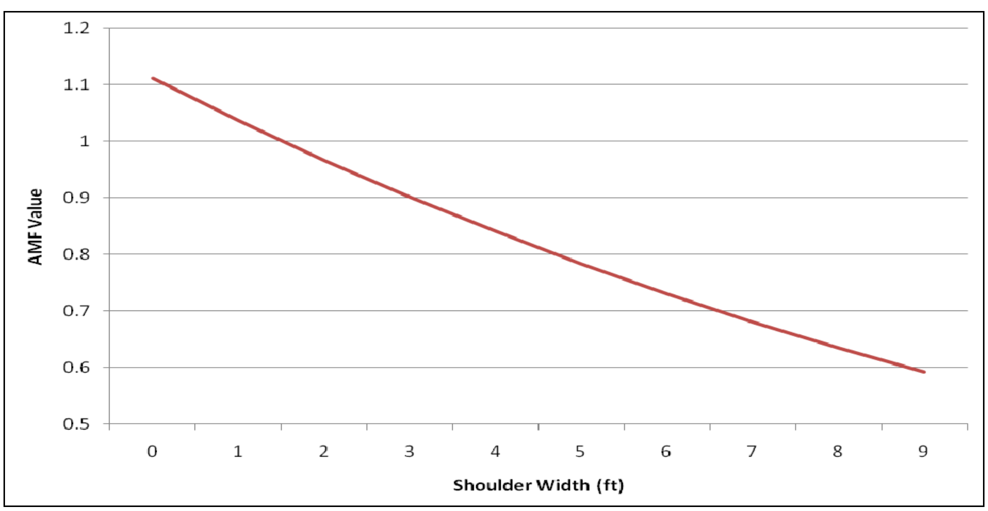
Rural frontage roads typically consist of an environment that is slightly more complex than a traditional rural two-lane highway. Equation 13-2 presents an AMF for shoulder width on rural frontage roads,[22] Exhibit 13-14 is based on Equation 13-2. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is a shoulder width (SW) of 1.5-ft.

$$AMF_{SW} = e^{-0.070(SW-1.5)} \qquad (13\text{-}2)$$

where:

SW = average paved shoulder width ([left shoulder width + right shoulder width]/2) (ft)

Exhibit 13-14: Potential Crash Effects of Paved Shoulder Width on Rural Frontage Roads[22]



NOTE: The standard error of the AMF is unknown.

To determine the AMF for changing lane width and/or AADT, divide the "new" condition AMF by the "existing" condition AMF.

The average paved shoulder width represents the sum of the left shoulder width and the right shoulder width on the frontage road divided by two. Both one-way and two-way frontage roads were considered in the development of this AMF. Development of this AMF was limited to shoulder widths ranging from 0 to 9 ft and AADT values from 100 to 6,200.

13.4.2.5. Modify Shoulder Type

Rural two-lane roads

The crash effect of modifying the shoulder type on rural two-lane roads is shown in Exhibit 13-15.[16,33,36] The crash effect varies by shoulder width and type, assuming that a paved shoulder is the base condition (i.e., the condition in which the AMF = 1.00) and that some type of shoulder is currently in place. Note that this AMF cannot be applied for a single shoulder type (horizontally across the table), the AMF in Exhibit 13-15 is exclusively for application to a situation that consists of modification from one shouler type to another shoulder type (vertically in the table for one given shoulder width).

Exhibit 13-15: Potential Crash Effects of Shoulder Type on Rural Two-Lane Roads for Related Accident Types [16,33,36]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF							
				Shoulder type	Shoulder width (ft)						
					1	2	3	4	6	8	10
Modify Shoulder Type	Rural (Two-lane Roads)	Unspecified	Single-vehicle run-off-road accidents and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe collisions (Unspecified)	Paved	1.00	1.00	1.00	1.00	1.00	1.00	1.00
				Gravel	1.00	1.01	1.01	1.01	1.02	1.02	1.03
				Composite	1.01	1.02	1.02	1.03	1.04	1.06	1.07
				Turf	1.01	1.03	1.04	1.05	1.08	1.11	1.14
Base Condition: Paved shoulder											

NOTE: Composite shoulders are 50 percent paved and 50 percent turf.

Standard error of the crash effect is unknown.

The related accident types to which this AMF applies include single-vehicle run-off-road accidents and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe collisions.

To determine the AMF for changing the shoulder type, divide the "new" condition AMF by the "existing" condition AMF.

This AMF cannot be applied for a single shoulder type to identify a change in shoulder width (horizontally in the table). This AMF is to be applied exclusively to a situation that consists of modification from one shoulder type to another shoulder type (vertically in the table for one given shoulder width).

If the shoulder types for two travel directions on a roadway segment differ, the AMF is determined separately for the shoulder type in each direction of travel and then averaged.[16]

Exhibit 13-23 and Equation 13-3 in Section 13.4.3 (Conversion Factor for Total-Crashes) may be used to determine the crash effect of shoulder type on total accidents, rather than just the accident types identified in Exhibit 13-15.

13.4.2.6. Provide a Raised Median

Urban two-lane roads

The crash effects of a raised median on urban two-lane roads are shown in Exhibit 13-16.[8] This effect may be related to the restriction of turning maneuvers at minor intersections and access points.[8] The type of raised median was unspecified.

The base condition of the AMF (i.e., the condition in which the AMF = 1.00) is the absence of a raised median.

Exhibit 13-16: Potential Crash Effects of Providing a Median on Urban Two-Lane Roads [8]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Provide a raised median	Urban (Two-lane)	Unspecified	All types (Injury)	**0.61**	0.1
Base Condition: Absence of raised median.					

NOTE: Based on International studies: Leong 1970; Thorson and Mouritsen 1971; Muskaug 1985; Blakstad and Giaever 1989

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Rural multi-lane highways and urban arterials

The crash effects of providing a median on urban arterial multi-lane roads are shown in Exhibit 13-17.[8] Providing a median on rural multi-lane roads reduces both injury and non-injury crashes, as shown in Exhibit 13-17.[8] The base condition of the AMF (i.e., the condition in which the AMF = 1.00) is the absence of a raised median.

Exhibit 13-17: Potential Crash Effects of Providing a Median on Multi-Lane Roads [8]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Provide a median	Urban (Arterial Multi-lane[(a)])	Unspecified	All types (Injury)	**0.78**[?]	0.02
			All types (Non-injury)	**1.09**[?]	0.02
	Rural (Multi-lane[(a)])		All types (Injury)	**0.88**	0.03
			All types Non-(Injury)	**0.82**	0.03
Base Condition: Absence of raised median.					

NOTE: Based on US studies: Kihlberg and Tharp 1968; Garner and Deen 1973; Harwood 1986; Squires and Parsonson 1989; Bowman and Vecellio 1994; Bretherton 1994; Bonneson and McCoy 1997 and International studies: Leon 1970; Thorson and Mouritsen 1971; Andersen 1977; Muskaug 1985; Scriven 1986; Blakstad and Giaever 1989; Dijkstra 1990; Kohler and Schwamb 1993; Claessen and Jones 1994

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

(a) Includes minor intersections

? Treatment results in a decrease in injury crashes and an increase in non-injury crashes. See Part D Introduction and Applications Guide.

13.4.2.7. Change the Width of an Existing Median

The main objective of widening medians is to reduce the frequency of severe cross-median collisions.

Rural multilane highways and urban arterials

Exhibit 13-18 through Exhibit 13-22 present AMFs for changing the median width on divided roads with traversable medians. These AMFs are based on the work by Harkey et al.[15]. Separate AMFs are provided for roads with full access control and with partial or no access control. For urban arterials, the AMFs are also dependent upon whether the arterial has four lanes or more. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the presence of traversable median width of 10-ft. The type of traversable median (grass, depressed) was not identified.

Exhibit 13-18: Potential Crash Effects of Median Width on Rural Four-Lane Roads with Full Access Control[15]

Median width (ft)	Setting (Road type)	Traffic Volume AADT	Accident type (Severity)	AMF	Std. Error
10 to 20-ft conversion	Rural (4 lanes with full access control)	2,400 to 119,000	Cross Median Crashes (Unspecified)	**0.86**	**0.02**
10 to 30-ft conversion				**0.74**	**0.04**
10 to 40-ft conversion				**0.63**	**0.05**
10 to 50-ft conversion				**0.54**	**0.06**
10 to 60-ft conversion				**0.46**	**0.07**
10 to 70-ft conversion				**0.40**	**0.07**
10 to 80-ft conversion				**0.34**	**0.07**
10 to 90-ft conversion				**0.29**	**0.07**
10 to 100-ft conversion				**0.25**	**0.06**
Base condition: Traversable median width of 10-ft					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Exhibit 13-19: Potential Crash Effects of Median Width on Rural Four-Lane Roads with Partial or No Access Control[15]

Median width (ft)	Setting (Road type)	Traffic Volume AADT	Accident type (Severity)	AMF	Std. Error
10 to 20-ft conversion	Rural (4 lanes with partial or no access control)	1,001 to 90,000	Cross Median Crashes (Unspecified)	**0.84**	**0.03**
10 to 30-ft conversion				**0.71**	**0.06**
10 to 40-ft conversion				**0.60**	**0.07**
10 to 50-ft conversion				**0.51**	**0.08**
10 to 60-ft conversion				**0.43**	**0.09**
10 to 70-ft conversion				**0.36**	**0.09**
10 to 80-ft conversion				**0.31**	**0.09**
10 to 90-ft conversion				**0.26**	**0.08**
10 to 100-ft conversion				**0.22**	**0.08**
Base condition: Traversable median width of 10-ft					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Exhibit 13-20: Potential Crash Effects of Median Width on Urban Four-Lane Roads with Full Access Control(15)

Median width (ft)	**Setting (Road type)**	**Traffic Volume AADT**	**Accident type (Severity)**	**AMF**	**Std. Error**
10 to 20-ft conversion	Urban (4 lanes with full access control)	4,410 to 131,000	Cross Median Crashes (Unspecified)	**0.89**	**0.04**
10 to 30-ft conversion				**0.80**	**0.07**
10 to 40-ft conversion				**0.71**	**0.09**
10 to 50-ft conversion				**0.64**	**0.1**
10 to 60-ft conversion				**0.57**	**0.1**
10 to 70-ft conversion				**0.51**	**0.1**
10 to 80-ft conversion				**0.46**	**0.1**
10 to 90-ft conversion				**0.41**	**0.1**
10 to 100-ft conversion				**0.36**	**0.1**
Base condition: Traversable median width of 10-ft					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Exhibit 13-21: Potential Crash Effects of Median Width on Urban Roads with at least Five Lanes with Full Access Control(15)

Median width (ft)	**Setting (Road type)**	**Traffic Volume AADT**	**Accident type (Severity)**	**AMF**	**Std. Error**
10 to 20-ft conversion	Urban (5 or more lanes with full access control)	2,555 to 282,000	Cross Median Crashes (Unspecified)	**0.89**	**0.04**
10 to 30-ft conversion				**0.79**	**0.07**
10 to 40-ft conversion				**0.71**	**0.1**
10 to 50-ft conversion				**0.63**	**0.1**
10 to 60-ft conversion				**0.56**	**0.1**
10 to 70-ft conversion				**0.50**	**0.1**
10 to 80-ft conversion				**0.45**	**0.1**
10 to 90-ft conversion				*0.40*	*0.2*
10 to 100-ft conversion				*0.35*	*0.2*
Base condition: Traversable median width of 10-ft					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

Exhibit 13-22: Potential Crash Effects of Median Width on Urban Four-Lane Roads with Partial or No Access Control(15)

Median width (ft)	Setting (Road type)	Traffic Volume AADT	Accident type (Severity)	AMF	Std. Error
10 to 20-ft conversion	Urban (4 lanes with partial or No access control)	1,880 to 150,000	Cross Median Crashes (Unspecified)	**0.87**	**0.04**
10 to 30-ft conversion				**0.76**	**0.06**
10 to 40-ft conversion				**0.67**	**0.08**
10 to 50-ft conversion				**0.59**	**0.1**
10 to 60-ft conversion				**0.51**	**0.1**
10 to 70-ft conversion				**0.45**	**0.1**
10 to 80-ft conversion				**0.39**	**0.1**
10 to 90-ft conversion				**0.34**	**0.1**
10 to 100-ft conversion				**0.30**	**0.1**
Base condition: Traversable median width of 10-ft					

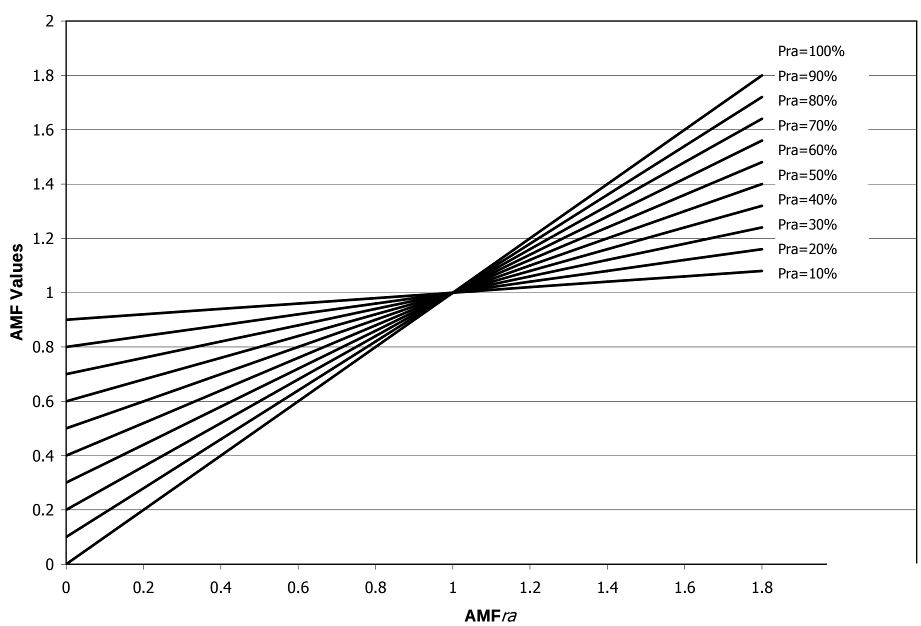
NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

13.4.3. Conversion Factor for Total-Crashes

This section presents an equation for the conversion of AMFs for crashes related to specific accident types into AMFs for total crashes.

Exhibit 13-23 and Equation 13-3 may be used to express the lane width AMF (Section 13.4.2.1), add or widen paved shoulder AMF (Section 13.4.2.4), and modify shoulder type AMF (Section 13.4.2.5) in terms of the crash effect on total accidents, rather than just the related accident types identified in the respective sections. (10,16,33)

Exhibit 13-23: Potential Crash Effects of Lane Width on Rural Two-Lane Roads on Total Accidents(16)



$$AMF = (AMF_{ra} - 1.0) \times p_{ra} + 1.0 \tag{13-3}$$

Where,

AMF = accident modification factor for total accidents

AMF_{ra} = accident modification factor for related accidents, i.e., single-vehicle run-off-road accidents and multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe collisions

P_{ra} = related accidents expressed as a proportion of total accidents

13.5. CRASH EFFECTS OF ROADSIDE ELEMENTS

13.5.1. Background and Availability of AMFs

This section presents roadside element treatments with AMFs.

The roadside is defined as the *"area between the outside shoulder edge and the right-of-way limits. The area between roadways of a divided highway may also be considered roadside"*.(23) The AASHTO Roadside Design Guide is an invaluable resource for roadside design, including clear zones, geometry, features and barriers.(23)

The knowledge presented here may be applied to roadside elements as well as to the median of divided highways. Exhibit 13-24 summarizes common treatments related to roadside elements and the corresponding AMF availability.

Exhibit 13-24: Summary of Treatments Related to Roadside Elements

HSM Section	Treatment	Rural Two-Lane Road	Rural Multi-Lane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
13.5.2	Flatten sideslopes	✓	✓	-	-	-	-
13.5.2.2	Increase distance to roadside features	✓	-	✓	-	-	-
13.5.2.3	Change roadside barrier along embankment to less rigid type	✓	✓	✓	✓	✓	✓
13.5.2.4	Install median barrier	N/A	✓	T	-	-	-
13.5.2.5	Install crash cushions at fixed roadside features	✓	✓	✓	✓	✓	✓
13.5.2.6	Reduce roadside hazard rating	✓	-	-	-	-	-
Appendix	Increase clear roadside recovery distance	T	-	-	-	-	-
Appendix	Install curbs	-	-	-	-	T	T
Appendix	Increase the distance to utility poles and decrease utility pole density	T	T	T	T	T	T
Appendix	Install roadside barrier along embankments	T	T	T	T	T	T

NOTE:
✓ = Indicates that an AMF is available for this treatment.
T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.
- = Indicates that an AMF is not available and a trend is not known.
N/A = Indicates that the treatment is not applicable to the corresponding setting.

13.5.2. Roadside Element Treatments with AMFs

13.5.2.1. Flatten Sideslopes

Rural two-lane roads

The effect on total accidents of flattening the roadside slope of a rural two-lane road is shown in Exhibit 13-25.[15] The effect on single-vehicle accidents of flattening side slopes is shown in Exhibit 13-26. [15] The base conditions of the AMFs (i.e., the condition in which the AMF = 1.00) is the sideslope in the *before* condition.

Exhibit 13-25: Potential Crash Effects on Total Accidents of Flattening Sideslopes[15]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF				
				Sideslope in Before Condition	Sideslope in After Condition			
					1V:4H	1V:5H	1V:6H	1V:7H
Flatten Sideslopes	Rural (Two-lane Road)	Unspecified	All types (Unspecified)	1V:2H	0.94	0.91	0.88	0.85
				1V:3H	0.95	0.92	0.89	0.85
				1V:4H		0.97	0.93	0.89
				1V:5H			0.97	0.92
				1V:6H				0.95
Base Condition: Existing sideslope in *before* condition.								

NOTE: Standard error of the AMF is unknown.

Exhibit 13-26: Potential Crash Effects on Single Vehicle Accidents of Flattening Sideslopes [15]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF				
				Sideslope in Before Condition	Sideslope in After Condition			
					1V:4H	1V:5H	1V:6H	1V:7H
Flatten Sideslopes	Rural (Two-lane Road)	Unspecified	Single Vehicle (Unspecified)	1V:2H	0.90	0.85	0.79	0.73
				1V:3H	0.92	0.86	0.81	0.74
				1V:4H		0.94	0.88	0.81
				1V:5H			0.94	0.86
				1V:6H				0.92
Base Condition: Existing sideslope in *before* condition.								

NOTE: Standard error of the AMF is unknown.

The gray box below presents an example of how to apply the preceding AMFs to assess the crash effects of modifying the sideslope on a rural two-lane highway.

Effectiveness of Modifying Sideslope

Question:

A high crash frequency segment of a rural two-lane highway is being analyzed for a series of improvements. Among the improvements, the reduction of the 1V:3H sideslope to a 1V:7H sideslope is being considered. What will be the likely reduction in expected average crash frequency for single vehicle crashes and total crashes?

Given Information:

- Existing roadway = rural two-lane
- Existing sideslope = 1V:3H
- Proposed sideslope = 1V:7H
- Expected average crash frequency without treatment for the segment (See Part C Predictive Method):

  a) 30 total crashes/year

  b) 8 single vehicle crashes/year

Find:

- Expected average total crash frequency with the reduction in sideslope
- Expected average single vehicle crash frequency with the reduction in sideslope
- Expected average total crash frequency reduction
- Expected average single vehicle crash frequency reduction

Answer:

1) Identify the AMFs corresponding to the change in sideslope from 1V:3H to 1V:7H

   a) For total crashes

   AMF_{Total} = 0.85 (Exhibit 13-25)

   b) For single vehicle crashes

   $AMF_{Single\ Vehicle}$ = 0.74 (Exhibit 13-26)

2) Apply the treatment AMF ($AMF_{Treatment}$) to the expected number of crashes on the rural two-lane highway without the treatment.

   a) For total crashes

   = 0.85 x 30 crashes/year = 25.5 crashes/year

   b) For single vehicle crashes

   = 0.74 x 8 crashes/year = 5.9 crashes/year

3) Calculate the difference between the expected number of crashes without the treatment and the expected number with the treatment.

   Change in Expected Average Crash Frequency

   a) For total crashes

   30.0 – 25.5 = 4.5 crashes/year reduction

   b) For single vehicle crashes

   8.0 – 5.9 = 2.1 crashes/year reduction

4) **Discussion: The change in sideslope from 1V:3H to 1V:7H may potentially cause a reduction of 4.5 total crashes/year and 2.1 single vehicle crashes/year. A standard error is not available for these AMFs.**

Rural multi-lane highways

Exhibit 13-27 presents AMFs for the effect of sideslopes on multi-lane undivided roadway segments. These AMFs were developed by Harkey et al. [10] from the work of Zegeer et al.[6] The base condition for this AMF (i.e., the condition in which the AMF = 1.00) is a sideslope of 1V:7H or flatter.

Exhibit 13-27: Potential Crash Effects of Sideslopes on Undivided Segments[15,34]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
1V:7H or Flatter	Rural (Multi-lane Highway)	Unspecified	All types (Unspecified)	1.00	N/A
1V:6H				1.05	
1V:5H				1.09	
1V:4H				1.12	
1V:2H or Steeper				1.18	
Base Condition: Provision of a 1V:7H sideslope.					

13.5.2.2. Increase the Distance to Roadside Features

Rural two-lane roads and Freeways

The crash effects of increasing the distance to roadside features from 3.3-ft to 16.7-ft, or from 16.7-ft to 30.0-ft are shown in Exhibit 13-28.[8] AMF values for other increments may be interpolated from the values presented in Exhibit 13-28.

The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is a distance of either 3.3-ft or 16.7-ft to roadside features depending on original geometry.

Exhibit 13-28: Potential Crash Effects of Increased Distance to Roadside Features [8]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Increase distance to roadside features from 3.3-ft to 16.7-ft	Rural (Two-lane roads and Freeways)	Unspecified	All types (All severities)	**0.78**	**0.02**
Increase distance to roadside features from 16.7-ft to 30.0-ft				**0.56**	**0.01**
Base Condition: Distance to roadside features of 3.3-ft or 16.7-ft depending on original geometry.					

NOTE: Based on US studies: Cirillo (1967), Zegeer et al. (1988)
Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Distance measured from the edgeline or edge of travel lane

13.5.2.3. Change Roadside Barrier along Embankment to Less Rigid Type

The type of roadside barrier applied can vary from very rigid to less rigid. In order of rigidity, the following generic types of barriers are available: [8]

- Concrete (most rigid)
- Steel
- Wire or cable (least rigid)

Rural two-lane roads, rural multi-lane highways, freeways, expressways, urban and suburban arterials

Changing the type of roadside barrier along an embankment to a less rigid type reduces the number of injury run-off-road accidents, as shown in Exhibit 13-29.[8] The AMF for fatal run-off-road accidents is shown in Exhibit 13-29.[8] A less rigid barrier type may not be suitable in certain circumstances.

The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the use of rigid barrier.

Exhibit 13-29: Potential Crash Effects of Changing Barrier to Less Rigid Type[8]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Change barrier along embankment to less rigid type	Unspecified (Unspecified)	Unspecified	Run-off-road (Injury)	**0.68**	**0.1**
			Run-off-road (Fatal)	*0.59*	*0.3*
Base Condition: Provision of a rigid roadside barrier.					

NOTE: Based on US studies: Glennon and Tamburri 1967; Tamburri, Hammer, Glennon, Lew 1968; Williston 1969; Woods, Bohuslav and Keese 1976; Ricker, Banks, Brenner, Brown and Hall 1977; Perchonok, Ranney, Baum, Morris and Eppick 1978; Hall 1982; Bryden and Fortuniewicz 1986; Schultz 1986; Ray, Troxel and Carney 1991; Hunter, Stewart and Council 1993; Gattis, Alguire and Narla 1996; Short and Robertson 1998; and International studies: Good and Joubert 1971; Pettersson 1977; Schandersson 1979; Boyle and Wright 1984; Domhan 1986; Corben, Deery, Newstead, Mullan and Dyte 1997; Ljungblad 2000

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

Distance to roadside barrierl is unspecified.

13.5.2.4. Install Median Barrier

A median barrier is *"a longitudinal barrier used to prevent an errant vehicle from crossing the highway median."*[8] The AASHTO *Roadside Design Guide* provides performance requirements, placement guidelines, and structural and safety characteristics of different median barrier systems.[1]

Rural multi-lane highways

Installing any type of median barrier on rural multi-lane highways reduces fatal and injury accidents of all types, as shown in Exhibit 13-30.[8]

The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence of a median barrier.

Exhibit 13-30: Potential Crash Effects of Installing a Median Barrier (8)

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Install any type of median barrier	Unspecified (Multi-lane divided highways)	AADT of 20,000 to 60,000	All types (Fatal)	**0.57?**	**0.1**
			All types (Injury)	**0.70?**	**0.06**
			All types (All severities)	**1.24?**	**0.03**
Install steel median barrier			All types (Injury)	**0.65**	**0.08**
Install cable median barrier				**0.71**	**0.1**
Base Condition: Absence of a median barrier.					

NOTE: Based on US studies: Billion 1956; Moskowitz and Schaefer 1960; Beaton, Field and Moskowitz 1962; Billion and Parsons 1962; Billion, Taragin and Cross 1962; Sacks 1965; Johnson 1966; Williston 1969; Galati 1970; Tye 1975; Ricker, Banks, Brenner, Brown and Hall 1977; Hunter, Steward and Council 1993; Sposito and Johnston 1999; Hancock and Ray 2000; Hunter et al 2001; and International studies: Moore and Jehu 1968; Good and Joubert 1971; Andersen 1977; Johnson 1980; Statens vagverk 1980; Martin et al 1998; Nilsson and Ljungblad 2000

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

? Treatment results in a decrease in fatal and injury crashes and an increase in crashes of all severities. See Part D Introduction and Applications Guide.

Width of the median where the barrier was installed and the use of barrier warrants are unspecified.

13.5.2.5. Install Crash Cushions at Fixed Roadside Features

Rural two-lane roads, rural multi-lane highways, freeways, expressways, urban and suburban arterials

The crash effects of installing crash cushions at fixed roadside features are shown in Exhibit 13-31.(8) The crash effects for fatal and non-injury crashes with fixed objects are also shown in Exhibit 13-31.(12) The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence of crash cushions.

Exhibit 13-31: Potential Crash Effects of Installing Crash Cushions at Fixed Roadside Features (8)

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Install crash cushions at fixed roadside features	Unspecified (Unspecified)	Unspecified	Fixed object (Fatal)	*0.31*	*0.3*
			Fixed object (Injury)	**0.31**	**0.1**
			Fixed object (Non-injury)	*0.54*	*0.3*
Base Condition: Absence of crash cushions.					

NOTE: Based on US studies: Viner and Tamanini 1973; Griffin 1984; Kurucz 1984; and International studies: Schoon 1990; Proctor 1994

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

The placement and type of crash cushions and fixed objects are unspecified.

13.5.2.6. Reduce Roadside Hazard Rating

For reference, the quantitative descriptions of the seven roadside hazard rating (RHR) levels are summarized in Exhibit 13-32. Photographs that illustrate the roadside design for each RHR level are presented in Appendix A.

Exhibit 13-32: Quantitative Descriptors for the Seven Roadside Hazard Ratings [(16)]

Rating	Clear zone width	Sideslope	Roadside
1	Greater than or equal to 30 ft	Flatter than 1V:4H; recoverable	N/A
2	Between 20 and 25 ft	About 1V:4H; recoverable	
3	About 10 ft	About 1V:3H or 1V:4H; marginally recoverable	Rough roadside surface
4	Between 5 and 10 ft	About 1V:3H or 1V:4H; marginally forgiving, increased chance of reportable roadside crash	May have guardrail (offset 5 to 6.5 ft) May have exposed trees, poles, other objects (offset 10 ft)
5		About 1V:3H; virtually non-recoverable	May have guardrail (offset 0 to 5 ft) May have rigid obstacles or embankment (offset 6.5 to 10 ft)
6	Less than or equal to 5 ft	About 1V:2H; non-recoverable	No guardrail Exposed rigid obstacles (offset 0 to 6.5 ft)
7		1V:2H or steeper; non-recoverable with high likelihood of severe injuries from roadside crash	No guardrail Cliff or vertical rock cut

NOTE: Clear zone width, guardrail offset, and object offset are measured from the pavement edgeline
N/A = no description of roadside is provided.

Rural two-lane roads

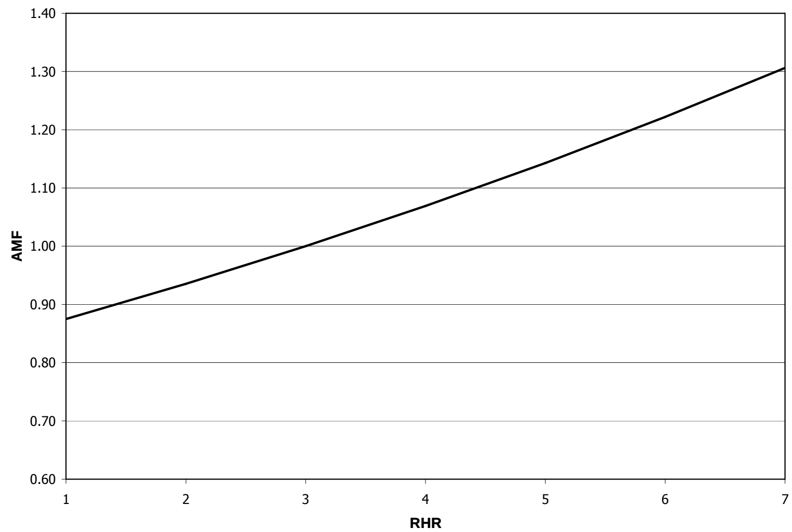
The AMFs for roadside design are presented in Equation 13-4 and Exhibit 13-33, using RHR equal to 3 as the base condition (i.e., the condition in which the AMF = 1.00).

$$AMF = \frac{e^{-0.6869+0.0668\times RHR}}{e^{-0.4865}} \qquad (13\text{-}4)$$

Where,

RHR = Roadside hazard rating for the roadway segment.

Exhibit 13-33: Potential Crash Effects of Roadside Hazard Rating for Total Accidents on Rural Two-Lane Highways(16)



NOTE: Standard error of AMF is unknown.
To determine the AMF for changing RHR, divide the "new" condition AMF by the "existing" condition AMF.
RHR = Roadside Hazard Rating

Section 13.6 summarizes treatments with AMFs related to alignment elements.

13.6. CRASH EFFECTS OF ALIGNMENT ELEMENTS

13.6.1. Background and Availability of AMFs

Exhibit 13-34 summarizes common treatments related to alignment elements and the corresponding AMF availability.

Exhibit 13-34: Summary of Treatments Related to Alignment Elements

HSM Section	Treatment	Rural Two-Lane Road	Urban Two-Lane Road	Rural Multi-Lane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
13.6.2.1	Modify horizontal curve radius and length, and provide spiral transitions	✓	-	-	-	-	-	-
13.6.2.2	Improve superelevation of horizontal curve	✓	-	-	-	-	-	-
13.6.2.3	Change vertical grade	✓	-	-	-	-	-	-
Appendix A	Modify Tangent Length Prior to Curve	T	T	T	T	T	T	T
Appendix A	Modify Horizontal Curve Radius	-	-	-	-	-	T	T

NOTE:
✓ = Indicates that an AMF is available for this treatment.
T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.
- = Indicates that an AMF is not available and a trend is not known.

13.6.2. Alignment Treatments with AMFs

13.6.2.1. Modify Horizontal Curve Radius and Length, and Provide Spiral Transitions

Rural two-lane roads

The probability of an accident generally decreases with longer curve radii, longer horizontal curve length, and the presence of spiral transitions.*(16)* The crash effect for horizontal curvature, radius, and length of a horizontal curve and presence of spiral transition curve is presented as an Accident Modification Function, as shown in Equation 13-5, the standard error of this AMF is unknown. This equation applies to all types of roadway segment accidents.*(16,35)* Exhibit 13-35 illustrates a graphical representation of Equation 13-5. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence of curvature.

$$AMF_{3r} = \frac{(1.55 \times L_c) + \left(\frac{80.2}{R}\right) - (0.012 \times S)}{(1.55 \times L_c)} \qquad (13\text{-}5)$$

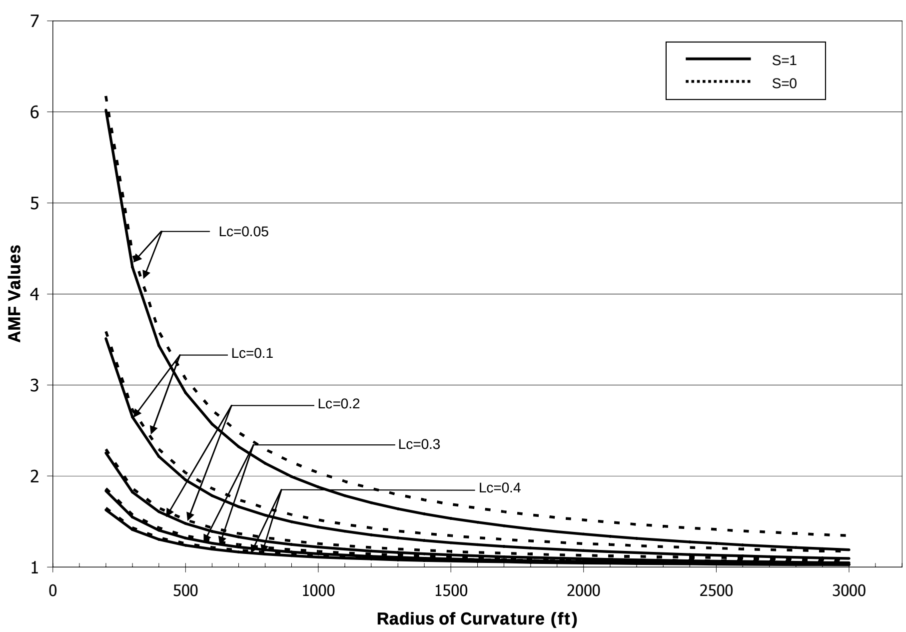
Where,

L_c= Length of horizontal curve including length of spiral transitions, if present (mi)

R= Radius of curvature (ft)

S= 1 if spiral transition curve is present; 0 if spiral transition curve is not present

Exhibit 13-35: Potential Crash Effect of the Radius, Length, and Presence of Spiral Transition Curves in a Horizontal Curve



13.6.2.2. Improve Superelevation of Horizontal Curves

Rural two-lane roads

Crash effects of superelevation variance on a horizontal curve are shown in Exhibit 13-36.(16,35) The base condition of the AMFs summarized in Exhibit 13-36 (i.e., the condition in which the AMF = 1.00) is an SV value that is less than 0.01.

Exhibit 13-36: Potential Crash Effects of Improving Superelevation Variance (SV) of Horizontal Curves on Rural Two-Lane Roads(16,35)

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF
Improve SV < 0.01	Rural (Two-lane)	Unspecified	All types (All severities)	1.00
Improve 0.01 ≤ SV < 0.02				= 1.00 + 6 (SV - 0.01)
Improve SV > 0.02				= 1.06 + 3 (SV - 0.02)
Base Condition: Superelevation variance < 0.01.				

NOTE: Standard error of AMF is unknown.

Based on a horizontal curve radius of 842.5 ft.

SV = Superelevation variance. Difference between recommended design value for superelevation and existing superelevation on a horizontal curve, where existing superelevation is less than recommended.

To determine the AMF for changing superelevation, divide the "new" condition AMF by the "existing" condition AMF.

13.6.2.3. Change Vertical Grade

Rural two-lane roads

Crash effects of increasing the vertical grade of a rural two-lane road, with a posted speed of 55 mph and a surfaced or stabilized shoulder, are shown in Exhibit 13-37.(35) The crash effect of increasing the vertical grade for accidents of all types and severities relative to a flat roadway (i.e., 0% grade) is also shown in Exhibit 13-37.(16)

These AMFs may be applied to each individual grade section on the roadway, without respect to the sign of the grade (i.e., upgrade or downgrade). These AMFs may be applied to the entire grade from one point of vertical intersection (PVI) to the next.(16)

The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is a level (0% grade) roadway.

Exhibit 13-37: Potential Crash Effects of Changing Vertical Grade on Rural Two-Lane Roads[16,24]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Increase vertical grade by 1%	Rural (Two-lane)	Unspecified	SV ROR (All severities[24])	**1.04^**	**0.02**
			All types (All severities[16])	1.02	N/A
Base Condition: Level roadway (0% grade)					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
SVROR = single-vehicle run-off-road accidents
AMFs are based on roads with 55 mph posted speed limit, 12 ft lanes, and no horizontal curves.
^ Observed variability suggests that this treatment could result in no crash effect. See Part D Applications Guidance.
N/A = Standard error of AMF is unknown.

13.7. CRASH EFFECTS OF ROADWAY SIGNS

13.7.1. Background and Availability of AMFs

Traffic signs are typically classified into three categories: regulatory signs, warning signs, and guide signs. As defined in the Manual on Uniform Traffic Control Devices (MUTCD),[19] regulatory signs provide notice of traffic laws or regulations, warning signs give notice of a situation that might not be readily apparent, and guide signs show route designations, destinations, directions, distances, services, points of interest, and other geographical, recreational or cultural information.

The MUTCD provides standards and guidance for signing within the right-of-way of all types of highways open to public travel. Many agencies supplement the MUTCD with their own guidelines and standards.

Exhibit 13-38 summarizes common treatments related to signs and the corresponding AMF availability.

Exhibit 13-38: Summary of Treatments Related to Roadway Signs

HSM Section	Treatment	Rural Two-Lane Road	Rural Multi-Lane Highway	Freeway	Expressway	Urban Local Street or Arterial	Suburban Arterial
13.7.2.1	Install combination horizontal alignment/advisory speed signs (W1-1a, W1-2a)	✓	✓	✓	✓	✓	✓
13.7.2.2	Install changeable accident ahead warning signs	-	-	✓	-	-	-
13.7.2.3	Install changeable "Queue Ahead" warning signs	-	-	✓	-	-	-
13.7.2.4	Install changeable speed warning signs	✓	✓	✓	✓	✓	✓
Appendix A	Install signs to conform to MUTCD	-	-	-	-	T	-

NOTE:
✓ = Indicates that an AMF is available for this treatment.
T = T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.
- = Indicates that an AMF is not available and a trend is not known.

13.7.2. Roadway Sign Treatments with AMFs

13.7.2.1. Install Combination Horizontal Alignment/Advisory Speed Signs (W1-1a, W1-2a)

Combination horizontal alignment/advisory speed signs are installed prior to a change in the horizontal alignment to indicate that drivers need to reduce speed.[9]

Rural two-lane roads, rural multi-lane highways, expressways, freeways, urban and suburban arterials

Compared to no signage, providing combination horizontal alignment/advisory speed signs reduces the number of all types of injury accidents, as shown in Exhibit 13-39.[8] The crash effect on all types of non-injury accidents is also shown in Exhibit 13-39.

The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence of any signage.

Exhibit 13-39: Potential Crash Effects of Installing Combination Horizontal Alignment/ Advisory Speed Signs (W1-1a, W1-2a) [(8)]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Install combination horizontal alignment/ advisory speed signs	Unspecified (Unspecified)	Unspecified	All types (Injury)	**0.87**	**0.09**
			All types (Non-injury)	*0.71*	*0.2*
Base Condition: Absence of any signage.					

NOTE: Based on US studies: McCamment 1959; Hammer 1969 and international study: Rutley 1972
Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

13.7.2.2. Install Changeable Accident Ahead Warning Signs

Freeways

Changeable accident warning signs on freeways inform drivers of an accident on the roadway ahead. The crash effect of installing changeable accident ahead warning signs on urban freeways is shown in Exhibit 13-40.[(8)] The base condition of the AMF (i.e., the condition in which the AMF = 1.00) is the absence of accident ahead warning signs.

Exhibit 13-40: Potential Crash Effects of Installing Changeable Accident Ahead Warning Signs [(8)]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Install changeable accident ahead warning signs	Urban (Freeways)	Unspecified	All types (Injury)	*0.56*	*0.2*
Base Condition: Absence of changeable accident ahead warning signs.					

NOTE: Based on International study: Duff 1971
Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

13.7.2.3. Install Changeable "Queue Ahead" Warning Signs

Changeable "Queue Ahead" warning signs give road users real-time information about queues on the road ahead.

Freeways

Crash effects of installing changeable "Queue Ahead" warning signs are shown in Exhibit 13-41.[(8)] The crash effect on rear-end non-injury accidents is also shown in Exhibit 13-41.[(8)] The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence changeable "Queue Ahead" warning signs.

Exhibit 13-41: Potential Crash Effects of Installing Changeable "Queue Ahead" Warning Signs [8]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Install changeable "Queue Ahead" warning signs	Urban (Freeways)	Unspecified	Rear-end (Injury)	**0.84**[?]	**0.1**
			Rear-end (Non-Injury)	*1.16*[?]	*0.2*
Base Condition: Absence of changeable "Queue Ahead" warning signs.					

NOTE: Based on International studies: Erke and Gottlieb 1980; Cooper, Sawyer and Rutley 1992; Persaud, Mucsi and Ugge 1995

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

? Treatment results in a decrease in injury crashes and an increase in non-injury crashes. See Part D Introduction and Applications Guide.

13.7.2.4. Install Changeable Speed Warning Signs

Individual changeable speed warning signs give individual drivers real-time feedback regarding their speed.

Rural two-lane roads, rural multi-lane highways, expressways, freeways, urban and suburban arterials

The crash effect of installing individual changeable speed warning signs is shown in Exhibit 13-42. The base condition of the AMF (i.e., the condition in which the AMF = 1.00) is the absence of changeable speed warning signs.

Exhibit 13-42: Potential Crash Effects of Installing Changeable Speed Warning Signs for Individual Drivers [8]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Install changeable speed warning signs for individual drivers	Unspecified (Unspecified)	Unspecified	All types (All severities)	*0.54*	*0.2*
Base Condition: Absence of changeable speed warning signs.					

NOTE: Based on International study: Van Houten and Nau 1981

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

13.8. CRASH EFFECTS OF ROADWAY DELINEATION

13.8.1. Background and Availability of AMFs

Delineation includes all methods of defining the roadway operating area for drivers, and has long been considered an essential element for providing guidance to drivers. Methods of delineation include devices such as pavement markings (made from a variety of materials), raised pavement markers (RPMs), chevron signs, object markers, and post-mounted delineators (PMDs).[11] Delineation may be used alone to convey regulations, guidance, or warnings.[19] Delineation may also be used to supplement other traffic control devices, such as signs and signals. The MUTCD provides guidelines for retroreflectivity, color, placement, types of materials and other delineation issues.[19]

Exhibit 13-43 summarizes common treatments related to delineation and the corresponding AMF availability.

Exhibit 13-43: Summary of Treatments Related to Delineation

HSM Section	Treatment	Rural Two-Lane Road	Rural Multi-Lane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
13.8.2.1	Install post-mounted delineators (PMDs)	✓	-	-	-	-	-
13.8.2.2	Place standard edgeline markings	✓	-	-	-	-	-
13.8.2.3	Place wide edgeline markings	✓	-	-	-	-	-
13.8.2.4	Place centerline markings	✓	-	N/A	N/A	-	-
13.8.2.5	Place edgeline and centerline markings	✓	✓	N/A	N/A	-	-
13.8.2.6	Install edgelines, centerlines and post-mounted delineators	✓	✓	N/A	N/A	-	-
13.8.2.7	Install snowplowable permanent raised pavement markers (RPMs)	✓	-	✓	-	-	-
Appendix A	Install chevron signs on horizontal curves	-	-	-	-	T	T
Appendix A	Provide distance markers	-	-	T	-	-	-
Appendix A	Place converging chevron pattern markings	-	-	-	-	T	T
Appendix A	Place edgeline and directional pavement markings on horizontal curves	T	-	-	-	-	-

NOTE:
✓ = Indicates that an AMF is available for this treatment.
T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.
- = Indicates that an AMF is not available and a trend is not known.
N/A = Indicates that the treatment is not applicable to the corresponding setting.

13.8.2. Roadway Delineation Treatments with AMFs

13.8.2.1. Install Post-Mounted Delineators (PMDs)

Post-mounted delineators (PMDs) are considered guidance devices rather than warning devices.[9] PMDs are typically installed in addition to existing edgeline and centerline markings.

Rural two-lane roads

The crash effects of installing PMDs on rural two-lane roads, including tangent and curved road sections, is shown in Exhibit 13-44. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence of post-mounted delineators.

Exhibit 13-44: Potential Crash Effects of Installing Post-Mounted Delineators (8)

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Install post-mounted delineators	Rural (Two-lane undivided)	Unspecified	All types (Injury)	1.04*	0.1
			All types (Non-injury)	**1.05***	**0.07**
Base Condition: Absence of post-mounted delineators					

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

13.8.2.2. Place Standard Edgeline Markings (4 to 6 in)

The MUTCD contains guidance on installing edgeline pavement markings.(9)

Rural two-lane roads

The crash effects of installing standard edgeline markings, 4 to 6 inches wide, on rural two-lane roads that currently have centerline markings is shown in Exhibit 13-45. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence of standard edgeline markings.

Exhibit 13-45: Potential Crash Effects of Placing Standard Edgeline Markings(8)

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Place standard edgeline marking	Rural (Two-lane)	Unspecified	All types (Injury)	**0.97***	**0.04**
			All types (Non-injury)	**0.97***	**0.1**
Base Condition: Absence of standard edgeline markings.					

NOTE: Based on US studies: Thomas 1958; Musick 1960; Williston 1960; Basile 1962; Tamburri, Hammer, Glennon and Lew 1968; Roth 1970; Bali, Potts, Fee, Taylor and Glennon 1978 and International studies: Charnock and Chessell 1978, McBean 1982; Rosbach 1984; Willis, Scott and Barnes 1984; Corben, Deery, Newstead, Mullan and Dyte 1997

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

13.8.2.3. Place Wide (8-in) Edgeline Markings

The MUTCD indicates that wide (8 inch) solid edgeline markings can be installed for greater emphasis.(9)

Rural two-lane roads

The crash effects of placing 8 inches wide edgeline markings on rural two-lane roads that currently have standard edgeline markings are shown in Exhibit 13-46.(8) The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the use of standard edgeline markings (4 to 6 inches wide).

Exhibit 13-46: Potential Crash Effects of Placing Wide Edgeline Markings[(8)]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Place wide edgeline markings	Rural (Two-lane)	Unspecified	All types (Injury)	**1.05*?**	**0.08**
			All types (Non-injury)	_0.99*?_	_0.2_
Base Condition: Standard edgeline markings (4 to 6 inches wide).					

NOTE: Based on U.S. studies: Hall 1987; Cottrell 1988; Lum and Hughes 1990

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

? Treatment results in an increase in injury crashes and a decrease in non-injury crashes. See Part D Introduction and Applications Guide.

13.8.2.4. Place Centerline Markings

The MUTCD provides guidelines and warrants for centerline marking application.[(9)]

Rural two-lane roads

The crash effects of placing centerline markings on rural two-lane roads that currently do not have centerline markings are shown in Exhibit 13-47.[(8)] The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence of centerline markings.

Exhibit 13-47: Potential Crash Effects of Placing Centerline Markings[(8)]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Place centerline markings	Rural (Two-lane)	Unspecified	All types (Injury)	**0.99*?**	**0.06**
			All types (Non-injury)	**1.01*?**	**0.05**
Base Condition: Absence of centerline markings.					

NOTE: Based on US studies: Tamburri, Hammer, Glennon and Lew 1968; Glennon 1986 and International studies: Engel and Krogsgard Thomsen 1983

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

? Treatment results in a decrease in injury crashes and an increase in non-injury crashes. See Part D Introduction and Applications Guide.

Study does not report if the roadway segments meet MUTCD guidelines for centerline marking application

13.8.2.5. Place Edgeline and Centerline Markings

The MUTCD provides guidelines and warrants for edgeline and centerline marking application.[9]

Rural two-lane roads and rural multi-lane highways

Placing edgeline and centerline markings where no markings exist decreases injury accidents of all types, as shown in Exhibit 13-48. The base condition of the AMF (i.e., the condition in which the AMF = 1.00) is the absence of markings.

Exhibit 13-48: Potential Crash Effects of Placing Edgeline and Centerline Markings [8]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Place edgeline and centerline markings	Rural (Two-lane/ Multilane undivided)	Unspecified	All types (Injury)	**0.76**	**0.1**
Base Condition: Absence of markings.					

NOTE: Based on US study: Tamburri, Hammer, Glennon and Lew, 1968
Study does not report if the roadway segments meet MUTCD guidelines for edgeline and centerline marking application
Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

13.8.2.6. Install Edgelines, Centerlines and Post-Mounted Delineators

Edgeline markings, centerline markings and post-mounted delineators are often combined on roadway segments.

Rural two-lane road and rural multi-lane highways

The crash effects of installing edgelines, centerlines, and post mounted delineators where no markings exist are shown in Exhibit 13-49. The base condition of the AMF (i.e., the condition in which the AMF = 1.00) is the absence of markings.

Exhibit 13-49: Potential Crash Effects of Installing Edgelines, Centerlines and Post-Mounted Delineators [8]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Install edgelines, centerlines and delineators	Urban/ Rural (Two-lane/ multilane Undivided)	Unspecified	All types (Injury)	**0.55**	**0.1**
Base Condition: Absence of markings.					

NOTE: Based on US studies: Tamburri, Hammer, Glennon and Lew 1968, Roth 1970
Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

13.8.2.7. Install Snowplowable Permanent Raised Pavement Markers (RPMs)

Implementing snowplowable permanent raised pavement markers (RPMs) requires consideration of traffic volumes and horizontal curvature.[2]

Rural two-lane roads

The crash effects of installing snowplowable RPMs on low volume (AADT of 0 to 5,000), medium volume (AADT of 5,001 to 15,000), and high volume (AADT of 15,001 to 20,000) roads are shown in Exhibit 13-50.(2)

The varying crash effect by traffic volume is likely due to the lower design standards (e.g., narrower lanes, narrower shoulders, etc.) associated with low-volume roads.(2) Providing improved delineation, such as RPMs, may cause drivers to increase their speeds. The varying crash effect by curve radius is likely related to the negative impact of speed increases.(2) The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence raised pavement markers.

Exhibit 13-50: Potential Crash Effects of Installing Snowplowable Permanent Raised Pavement Markers (RPMs) (2)

Treatment	Setting (Road type)	Traffic Volume AADT	Accident type (Severity)	AMF	Std. Error
Install snowplowable permanent RPMs	Rural (Two-lane with radius > 1640 ft)	0 to 5,000	Nighttime All types (All severities)	**1.16**	**0.03**
		5,001 to 15,000		**0.99***	**0.06**
		15,001 to 20,000		**0.76**	**0.08**
	Rural (Two-lane with radius ≤ 1640 ft)	0 to 5,000		**1.43**	**0.1**
		5,001 to 15,000		**1.26**	**0.1**
		15,001 to 20,000		**1.03***	**0.1**
Base Condition: Absence of raised pavement markers.					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

Freeways

The crash effects of installing snowplowable RPMs on rural four-lane freeways for nighttime crashes by traffic volume is shown in Exhibit 13-51.(2) The varying crash effect by traffic volume is likely due to the lower design standards (e.g., narrower lanes, narrower shoulders, etc.) associated with lower-volume roads.(2) The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence raised pavement markers.

Exhibit 13-51: Potential Crash Effects of Installing Snowplowable Permanent Raised Pavement Markers (RPMs) [2]

Treatment	Setting (Road type)	Traffic Volume AADT	Accident type (Severity)	AMF	Std. Error
Install snowplowable PRPMs	Rural (Four-lane freeways)	≤ 20,000	Nighttime All types (All severities)	*1.13**	*0.2*
		20,001 to 60,000		*0.94**	*0.3*
		> 60,000		*0.67*	*0.3*
Base Condition: Absence of raised pavement markers.					

NOTE: *Italic* text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

13.9. CRASH EFFECTS OF RUMBLE STRIPS

13.9.1. Background and Availability of AMFs

Rumble strips warn drivers by creating vibration and noise when driven over. The objective of rumble strips is to reduce crashes caused by drowsy or inattentive drivers. In general, rumble strips are used in areas where the noise generated is unlikely to disturb adjacent residents; that is, in non-residential areas. The decision to incorporate rumble strips may also depend on the presence of bicyclists on the roadway segment.

Jurisdictions have not identified additional maintenance requirements with respect to rumble strips.[23] The vibratory effects of rumble strips can be felt in snow and icy conditions and may act as a guide to drivers in inclement weather.[13] Analysis of downstream crash data for shoulder rumble strips found migration and/or spillover of crashes to be unlikely.[13]

Exhibit 13-52 summarizes common treatments related to rumble strips and the corresponding AMF availability.

Exhibit 13-52: Summary of Treatments Related to Rumble Strips

HSM Section	Treatment	Rural Two-Lane Road	Urban Two-Lane Road	Rural Multi-Lane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
13.9.2.1	Install continuous shoulder rumble strips	-	-	✓	✓	-	-	-
13.9.2.2	Install centerline rumble strips	✓	-	-	N/A	N/A	-	-
Appendix A	Install continuous shoulder rumble strips and wider shoulders	-	-	-	T	-	-	-
Appendix A	Install transverse rumble strips	T	-	-	-	-	-	-

NOTE:
✓ = Indicates that an AMF is available for this treatment.
T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.
- = Indicates that an AMF is not available and a trend is not known.
N/A = Indicates that the treatment is not applicable to the corresponding setting.

13.9.2. Rumble Strip Treatments with AMFs

13.9.2.1. Install Continuous Shoulder Rumble Strips

Shoulder rumble strips are installed on a paved roadway shoulder near the travel lane. Shoulder rumble strips are made of a series of indented, milled, or raised elements intended to alert inattentive drivers, through vibration and sound, that their vehicles have left the roadway. On divided highways, shoulder rumble strips are typically installed on both the inner and outer shoulders (i.e., median and right shoulders).[28]

The impact of shoulder rumble strips on motorcycles or bicyclists has not been quantified in terms of crash experience.[29]

Continuous shoulder rumble strips are applied with consistently small spacing between each groove (generally less than 1-ft. There are no gaps of smooth pavement longer than about 1-ft.

Rural multi-lane highways

The crash effects of installing continuous shoulder rumble strips on rural multi-lane divided highways, with posted speeds of 55 to 70 mph are shown in Exhibit 13-53.[6] The crash effects on all types of injury severity and single-vehicle run-off-road accidents are also shown in Exhibit 13-53. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence shoulder rumble strips.

Exhibit 13-53: Potential Crash Effects of Installing Continuous Shoulder Rumble Strips on Multi-Lane Highways (6)

Treatment	Setting (Road type)	Traffic Volume (AADT)	Accident type (Severity)	AMF	Std. Error
Install continuous milled-in shoulder rumble strips	Rural (Multi-lane divided)	2,000 to 50,000	All types (All severities)	**0.84**	**0.1**
			All types (Injury)	*0.83*	*0.2*
			SV ROR (All severities)	*0.90**	*0.3*
			SV ROR (Injury)	*0.78**	*0.3*
Base Condition: Absence of shoulder rumble strips.					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.
SV ROR = Single-Vehicle Run-off-road accidents
* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

Freeways

There are specific circumstances in which installing continuous shoulder rumble strips on all four shoulders reduces single-vehicle run-off-road (SV ROR) accidents. The specific circumstances are SV ROR crashes with contributing factors including alcohol, drugs, inattention, inexperience, fatigue, illness, distraction, and glare. The AMFs are presented in Exhibit 13-54.(25)

The crash effects on all SV ROR accidents of all severities and injury severity are also shown in Exhibit 13-54. There is no evidence that shoulder rumble strips have an effect on multi-vehicle accidents within the boundaries of the treatment area.(13) The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence of shoulder rumble strips.

Exhibit 13-54: Potential Crash Effects of Installing Continuous Shoulder Rumble Strips on Freeways (25,13)

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Install continuous milled-in shoulder rumble strips (6)	Urban/Rural (Freeway)	Unspecified	Specific SV ROR (All severities)	**0.21**	**0.07**
Install continuous rolled-in shoulder rumble strips (11)	Urban/Rural (Freeway)		SV ROR (All severities)	**0.82**	**0.1**
			SV ROR (Injury)	*0.87*	*0.2*
	Rural (Freeway)		SV ROR (All severities)	*0.79*	*0.2*
			SV ROR (Injury)	*0.93**	*0.3*
Base Condition: Absence of shoulder rumble strips.					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.
SV ROR = Single Vehicle Run-off-road accidents
Specific SV ROR crashes have certain causes including alcohol, drugs, inattention, inexperience, fatigue, illness, distraction, and glare.
* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

The gray box below presents an example of how to apply the preceding AMFs to assess the crash effects of implementing rumble strips on an urban freeway.

Part D Introductions and Applications Guidance provides information about the application of AMFs.

Effectiveness of Implementing Rumble Strips

Question:

The installation of rumble strips is being considered along an urban freeway segment to reduce single vehicle run of the road (SV ROR) crashes. What will be the likely change in expected average crash frequency?

Given Information:

- Existing roadway = urban freeway
- Average crash frequency without treatment = 22 crashes/year

Find:

- Average crash frequency with installation of rumble strips
- Change in average crash frequency

Answer:

1) Identify the applicable AMF

   AMF = 0.82 (Exhibit 13-54)

2) Calculate the 95th percentile confidence interval estimation of crashes with the treatment

   = (0.82 ± 2 x 0.10) x (22 crashes/year) = 13.6 or 22.4 crashes/year

   A standard error is provided for this AMF in Exhibit 13-54 as 0.10. The multiplication of the standard error by 2 yields a 95% probability that the true value is between 13.6 and 22.4 crashes/year. See Section 3.5.3 in *Chapter 3 Fundamentals* for a detailed explanation.

3) Calculate the difference between the number of crashes without the treatment and the number of crashes with the treatment.

   Change in Average Crash Frequency:

   Low Estimate = 22.4 – 22.0 = -0.4 crashes/year increment

   High Estimate = 22.4 – 13.6 = 8.8 crashes/year reduction

4) **Discussion: This example illustrates that the installation of rumble strips is more likely to result in a decrease in expected average crash frequency. However, there is also a probability that crashes will remain unchanged or experience a slight increase.**

13.9.2.2. Install Centerline Rumble Strips

Centerline rumble strips are installed on undivided roadways, along the centerline that divides opposing traffic. Centerline rumble strips target head-on and opposite-direction sideswipe crashes. A secondary target is drift-off run-off-road-to-the-left crashes. Centerline rumble strips may reduce risky passing, but this is not their primary intent and the effect on risky passing is not known.

National guidelines do not exist for the application of centerline rumble strips. Appendix A contains information about the placement of centerline rumble strips in relation to centerline markings.

Rural two-lane roads

The crash effects of installing centerline rumble strips on rural two-lane roads are shown in Exhibit 13-55.[8] The crash effects for frontal and opposing-direction sideswipe accidents are also shown in Exhibit 13-55.

The AMFs are applicable to a range of centerline rumble strip designs (e.g., milled in, rolled in, formed, raised) and placements (e.g., continuous, intermittent).[26] The AMFs are also applicable to horizontal curves and tangent sections, passing and no-passing zones.[26] The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence of centerline rumble strips.

Exhibit 13-55: Potential Crash Effects of Installing Centerline Rumble Strips [14]

Treatment	Setting (Road type)	Traffic Volume AADT	Accident type (Severity)	AMF	Std. Error
Install centerline rumble strips	Rural (Two-lane)	5,000 to 22,000	All types (All severities)	**0.86**	**0.05**
			All types (Injury)	**0.85**	**0.08**
			Frontal and opposing-direction sideswipe (All severities)	**0.79**	**0.1**
			Frontal and opposing-direction sideswipe (Injury)	*0.75*	*0.2*
Base Condition: Absence of centerline rumble strips.					

NOTE: Based on centerline rumble strip installation in seven states: California, Colorado, Delaware, Maryland, Minnesota, Oregon, and Washington

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

13.10. CRASH EFFECTS OF TRAFFIC CALMING

13.10.1. Background and Availability of AMFs

Some objectives of traffic calming are to reduce traffic speed and/or traffic volume, in order to reduce conflicts between local traffic and through traffic, make it easier for pedestrians to cross the road, and reduce traffic noise. Traffic calming measures and devices are applied in different combinations to suit the specific road environment and the specific result desired.

Traffic calming measures have grown in application over the past 15 years in North America. Various factors have contributed including the desire to provide a shared space between vehicular, pedestrian, and bicyclist traffic.

Exhibit 13-56 summarizes common treatments related to traffic calming and the corresponding AMF availability.

Exhibit 13-56: Summary of Treatments Related to Traffic Calming

HSM Section	Treatment	Rural Two-Lane Road	Rural Multi-Lane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
13.10.2.1	Install speed humps	N/A	N/A	N/A	N/A	✓	✓
Appendix A	Install transverse rumble strips	-	-	N/A	N/A	T	T
Appendix A	Apply several traffic calming measures to a road segment	N/A	N/A	N/A	N/A	✓	-

NOTE:
✓ = Indicates that an AMF is available for this treatment.
T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.
- = Indicates that an AMF is not available and a trend is not known.
N/A = Indicates that the treatment is not applicable to the corresponding setting.

13.10.2. Traffic Calming Treatments with AMFs

This section provides a summary of traffic calming treatments with AMFs.

13.10.2.1. Install Speed Humps

Speed humps are most commonly used on residential roads in urban or suburban environments to reduce speeds and, in some cases, to reduce traffic volumes.

Urban and suburban arterials

The crash effects of installing speed humps for treated roads and for adjacent untreated roads are shown in Exhibit 13-57.(8) The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence of speed humps.

Exhibit 13-57: Potential Crash Effects Of Installing Speed Humps (8)

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Adjacent to roads with speed humps	Urban/ Suburban (Residential Two-lane)	Unspecified	All types (Injury)	**0.95***	**0.06**
Install speed humps				*0.60*	*0.2*
Base Condition: Absence of speed humps.					

NOTE: Based on U.S. studies: Ewing 1999 and International studies: Baguley 1982; Blakstad and Giæver 1989; Giæver and Meland 1990; Webster 1993; Webster and Mackie 1996; ETSC 1996; Al Masaeid 1997; Eriksson and Agustsson 1999; Agustsson 2001

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

13.11. CRASH EFFECTS OF ON-STREET PARKING

13.11.1. Background and Availability of AMFs

There are two broad types of parking facilities: at the curb or on-street parking, and off-street parking in lots or parking structures.(22) Parking safety is influenced by a complex set of driver and pedestrian attitudinal and behavioral patterns.(32)

Certain kinds of accidents may be caused by curb or on-street parking operations, these include:

- Sideswipe and rear-end crashes resulting from lane changes due to the presence of a parking vehicle or contact with a parked car;
- Sideswipe and rear-end crashes resulting from vehicles stopping prior to entering the parking stall;
- Sideswipe and rear-end crashes resulting from vehicles exiting parking stalls and making lane changes; and
- Pedestrian crashes resulting from passengers alighting from the street-side doors of parked vehicles, or due to pedestrians obscured by parked vehicles.

Exhibit 13-58 summarizes common treatments related to on-street parking and the corresponding AMF availability.

Section 13.11 provides a summary of parking treatments with AMFs.

Exhibit 13-58: Summary of Treatments Related to On-Street Parking

HSM Section	Treatment	Rural Two-Lane Road	Rural Multi-Lane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
13.11.2.1	Prohibit on-street parking	N/A	N/A	N/A	N/A	✓	N/A
13.11.2.2	Convert free to regulated on-street parking	N/A	N/A	N/A	N/A	✓	N/A
13.11.2.3	Implement time-limited on-street parking restrictions	N/A	N/A	N/A	N/A	✓	N/A
13.11.2.4	Convert angle parking to parallel parking	N/A	N/A	N/A	N/A	✓	N/A

NOTE:
✓ = Indicates that an AMF is available for this treatment.
- = Indicates that an AMF is not available and a trend is not known.
N/A = Indicates that the treatment is not applicable to the corresponding setting.

13.11.2. Parking Treatments with AMFs

13.11.2.1. Prohibit On-Street Parking

Many factors may be considered before removing or altering on-street parking. These factors include parking demand, road geometry, traffic operations, and safety.

Urban arterials

Crash effects of prohibiting on-street parking on urban arterials with AADT traffic volumes from 30,000 to 40,000 are shown in Exhibit 13-59. The base condition of the AMFs summarized in Exhibit 13-59 (i.e., the condition in which the AMF = 1.00) is the provision of on-street parking.

Exhibit 13-59: Potential Crash Effects of Prohibiting On-Street Parking[22,19]

Treatment	Setting (Road type)	Traffic Volume AADT	Accident type (Severity)	AMF	Std. Error
Prohibit on-street parking	Urban (Arterial (64 ft wide))	30,000	All types (All severities)	**0.58**	**0.08**
Prohibit on-street parking	Urban (Arterial)	30,000 to 40,000	All types (Injury)	**0.78+**	**0.05**
			All types (Non-injury)	**0.72+**	**0.02**
Base Condition: Provision of on-street parking.					

NOTE: (10) Based on U.S. studies: Crossette and Allen 1969; Bonneson and McCoy 1997 and International studies: Madelin and Ford 1968; Good and Joubert 1973; Main 1983; Westman 1986; Blakstad and Giaever 1989

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

+ Combined AMF, See Part D Introduction and Applications Guide.

Accident migration is a possible result of prohibiting on-street parking.[19] Drivers may use different streets to find on-street parking, or they may take different routes to off-street parking. Shifts in travel modes may also occur as a result of the reduction in parking spaces caused by prohibiting on-street parking. Drivers may choose to walk, cycle, or use public transportation. However, the crash effects are not certain at this time.

13.11.2.2. Convert Free to Regulated On-Street Parking

Regulated on-street parking includes time-limited parking, reserved parking, area/place-limited parking, and paid parking.

Urban arterials

The crash effects of converting free parking to regulated on-street parking on urban arterials are shown in Exhibit 13-60.[8] The crash effect on injury accidents of all types is also shown in Exhibit 13-60. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the provision of free parking.

Exhibit 13-60: Potential Crash Effects of Converting from Free to Regulated On-Street Parking [8]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Convert free to regulated parking	Urban (Arterial)	Unspecified	All types (Injury)	**0.94*?**	**0.08**
			All types (Non-injury)	**1.19?**	**0.05**
Base Condition: Provision of free parking.					

NOTE: Based on U.S. studies: Cleveland, Huber and Rosenbaum 1982 and International studies: Dijkstra 1990
Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.
? Treatment results in a decrease in injury crashes and an increase in non-injury crashes. See Part D Introduction and Applications Guide.

13.11.2.3. Implement Time-Limited On-Street Parking Restrictions

Time-limited on-street parking may consist of parking time limitations ranging from 15 minutes to several hours.

Urban arterials

The crash effects of implementing time-limited parking restrictions to regulate previously unrestricted parking on urban arterials and collectors are shown in Exhibit 13-61.[8] The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the provision of unrestricted parking.

Exhibit 13-61: Potential Crash Effects of Implementing Time-Limited On-Street Parking [8]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Implement time-limited parking restrictions	Urban (Arterial and Collector)	Unspecified	All types (All severities)	**0.89**	**0.06**
			Parking-related accidents (All severities)	**0.21**	**0.09**
Base Condition: Provision of unrestricted parking.					

NOTE: Based on U.S. studies: DeRose 1966; LaPlante 1967
Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

13.11.2.4. Convert Angle Parking to Parallel Parking

In recent years, some agencies have replaced angle curb parking configurations with parallel parking for safety and operational reasons. Converting angle parking to parallel parking reduces the number of parking spaces, but increases the sightlines for drivers exiting the parking position and reduces weaving time.

Urban arterials

The crash effect of converting angle parking to parallel parking on urban arterials is incorporated in an AMF for on-street parking that includes the crash effects not only of angle vs. parallel parking, but also of the type of development along the arterial and the proportion of curb length with on-street parking.[5] The base condition of the AMF (i.e., the condition in which the AMF = 1.00) is the absence of on-street parking. An AMF for changing from angle parking to parallel parking can be determined by dividing the AMF determined for parallel parking by the AMF determined for angle parking. This AMF applies to total roadway segment accidents. The standard error for this AMF is unknown.

The AMF is determined as:

$$AMF_{1r} = 1.00 + p_{pk}\ (f_{pk} - 1.00) \qquad (13\text{-}6)$$

Where,

AMF_{1r} = accident modification factor for the effect of on-street parking on total accidents;

f_{pk} = factor from Exhibit 13-63;

p_{pk} = proportion of curb length with on-street parking = (0.5 Lpk/L');

L_{pk} = sum of curb length with on-street parking for both sides of the road combined; and

L' = total roadway segment length with deductions for intersection widths, crosswalks, and driveway widths.

Exhibit 13-62: Potential Crash Effects of Implementing On-Street Parking[5]

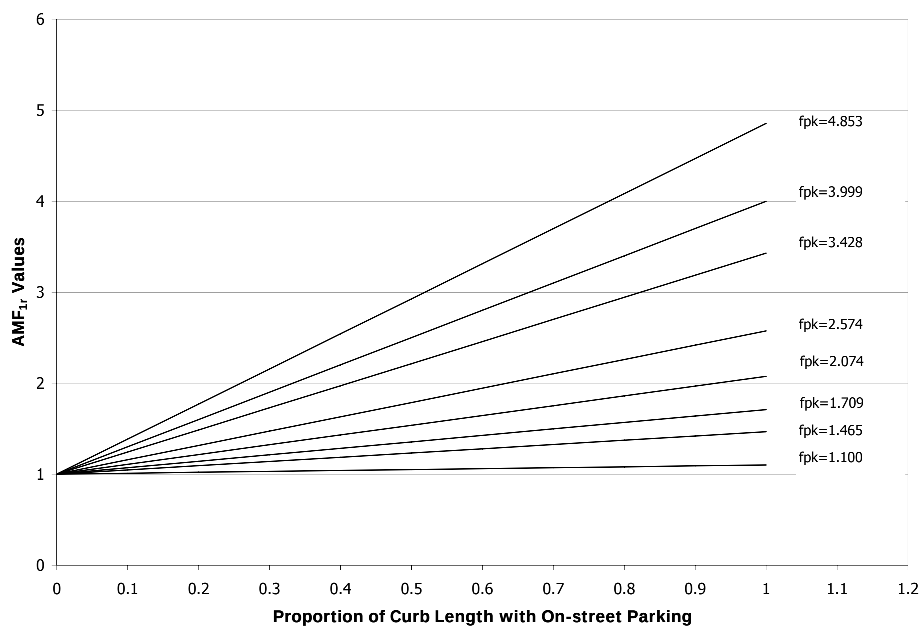


Exhibit 13-63: Type of Parking and Land Use Factor (f_{pk} in Equation 13-6)

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

NOTE: 2U = Two-lane undivided arterials. 3T = Three-lane arterial including a center two-way left-turn lane (TWLTL). 4U = Four-lane undivided arterial. 4D = Four-lane divided arterial (i.e., including a raised or depressed median). 5T = Five-lane arterial including a center TWLTL.

Accident migration is a possible result of converting angle parking to parallel parking, in part because of the reduced number of parking spaces. Drivers may use different streets to find on-street parking, or take different routes to off-street parking. Shifts in travel modes may also occur as a result of the reduction in parking spaces as a result of converting angle parking to parallel parking. However, the crash effect is not certain at this time.

The gray box below presents an example of how to apply the preceding equations and graphs to assess the crash effects of modifying angle to parallel parking on a residential two-lane arterial road.

Effectiveness of Modifying Angle to Parallel Parking

Question:

A 3,000-ft segment of a two-lane undivided arterial in a residential area currently provides angle parking for nearby residents on about 80% of its total length. The local jurisdiction is investigating the impacts of modifying the parking scheme to parallel parking. What will be the likely reduction in expected average crash frequency for the entire 3,000-ft segment?

Given Information:

- Existing roadway = Two-lane undivided arterial (2U in Exhibit 13-63)
- Setting = Residential area
- Length of roadway = 3,000-ft
- Percent of roadway with Parking = 80%
- Expected average crash frequency with angle parking for the entire 3,000-ft segment (Seep Part C Predictive Method) = 8 crashes/year

The Part D Introduction and Applications Guide provides information about applying AMFs.

Find:

- Expected average crash frequency with modification from angle to parallel parking
- Change in expected average crash frequency

Answer:

5) Identify the parking and land use factor for existing condition angle parking
f_{pk} = 3.428 (Exhibit 13-63)

6) Identify the parking and land use factor for proposed condition parallel parking
f_{pk} = 1.465 (Exhibit 13-63)

7) Calculate the AMF for the existing condition
AMF = 2.94 (Equation 13-6 or Exhibit 13-62)

8) Calculate the AMF for the proposed condition
AMF= 1.37 (Equation 13-6 or Exhibit 13-62)

9) Calculate the treatment AMF ($AMF_{Treatment}$) corresponding to the change in parking scheme

$AMF_{Treatment}$ = 1.37/2.94 = 0.47

The treatment AMF is calculated as the ratio between the existing condition AMF and the proposed condition AMF. Whenever the existing condition is not equal to the base condition for a given AMF, a division of existing condition AMF (where available) and proposed condition AMF will be required.

10) Apply the treatment AMF ($AMF_{Treatment}$) to the expected number of crashes along the roadway segment without the treatment.

= 0.47 x 8 crashes/year = 3.8 crashes/year

11) Calculate the difference between the expected number of crashes without the treatment and the expected number of crashes with the treatment.

Change in Expected Average Crash Frequency:

= 8.0 – 3.8 = 4.2 crashes/year reduction

12) Discussion: the change in parking scheme may potentially result in a reduction of 4.2 crashes/year. A standard error was not available for this AMF.

1113

Section 13.12 presents information about pedestrian and bicyclist treatments with AMFs.

13.12. CRASH EFFECTS OF ROADWAY TREATMENTS FOR PEDESTRIANS AND BICYCLISTS

13.12.1. Background and Availability of AMFs

Pedestrians and bicyclists are considered vulnerable road users as they are more susceptible to injury than vehicle occupants when involved in a traffic crash. Vehicle occupants are usually protected by the vehicle.

The design of accessible pedestrian facilities is required and is governed by implementing regulations under the Rehabilitation Act of 1973 and the Americans with Disabilities Act (ADA) of 1990. These two acts reference specific design and construction standards for usability.[(6)] Appendix A presents a discussion of design guidance resources including the PEDSAFE Guide.

For most treatments concerning pedestrian and bicyclist safety at intersections, the road type is unspecified. Where specific site characteristics are known, they are stated.

Exhibit 13-64 summarizes common roadway treatments for pedestrians and bicyclists, there are currently no AMFs available for these treatments. Appendix A presents general information and potential trends in crashes and user behavior for applicable roadway types.

Exhibit 13-64: Summary of Roadway Treatments for Pedestrians and Bicyclists

HSM Section	Treatment	Rural Two-Lane Road	Rural Multi-Lane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
Appendix A	Provide a sidewalk or shoulder	N/A	N/A	N/A	N/A	T	-
Appendix A	Install raised pedestrian crosswalks	N/A	N/A	N/A	N/A	T	T
Appendix A	Install pedestrian-activated flashing yellow beacons with overhead signs and advance pavement markings	N/A	N/A	N/A	N/A	T	T
Appendix A	Install overhead electronic signs with pedestrian-activated crosswalk flashing beacons	N/A	N/A	N/A	N/A	T	-
Appendix A	Reduce posted speed limit through school zones during school times	T	T	N/A	N/A	T	T
Appendix A	Provide pedestrian overpasses and underpasses	-	-	N/A	N/A	-	T
Appendix A	Mark crosswalks at uncontrolled locations, intersection or mid-block	-	N/A	N/A	N/A	T	T
Appendix A	Use alternative crosswalk markings at mid-block locations	-	N/A	N/A	N/A	T	T
Appendix A	Use alternative crosswalk devices at mid-block locations	-	N/A	N/A	N/A	T	T
Appendix A	Provide a raised median or refuge island at marked and unmarked crosswalks	N/A	N/A	N/A	N/A	T	T
Appendix A	Provide a raised or flush median or center two-way left-turn lane at marked and unmarked crosswalks	N/A	N/A	N/A	N/A	T	T
Appendix A	Install pedestrian refuge islands or split pedestrian crossovers	N/A	N/A	N/A	N/A	T	T
Appendix A	Widen median	N/A	-	N/A	N/A	T	T
Appendix A	Provide dedicated bicycle lanes (BLs)	N/A	N/A	N/A	N/A	T	-
Appendix A	Provide wide curb lanes (WCLs)	N/A	N/A	N/A	N/A	T	-
Appendix A	Provide shared bus/bicycle lanes	N/A	N/A	N/A	N/A	T	-
Appendix A	Re-stripe roadway to provide bike lane	N/A	N/A	N/A	N/A	T	-
Appendix A	Pave highway shoulders for bicyclist use	T	T	N/A	N/A	N/A	-
Appendix A	Provide bicycle boulevards	N/A	N/A	N/A	N/A	T	-
Appendix A	Provide separate bicycle facilities	N/A	N/A	N/A	N/A	T	-

NOTE:
T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.
N/A = Indicates that the treatment is not applicable to the corresponding setting.

13.13. CRASH EFFECTS OF HIGHWAY LIGHTING

13.13.1. Background and Availability of AMFs

Artificial illumination, also known as lighting, is often provided on roadway segments in urban and suburban areas. Lighting is also often provided at rural locations where road users may need to make a decision.

Exhibit 13-65 summarizes common treatments related to highway lighting and the corresponding AMF availability.

Exhibit 13-65: Summary of Treatments Related to Highway Lighting

HSM Section	Treatment	Rural Two-Lane Road	Rural Multi-Lane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
13.13.2.1	Provide highway illumination	✓	✓	✓	✓	✓	✓
NOTE: ✓ = Indicates that an AMF is available for this treatment.							

13.13.2. Highway Lighting Treatments with AMFs

13.13.2.1. Provide Highway Lighting

Rural two-lane roads, rural multilane highways, freeways, expressways, urban and suburban arterials

The crash effects of providing highway lighting on roadway segments that previously had no lighting are shown in Exhibit 13-66. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence of lighting.

Exhibit 13-66: Potential Crash Effects of Providing Highway Lighting[7,8,12,27]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Provide highway lighting	All settings (All types)	Unspecified	All types (Nighttime injury) [8]	**0.72**	**0.06**
			All types (Nighttime Non-injury) [8]	**0.83**	**0.07**
			All types (Nighttime injury)[15]	0.71	N/A°
			All types (Nighttime all severities)[15]	0.80	N/A°
Base Condition: Absence of lighting.					

NOTE: Based on U.S. studies: Harkey at al., 2008; and International studies: Elvik and Vaa 2004

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

° Standard error of the AMF is unknown.

The AMFs for nighttime injury accidents and nighttime accidents for all severity levels were derived by Harkey et al.[15] using the results from Elvik and Vaa[8] along with information on the distribution of accidents by injury severity and time of day from the states of Minnesota and Michigan.

13.14. CRASH EFFECTS OF ROADWAY ACCESS MANAGEMENT

13.14.1. Background and Availability of AMFs

Section 13.14.1 presents information about access management treatments with AMFs.

Access management is a set of techniques designed to manage the frequency and magnitude of conflict points at residential and commercial access points. The purpose of an access management program is to balance the mobility required from a roadway facility with the accessibility needs of adjacent land uses.[31]

The management of access, namely the location, spacing, and design of driveways and intersections, is considered to be one of the most critical elements in roadway planning and design. Access management provides or manages access to land development while simultaneously preserving traffic safety, capacity, and speed on the surrounding road system, thus addressing congestion, capacity loss, and accidents on the nation's roadways.[21]

This section presents the crash effects of access density, or the number of access points per unit length, along a roadway segment. An extensive TRB website containing access management information is available at www.accessmanagement.gov.

Separate predictive methods are provided in *Part C* of the HSM for public-road intersections. However, where intersection characteristics or side-road traffic volume data are lacking, some minor, very-low-volume intersections may be treated as driveways for analysis purposes.

Exhibit 13-67 summarizes common treatments related to access points and the corresponding AMF availability.

Exhibit 13-67: Summary of Treatments Related to Access Management

HSM Section	Treatment	Rural Two-Lane Road	Urban Two-Lane Road	Suburban Two-Lane Roads	Rural Multi-Lane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
13.14.2.1	Modify access point density	✓	N/A	N/A	-	N/A	-	✓	✓
Appendix	Reduce number of median crossings and intersections	-	N/A	N/A	-	N/A	-	T	T

NOTE:
✓ = Indicates that an AMF is available for this treatment.
T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.
- = Indicates that an AMF is not available and a trend is not known.
N/A = Indicates that the treatment is not applicable to the corresponding setting.

13.14.2. Access Management Treatments with AMFs

13.14.2.1. Modify Access Point Density

Access point density refers to the number of access points per mile.

Rural two-lane roads

The crash effects of decreasing access point density on rural two-lane roads are presented in Equation 13-5[16] and Exhibit 13-68. The base condition (i.e., the condition in which the AMF = 1.00) for access point density is five access points per mile. The standard error of the AMF is unknown.

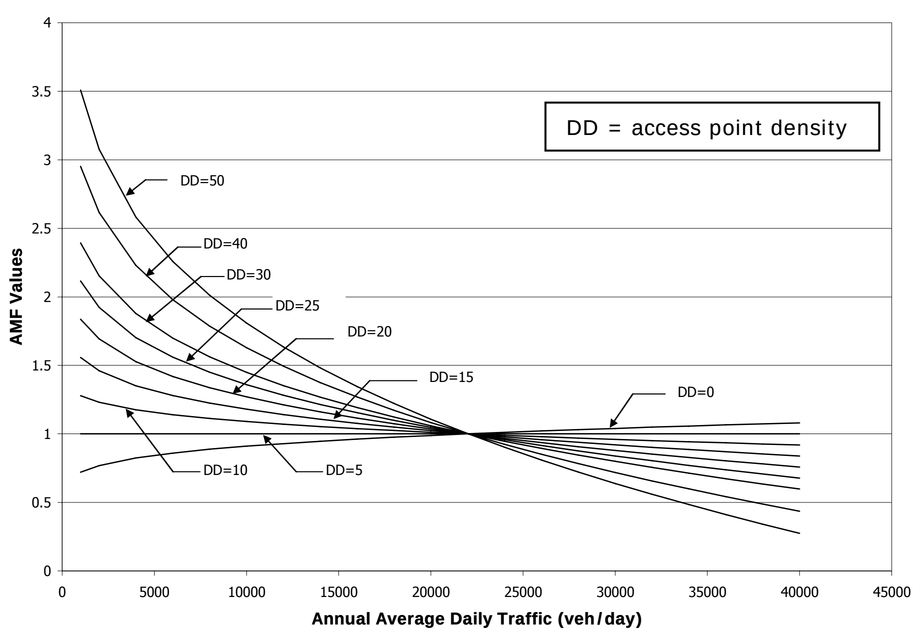
$$AMF = \frac{0.322 + DD \times [0.05 - 0.005 \times \ln(AADT)]}{0.322 + 5 \times [0.05 - 0.005 \times \ln(AADT)]} \tag{13-7}$$

Where,

AADT = average annual daily traffic volume of the roadway being evaluated; and

DD = access point density measured in driveways per mile.

Exhibit 13-68: Potential Crash Effects of Access Point Density on Rural Two-lane Roads



Urban and Suburban Arterials

The crash effects of decreasing access point density on urban and suburban arterials are shown in Exhibit 13-69. [8]

The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the initial driveway density prior to the implementation of the treatment as presented in Exhibit 13-69.

Exhibit 13-69: Potential Crash Effects of Reducing Access Point Density[(8)]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Reduce driveways from 48 to 26-48 per mile				**0.71**	**0.04**
Reduce driveways from 26-48 to 10-24 per mile	Urban and suburban (Arterial)	Unspecified	All types (Injury)	**0.69**	**0.02**
Reduce driveways from 10-24 to less than 10 per mile				**0.75**	**0.03**
Base Condition: Initial driveway density per mile based on values in this table (48, 26-48, and 10-24 per mile).					

NOTE: Based on International studies: Jensen 1968; Grimsgaard 1976; Hvoslef 1977; Amundsen 1979; Grimsgaard 1979; Hovd 1979; Muskaug 1985

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

13.15. CRASH EFFECTS OF WEATHER ISSUES

Section 13.15 presents information about weather related treatements with AMFs.

13.15.1. Background and Availability of AMFs

The weather cannot be controlled, but measures are available to mitigate inclement weather and the resulting impact on roadways.

Exhibit 13-70 summarizes common treatments related to weather issues and the corresponding AMF availability.

Exhibit 13-70: Summary of Treatments Related to Weather Issues

HSM Section	Treatment	Rural Two-Lane Road	Rural Multilane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
13.15.2.1	Implement faster response times for winter maintenance	✓	✓	✓	✓	✓	✓
Appendix A	Apply preventive chemical anti-icing during the whole winter season	T	T	T	T	T	T
Appendix A	Install changeable fog warnings signs	-	-	T	-	-	-
Appendix A	Install snow fences for the whole winter season	T	T	-	-	N/A	N/A
Appendix A	Raise the state of preparedness for winter maintenance	T	T	T	T	T	T

NOTE:
✓ = Indicates that an AMF is available for this treatment.
T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.
- = Indicates that an AMF is not available and a trend is not known.
N/A = Indicates that the treatment is not applicable to the corresponding setting.

13.15.2. Weather Related Treatments with AMFs

13.15.2.1. Implement Faster Response Times for Winter Maintenance

Most jurisdictions that experience regular snowfall have developed acceptable response times for snow, slush, and ice control. For example, a jurisdiction may clear or plow the road before snow depth exceeds two inches. Standards for snow clearance vary by road type or function and traffic volume. Depending on snowfall intensity, the maximum snow depth standard implies a certain maximum response time before snow is cleared. If snow falls very intensely, the response is faster than when snow falls as scattered snowflakes.

As it starts to snow, road surface conditions worsen and it is generally expected that the accident rate will increase. After snow clearance or reapplication of de-icing treatments, the action of traffic continues to melt whatever snow or ice might be left, and the accident rate is generally expected to return to the before-snow rate.

If maintenance crews operate with a faster response time or if maintenance crews are deployed when less snow has accumulated (i.e., maintenance standards are raised), the expected increase in the accident rate could be reversed at an earlier time, possibly resulting in fewer total crashes. [1]

The effects of different winter maintenance standards for different road types on accidents during winter are likely a function of the season's duration and severity. The longer the winter season, and the more often there is adverse weather, the more important the standard of winter maintenance becomes for safety.

Rural two-lane roads, rural multilane highways, freeways, expressways, urban and suburban arterials

A jurisdiction's road system is usually classified into a hierarchy with respect to the minimum standards for winter maintenance. The hierarchy is based on traffic volume and road function. The strictest standards usually apply to freeways or arterial roads, whereas local residential roads may not be cleared at all. The crash effects of taising a road's standards for winter maintenance by one class are shown in Exhibit 13-71.[(8)] The base conditions of the AMFs (i.e., the condition in which the AMF = 1.00) consist of the original maintenance hierarchy assigned to a roadway prior to the implementation of the treatment.

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[1] Accident rate is used in this discussion as the number of accidents that occur prior to snow maintenance. The number of accidents depends on the amount of traffic on the roads between the start of snowfall and snow maintenance.

Exhibit 13-71: Potential Crash Effects of Raising Standards for Winter Maintenance for the Whole Winter Season[(8)][2]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Raise standard for winter maintenance	All settings (All types)	Any volume	All types (Injury)	**0.89**	**0.02**
			All types (Non-injury)	**0.73**	**0.02**
Base Condition: Original maintenance hierarchy assigned to a roadway prior to the implementation of the treatment.					

NOTE: Based on International studies: Ragnøy 1985; Bertilsson 1987; Schandersson 1988; Eriksen and Vaa 1994; Vaa 1996

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

13.16. CONCLUSION

The treatments discussed in this chapter focus on the potential crash effects of roadway segment factors such as roadway and roadside objects, roadway alignment, traffic calming, on-street parking, pedestrian and bicycle factors, illumination, access management, and weather. The information presented is the AMFs known to a degree of statistical stability and reliability for inclusion in this edition of the HSM. Additional qualitative information regarding potential roadway treatments is contained in Appendix A.

The remaining chapters in *Part D* present treatments related to other site types such as intersections and interchanges. The material in this chapter can be used in conjunction with activities in *Chapter 6 Select Countermeasures*, and *Chapter 7 Economic Appraisal*. Some *Part D* AMFs are included in *Part C* for use in the predictive method. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7 of the *Part C Introduction and Applications Guidance*.

[2] Nearly all studies were conducted in the Scandinavian countries. The length and severity of the winter season varies substantially between regions of these countries. In the south of Sweden, for example, there may not be any snow at all during winter and only a few days with freezing rain or ice on the road. In the northern parts of Finland, Norway and Sweden, snow usually falls in October and remains on the ground until late April. Most roads in these areas, at least in rural areas, are fully or partly covered by snow throughout the winter.

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APPENDIX A

A.1 INTRODUCTION

The appendix presents general information, trends in crashes and/or user-behavior as a result of the treatments, and a list of related treatments for which information is not currently available. Where AMFs are available, a more detailed discussion can be found within the chapter body. The absence of an AMF indicates that at the time this edition of the HSM was developed, completed research had not developed statistically reliable and/or stable AMFs that passed the screening test for inclusion in the HSM. Trends in crashes and user behavior that are either known or appear to be present are summarized in this appendix.

This appendix is organized into the following sections:

- Roadway Elements (Section A.2)
- Roadside Elements (Section A.3)
- Alignment Elements (Section A.4)
- Roadway Signs (Section A.5)
- Roadway Delineation (Section A.6)
- Rumble Strips (Section A.7)
- Traffic Calming (Section A.8)
- Roadway Treatments for Pedestrians and Bicyclists (Section A.9)
- Roadway Access Management (Section A.10)
- Weather Issues (Section A.11)
- Treatments with Unknown Crash Effects (Section A.12)

A.2 ROADWAY ELEMENTS

A.2.1 General Information

Lanes

Lane width and the number of lanes are generally determined by the roadway's traffic volume and the road type and function.

In the past, wider lanes were thought to reduce crashes for two reasons. First, wider lanes increase the average distance between vehicles in adjacent lanes, providing a wider buffer for vehicles that deviate from the lane.[20] Second, wider lanes provide more room for driver correction in near-accident circumstances.[20] For example, on a roadway with narrow lanes, a moment of driver inattention may lead a vehicle over the pavement edge-drop and onto a gravel shoulder. A wider lane width provides greater opportunity to maintain the vehicle on the paved surface in the same moment of driver inattention.

Drivers, however, adapt to the road. Wider lanes appear to induce somewhat faster travel speeds, as shown by the relationship between lane width and free flow speed documented in the Highway Capacity Manual.[50] Wider lanes may also lead to closer following.

It is difficult to separate the effect of lane width from the crash effect of other cross-section elements, e.g., shoulder width, shoulder type, etc.[20] In addition, lane width likely plays a different role for two-lane versus multilane roads.[20] Finally, increasing the number of lanes on a roadway segment increases the crossing distance for pedestrians, and increases the exposure of pedestrians to vehicles.

Shoulders

Shoulders are intended to perform several functions including: provide a recovery area for out-of-control vehicles, provide an emergency stopping area, and improve the pavement surface's structural integrity.[23]

The main purposes of paving shoulders are: to protect the physical road structure from water damage, to protect the shoulder from erosion by stray vehicles, and to enhance the controllability of stray vehicles. Fully paved shoulders, however, generate some voluntary stopping. More than 10% of all fatal freeway accidents are associated with stopped-on-shoulder vehicles or maneuvers associated with leaving and returning to the outer lane. [23]

Some concerns when increasing shoulder width include:

- Wider shoulders may result in higher operating speeds which, in turn, may impact accident severity;
- Steeper side or backslopes may result from wider roadway width and limited right-of-way; and,
- Drivers may choose to use the wider shoulder as a travel lane.

Medians

Medians are intended to perform several functions. Some of the main functions are: separate opposing traffic, provide a recovery area for out-of-control vehicles, provide an emergency stopping area, and allow space for speed change lanes and storage of left-turning and U-turning vehicles.[2] Medians may be depressed, raised, or flush with the road surface.

Some additional considerations when providing medians or increasing median width include:

- Wider grassed medians may result in higher operating speeds which, in turn, may impact accident severity;
- The buffer area between private development along the road and the traveled way may have to be narrowed; and,
- Vehicles require increased clearance time to cross the median at signalized intersections.

Geometric design standards for medians on roadway segments are generally based on the setting, amount of traffic, right-of-way constraints and, over time, the revision of design standards towards more generous highway design standards.[3] Median design decisions include whether a median should be provided, how wide the median should be, the shape of the median, and whether to provide a median barrier.[24] These interrelated design decisions make it difficult to extract the effect on expected average crash frequency of median width and/or median type from the effect of other roadway and roadside elements.

In addition, median width and type likely play a different role in urban versus rural areas, and for horizontal curves versus tangent sections.

The effects on expected average crash frequency of two-way left-turn lanes (a type of "median") are discussed in *Chapter 16*.

A.2.2 Roadway Element Treatments with no AMFs - Trends in Crashes or User Behavior

A.2.2.1 Increase Median Width

On divided highways, median width includes the left shoulder, if any.

Freeways, and expressways

Increasing median width appears to decrease cross-median collisions.[(24)] However, no conclusive results about the crash effects for other collision types were found for this edition of the HSM.

A.3 ROADSIDE ELEMENTS

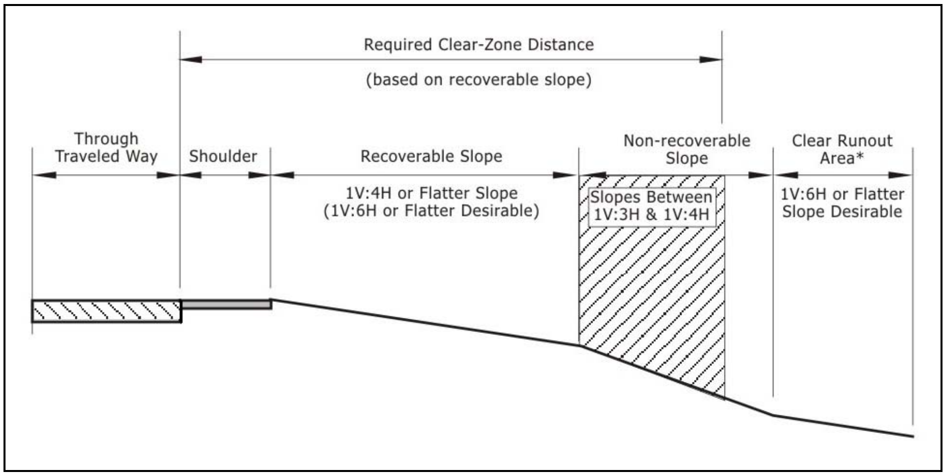
A.3.1 General Information

Roadside Geometry

Roadside geometry refers to the physical layout of the roadside, such as curbs, foreslopes, backslopes, and transverse slopes.

The AASHTO *Roadside Design Guide* defines the "clear zone" as the "*total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear run-out area."*[(3)] The clear zone is illustrated in Exhibit 13-72.

Exhibit 13-72: Clear Zone Distance with Example of a Parallel Foreslope Design[3]



NOTE: * The Clear Runout Area is additional clear-zone space that is needed because a portion of the required Clear Zone (shaded area) falls on a non-recoverable slope. The width of the Clear Runout Area is equal to that portion of the Clear Zone Distance located on the non-recoverable slope.

Designing a roadside environment to be clear of fixed objects with stable flattened slopes is intended to increase the opportunity for errant vehicles to regain the roadway safely, or to come to a stop on the roadside. This type of roadside environment, called a “forgiving roadside”, is also designed to reduce the chance of serious consequences if a vehicle leaves the roadway. The concept of a “forgiving roadside” is explained in the AASHTO Roadside Design Guide.[3]

The AASHTO Roadside Design Guide contains substantial information that can be used to determine the clear zone distance for roadways based on traffic volumes and speeds. The AASHTO Roadside Design Guide also presents a decision process that can be used to determine whether a treatment is suitable for a given fixed object or non-traversable terrain feature.[3]

While there are positive safety benefits to the clear zone, there is no single clear zone width that defines maximum safety since the distance traveled by errant vehicles may exceed any given width. It is generally accepted that a wider clear zone creates a safer environment for potentially errant vehicles, up to some cost-effective limit beyond which very few vehicles will encroach.[42] In most cases, however, numerous constraints limit the available clear zone.

Roadside Features

Roadside features include signs, signals, luminaire supports, utility poles, trees, driver aid call boxes, railroad crossing warning devices, fire hydrants, mailboxes, and other similar roadside features.

The AASHTO *Roadside Design Guide* contains information about the placement of roadside features, criteria for breakaway supports, base designs, etc.[3] When removal of hazardous roadside features is not possible, the objects may be relocated farther from the traffic flow, shielded with roadside barriers, or replaced with breakaway devices.[42]

Providing barriers in front of roadside features that cannot be relocated is discussed in Section 13.5.2.5.

Roadside Barriers

Roadside barriers are also known as guardrails or guiderails.

A roadside barrier is *"a longitudinal barrier used to shield drivers from natural or man-made obstacles located along either side of a traveled way. It may also be used to protect bystanders, pedestrians, and cyclists from vehicular traffic under special conditions."*[3] Warrants for barrier installation can be found in the AASHTO Roadside Design Guide. The AASHTO Roadside Design Guide also sets out performance requirements, placement guidelines, and a methodology for identifying and upgrading existing installations.[3]

Barrier end treatments or terminals are *"normally used at the end of a roadside barrier where traffic passes on one side of the barrier and in one direction only. A crash cushion is normally used to shield the end of a median barrier or a fixed object located in a gore area. A crash cushion may also be used to shield a fixed object on either side of a roadway if a designer decides that a crash cushion is more cost-effective than a traffic barrier."*[3]

The AASHTO Roadside Design Guide contains information about barrier types, barrier end treatment and crash cushion installation warrants, structural and performance requirements, selection guidelines, and placement recommendations.[3]

Roadside Hazard Rating

The AASHTO *Roadside Design Guide* discusses clear zone widths related to speed, traffic volume, and embankment slope. The Roadside Hazard Rating (RHR) system considers the clear zone in conjunction with the roadside slope, roadside surface roughness, recoverability of the roadside, and other elements beyond the clear zone such as barriers or trees.[19] As the RHR increases from 1 to 7, the crash risk for frequency and/or severity increases.

Exhibit 13-73 through Exhibit 13-79 are photographs illustrating the seven RHR levels. In the safety prediction procedure for two-lane rural roads (*Chapter 10*), roadside design is described by the RHR.

Exhibit 13-73: Typical Roadway with Roadside Hazard Rating of 1

Clear zone greater than or equal to 30 ft sideslope flatter than 1V:4H, recoverable.

Exhibit 13-74: Typical Roadway with Roadside Hazard Rating of 2

Clear zone between 20 and 25 ft; sideslope about 1V:4H, recoverable.

Exhibit 13-75: Typical Roadway with Roadside Hazard Rating of 3

Clear zone about 10 ft; sideslope about 1V:3H, marginally recoverable.

Exhibit 13-76: Typical Roadway with Roadside Hazard Rating of 4

Clear zone between 5 and 10 ft; sideslope about 1V:3H or 1V:4H, marginally forgiving, increased chance of reportable roadside crash.

Exhibit 13-77: Typical Roadway with Roadside Hazard Rating of 5

Clear zone between 5 and 10 ft; sideslope about 1V:3H, virtually non-recoverable.

Exhibit 13-78: Typical Roadway with Roadside Hazard Rating of 6

Clear zone less than or equal to 5 ft; sideslope about 1V:2H, non-recoverable.

Exhibit 13-79: Typical Roadway with Roadside Hazard Rating of 7

Clear zone less than or equal to 5 ft; sideslope about 1V:2H or steeper, non-recoverable with high likelihood of severe injuries from roadside crash.

A.3.2 Roadside Element Treatments with no AMFs - Trends in Crashes or User Behavior

A.3.2.1 Install Median Barrier

Freeways

Installing a median barrier appears to have a positive crash effect in narrow medians up to 36-ft wide. The crash effect appears to diminish on wider medians.[24] However, the magnitude of the crash effect is not certain at this time.

A.3.2.2 Increase Clear Roadside Recovery Distance

Rural two-lane roads

Increasing the clear roadside recovery distance appears to reduce related accident types (i.e., run-off-road, head-on, and sideswipe accidents).[40,42] The magnitude of the crash effect is not certain at this time but depends on the clear roadside recovery distance before and after treatment. Current guidance on the roadside design and clear zones is provided in the AASHTO Roadside Design Guide.[3]

A.3.2.3 Install Curb

The AASHTO Policy on Geometric Design of Highways and Streets states that *"a curb, by definition, incorporates some raised or vertical element."*[20] Curbs are used primarily on low-speed urban highways, generally with a design speed of 45 mph or less.[20]

There are two curb design types: vertical and sloping. Vertical curbs are designed to deter vehicles from leaving the roadway. Sloping curbs, also called "mountable curbs," are designed to permit vehicles to cross the curbs readily when needed.[1] Materials that may be used to construct curbs include cement concrete, granite, and bituminous (asphalt) concrete.

While cement concrete and bituminous (asphalt) concrete curbs are used extensively, it should be noted that the appearance of these types of curbs offers little visible contrast to normal pavements particularly during foggy conditions or at night when surfaces are wet. The visibility of curbs may be improved by attaching reflectorized markers to the top of the curb. Visibility may also be improved by marking curbs with reflectorized materials such as paints and thermoplastics in accordance with MUTCD guidelines.[1]

Urban arterials and suburban arterials

Installing curbs instead of narrow (2 to 3-ft) flush shoulders on urban four-lane undivided roads appears to increase off-the-road and on-the-road accidents of all severities.[25] Installing curbs instead of narrow flush shoulders on suburban multi-lane highways appears to increase accidents of all types and severities.[25] However, the magnitude of the crash effect is not certain at this time.

A.3.2.4 Increase Distance to Utility Poles and Decrease Utility Pole Density

Rural two-lane roads, rural multi-lane highways, freeways, expressways, urban and suburban arterials

As the distance between the roadway edgeline and the utility pole, or utility pole offsets, is increased and utility pole density is reduced, utility pole accidents appear to be reduced.[35] Relocating utility poles from less than 10-ft to more than 10-ft from the roadway appears to provide a greater decrease in crashes than relocating utility poles that are beyond 10-ft from the roadway edge.[35] As the pole offset increases beyond 10-ft, the safety benefits appear to continue.[35] However, the magnitude of the crash effect is not certain at this time.

Placing utility lines underground, increasing pole offsets, and reducing pole density through multiple-use poles results in fewer roadside features for an errant vehicle to strike. These treatments may also reduce utility pole accidents.[53] However, the magnitude of the crash effect is not certain at this time.

A.3.2.5 Install Roadside Barrier along Embankment

Rural two-lane roads, rural multi-lane highways, freeways, expressways, urban and suburban arterials

Installing roadside barriers along embankments appears to reduce the number of fatal and injury run-off-road accidents and the number of run-off-road accidents of all severities.[(13)] However, the magnitude of the crash effect is not certain at this time.

It is expected that the crash effect of installing roadside barriers is related to existing roadside features and roadside geometry.

The AASHTO Roadside Design Guide contains information about barrier types, barrier end treatment and crash cushion installation warrants, structural and performance requirements, selection guidelines, and placement recommendations.[(3)]

A.4 ALIGNMENT ELEMENTS

A.4.1 General Information

Horizontal Alignment

Several elements of horizontal alignment are believed to be associated with crash occurrence on horizontal curves. These elements include internal features (e.g., radius or degree of curve, superelevation, spiral, etc.) and external features (e.g., density of curves upstream, length of preceding tangent sections, sight distance, etc.).[(22)]

Vertical Alignment

Vertical alignment is also known as grade, gradient, or slope. The vertical alignment of a road is believed to affect crash occurrence in several ways. These include:[(21)]

- Average speed: Vehicles tend to slow down going upgrade and speed up going downgrade. Speed is known to affect accident severity. As more severe accidents are more likely than minor accidents to be reported to the police and to be entered into crash databases, the number of reported accidents likely depends on speed and grade.
- Speed differential: It is generally believed that accident frequency increases when speed differential increases. Since road grade affects speed differential, vertical alignment may also affect accident frequency through speed differentials.
- Braking distance: This is also affected by grade. Braking distance may increase on a downgrade and decrease on an upgrade. A longer braking distance consumes more of the sight distance available before the driver reaches the object that prompted the braking. In other words, the longer braking distances associated with downgrades require the driver to perceive, decide and react in less time.
- Drainage: Vertical alignment influences the way water drains from the roadway or may pond on the road. A roadway surface that is wet or subject to ponding may have an effect on safety.

For some of these elements (e.g., drainage) the distinction between upgrade and downgrade is not necessary. For others, e.g., average speed, the distinction between upgrade and downgrade may be more relevant, although for many roads, an upgrade for one direction of travel is a downgrade for the other.

Grade length may also influence the grade's safety. While speed may not be affected by a short downgrade, it may be substantially affected by a long downgrade.[21]

In short, the crash effect of grade can be understood only in the context of the road profile and its influence on the speed distribution profile.[21]

A.4.2 Alignment Treatments with no AMFs - Trends in Crashes or User Behavior

A.4.2.1 Modify Tangent Length Prior to Curve

When a long tangent is followed by a sharp curve (i.e. radius less than 1,666-ft), the number of accidents on the horizontal curve appears to increase.[21] The crash effect appears to be related to the length of the tangent in advance of the curve and the curve radius. However, the magnitude of the crash effect is not certain at this time.

A.4.2.2 Modify Horizontal Curve Radius

Urban and suburban arterials

Increasing the degree of horizontal curvature has been shown to increase injury and non-injury off-the-road accidents on urban and suburban arterials.[25]

A.5 ROADWAY SIGNS

A.5.1 Roadway Sign Treatments with no AMFs - Trends in Crashes or User Behavior

A.5.1.1 Install Signs to Conform to MUTCD

The MUTCD defines the standards highway agencies nationwide use to install and maintain traffic control devices on all streets and highways, but not all signs meet MUTCD standards. For example, the signs may have been installed several years ago.

Urban local street

Replacing older, non-standard signs to conform to current MUTCD standards has been shown to reduce the number of injury accidents.[7] The crash effect on non-injury accidents may consist of an increase, decrease, or no change in non-injury accidents. [7]

A.6 ROADWAY DELINEATION

A.6.1 Roadway Delineation Treatments with no AMFs - Trends in Crashes or User Behavior

A.6.1.1 Install Chevron Signs on Horizontal Curves

Curve radius and curve angle are important predictors of travel speed through horizontal curves.(6) Driver responses indicate that the deflection angle of a curve is more important than the radius in determining approach speed.(6)

For these reasons, chevron markers which delineate the entire curve angle are generally recommended on sharp curves (with deflection angles greater than 7 degrees) and are preferable to RPMs on sharp curves.(6)

Urban and suburban Arterials

Installing chevron signs on horizontal curves in an urban or suburban arterials appears to reduce accidents of all types. However, the magnitude of the crash effect is not certain at this time.

A.6.1.2 Provide Distance Markers

Distance markers are chevrons or other symbols painted on the travel lane pavement surface to help drivers maintain an adequate following distance from vehicles traveling ahead.(13)

Freeways

On freeways (with unspecified traffic volumes) this treatment appears to reduce injury accidents.(13) However, the magnitude of the crash effect is not certain at this time.

A.6.1.3 Place Converging Chevron Pattern Markings

A converging chevron pattern marking may be applied to the travel lane pavement surface to reduce speeds by creating the illusion that the vehicle is speeding and the road is narrowing. The chevron is in the shape of a “V” that points in the direction of travel.

Urban and suburban Arterials

On urban and suburban arterials with unspecified traffic volumes, converging chevron pattern markings appear to reduce all types of accidents of all severities.(16) However, the magnitude of the crash effect is not certain at this time.

A.6.1.4 Place Edgeline and Directional Pavement Markings on Horizontal Curves

Rural two-lane roads

On rural two-lane roads with AADT volumes less than 5,000, edgeline with directional pavement markings appear to reduce injury accidents of the single-vehicle run-off-road type.(13) However, the magnitude of the crash effect is not certain at this time.

A.7 RUMBLE STRIPS

A.7.1 Rumble Strip Treatments with no AMFs - Trends in Crashes or User Behavior

A.7.1.1 Install Continuous Shoulder Rumble Strips and Wider Shoulders

Freeways

On freeways, this treatment appears to decrease accidents of all types and all severities.[(17)] However, the magnitude of the crash effect is not certain at this time.

A.7.1.2 Install Transverse Rumble Strips

Transverse rumble strips (also called "in-lane" rumble strips or "rumble strips in the traveled way") are installed across the travel lane perpendicular to the direction of travel to warn drivers of an upcoming change in the roadway. Transverse rumble strips are designed so that each vehicle will encounter them. Transverse rumble strips have been used as part of traffic calming or speed management programs, in work zones, and in advance of toll plazas, intersections, railroad-highway grade crossings, bridges and tunnels.

There are currently no national guidelines for the application of transverse rumble strips. There are concerns that drivers will cross into opposing lanes of traffic in order to avoid transverse rumble strips. As in the case of other rumble strips, there are concerns about noise, motorcyclists, bicyclists, and maintenance.

Rural two-lane roads

Installing transverse rumble strips in conjunction with raised pavement markers on rural two-lane roads on the approach to horizontal curves appears to reduce all accident types combined, as well as wet and nighttime accidents of all severities. However, the magnitude of the crash effect is not certain at this time.[(4)]

A.7.1.3 Install Centerline Rumble Strips and Centerline Markings

There is debate about the effect of placing centerline markings on top of centerline rumble strips. According to some, the retroreflectivity of the centerline marking is not reduced if the line is painted on top of the rumble strip; it may even be enhanced. Others conclude it can make it harder to see the centerline marking, particularly if debris (e.g. snow, salt, sand) settles in the rumble strip groove. No conclusive results about the crash effects of the placement of centerline markings in relation to centerline rumble strips were found for this edition of the HSM.

A.8 TRAFFIC CALMING

A.8.1 General Information

Traffic calming elements are generally applied to two-lane roads with a speed limit of 30 to 35 mph. The environment is urban, often consisting of a mixture of residential and commercial land use. The road segments treated are typically about 0.6 miles long with two lanes and a high access density. Common traffic calming elements include:

- Narrowing driving lanes;

- Installing chokers or curb bulbs (curb extensions);
- Using cobblestones in short sections of the road;
- Providing raised crosswalks or speed humps;
- Installing transverse rumble strips, usually at the start of the treated roadway segment; and,
- Providing on-street parking.

A.8.2 Traffic Calming Treatments with no AMFs - Trends in Crashes or User Behavior

A.8.2.1 Install Transverse Rumble Strips on Intersection Approaches

Urban and suburban arterials

On urban and suburban two-lane roads, this treatment appears to reduce accidents of all severities.(13) However, the magnitude of the crash effect is not certain at this time.

A.8.2.2 Apply Several Traffic Calming Measures to a Road Segment

Urban arterials

Applying traffic calming measures on two-lane urban roads with AADT traffic volumes of 6,000 to 8,000 appears to decrease the number of accidents of all severities and of injury severity.(13) Non-injury accidents may also experience a reduction with the implementation of traffic calming.

Accident migration is a possible result of traffic calming. Drivers who are forced by traffic calming measures to slow down may try to "catch up" by speeding once they have passed the traffic calmed area. However, the crash effects are not certain at this time.

A.9 ROADWAY TREATMENTS FOR PEDESTRIANS AND BICYCLISTS

A.9.1 Pedestrian and Bicycle Treatments with no AMFs - Trends in Crashes or User Behavior

A.9.1.1 Provide a Sidewalk or Shoulder

"Walking along roadway" pedestrian crashes tend to occur at night where no sidewalks or paved shoulders exist. Higher speed limits and higher traffic volumes are believed to increase the risk of "walking along roadway" pedestrian crashes on roadways without a sidewalk or wide shoulder.(39)

Urban arterials

Compared to roadways without a sidewalk or wide shoulder, the provision of a sidewalk or wide shoulder (4-ft or wider) on urban roads appears to reduce the risk of "walking along roadway" pedestrian crashes.(39) Providing sidewalks, shoulders or walkways is likely to reduce certain types of pedestrian crashes, e.g. where pedestrians walk along roadways and may be struck by a motor vehicle.(30)

Residential streets and streets with higher pedestrian exposure have been shown to benefit most from the provision of pedestrian facilities such as sidewalks or wide grassy shoulders.[33,39]

Compared to roads with sidewalks on one side, roads with sidewalks on both sides appear to reduce the risk of pedestrian crashes.[48]

Compared to roads with no sidewalks at all, roads with sidewalks on one side appear to reduce the risk of pedestrian crashes.[48]

A.9.1.2 Install Raised Pedestrian Crosswalks

Raised pedestrian crosswalks are applied most often on local urban two-lane streets in residential or commercial areas. Raised pedestrian crosswalks may be applied at intersections or mid-block. Raised pedestrian crosswalks are one of many traffic calming treatments.

Urban and suburban arterials

On urban and suburban two-lane roads, raised pedestrian crosswalks appear to reduce injury accidents.[13] It is reasonable to conclude that raised pedestrian crosswalks have an overall positive effect on crash occurrence since they are designed to reduce vehicle operating speed.[13] However, the magnitude of the crash effect is not certain at this time.

Combining a raised pedestrian crosswalk with an overhead flashing beacon appears to increase driver yielding behavior. [27]

A.9.1.3 Install Pedestrian-Activated Flashing Yellow Beacons with Overhead Signs

Urban and suburban arterials

Pedestrian-activated yellow beacons are sometimes used in Europe to alert drivers to pedestrians who are crossing the roadway. Overhead pedestrian signs with flashing yellow beacons appear to result in drivers yielding to pedestrians more often.[28,43,44] The impact appears to be minimal, possibly because:

- Yellow warning beacons are not exclusive to pedestrian crossings, and drivers do not necessarily expect a pedestrian when they see an overhead flashing yellow beacon.

- Drivers learn that many pedestrians are able to cross the road more quickly than the timing on the beacon provides. Motorists may come to think that a pedestrian has already finished crossing the road if a yielding or stopped vehicle blocks the pedestrian from sight.

A.9.1.4 Install Pedestrian-Activated Flashing Yellow Beacons with Overhead Signs and Advance Pavement Markings

Urban and suburban arterials

Pedestrian-activated yellow beacons with overhead signs and advance pavement markings are sometimes used to alert drivers to pedestrians who are crossing the roadway. The pavement markings consist of a large white "X" in each traffic lane. The "X" is 20-ft long and each line is 12 to 20 inches wide. The "X" is positioned approximately 100-ft in advance of the crosswalk. The crosswalk is at least 8-ft wide with edgelines 6 to 8 inches wide.[9]

Compared to previously uncontrolled crosswalks, this type of pedestrian crossing may decrease pedestrian fatalities.[9] However, the magnitude of the crash effect is not certain at this time. The following undesirable behavior patterns were observed at these crossings:[9]

- Some pedestrians step off the curb without signaling to drivers that they intend to cross the road. These pedestrians appear to assume that vehicles will stop very quickly.
- Some drivers initiate overtaking maneuvers before reaching the crosswalk. This behavior suggests that improved education and enforcement are needed.

A.9.1.5 Install overhead electronic signs with pedestrian-activated crosswalk flashing beacons

Urban arterials

Overhead electronic pedestrian signs with pedestrian-activated crosswalk flashing beacons are generally used at marked crosswalks, usually in urban areas.

The overhead electronic pedestrian signs have animated light-emitting diode (LED) eyes that indicate to drivers the direction from which a pedestrian is crossing. The provision of pedestrian crossing direction information appears to increase driver yielding behavior.[41,51] This treatment is generally implemented at marked crosswalks, usually in urban areas.

Pedestrian-activated crosswalk flashing beacons located at the crosswalk or in advance of the crosswalk may increase the percentage of drivers that yield to pedestrians in the crosswalk. Two options for this treatment are:

- An illuminated sign with the standard pedestrian symbol next to the beacons; and,
- Signs placed 166.7-ft before the crosswalk. The signs display the standard pedestrian symbol and request drivers to yield when the beacons are flashing.

Both options appear to increase driver yielding behavior. Both options together appear to have more effect on behavior than either option alone. Only the second option appears to be effective in reducing vehicle-pedestrian conflicts.[51]

The effectiveness of specific variations of this treatment is likely a result of:

- Actuation: By displaying the pedestrian symbol and having the beacons flash only when a pedestrian is in the crosswalk, the treatment may have more impact than continuously flashing signs.
- Pedestrian crossing direction information: By indicating the direction from which a pedestrian is crossing, the treatment prompts drivers to be alert and to look in the appropriate direction.
- Multiple pedestrians: By indicating multiple directions when pedestrians are crossing from two directions simultaneously, the treatment prompts drivers to be alert and to be aware of the presence of multiple pedestrians.[51]

A.9.1.6 Reduce Posted Speed Limit through Schools Zones during School Times

Rural two-lane road, rural multi-lane highway, urban and suburban arterial

Reducing the posted speed through school zones is accomplished using signage, such as "25 MPH WHEN FLASHING," in conjunction with yellow flashing beacons.[9] No conclusive results about the crash effects of this treatment were found for this edition of the HSM. The treatment appears to result in a small reduction of vehicle operating speeds, and may not be effective in reducing vehicle speeds to the reduced posted speed limit.[9] In rural locations, this treatment may increase speed variance which is an undesirable result.[9]

School crossing guards and police enforcement used in conjunction with this treatment may increase driver compliance with speed limits.[9]

A.9.1.7 Provide Pedestrian Overpass and Underpass

Urban arterials

Overpass usage depends on walking distances and the convenience of the overpass to potential users.[9] The convenience of using a pedestrian overpass can be determined from the ratio of the time it takes to cross the street on an overpass divided by the time it takes to cross at street level. It appears that about 95% of pedestrians will use an overpass if this ratio is 1, meaning that it takes the same amount of time to cross using the overpass as the time to cross at street level. It appears that if the overpass route takes 50% longer, very few pedestrians will use it. Similar time ratios suggest that the use of underpasses by pedestrians is less than the use of overpasses.[9]

Pedestrian overpasses and underpasses provide grade-separation, but they are expensive structures and may not be used by pedestrians if they are not perceived to be safer and more convenient than street-level crossing.

Providing pedestrian overpasses appears to reduce pedestrian crashes, although vehicular accidents may increase slightly near the overpass.[9] However, the magnitude of the crash effect is not certain at this time.

A.9.1.8 Mark Crosswalks at Uncontrolled Locations, Intersections, or Mid-block

Urban and suburban arterial

At uncontrolled locations on two-lane roads and multi-lane roads with AADT less than 12,000, a marked crosswalk alone, compared with an unmarked crosswalk, appears to have no statistically significant effect on the pedestrian crash rate, measured as pedestrian crashes per million crossings.[9] Marking pedestrian crosswalks at uncontrolled locations on two- or three-lane roads with speed limits 35 to 40 mph and less than 12,000 AADT appears to have no measurable effect on either pedestrian or motorist behavior.[34] Crosswalk usage appears to increase after markings are installed. Pedestrians walking alone appear to tend to stay within the marked lines of the crosswalk, especially at intersections, while pedestrian groups appear to take less notice of the markings. There is no evidence that pedestrians are less vigilant or more assertive in the crosswalk after markings are installed.[34]

At uncontrolled locations on multi-lane roads with AADT greater than 12,000, a marked crosswalk alone, without other crosswalk improvements, appears to result in a statistically significant increase in pedestrian crash rates compared to uncontrolled sites with an unmarked crosswalk.(54)

Marking pedestrian crosswalks at uncontrolled intersection approaches with a 35 mph speed limit on recently resurfaced roadways appears to slightly reduce vehicle approach speeds.(52) Drivers at lower speeds are generally more likely to stop and yield to pedestrians than higher-speed motorists. (7)

When deciding whether to mark or not mark crosswalks, these results indicate the need to consider the full range of other elements related to pedestrian needs when crossing the roadway .(54)

A.9.1.9 Use Alternative Crosswalk Markings at Mid-block Locations

Urban and suburban arterials

Crosswalk markings may consist of zebra markings, ladder markings, or simple parallel bars. There appears to be no statistically significant difference in pedestrian crash risk between the alternative crosswalk markings available.

A.9.1.10 Use Alternative Crosswalk Devices at Mid-block Locations

Urban and suburban arterials

Zebra and Pelican

Signalized Pelican crossings allow for the smooth flow of vehicular traffic in areas of heavy pedestrian activity. Both traffic engineers and the public seem to feel that Pelican crossings reduce the risk to pedestrians because drivers are controlled by signals.

Replacing Zebra crossings with Pelican crossings does not necessarily cause a reduction in crashes or increase convenience for pedestrians, and may sometimes increase accidents due to increased pedestrian activity at one location, among other factors.(12) In traffic-calmed areas, Zebra crossings seem to be gaining in popularity as they give pedestrians priority over vehicles, are less expensive than signalization, and are more visually appealing.

Exhibit 13-80 and Exhibit 13-81 present examples of Zebra and Pelican crossings.

Exhibit 13-80: Zebra Crossing

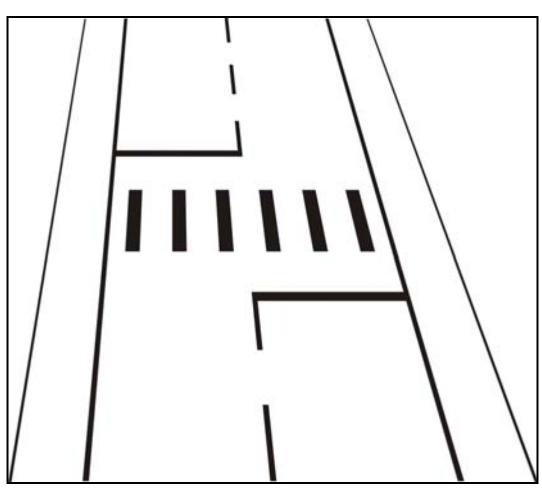
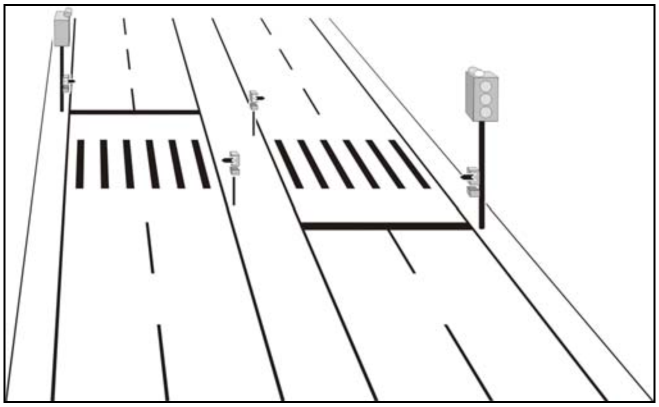


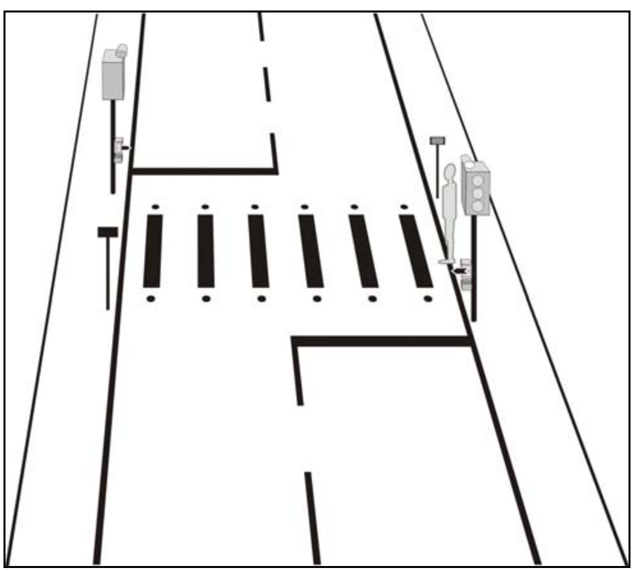
Exhibit 13-81: Pelican Crossing



Puffin

It appears that, with some modifications at Puffin crossings, pedestrians are more likely to look at on-coming traffic rather than looking across the street to where the pedestrian signal head would be located on a Pelican crossing signal.[12] Puffin crossings may result in fewer major pedestrian crossing errors, such as crossing during the green phase for vehicles. This may be a result of the reduced delay to pedestrians at Puffin crossings. Minor pedestrian crossing errors, such as starting to cross at the end of the pedestrian phase, may increase.[12] Exhibit 13-82 presents an example of a Puffin crossing.

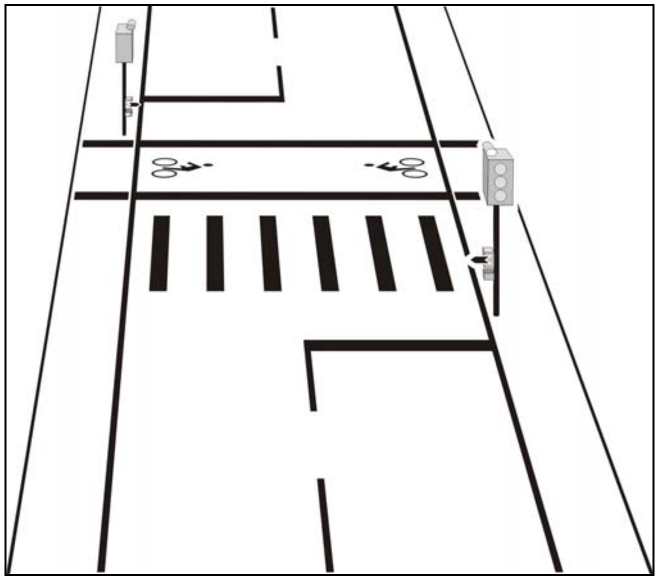
Exhibit 13-82: Puffin Crossing



Toucan

Responses from pedestrians and cyclists using Toucan crossings have been generally favorable despite problems with equipment reliability. No safety or practical issues have been reported for pedestrians where bicyclists are allowed to share a marked pedestrian crosswalk.[12] Exhibit 13-83 presents an example of a Toucan crossing.

Exhibit 13-83: Toucan Crossing



A.9.1.11 Provide a Raised Median or Refuge Island at Marked and Unmarked Crosswalks

Urban and Suburban Arterials

On multi-lane roads with either marked or unmarked crosswalks at both midblock and intersection locations, providing a raised median or refuge island appears to reduce pedestrian crashes.

On urban or suburban multi-lane roads with marked crosswalks, 4 to 8 lanes wide with an AADT of 15,000 or more, the pedestrian crash rate is lower with a raised median than without a raised median.[54] However, the magnitude of the crash effect is not certain at this time.

For similar sites at unmarked crosswalk locations, the pedestrian crash rate[3] is lower with a raised median than without a raised median.[54] However, the magnitude of the crash effect is not certain at this time.

A.9.1.12 Provide a Raised or Flush Median or Center Two-way Left-turn Lane at Marked and Unmarked Crosswalks

Urban and Suburban Arterials

A flush median (painted but not raised) or a center two-way-left-turn lane (TWLTL) on urban or suburban multi-lane roads with 4 to 8 lanes and AADT of 15,000 or more do not appear to provide a crash benefit to pedestrians when compared to multi-lane roads with no median at all.[54]

[3] Pedestrian crash rate is calculated as the number of pedestrian crashes per million crossings

Suburban arterial streets with raised curb medians appear to have lower pedestrian crash rates as compared to TWLTL medians.[8] However, the magnitude of the crash effect is not certain at this time.

Replacing a 6-ft painted median with a wide raised median appears to reduce pedestrian crashes.[11] However, the magnitude of the crash effect is not certain at this time.

A.9.1.13 Install Pedestrian Refuge Islands or Split Pedestrian Crossovers

Urban and suburban arterials

Raised pedestrian refuge islands (PRIs) may be located in the center of roads that are 52-ft wide. The islands are approximately 6-ft wide and 36-ft long. Pedestrian warning signs alert approaching drivers of the island. Further guidance is provided by end island markers and keep right signs posted at both ends of the island. Pedestrians who use the islands are advised with "Wait for Gap" and "Cross Here" signs. Pedestrians do not have the legal right-of-way.[5]

Split pedestrian crossovers (SPXOs) provide a refuge island, static traffic signs, an internally illuminated overhead "pedestrian crossing" sign and pedestrian-activated flashing amber beacons. Drivers approaching an activated SPXO must yield the right-of-way to the pedestrian until the pedestrian clears the driver's half of the road and reaches the island. Like the pedestrian refuges described above, SPXOs include pedestrian warning signs, keep right signs and end island markers to guide drivers; however, the pedestrian signing reads, "Caution Push Button to Activate Early Warning System."[5]

PRIs appear to experience more vehicle-island crashes while SPXOs appear to experience more vehicle-vehicle accidents.[5]

Providing a PRI appears to reduce pedestrian crashes but may increase total accidents, as vehicles collide with the island.[5] However, the magnitude of the crash effect is not certain at this time.

A.9.1.14 Widen Median

Urban and suburban arterials

Increasing median width on arterial roads from 4-ft to 10-ft appears to reduce pedestrian crash rates.[46] However, the magnitude of the crash effect is not certain at this time.

A.9.1.15 Provide Dedicated Bicycle Lanes

Urban arterials

Providing dedicated bicycle lanes in urban areas appears to reduce bicycle-motor vehicle crashes and total crashes on roadway segments.[10,29,32,37,45,47] However, the magnitude of the crash effect is not certain at this time.

Installing pavement markings at the side of the road to delineate a dedicated bicycle lane appears to reduce erratic maneuvers by drivers and bicyclists. Compared to a wide curb lane (WCL), the dedicated bicycle lane may also lead to higher levels of comfort for both bicyclists and motor vehicle drivers.[18]

Three types of bicycle-motor vehicle accidents may be unaffected by bicycle lanes: (1) where a bicyclist fails to stop or yield at a controlled intersection, (2) where a driver fails to stop or yield at a controlled intersection, and (3) where a driver makes an improper left-turn.[37]

A.9.1.16 Provide Wide Curb Lanes (WCLs)

Urban arterials

One alternative to providing a dedicated bicycle lane is to design a wider curb lane to accommodate both bicyclists and motor vehicles. A curb lane 12-ft wide or more appears to improve the interaction between bicycles and motor vehicles in the shared lane.[38] It is likely, however, that there is a lane width beyond which safety may decrease due to driver and bicyclist misunderstanding of the shared space.[38]

Vehicles passing bicyclists on the left appear to encroach into the adjacent traffic lane on roadway segments with WCLs more than on roadway segments with bicycle lanes.[29,18]

Compared to WCLs with the same motor vehicle traffic volume, bicyclists appear to ride further from the curb in bicycle lanes 5.2-ft wide or greater.[29]

A.9.1.17 Provide Shared Bus/Bicycle Lanes

Urban arterials

Compared to streets with general use lanes, and although bicycle traffic may increase after installation of the shared bus/bicycle lane, providing shared bus/bicycle lanes appears to reduce total crashes. [29] However, the magnitude of the crash effect is not certain at this time.

Installing unique pavement markings to highlight the conflict area between bicyclists and transit users at bus stops appears to encourage bicyclists to slow down when a bus is present at the bus stop. [29] The pavement markings may reduce the number of serious conflicts between bicyclists and transit users loading or unloading from the bus. [29]

A.9.1.18 Re-stripe Roadway to Provide Bicycle Lane

Urban arterials

Where on-street parking exists, retrofitting the roadway to accommodate a bicycle lane may result in the traffic lane next to the bicycle lane being somewhat narrower than standard.

Re-striping the roadway to narrow the traffic lane to 10.5-ft (from 12-ft) in order to accommodate a 5-ft BL next to on-street parallel parking does not appear to increase conflicts between curb lane vehicles and bicycles. [29] The narrower curb lane does not appear to alter bicycle lateral positioning. [29]

A.9.1.19 Pave Highway Shoulders for Bicyclist Use

Rural two-lane road and rural multi-lane highways

A paved shoulder for bicyclists is similar to a dedicated bicycle lane. The shoulder provides separation between the bicyclists and drivers.[18]

When a paved highway shoulder is available for bicyclists and provides an alternative to sharing a lane with drivers, the expected number of bicycle-motor vehicle crashes appears to be reduced. However, the magnitude of the crash effect is not certain at this time.

Bicyclists using a paved shoulder may be at risk if drivers inadvertently drift off the road. Shoulder rumble strips are one treatment that may be used to address this issue.(14) Rumble strips may be designed to accommodate bicyclists.(49)

A.9.1.20 Provide Separate Bicycle Facilities

Urban arterials

Separate bicycle facilities may be provided where motor vehicle speeds or volumes are high.(29) Providing separate off-road bicycle facilities reduces the potential interaction between motor vehicles and bicycles.

Although bicyclists may feel safer on separate bicycle facilities compared to bicycle lanes, the crash effects appear to be comparable along roadway segments.(36) The crossing of separate bicycle facilities at intersections may result in an increase in vehicle-bicycle crashes.(29) However, the magnitude of the crash effect is not certain at this time.

A.10 ROADWAY ACCESS MANAGEMENT

A.10.1 Roadway Access Management Treatments with no AMFs – Trends in Crashes or User Behavior

A.10.1.1 Reduce Number of Median Crossings and Intersections

Urban and suburban arterials

On urban and suburban arterials, reducing the number of median openings and intersections appears to reduce the number of intersection and driveway-related crashes.(15) However, the magnitude of the crash effect is not certain at this time.

A.11 WEATHER ISSUES

A.11.1 General Information

Adverse Weather and Low Visibility Warning Systems

Some transportation agencies employ advanced highway weather information systems that warn drivers of adverse weather including icy conditions or low visibility. These systems may include on-road systems such as flashing lights, changeable message signs, static signs, e.g., “snow belt area”, “heavy fog area”, or in-vehicle information systems, or some combination of these elements. These warning systems are most commonly used on freeways and on roads passing through mountains or other locations that may experience unusually severe weather.

Snow, Slush, and Ice Control

It is generally accepted that snow, slush or ice on a road increases the number of expected accidents. By improving winter maintenance standards, it may be possible to mitigate the expected increase in accidents. A number of treatments can be applied to control snow, slush, and ice.

A.11.2 Weather Issue Treatments with No AMFs – Trends in Crashes or User Behavior

A.11.2.1 Install Changeable Fog Warnings Signs

Freeways

Traffic congestion in dense fog can lead to safety issues as reduced visibility results in following drivers being unable to see vehicles that are moving slowly or that have stopped downstream. In dense fog on freeways, crashes often involve multiple vehicles.

On freeways, installing changeable fog warning signs appears to reduce the number of accidents that occur during fog conditions.[26,31] However, the magnitude of the crash effect is not certain at this time.

A.11.2.2 Install Snow Fences for the Whole Winter Season

Rural two-lane road and rural Multi-Lane Highway

Snow fences may be installed on highways that are exposed to snow drifts. On mountainous highways, installing snow fences appears to reduce all types of crashes of all severities.[13] However, the magnitude of the crash effect is not certain at this time.

A.11.2.3 Raise the State of Preparedness for Winter Maintenance

The crash effect of raising the state of preparedness during the entire winter season - for example, putting maintenance crews on standby or by having inspection vehicles drive around the road system - is shown to increase, decrease or cause no change in crash frequency.[13]

A.11.2.4 Apply Preventive Chemical Anti-icing During Entire Winter Season

Salt, also known as chemical de-icing, is generally used to prevent snow from sticking to the road surface. As the salt is cleared from the road by melting snow, a jurisdiction may have to reapply salt through the winter season depending on the amount and frequency of snowfall. In cold winter climates, de-icing treatments are not feasible as salt is effective only at temperatures above about 21F (-6°C). [13]

Preventive salting or chemical anti-icing refers to the spread of salt or liquid chemicals before snow starts in order to prevent snow from sticking to the road surface.

Rural two-lane roads, rural multi-lane highways, freeways, expressways, urban and suburban arterials

The use of preventive salting or chemical anti-icing (i.e., application of chemicals before the onset of a winter storm), in contrast to conventional salting or chemical de-icing (e.g., application of chemicals after a winter storm has begun) appears to reduce injury accidents.[7] The crash effects of applying preventive anti-icing and terminating salting or chemical de-icing do not show a defined trend.

A.12 TREATMENTS WITH UNKNOWN CRASH EFFECTS

A.12.1 Treatments Related to Roadway Elements

- Increase lane width at horizontal curves;
- Increase shoulder width at horizontal curves;
- Change median shape, e.g., raised, level or depressed, or median type, e.g., paved, turf;

A.12.2 Treatments Related to Roadside Elements

- Remove roadside features, e.g., trees;
- Delineate roadside features;
- Install cable guardrails between lanes of opposing traffic;
- Modify backslopes;
- Modify transverse slopes;
- Install curbs and barriers;
- Change curb design, e.g., vertical curb, sloping curb, curb height, or material;
- Replace curbs with other roadside treatments
- Modify drainage structures or features including ditches, drop inlets and channels
- Modify location and support type of signs, signals, and luminaires
- Install breakaway devices
- Modify location and type of driver-aid call boxes, mailboxes, fire hydrants
- Modify barrier end treatments, including breakaway cable terminal (BCT) and modified eccentric loader terminal (MELT).

A.12.3 Treatments Related to Alignment Elements

- Increase sight distance
- Modify lane and shoulder width at curves

A.12.4 Treatments Related to Roadway Signs

- Install active close-following warning signs
- Install limited sight distance warning signs
- Install changeable warning signs on horizontal curves
- Install advance curve warning signs

- Modify sign location, e.g., overhead or roadside;
- Install regulatory signs, such as speed limits;
- Install warning signs, such as stop ahead;
- Increase the daytime and nighttime conspicuity of signs;
- Modify sign materials, e.g., grade sheeting material, and retroreflectivity; and,
- Modify sign support material.

A.12.5 Treatments Related to Roadway Delineation

- Install flashing beacons at curves or other locations to supplement a warning or regulatory sign or marker
- Mount reflectors on guardrails, curbs, and other barriers
- Add delineation treatments at bridges, tunnels and driveways
- Place transverse pavement markings
- Install raised buttons
- Install non-permanent or temporary pavement markers

A.12.6 Treatments Related to Rumble Strips

- Install mid-lane rumble strips;
- Install rumble strips on segments with various lane and shoulder widths;
- Install rumble strips with different dimensions and patterns;

A.12.7 Treatments Related to Passing Zones

- Different passing sight distances;
- Presence of access points/driveways;
- Different length of no-passing zones;
- Different frequency of passing zones;
- Passing zones for various weather, cross-section and operational conditions;

A.12.8 Treatments Related to Traffic Calming

- Install chokers/curb bulb-outs
- Use pavement markings to narrow lanes
- Apply different textures to the road surface

A.12.9 Treatments Related to On-Street Parking

- Eliminate on-street parking on one side of the roadway
- Convert parallel parking to angle parking
- On-street parking with different configurations and adjacent land use

A.12.10 Roadway Treatments for Pedestrians and Bicyclists

- Modify sidewalk or walkway width
- Provide separation between the walkway and the roadway ("buffer zone")
- Change type of walking surface
- Modify sidewalk cross-slope, grade, curb ramp design
- Change the location of trees, poles, posts, news racks, and other roadside features
- Provide sidewalk illumination
- Presence of driveways
- Provide signage for pedestrian and bicyclist information
- Trail planning and design
- Install illuminated crosswalk signs
- Install in-pavement lighting at uncontrolled marked crosswalks
- Provide advance stop lines or yield lines
- Provide mid-block crossing illumination
- Modify median type
- Modify traffic control devices at refuge islands/medians, e.g., signs, striping, warning devices
- Widen bicycle lanes
- Install rumble strips adjacent to bicycle lane
- Provide bicycle boulevards

A.12.11 Treatments Related to Access Management

- Modify signalized intersection spacing

A.12.12 Treatments Related to Weather Issues

- Install changeable weather warnings signs (e.g., high winds, snow, freezing rain, low visibility)

- Install static warning signs for weather or road surface (e.g., bridge road surface freezes before road, high winds)
- Implement assisted platoon driving during inclement weather
- Apply sand or other material to improve road surface friction
- Apply chemical de-icing as a location-specific treatment

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PART D— ACCIDENT MODIFICATION FACTORS

CHAPTER 14— INTERSECTIONS

14.1. Introduction 14-1

14.2. Definition, Application, and Organization of AMFs 14-1

14.3. Definition of an Intersection 14-2

14.4. Crash Effects of Intersection Types 14-4

14.4.1. Background and Availability of AMFs 14-4

14.4.2. Intersection Type Treatments with Accident Modification Factors 14-5

14.4.2.1. Convert Four-Leg Intersection to Two Three-Leg Intersections 14-5

14.4.2.2. Convert Signalized Intersection to a Modern Roundabout 14-8

14.4.2.3. Convert a Stop-Controlled Intersection to a Modern Roundabout 14-9

14.4.2.4. Convert Minor-Road Stop Control to All-way Stop Control 14-11

14.4.2.5. Remove Unwarranted Signals on One-Way Streets 14-11

14.4.2.6. Convert Stop Control to Signal Control 14-12

14.5. Crash Effects of Access Management 14-13

14.5.1. Background and Availability of AMFs 14-13

14.6. Crash Effects of Intersection Design Elements 14-14

14.6.1. Background and Availability of AMFs 14-14

14.6.2. Intersection Design Element Treatments with Accident Modification Factors 14-16

14.6.2.1. Reduce Intersection Skew Angle 14-16

14.6.2.2. Provide a Left-Turn Lane on One or More Approaches to Three-Leg Intersections 14-20

14.6.2.3. Provide a Left-Turn Lane on One or More Approaches to Four-Leg Intersections 14-21

14.6.2.4. Provide a Channelized Left-Turn Lane at Four-Leg Intersections 14-26

14.6.2.5. Provide a Channelized Left-Turn Lane at Three-Leg Intersections 14-27

14.6.2.6. Provide a Right-Turn Lane on One or More Approaches to an Intersection 14-27

14.6.2.7. Increase Intersection Median Width 14-29

14.6.2.8. Provide Intersection Lighting 14-31

14.7. Crash Effects of Intersection Traffic Control and Operational Elements 14-32

14.7.1. Background and Availability of AMFs 14-32

14.7.2. Intersection Traffic Control and Operational Element Treatments with Accident Modification Factors 14-34

14.7.2.1. Prohibit Left-Turns and/or U-Turns by Installing “No Left Turn” and “No U-Turn” Signs 14-34

14.7.2.2. Provide "Stop Ahead" Pavement Markings 14-34
14.7.2.3. Provide Flashing Beacons at Stop-Controlled Intersections............................ 14-35
14.7.2.4. Modify Left-Turn Phase 14-37
14.7.2.5. Replace Direct Left-Turns with Right-Turn/U-turn Combination 14-40
14.7.2.6. Permit Right-Turn-on-Red Operation 14-41
14.7.2.7. Modify Change plus Clearance Interval 14-43
14.7.2.8. Install Red-Light Cameras at Intersections... 14-44

14.8. Conclusion... 14-45

14.9. References 14-46

EXHIBITS

Exhibit 14-1: Intersection Physical and Functional Areas [1] 14-3
Exhibit 14-2: Elements of the functional area of an intersection [1] 14-3
Exhibit 14-3: Treatments Related to Intersection Types 14-4
Exhibit 14-4: Two Ways of Converting Four-Leg Intersection into Two Three-Leg Intersections 14-5
Exhibit 14-5: Potential Crash Effects of Converting Four-Leg Intersection to Two Three-Leg Intersections[9] 14-6
Exhibit 14-6: Modern Roundabout Elements[11] 14-8
Exhibit 14-7: Potential Crash Effects of Converting Signalized Intersections into Modern Roundabout[31] 14-9
Exhibit 14-8: Potential Crash Effects of Converting Stop-Controlled Intersections to Modern Roundabout[31] 14-10
Exhibit 14-9: Potential Crash Effects of Converting Minor-Road Stop-Control to All-way Stop-Control [22] 14-11
Exhibit 14-10: Potential Crash Effects of Removing Unwarranted Signals[25] 14-12
Exhibit 14-11: Potential Crash Effects of Converting from Stop to Signal Control[8,15] . 14-13
Exhibit 14-12: Treatments Related to Access Management 14-14
Exhibit 14-13: Treatments Related to Intersection Design Elements 14-15
Exhibit 14-14: Skewed Intersection 14-16
Exhibit 14-15: Potential Crash Effects of Skew Angle for Intersections with Minor-Road Stop Control on Rural Two-Lane Highways 14-17
Exhibit 14-16: Potential Crash Effects of Skew Angle of Three- and Four-leg Intersections with Minor-road Stop Control on Rural Multilane Highways 14-19
Exhibit 14-17: Potential Crash Effects of Skew Angle on Fatal and Injury Accidents for Three- and Four-leg Intersections with Minor-road Stop Control 14-20
Exhibit 14-18: Potential Crash Effects of Providing a Left-Turn Lane on One Approach to Three-Leg Intersections[15,16] 14-21
Exhibit 14-19: Potential Crash Effects of Providing a Left-Turn Lane on One Approach to Four-Leg Intersections[16] 14-23
Exhibit 14-20: Potential Crash Effects of Providing a Left-Turn lane on Two Approaches to Four-Leg Intersections[16] 14-24
Exhibit 14-21: Potential Crash Effects of a Channelized Left-Turn Lane on Both Major and Minor-Road Approaches at Four-Leg Intersections[9] 14-26
Exhibit 14-22: Potential Crash Effects of a Channelized Left-Turn Lane at Three-Leg Intersections[9] 14-27
Exhibit 14-23: Potential Crash Effects of Providing a Right-Turn Lane on One Approach to an Intersection[16] 14-28
Exhibit 14-24: Potential Crash Effects of Providing a Right-Turn Lane on Two Approaches to an Intersection[16] 14-29
Exhibit 14-25: Median Width, Median Roadway, Median Opening Length, and Median Area [18] 14-30

Exhibit 14-26: Potential Crash Effects of Increasing Intersection Median Width (18) 14-31

Exhibit 14-27: Potential Crash Effects of Providing Intersection Illumination (9,12,10,26) .. 14-32

Exhibit 14-28: Treatments Related to Intersection Traffic Control and Operational Elements 14-33

Exhibit 14-29: Potential Crash Effects of Prohibiting Left-Turns and/or U-Turns by Installing “No Left Turn” and “No U-Turn” Signs (6) 14-34

Exhibit 14-30: Potential Crash Effects of Providing Stop Ahead Pavement Markings (13) 14-35

Exhibit 14-31: Potential Crash Effects of Providing Flashing Beacons at Stop-Controlled Intersections on Two-Lane Roads (31) 14-36

Exhibit 14-32: Potential Crash Effects of Modifying Left-Turn phase at Urban Signalized Intersections (8,15,22) 14-38

Exhibit 14-33: Potential Crash Effects of Modifying Left-Turn Phase on One Intersection Approach(17,19) 14-38

Exhibit 14-34: Right-Turn/U-Turn Combination 14-40

Exhibit 14-35: Potential Crash Effects of Replacing Direct Left-Turns with Right-Turn/U-turn Combination (32) 14-41

Exhibit 14-36: Potential Crash Effects of Permitting Right-Turn-On-Red Operation (7,27) 14-42

Exhibit 14-37: Potential Crash Effects of Modifying Change Plus Clearance Interval (28) 14-44

Exhibit 14-38: Potential Crash Effects of Installing Red-Light Cameras at Intersections(23,30) 14-45

Exhibit 14-39: Summary of Bicycle Lanes and Wide Curb Lanes Crash Effects 14-52

Exhibit 14-40: Potential Crash Effects of Marked Crosswalks at Uncontrolled Locations, Intersections or Midblock 14-54

Exhibit 14-41: Potential Crash Effects of Providing a Raised Median or Refuge Island at Marked 14-54

Exhibit 14-42: Potential Crash Effects of Modifying Pedestrian Signal Heads 14-55

Exhibit 14-43: Potential Crash Effects of Installing Additional Pedestrian Signs 14-58

APPENDIX A

A.1 Introduction 14-49

A.2 Intersection Types 14-49

A.2.1 Intersection Type Elements with No AMFs - Trends in Crashes or User Behavior 14-49

A.2.1.1 Convert a Signalized Intersection to a Modern Roundabout 14-49

A.2.1.2 Convert a Stop- Control Intersection to a Modern Roundabout 14-49

A.3 Access Management 14-50

A.3.1 Access Management Elements with No AMFs - Trends in Crashes or User Behavior 14-50

A.3.1.1 Close or Relocate Access Points in Intersection Functional Area 14-50

A.3.1.2 Provide Corner Clearance 14-50

A.4 Intersection Design Elements 14-50

A.4.1 General Information 14-50

A.4.2 Intersection Design Elements with No AMFs - Trends in Crashes and/or User Behavior 14-52

A.4.2.1 Provide bicycle lanes or wide curb lanes at intersections 14-52

A.4.2.2 Narrow Roadway at Pedestrian Crossing 14-52

A.4.2.3 Install Raised Pedestrian Crosswalk 14-53

A.4.2.4 Install Raised Bicycle Crossing 14-53

A.4.2.5 Mark Crosswalks at Uncontrolled Locations, Intersection or Midblock 14-53

A.4.2.6 Provide a Raised Median or Refuge Island at Marked and Unmarked Crosswalks 14-54

A.5 Traffic Control and Operational Elements 14-55

A.5.1 Traffic Control and Operational Elements with No AMFs - Trends in Crashes or User Behavior 14-55

A.5.1.1 Place Transverse Markings on Roundabout Approaches 14-55

A.5.1.2 Install Pedestrian Signal Heads at Signalized Intersections 14-55

A.5.1.3 Modify Pedestrian Signal Heads 14-55

A.5.1.4 Install Pedestrian Countdown Signals 14-56

A.5.1.5 Install Automated Pedestrian Detectors 14-56

A.5.1.6 Install Stop Lines and Other Crosswalk Enhancements 14-56

A.5.1.7 Provide Exclusive Pedestrian Signal Timing Pattern 14-56

A.5.1.8 Provide Leading Pedestrian Interval Signal Timing Pattern 14-56

A.5.1.9 Provide Actuated Control 14-57

A.5.1.10 Operate Signals in “Night-Flash” Mode 14-57

A.5.1.11 Provide Advance Static Warning Signs and Beacons 14-57

A.5.1.12 Provide Advance Warning Flashers and Warning Beacons 14-57

A.5.1.13 Provide Advance Overhead Guide Signs 14-58

A.5.1.14 Install Additional Pedestrian Signs 14-58
A.5.1.15 Modify Pavement Color for Bicycle Crossings 14-59
A.5.1.16 Place "slalom" Profiled Pavement Markings at Bicycle Lanes 14-59
A.5.1.17 Install Rumble Strips on Intersection Approaches 14-59

A.6 Treatments with Unknown Crash Effects 14-60
A.6.1 Treatments Related to Intersection Types 14-60
A.6.2 Treatments Related to Intersection Design Elements 14-60
A.6.3 Treatments Related to Intersection Traffic Control and Operational Elements 14-62

A.7 Appendix References 14-64

CHAPTER 14 INTERSECTIONS

14.1. INTRODUCTION

Chapter 14 presents the Accident Modification Factors (AMFs) applicable to intersection types, access management characteristics near intersections, intersection design elements, and intersection traffic control and operational elements. Pedestrian and bicyclist related treatments and the corresponding effects on pedestrian and bicyclist crash frequency are integrated into the topic areas noted above. The information presented in this chapter is used to identify effects on expected average crash frequency resulting from treatments applied at intersections.

Chapter 14 presents intersection type, access management, intersection design elements, and intersection traffic control and operation treatments with AMFs.

The *Part D Introduction and Applications Guidance* section provides more information about the processes used to determine the AMFs presented in this chapter.

Chapter 14 is organized into the following sections:

- Definition, Application, and Organization of AMFs (Section 14.2)
- Definition of an Intersection (Section 14.3)
- Crash Effects of Intersection Types (Section 14.4)
- Crash Effects of Access Management (Section 14.5)
- Crash Effects of Intersection Design Elements (Section 14.6)
- Crash Effects of Intersection Traffic Control and Operational Elements (Section 14.7)
- Conclusion (Section 14.8)

Appendix A presents the crash trends for treatments for which AMFs are not currently known, and a listing of treatments for which neither AMFs nor trends are known.

14.2. DEFINITION, APPLICATION, AND ORGANIZATION OF AMFS

Chapter 3 provides a thorough definition and explanation of AMFs.

AMFs quantify the change in expected average crash frequency (crash effect) at a site caused by implementing a particular treatment (also known as a countermeasure, intervention, action, or alternative), design modification, or change in operations. AMFs are used to estimate the potential change in expected crash frequency or crash severity plus or minus a standard error due to implementing a particular action. The application of AMFs involves evaluating the expected average crash frequency with or without a particular treatment, or estimating it with one treatment versus a different treatment.

Specifically, the AMFs presented in this chapter can be used in conjunction with activities in *Chapter 6 Select Countermeasures*, and *Chapter 7 Economic Appraisal*. Some *Part D* AMFs are included in *Part C* for use in the predictive method. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7 of the *Part C Introduction and Applications Guidance*. *Chapter 3 Fundamentals*, Section 3.5.3 Accident Modification Factors provides a comprehensive discussion of AMFs including: an introduction to AMFs, how to interpret and apply AMFs, and applying the standard error associated with AMFs.

The treatments are organized into 3 categories: treatments with AMFs; treatments with trend information; and, no trend or AMF information.

In all *Part D* chapters, the treatments are organized into one of the following categories:

1. AMF is available;
2. Sufficient information is available to present a potential trend in crashes or user behavior, but not to provide an AMF;
3. Quantitative information is not available.

Treatments with AMFs (Category 1 above) are typically estimated for three accident severities: fatal, injury, and non-injury. In the HSM, fatal and injury are generally combined and noted as injury. Where distinct AMFs are available for fatal and injury severities, they are presented separately. Non-injury severity is also known as property-damage-only severity.

Treatments for which AMFs are not presented (Categories 2 and 3 above) indicate that quantitative information currently available did not meet the criteria for inclusion in the HSM. The absence of an AMF indicates additional research is needed to reach a level of statistical reliability and stability to meet the criteria set forth within the HSM. Treatments for which AMFs are not presented are discussed in Appendix A.

14.3. DEFINITION OF AN INTERSECTION

Section 14.3 defines an intersection in Part D.

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".[(1)] This chapter deals with at-grade intersections including signalized, stop-controlled, and roundabout intersections.

An at-grade intersection is defined "by both its physical and functional areas", as illustrated in Exhibit 14-1.[(1)] The functional area "extends both upstream and downstream from the physical intersection area and includes any auxiliary lanes and their associated channelization."[(1)] As illustrated in Exhibit 14-2, the functional area on each approach to an intersection consists of three basic elements:[(1)]

- Decision distance;
- Maneuver distance; and,
- Queue-storage distance.

Exhibit 14-1: Intersection Physical and Functional Areas [(1)]

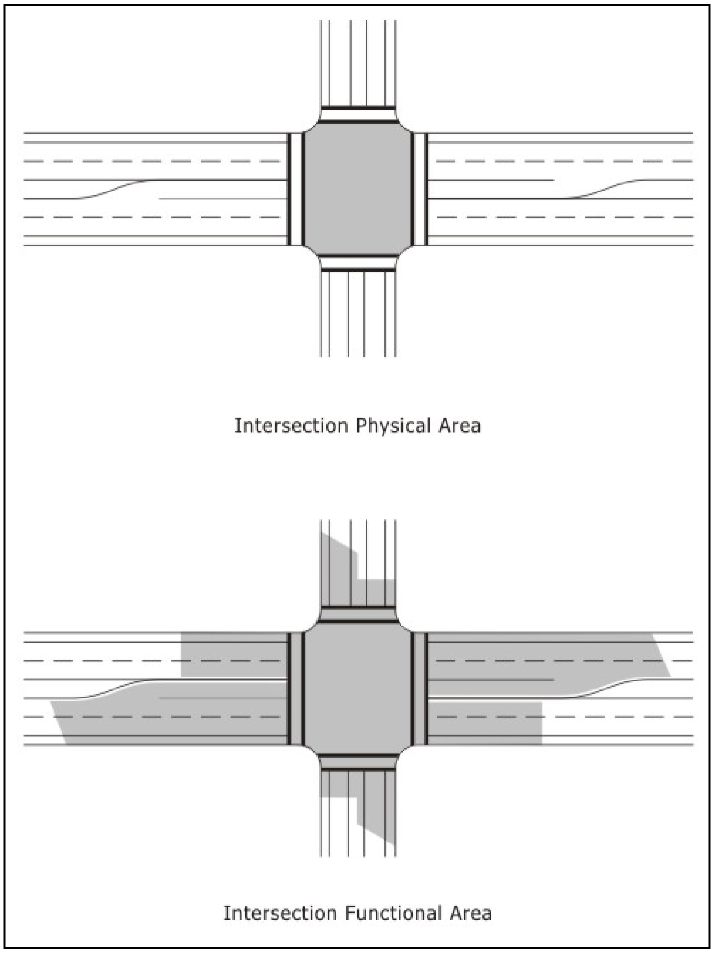
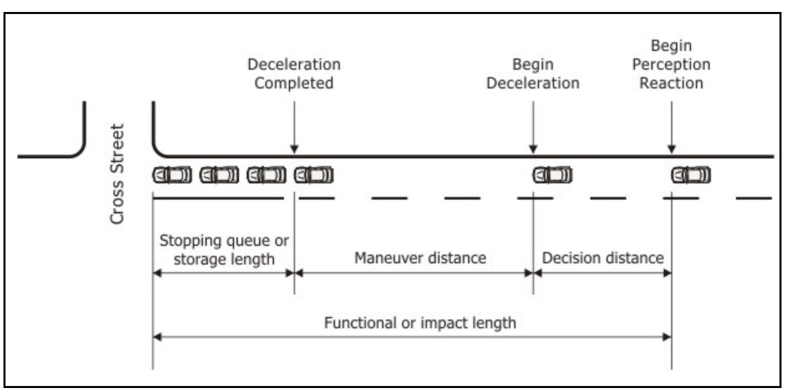


Exhibit 14-2: Elements of the functional area of an intersection [(1)]



The definition of an intersection accident tends to vary between agencies.[(5)] Some agencies define an intersection accident as one which occurs within the intersection crosswalk limits or physical intersection area. Other agencies consider all accidents within a specified distance, such as 250-ft, from the center of an intersection to be intersection accidents.[(5)] However, not all accidents occurring within 250-ft of an intersection can be considered intersection accidents, since some of these may have occurred regardless of the existence of an intersection. Consideration should be given to these differences in definitions when evaluating conditions and seeking solutions.

14.4. CRASH EFFECTS OF INTERSECTION TYPES

14.4.1. Background and Availability of AMFs

The following section provides information on the AMFs for different intersection types (e.g. a stop controlled, traffic signal, roundabout). The different intersection types are defined by their basic geometric characteristics and the governing traffic control device at the intersection. Types of traffic control for at-grade intersections include traffic control signals, stop-control, and yield-control.

Section 14.4.2 provides AMFs for treatments related to intersection types.

The AMFs are summarized in Exhibit 14-3. This exhibit also contains the section number where each AMF can be found.

Exhibit 14-3: Treatments Related to Intersection Types

HSM Section	Treatment	Urban				Suburban				Rural			
		Stop		Signal		Stop		Signal		Stop		Signal	
		Minor Road	All-Way	3-Leg	4-Leg	Minor Road	All-Way	3-Leg	4-Leg	Minor Road	All-Way	3-Leg	4-Leg
14.4.2.1	Convert four-leg intersection to two three-leg intersections	✓	-	-	-	-	-	-	-	-	-	-	-
14.4.2.2	Convert signalized intersection to a modern roundabout	N/A	N/A	✓	✓	N/A	N/A	✓	✓	N/A	N/A	✓	✓
14.4.2.3	Convert stop-controlled intersection to a modern roundabout	✓	✓	N/A	N/A	✓	✓	N/A	N/A	✓	✓	N/A	N/A
14.4.2.4	Convert minor-road stop control to all-way stop control	✓	-	-	-	-	-	-	-	✓	-	-	-
14.4.2.5	Remove unwarranted signal on one-way streets (i.e. convert from signal to stop control on one-way street)	-	-	✓	✓	-	-	-	-	-	-	-	-
14.4.2.6	Convert stop control to signal control	✓	T	N/A	N/A	-	-	N/A	N/A	✓	-	N/A	N/A

NOTE: ✓ = Indicates that an AMF is available for this treatment.

T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.

- = Indicates that an AMF is not available and a trend is not known.

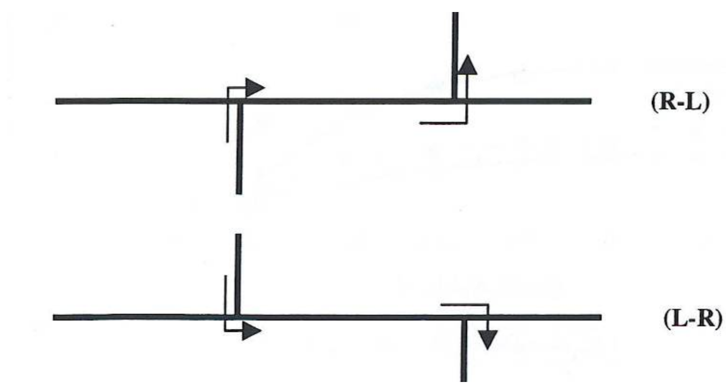
N/A = Indicates that the treatment is not applicable to the corresponding setting.

14.4.2. Intersection Type Treatments with Accident Modification Factors

14.4.2.1. Convert Four-Leg Intersection to Two Three-Leg Intersections

At specific sites where the opportunity exists, four-leg intersections with minor-road stop control can be converted into a pair of three-leg intersections.[(4)] These "offset" or "staggered" intersections can be constructed in one of two ways: right-left (R-L) staggering or left-right (L-R) staggering as shown in Exhibit 14-4.

Exhibit 14-4: Two Ways of Converting Four-Leg Intersection into Two Three-Leg Intersections



The effect on crash frequency of converting an urban four-leg intersection with minor-road stop control into a pair of three-leg intersections with minor-road stop control is dependent on the proportion of minor-road traffic at the intersection prior to conversion.[(9)] However, no conclusive results about the difference in crash effect between right-left or left-right staging of the two resulting three-leg intersections were found for this edition of the HSM.

Urban minor-road stop-controlled intersections

Exhibit 14-5 summarizes the AMFs known for converting an urban intersection from a four-leg intersection with minor-road stop control into a pair of three-leg intersections with minor-road stop control. The crash effects are organized based on the proportion of the minor-road traffic compared to the total entering volume as follows:

- Minor-road traffic > 30% of Total Entering Traffic
- Minor-road traffic =15% to 30% of Total Entering Traffic
- Minor-road traffic < 15% of Total Entering Traffic

The study from which this information was obtained did not indicate a distance or range of distances between the two three-leg intersections nor did it indicate whether or not the effect on crash frequency changed based on the distance between the two three-leg intersections.

The base condition for the AMFs summarized in Exhibit 14-5 (i.e., the condition in which the AMF = 1.00) is an urban four-leg two-way stop controlled intersection.

Exhibit 14-5: Potential Crash Effects of Converting Four-Leg Intersection to Two Three-Leg Intersections[9]

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Convert four-leg intersection into two T-intersections	Urban (Four-leg)	Minor-road traffic >30% of total entering	All types (Injury)	**0.67**	**0.1**
			All types (Non-injury)	**0.90***	**0.09**
		Minor-road traffic = 15-30% of total entering	All types (Injury)	**0.75**	**0.08**
			All types (Non-injury)	**1.00***	**0.09**
		Minor-road traffic <15% of total entering	All types (Injury)	*1.35*	*0.3*
			All types (Non-injury)	**1.15**	**0.1**
Base Condition: Urban four-leg intersection with minor-road stop control					

NOTE: Based on U.S. studies: Hanna, Flynn and Tyler 1976; Montgomery and Carstens 1987; and International studies: Lyager and Loschenkohl 1972; Johannessen and heir 1974; Vaa and Johannessen 1978; Brude and larsson 1978; Cedersund 1983; Vodahl and Giaever 1986; Brude and Larsson 1987

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

The gray box below illustrates how to apply the information in Exhibit 14-5 to calculate the crash frequency effects of converting a four-leg intersection to two three-leg intersections.

Effectiveness of Converting a Four-Leg Intersection to Two Three-Leg Intersections

Question:

A minor street crosses a major urban arterial forming a four-leg intersection. The minor street approaches are stop-controlled and account for approximately 10 percent of the total intersection entering traffic volume. A development project has requested that one approach of the minor street be vacated and replaced with a parallel connection at another location. The governing agency is investigating the effect of the replacement of the four-way intersection with two new three-way intersections. What will be the likely change in expected average crash frequency?

Given Information:

- Existing two-way stop-controlled intersection at a major urban road and a minor street
- Existing minor street intersection entering volume is approximately 10-percent of total intersection entering volume
- Expected average crash frequency without treatment (see Part C Predictive Method) = 7 crashes/year

Find:

- Expected average crash frequency with two three-way stop-controlled intersections
- Change in expected average crash frequency

Answer:

1) Identify the Applicable AMF

   AMF = 1.15 (Exhibit 14-5)

2) Calculate the 95th Percentile Confidence Interval Estimation of Crashes with the Treatment

   Expected Crashes with treatment: = [1.15 ± (2 x 0.10)] x (7 crashes/year) = 6.7 or 9.5 crashes/year

   The multiplication of the standard error by 2 yields a 95% probability that the true value is between 6.7 and 9.5 crashes/year. See Section 3.5.3 in *Chapter 3 Fundamentals* for a detailed explanation.

3) Calculate the difference between the expected number of crashes without the treatment and the expected number of crashes with the treatment.

   Change in Expected Average Crash Frequency:

   High Estimate = 7 – 6.7 = 0.3 crashes/year decrease

   Low Estimate = 9.5 – 7 = 2.5 crashes/year increment

4) Discussion: This example shows that it is more probable that the treatment will result in an increase in crashes, however, a slight crash decrease may also occur.

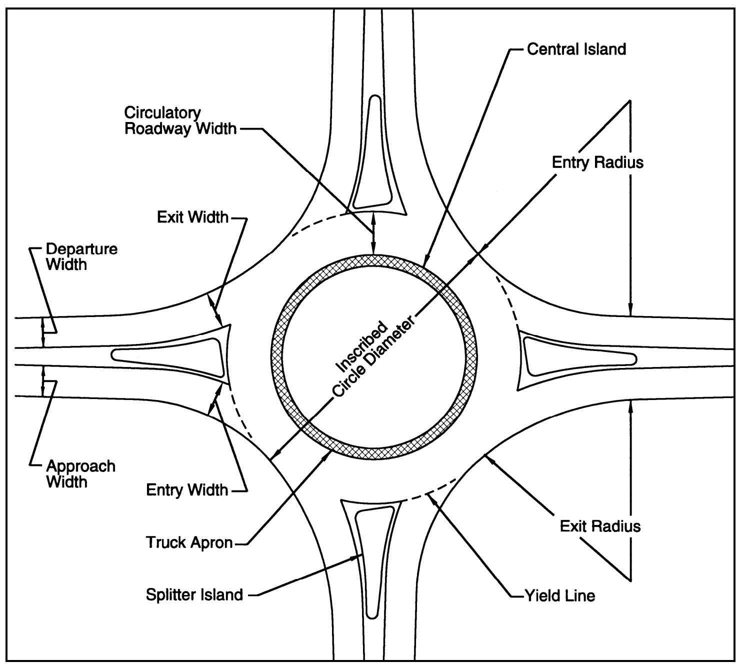
14.4.2.2. Convert Signalized Intersection to a Modern Roundabout

Roundabouts reduce traffic speeds as a result of their small diameters, deflection angle on entry, and circular configuration. Roundabouts also change conflict points from crossing conflicts to merging conflicts. Their circular configuration requires vehicles to circulate in a counterclockwise direction. The reduced speeds and conflict points contribute to the crash reductions experienced compared to signalized intersections.

The reduced vehicle speeds and motor vehicle conflicts are the reason roundabouts are also considered as a traffic calming treatment for locations experiencing characteristics such as higher than desired speeds and/or cut through traffic.

Exhibit 14-6 is a schematic figure of a modern roundabout with the key features labeled.

Exhibit 14-6: Modern Roundabout Elements[11]



Urban, suburban, and rural signalized intersections

Exhibit 14-7 summarizes the effects on crash frequency related to:

- Converting an urban signalized intersection to a single- or multilane modern roundabout; and
- Converting a signalized intersection in any setting (urban, rural or suburban) into a single- or multilane modern roundabout.

The predictive method for urban and suburban arterials in Chapter 12 includes a procedure for roundabouts at intersections that were previously signalized that is based on the AMF in Exhibit 14-7 for installing modern roundabouts in all settings.

The base condition for the AMFs summarized in Exhibit 14-7 is a signalized intersection.

Exhibit 14-7: Potential Crash Effects of Converting Signalized Intersections into Modern Roundabout(31)

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Convert signalized intersection to modern roundabout	Urban (One or two lanes)	Unspecified	All types (All severities)	0.99*	0.1
			All types (Injury)	**0.40**	0.1
	Suburban (Two lanes)		All types (All severities)	**0.33**	0.05
	All settings (One or two lanes)		All types (All severities)	**0.52**	0.06
			All types (Injury)	**0.22**	0.07
Base Condition: Signalized intersection					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.
The study from which this information was obtained does not contain information related to the posted or observed speeds at or on approach to the intersections that were converted to a modern roundabout.

The AMFs in Exhibit 14-7 are also used in Chapter 12: Urban and Suburban Arterials.

In this instance, the observed variability related to the AMF indicates that the treatment could result in an increase, decrease, or no change in crashes at the intersection (see Exhibit 14-7).(31)

Information regarding pedestrians and bicyclists at modern roundabouts is contained in Appendix A.

14.4.2.3. Convert a Stop-Controlled Intersection to a Modern Roundabout

Urban, suburban, and rural stop controlled intersections

Exhibit 14-8 summarizes the crash effects related to:

- Converting an intersection with minor-road stop control to a modern roundabout;
- Converting a rural intersection with minor-road stop control to a one-lane modern roundabout;
- Converting an urban intersection with minor-road stop control to a one-lane modern roundabout;
- Converting an urban intersection with minor-road stop control to a two-lane modern roundabout;
- Converting a suburban intersection with minor-road stop control to a one-lane or two-lane modern roundabout; and
- Converting an all-way stop-controlled intersection in any setting to a modern roundabout.

The predictive method for urban and suburban arterials in Chapter 12 includes a procedure for roundabouts at intersections that previously had minor-road stop control. This procedure is based on the AMF for installation of modern roundabouts in all settings presented in Exhibit 14-8.

The base condition for the AMFs shown in Exhibit 14-8 (i.e., the condition in which the AMF = 1.00) is a stop-controlled intersection.

AMFs with a setting described as "All or Any Setting" were developed from an aggregate of urban, suburban and rural data.

Exhibit 14-8: Potential Crash Effects of Converting Stop-Controlled Intersections to Modern Roundabout(31)

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Convert intersection with minor-road stop control to modern roundabout	All settings (One or Two lanes)	Unspecified	All types (All severities)	**0.56**	**0.05**
			All types (Injury)	**0.18**	**0.04**
	Rural (One lane)		All types (All severities)	**0.29**	**0.04**
			All types (Injury)	**0.13**	**0.04**
	Urban (One or Two lanes)		All types (All severities)	**0.71**	**0.1**
			All types (Injury)	**0.19**	**0.1**
	Urban (One lane)		All types (All severities)	**0.61**	**0.1**
			All types (Injury)	**0.22**	**0.1**
	Urban (Two lane)		All types (All severities)	*0.88*	*0.2*
	Suburban (One or Two lanes)		All types (All severities)	**0.68**	**0.08**
			All types (Injury)	**0.29**	**0.1**
	Suburban (One lane)		All types (All severities)	**0.22**	**0.07**
			All types (Injury)	**0.22**	**0.1**
	Suburban (Two lane)		All types (All severities)	**0.81**	**0.1**
			All types (Injury)	**0.32**	**0.1**
Convert all-way stop-controlled intersection to roundabout	All settings (One or Two lanes)		All types (All severities)	*1.03**	*0.2*
Base Condition: Stop-controlled intersection					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

The study from which this information was obtained does not contain information related to the posted or observed speeds at or on approach to the intersections that were converted to a modern roundabout.

In this instance, the observed variability of the AMF indicates that the conversion could result in an increase, decrease or no change in crashes (see Exhibit 14-8).[31]

Information regarding pedestrians and bicyclists at modern roundabouts is contained in Appendix A.

14.4.2.4. Convert Minor-Road Stop Control to All-way Stop Control

The Manual on Uniform Traffic Control Devices (MUTCD) contains warrants to determine when it is appropriate to convert an intersection with minor-road stop control intersection to an all-way stop control intersection. The effects on crash frequency described below assume that MUTCD warrants for converting a minor-road stop-controlled intersection to an all-way stop-control intersection are met.

Urban and rural minor-road stop-controlled intersections

Exhibit 14-9 provides specific information regarding the crash effects of converting urban intersections with minor-road stop control to all-way stop control when established MUTCD warrants are met. The effect on pedestrian crashes is also shown in Exhibit 14-9.

The base condition for the AMFs below (i.e., the condition in which the AMF = 1.00) is an intersection with minor-road stop control that meets MUTCD warrants to become an all-way stop controlled intersection.

Exhibit 14-9: Potential Crash Effects of Converting Minor-Road Stop-Control to All-way Stop-Control [22]

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Convert minor-road stop control to all-way stop control[22]	Urban (MUTCD warrants are met)	Unspecified	Right-angle (All severities)	**0.25**	**0.03**
			Rear-end (All severities)	**0.82**	**0.1**
			Pedestrian (All severities)	*0.57*	*0.2*
			All types (Injury)	**0.30**	**0.06**
Convert minor-road stop control to all-way stop control[16]	Rural (MUTCD warrants are met)		All types (All severities)	**0.52**	**0.04**

Base Condition: Intersection with minor-road stop control meeting MUTCD warrants for an all-way stop controlled intersection.

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.
Conversions from two-way to all-way stop-control meet established MUTCD warrants.

14.4.2.5. Remove Unwarranted Signals on One-Way Streets

Unwarranted signals are those that do not meet the warrants outlined in the MUTCD.

Urban Signalized Intersections

Exhibit 14-10 summarizes the specific AMFs related to removing unwarranted traffic signals. This AMF may not be applicable to major arterials and is not intended to indicate the crash effects of installing unwarranted signals.

The base condition for the AMFs summarized in Exhibit 14-10 (i.e., the condition in which the AMF = 1.00) is an unwarranted traffic signal located on an urban one-way street.

Exhibit 14-10: Potential Crash Effects of Removing Unwarranted Signals[25]

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Remove unwarranted signal	Urban (one-lane one-way streets, excluding major arterials)	Unspecified	All types (All severities)	**0.76**	**0.09**
			Right-angle and Turning (All severities)	**0.76**	**0.1**
			Rear-end (All severities)	*0.71*	*0.2*
			Pedestrian (All severities)	*0.82*	*0.3*
Base Condition: Unwarranted traffic signal on an urban one-way street					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

14.4.2.6. Convert Stop Control to Signal Control

Prior to installing a traffic signal, an engineering study of traffic conditions, pedestrian characteristics, and physical characteristics of the location is typically performed to determine whether installing a traffic signal is warranted at a particular location as outlined in the MUTCD. The satisfaction of a traffic signal warrant or warrants does not in itself require installing a traffic signal.

Urban and rural minor-road stop-controlled

Exhibit 14-11 summarizes the AMFs related to Converting a stop-controlled intersection to a signalized intersection. The AMF presented for urban intersections applies only for intersections with a major road speed limit at least 40 mph.

The base condition for the AMFs summarized in Exhibit 14-11 (i.e., the condition in which the AMF = 1.00) is a minor-road stop controlled intersection in an urban or rural area.

Exhibit 14-11: Potential Crash Effects of Converting from Stop to Signal Control[8,15]

Treatment	Setting (Intersection type)	Traffic Volume AADT (veh/day)	Accident type (Severity)	AMF	Std. Error
Install a traffic signal	Urban (major road speed limit at least 40 mph; 4 leg [8])	Unspecified	All types (All severities)	**0.95***	**0.09**
			Right-angle (All severities)	**0.33**	**0.06**
			Rear-end (All severities)	*2.43*	*0.4*
	Rural (3-leg and 4-leg [15])	Major road 3,261 to 29,926; Minor road 101 to 10,300	All types (All severities)	**0.56**	**0.03**
			Right-angle (All severities)	**0.23**	**0.02**
			Left-turn (All severities)	**0.40**	**0.06**
			Rear-end (All severities)	*1.58*	*0.2*
Base Condition: Minor-road stop-controlled intersection					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Italic text is used for less reliable AMFs. These AMFs have standard errors 0.2 or higher.
* Observed variability suggests that this treatment could result in an increase, decrease, or no change in crashes. See Part D Applications Guidance.

14.5. CRASH EFFECTS OF ACCESS MANAGEMENT

14.5.1. Background and Availability of AMFs

Access management is a set of techniques designed to manage the frequency and type of conflict points at public intersections and at residential and commercial access points. The management of access, namely the location, spacing, and design of private and public intersections, is an important element in roadway planning and design. Access management provides or manages access to land development while simultaneously preserving traffic safety, capacity, and speed on the surrounding road system, thus addressing congestion, capacity loss, and accidents on the nation's roadways while balancing mobility and access across various facility types.[12,26]

There are no access management treatments with AMFs. Trends related to these treatments are summarized in Appendix A.

The effects on crash frequency of access management at or near intersections are not known to a sufficient degree to present quantitative information in this edition of the HSM. Trends regarding the potential crash effects or changes in user behavior are discussed in Appendix A. The material focuses on the location of access points relative to the functional area of an intersection (see Exhibit 14-1 and Exhibit 14-2). AASHTO's Policy on Geometric Design states that *"driveways should not be situated within the functional boundary of at-grade intersections"*.[2] In the HSM, access points include minor or side-street intersections and private driveways. Exhibit 14-12 summarizes common access management treatments; there are currently no AMFs available for these treatments. Appendix A presents general information and potential change in crash trends for these treatments.

Exhibit 14-12: Treatments Related to Access Management

HSM Section	Treatment	Urban				Suburban				Rural			
		Stop		Signal		Stop		Signal		Stop		Signal	
		Minor Road	All-Way	3-Leg	4-Leg	Minor Road	All-Way	3-Leg	4-Leg	Minor Road	All-Way	3-Leg	4-Leg
Appendix A	Close or relocate access points in intersection functional area	T	T	T	T	T	T	T	T	T	T	T	T
Appendix A	Provide corner clearance	T	T	T	T	T	T	T	T	T	T	T	T

NOTE: T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.

14.6. CRASH EFFECTS OF INTERSECTION DESIGN ELEMENTS

14.6.1. Background and Availability of AMFs

The following sections provide information on the crash effects of treatments related to intersection design elements. The treatments discussed in this section and the corresponding AMFs available are summarized below in Exhibit 14-13.

Exhibit 14-13: Treatments Related to Intersection Design Elements

HSM Section	Treatment	Urban				Suburban				Rural			
		Stop		Signal		Stop		Signal		Stop		Signal	
		Minor Road	All-Way	3-Leg	4-Leg	Minor Road	All-Way	3-Leg	4-Leg	Minor Road	All-Way	3-Leg	4-Leg
14.6.2.1	Reduce intersection skew angle	-	-	-	-	-	-	-	-	✓	✓	-	-
14.6.2.2	Provide a left-turn lane on approach(es) to three -leg intersections	✓	-	✓	N/A	-	-	-	-	✓	-	✓	N/A
14.6.2.3	Provide a left-turn lane on approach(es) to four-leg intersections	✓	-	N/A	✓	-	-	-	-	✓	-	N/A	✓
14.6.2.4	Provide a channelized left-turn lane at four-leg intersections	-	-	N/A	-	-	-	N/A	-	✓	✓	N/A	✓
14.6.2.5	Provide a channelized left-turn lane at three-leg intersections	-	-	-	N/A	-	-	-	N/A	✓	✓	✓	N/A
14.6.2.6	Provide a right-turn lane on approach(es) to an intersection	✓	-	✓	✓	-	-	-	-	✓	-	✓	✓
14.6.2.7	Increase intersection median width	✓	✓	-	✓	✓	✓	-	✓	✓	✓	-	-
14.6.2.8	Provide intersection lighting	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Appendix	Provide bicycle lanes or wide curb lanes at intersections	T	T	T	T	T	T	T	T	T	T	T	T
Appendix	Narrow roadway at pedestrian crossing	T	T	T	T	T	T	T	T	T	T	T	T
Appendix	Install raised pedestrian crosswalk	T	T	-	-	T	T	-	-	-	-	-	-
Appendix	Install raised bicycle crossing	-	-	T	T	-	-	T	T	-	-	T	T
Appendix	Mark crosswalks at uncontrolled locations, intersection or midblock	T	-	-	-	T	-	-	-	T	-	-	-
Appendix	Provide a raised median or refuge island at marked and unmarked crosswalks	T	T	T	T	T	T	T	T	T	T	T	T

NOTE: ✓ = Indicates that an AMF is available for this treatment.

T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.

- = Indicates that an AMF is not available and a trend is not known.

N/A = Indicates that the treatment is not applicable to the corresponding setting.

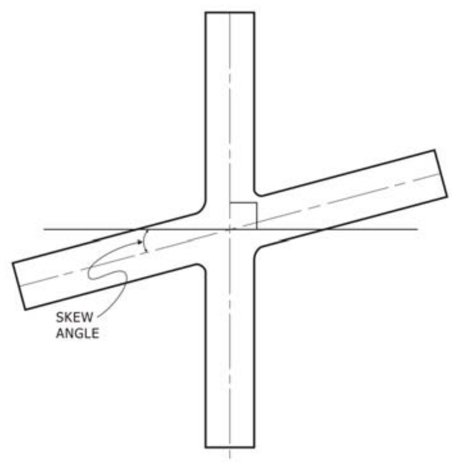
Crash effects of intersection design elements are summarized in section 14.6.2.

14.6.2. Intersection Design Element Treatments with Accident Modification Factors

14.6.2.1. Reduce Intersection Skew Angle

A skewed intersection has an angle of less than 90 degrees between the legs of the intersection; an intersection's skew is measured as the absolute value of the difference between 90 degrees and the actual intersection angle. Exhibit 14-14 illustrates a skewed intersection and how the skewed angle is measured.

Exhibit 14-14: Skewed Intersection



An intersection that is closer to perpendicular reduces the extent to which drivers must turn their head and neck to view approaching vehicles. This can be particularly beneficial to older drivers. Reducing the intersection skew angle can also result in increased sight distance. Drivers may then be better able to stay within the designated lane and better able to judge gaps in the crossing traffic flow.[(3)] Reducing the intersection skew angle can reduce crossing distances for pedestrians and vehicles, which reduces exposure to conflicts.

Intersection skew angle may be less important for signalized intersections than for stop-controlled intersections. A traffic signal separates most conflicting movements so the risk of accidents related to the skew angle between the intersecting approaches is limited.[(15)] The crash effect of the skew angle at a signalized intersection may, however, also depend on the operational characteristics of the traffic signal control.

Rural stop controlled intersections

Presented below are AMFs in the form of a function. One set is applicable to intersections on rural two-lane highways (Equations 14-1 and 14-2); the second set is applicable to intersections on rural multilane highways (Equations 14-3 through 14-6).

Intersections on Rural Two-Lane Highways

The crash effect of changing intersection skew angle at rural three-leg intersections with minor-road stop control is represented by the following AMF:[(16)]

$$AMF = e^{(0.0040 \times SKEW)} \qquad (14\text{-}1)$$

Where,

AMF = accident modification factor for total accidents; and

SKEW = intersection skew angle (in degrees); the absolute value of the difference between 90 degrees and the actual intersection angle

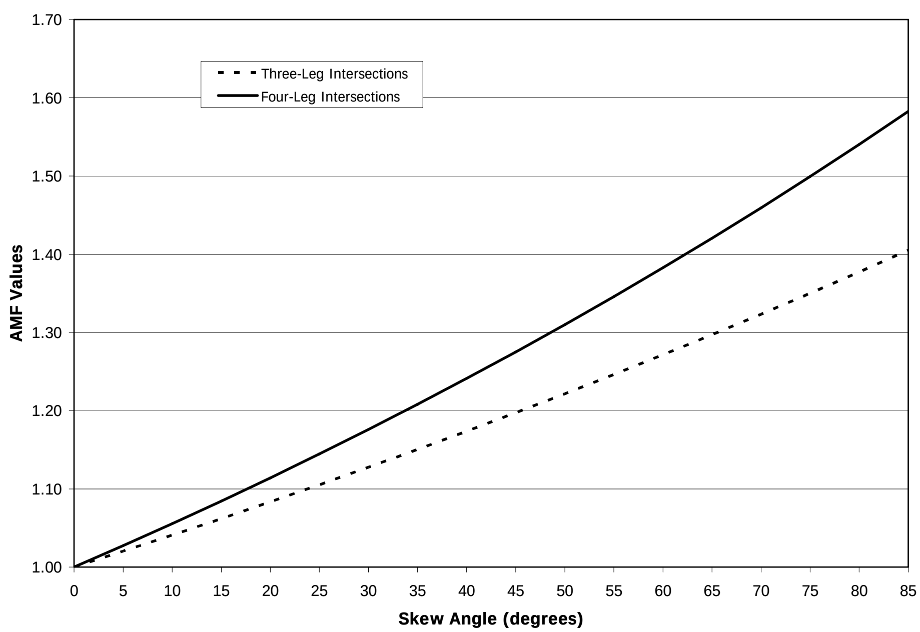
An analogous AMF for the crash effect of changing intersection skew angle at rural four-leg intersections with minor-road stop control is represented by:(16)

$$AMF = e^{(0.0054 \times SKEW)} \qquad (14\text{-}2)$$

The AMFs in Equations 14-1 and 14-2 are used in the predictive method for rural two-lane highways in *Chapter 10*. The base condition for these AMFs (i.e., the condition in which the AMF = 1.00) is the absence of intersection skew (i.e., a 90-degree intersection). The standard error of these AMFs is unknown.

Exhibit 14-15 below illustrates the relationship between the skew angle and the AMF value.

Exhibit 14-15: Potential Crash Effects of Skew Angle for Intersections with Minor-Road Stop Control on Rural Two-Lane Highways



The graph shown above indicates that, as the skew angle increases, the value of the AMF increases above 1.0, indicating an increase in crash frequency as the angle between the intersecting roadways deviates further from 90 degrees.

The gray box below presents an example of how to apply the preceding equations to assess the crash effects of reducing intersection skew angle at rural two-lane highway intersections with minor-road stop control.

Effectiveness of Reducing Intersection Skew Angles

Question:

A three-leg intersection with minor-road stop control on a rural two-lane highway has an intersection skew angle of approximately 45˚. Due to redevelopment adjacent to the intersection, the governing jurisdiction has an opportunity to reduce the skew angle to 10˚. What will be the likely change in expected average crash frequency?

Given Information:

- Existing intersection skew angle = 45˚
- Reduced intersection skew angle = 10˚
- Expected average crash frequency without treatment (See Part C Predictive Methods) = 15 crashes/year

Find:

- Expected average crash frequency with reduced skew angle
- Change in expected average crash frequency

Answer:

1) Identify the applicable AMF equation

   $AMF = e^{(0.0040 \times SKEW)}$ (Equation 14-1 or Exhibit 14-15)

2) Calculate the AMF for the existing condition

   $AMF = e^{(0.0040 \times 45)}$ = 1.20

3) Calculate the AMF for the after condition

   $AMF = e^{(0.0040 \times 10)}$ = 1.04

4) Calculate the treatment AMF ($AMF_{Treatment}$) corresponding to the change in SKEW angle

   $AMF_{Treatment}$ = 1.04/1.20 = 0.87

   The AMF corresponding to the treatment condition (reduced skew angle) is divided by the AMF corresponding to the existing condition yielding the treatment AMF ($AMF_{Treatment}$). The division is conducted to quantify the difference between the existing condition and the treatment condition. The *Part D Introduction and Applications Guidance* contains additional information.

5) Apply the $AMF_{Treatment}$ to the expected average crash frequency at the intersection without the treatment.

   Expected Crashes with Treatment = 0.87 x 15 crashes/year = 13.0 crashes/year

6) Calculate the difference between the expected average crash frequency without the treatment and with the treatment.

   Change in Expected Average Crash Frequency:
   15.0– 13.0 = 2.0 crashes/year reduction

7) Discussion: This example shows that expected average crash frequency may potentially be reduced by 2.0 crashes/year with the skew angle variation from 45 to 10 degrees. A standard error was not available for this AMF, therefore a confidence interval for the reduction cannot be calculated.

Intersections on Rural Multilane Highways

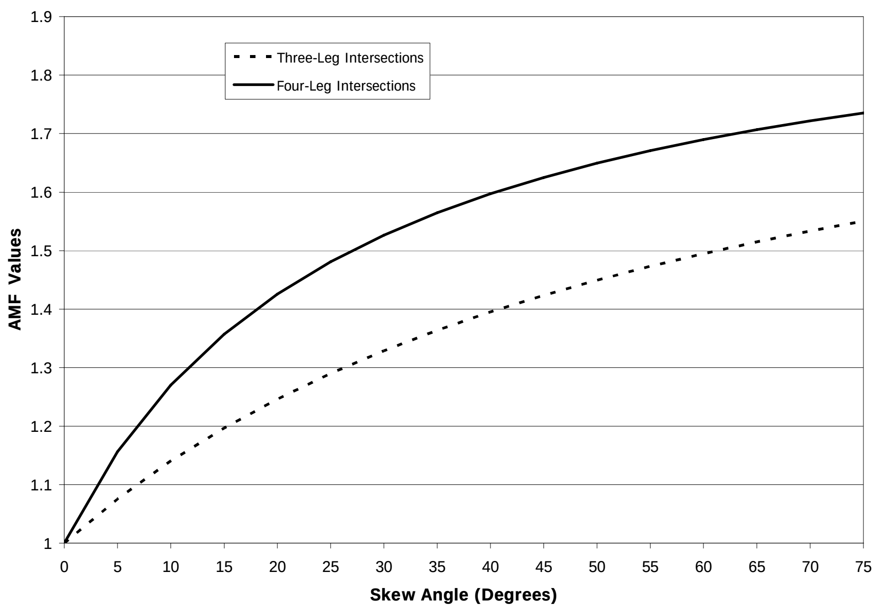
The crash effect of skew angle for three-leg intersections with minor-road stop control is represented by:[20]

$$AMF = \frac{0.016 \times SKEW}{(0.98 + 0.16 \times SKEW)} + 1.0 \qquad (14\text{-}3)$$

This AMF applies to total intersection accidents. The analogous AMF for four-leg intersections with minor-road stop control is: [20]

$$AMF = \frac{0.053 \times SKEW}{(1.43 + 0.53 \times SKEW)} + 1.0 \qquad (14\text{-}4)$$

Exhibit 14-16: Potential Crash Effects of Skew Angle of Three- and Four-leg Intersections with Minor-road Stop Control on Rural Multilane Highways



Equivalent AMFs for the crash effect of intersection skew on fatal and injury accidents (excluding possible-injury accidents, also known as C-injury accidents) for three-leg intersections with minor-road stop control are presented as Equations 14-5 and 14-6: [20]

$$AMF_{KAB} = \frac{0.017 \times SKEW}{(0.52 + 0.17 \times SKEW)} + 1.0 \qquad (14\text{-}5)$$

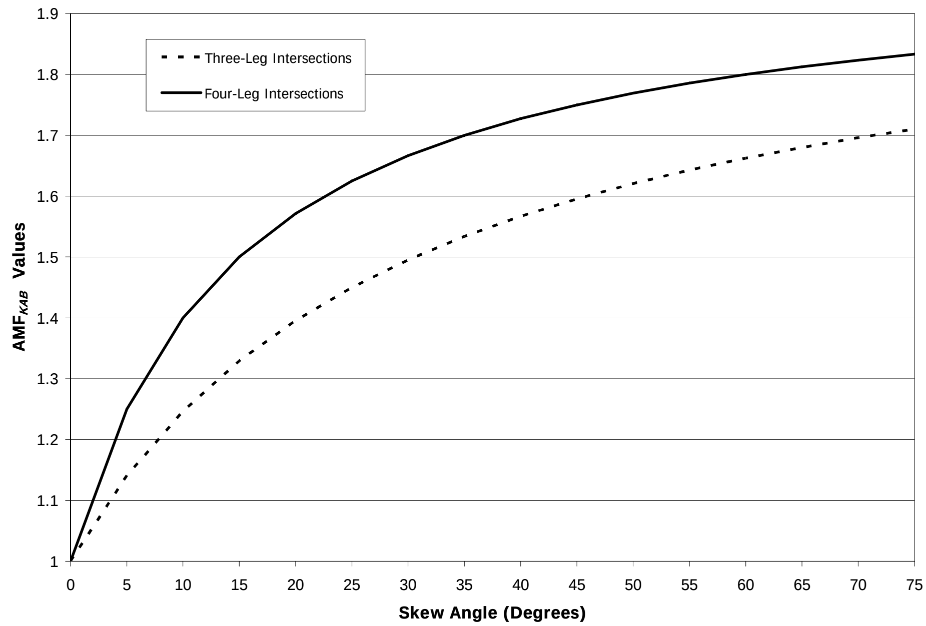
Where,

AMF_{KAB} = AMF for fatal-and-injury accidents (excluding possible-injury accidents, also known as C-injury accidents)

For four-leg intersections with minor-road stop control: [20]

$$AMF_{KAB} = \frac{0.048 \times SKEW}{(0.72 + 0.48 \times SKEW)} + 1.0 \qquad (14\text{-}6)$$

Exhibit 14-17: Potential Crash Effects of Skew Angle on Fatal and Injury Accidents for Three- and Four-leg Intersections with Minor-road Stop Control



The AMFs related to skew and presented in Equations 14-3 through 14-6 are used in the predictive method for rural multilane highways in Chapter 11.

The AMFs presented in Equations 14-3 through 14-6 are used in the predictive method for rural multilane highways in *Chapter 11* to represent the effect of intersection skew at intersections with minor-road stop control. The variability of these AMFs is unknown.

14.6.2.2. Provide a Left-Turn Lane on One or More Approaches to Three-Leg Intersections

Urban and rural 3-leg minor-road stop-controlled intersections, urban and rural 3-leg signalized intersections

By removing left-turning vehicles from the through-traffic stream, conflicts with through vehicles can be reduced or even eliminated depending on the signal timing and phasing scheme. Providing a left-turn lane allows drivers to wait in the turn lane until a gap in the opposing traffic allows them to turn safely. The left-turn lane helps to reduce conflicts with opposing through traffic.[(3)]

Exhibit 14-18 summarizes the crash effects of providing a left-turn lane on one approach of three-leg intersections under the following settings:

- Rural intersections with minor-road stop control;
- Urban intersections with minor-road stop control; and
- Rural or urban signalized intersections.

The AMFs in Exhibit 14-18 are used to represent the crash effects of providing left-turn lanes at three-leg intersections in the predictive method in *Chapters 10, 11,* and *12*. These AMFs apply to installing left-turn lanes on approaches without stop control at unsignalized intersections and on any approach at signalized intersections. The AMFs for installing left-turn lanes on two intersection approaches would be the AMF values shown in Exhibit 14-18 squared.

The base condition for the AMFs summarized in Exhibit 14-18 (i.e., the condition in which the AMF = 1.00) is a three-leg intersection approach without a left-turn lane.

Exhibit 14-18: Potential Crash Effects of Providing a Left-Turn Lane on One Approach to Three-Leg Intersections(15,16)

Treatment	Setting (Intersection type)	Traffic Volume AADT (veh/day)	Accident type (Severity)	AMF	Std. Error
Provide a left-turn lane on one major-road approach	Rural (minor-road stop-controlled three-leg intersection)(16)	Major road 1,600 to 32,400, Minor road 50 to 11,800	All types (All severities)	**0.56**	**0.07**
			All types (Injury)	**0.45**	**0.1**
	Urban (minor-road stop-controlled three-leg intersection)(16)	Major road 1,520 to 40,600, Minor road 200 to 8000	All types (All severities)	*0.67*	*0.2*
	Rural (Signal-controlled three-leg intersection)(16)	Unspecified	All types (All severities)	0.85	N/A°
	Urban (Signal-controlled three- leg intersection)(16)			0.93	N/A°
	Urban (Signal-controlled three-leg intersection)(15)	Unspecified	All types (Injury)	0.94	N/A°
	Urban (Minor-road stop-controlled three-leg intersection)(15)			0.65	N/A°

Base Condition: A three-leg intersection without left-turn lanes.

NOTE: AMFs apply to installation of left-turn lanes for uncontrolled approaches at unsignalized intersections and for any approach at signalized intersections.

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

° Standard error of the AMF is unknown.

14.6.2.3. Provide a Left-Turn Lane on One or More Approaches to Four-Leg Intersections

This section addresses the crash effects of providing a left-turn lane on one or two approaches to a four-leg intersection. The left-turn lanes addressed in this section may be defined by either painted or raised channelization.

Urban and rural 4-leg minor-road stop-controlled intersections, urban and rural 4-leg signalized intersections

By removing left-turning vehicles from the through-traffic stream, conflicts with through vehicles can be reduced or even eliminated depending on the signal timing and phasing scheme. Providing a left-turn lane allows drivers to wait in the turn lane until a gap in the opposing traffic allows them to turn safely. The left-turn lane helps to reduce conflicts with opposing through traffic.(3)

Left-turn lane on one approach

Providing a left-turn lane on one approach to a four-leg intersection reduces crashes of various types and severities under the following settings:

- Rural or urban intersection with minor-road stop control;
- Rural signalized intersection;
- Urban signalized intersection; and
- Urban intersection with recently implemented signal control (i.e. newly signalized). (16)

Exhibit 14-19 provides specific information regarding the AMFs that are used to calculate change in crashes. The AMFs in Exhibit 14-19 are used to represent the crash effects of providing left-turn lanes at four-leg intersections in the predictive method in *Chapters 10, 11,* and *12.* These AMFs apply to installing left-turn lanes on approaches without stop control at unsignalized intersections and on any approach at signalized intersections.

The base condition for the AMFs summarized in Exhibit 14-19 (i.e., the condition in which the AMF = 1.00) is a four-leg intersection without left-turn lanes on the major-road approaches.

Exhibit 14-19: Potential Crash Effects of Providing a Left-Turn Lane on One Approach to Four-Leg Intersections(16)

The AMFs in Exhibit 14-19 are used to represent the crash effects of providing left-turn lanes at four-leg intersections in the predictive methods in Chapters 10, 11, and 12.

Treatment	Setting (Intersection type)	Traffic Volume AADT (veh/day)	Accident type (Severity)	AMF	Std. Error
Provide a left-turn lane on one major-road approach	Rural (four-leg Minor-road stop-controlled intersection)	Major road 1,600 to 32,400, Minor road 50 to 11,800	All types (All severities)	**0.72**	**0.03**
			All types (Injury)	**0.65**	**0.04**
	Urban (four-leg minor-road stop-controlled four-leg intersection)	Major road 1,520 to 40,600, Minor road 200 to 8000	All types (All severities)	**0.73**	**0.04**
			All types (Injury)	**0.71**	**0.05**
	Rural (four-leg signalized intersection)	Unspecified	All types (All severities)	0.82	N/A°
	Urban (four-leg Signalized intersection)	Major road 7,200 to 55,100, Minor road 550 to 2,600	All types (All severities)	**0.90***	**0.1**
			All types (Injury)	**0.91**	**0.02**
	Urban (four-leg Newly signalized Intersection)	Major road 4,600 to 40,300, Minor road 100 to 13,700	All types (All severities)	**0.76**	**0.03**
			All types (Injury)	**0.72**	**0.06**

Base Condition: A four-leg intersection without left-turn lanes

NOTE: AMFs apply to installing left-turn lanes for uncontrolled approaches at unsignalized intersections and for any approach at signalized intersections.

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

° Standard error of AMF is unknown.

* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

Left-turn lanes on two approaches

When installing a left-turn lane on more than one approach, the AMF is raised to a power equal to the number of approaches.

Exhibit 14-20 provides AMFs, analogous to those in Exhibit 14-19, for installing left-turn lanes on two approaches to a four-leg intersection. The AMFs in Exhibit 14-20 are generally equivalent to the AMF values for one approach, shown in Exhibit 14-19, squared. For four-leg signalized intersections where left-turn lanes are provided on three or four approaches, the AMF for providing left-turn lanes on three or four approaches is equal to the AMF for installing left-turn lanes on one approach, from Exhibit 14-19, raised to the third or fourth power, respectively.

The base condition for the AMFs summarized in Exhibit 14-20 (i.e., the condition in which the AMF = 1.00) is a four-leg intersection without left-turn lanes on the major-road approaches.

Exhibit 14-20: Potential Crash Effects of Providing a Left-Turn lane on Two Approaches to Four-Leg Intersections(16)

Treatment	Setting (Intersection type)	Traffic Volume AADT (veh/day)	Accident type (Severity)	AMF	Std. Error
Provide a left-turn lane on both major-road approaches	Rural (four-leg Minor-road stop-controlled intersection)	Major road 1,500 to 32,400, Minor road 50 to 11,800	All types (All severities)	**0.52**	**0.04**
			All types (Injury)	**0.42**	**0.04**
	Urban (four-leg Minor-road stop-controlled intersection)	Major road 1,500 to 40,600, Minor road 200 to 8000	All types (All severities)	**0.53**	**0.04**
			All types (Injury)	**0.50**	**0.06**
	Rural (four-leg Signalized intersection)	Unspecified	All types (All severities)	0.67	N/A°
	Urban (four-leg Signalized intersection)	Major road 7,200 to 55,100, Minor road 550 to 2,600	All types (All severities)	**0.81**	**0.1**
			All types (Injury)	**0.83**	**0.02**
	Urban (four-leg Newly signalized[1] Intersection)	Major road 4,600 to 40,300, Minor road 100 to 13,700	All types (All severities)	**0.58**	**0.04**
			All types (Injury)	**0.52**	**0.07**
Base Condition: A four-leg intersection without a left-turn lane					

NOTE: AMFs apply to installing left-turn lanes for uncontrolled approaches at unsignalized intersections and for any approach at signalized intersections.

(1) A newly signalized intersection is an intersection where the signal was installed in conjunction with left-turn installation.

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

° Standard error of AMF is unknown.

The gray box example below illustrates how the information in Exhibit 14-19 is used to estimate the crash effects of providing a left-turn lane on two approaches to a four-leg intersection.

Effectiveness of Installing Left-Turn Lanes on Two Approaches of a Four-Leg Intersection

Question:

A urban minor street with an estimated 2,000 vpd traffic volume intersects a major arterial with an estimated 35,000 vpd traffic volume. The minor street is stop-controlled. The governing jurisdiction has an opportunity to add left-turn lanes to both major street approaches as part of a redevelopment project. What will be the likely change in the expected average injury crash frequency?

Given Information:

- Existing roadways = an urban minor street and a major arterial
- Existing intersection type = four-leg intersection
- Existing intersection control = minor-street stop-controlled
- Expected average injury crash frequency without treatment (See Part C Predictive Method) = 12 crashes/year

Find:

- Expected average injury crash frequency with installation of left-turn lanes
- Change in expected average injury crash frequency

Answer:

1) Identify the applicable AMF

   AMF = 0.50 (Exhibit 14-20)

2) Calculate the 95th percentile confidence interval estimation of injury crashes with the treatment standard error

   = [0.50 ± (2 x 0.06)] x (12 crashes/year) = 4.6 or 7.4 crashes/year

   The multiplication of the standard error by 2 yields a 95% probability that the true value is between 4.6 and 7.4 crashes/year. See Section 3.5.3 in *Chapter 3 Fundamentals* for a detailed explanation of standard error application.

3) Calculate the difference between the expected number of injury crashes without the treatment and the expected number of injury crashes with the treatment.

   Change in Expected Average Crash Frequency:

   Low Estimate = 12 - 7.4 = 4.6 crashes/year reduction

   High Estimate = 12 - 4.6 = 7.4 crashes/year reduction

4) **Discussion: This example illustrates that the construction of left-turn lanes on both approaches of the major arterial may potentially cause a reduction of 4.6 to 7.4 crashes per year. The confidence interval estimation yields a 95% probability that the reduction will be between 4.6 and 7.4 crashes per year.**

14.6.2.4. Provide a Channelized Left-Turn Lane at Four-Leg Intersections

Channelization is the separation of conflicting traffic movements into definite travel paths. Channelization is achieved by traffic islands, i.e. physical channelization, or by pavement markings, i.e. painted channelization.[1,9] Both physical and painted channelization are used to demarcate shared and exclusive lanes.

Rural 4-leg signalized, minor-road stop-controlled, and all-way stop controlled intersections

The crash effects of providing a physically channelized left-turn lane on both major and minor-road approaches to a rural four-leg intersection are shown Exhibit 14-21.[9]

The crash effect of providing a physically channelized left-turn lane on only the major-road approaches to a rural four-leg intersection is also shown in Exhibit 14-21.[9]

The base condition for the AMFs summarized in Exhibit 14-21 (i.e., the condition in which the AMF = 1.00) is a rural four-leg intersection without channelized left-turn lanes.

Exhibit 14-21: Potential Crash Effects of a Channelized Left-Turn Lane on Both Major and Minor-Road Approaches at Four-Leg Intersections[9]

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Provide a channelized left-turn lane on both major and minor-road approaches	Rural (four-leg intersection Two-lane roads)	5,000 to 15,000 vpd	All types (Injury)	**0.73**	**0.1**
Provide a channelized left-turn lane on both major-road approaches				*0.96**	*0.2*
Base Condition: Rural four-leg intersection without channelized left-turn lanes.					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.
* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.
"vpd"= vehicles per day

14.6.2.5. Provide a Channelized Left-Turn Lane at Three-Leg Intersections

Rural 3-leg signalized, minor-road stop-controlled, and all-way stop controlled intersections

Exhibit 14-22 summarizes the crash effects of providing a physically channelized left-turn lane on:

1. One major-road approach, and
2. One major-road approach and the minor-road approach to a rural three-leg intersection.[9]

The base condition for the AMFs below (i.e., the condition in which the AMF = 1.00) is a rural three-leg intersection without channelized left-turn lanes.

Exhibit 14-22: Potential Crash Effects of a Channelized Left-Turn Lane at Three-Leg Intersections[9]

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Provide a channelized left-turn lane on major-road approach	Rural (three-leg intersection Two-lane roads)	5,000 to 15,000 vpd	All types (Injury)	*0.73*	*0.2*
Provide a channelized left-turn lane on major-road approach and minor-road approach			All types (Injury)	*1.16*	*0.2*
Base Condition: Rural three-leg intersection without channelized left-turn lanes					

NOTE: *Italic* text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.
"vpd"= vehicles per day

14.6.2.6. Provide a Right-Turn Lane on One or More Approaches to an Intersection

This section addresses the effects on crash frequency of providing a right-turn lane on one approach to an intersection. The right-turn lanes addressed in this section may be defined by either painted or raised channelization.

Urban and rural signalized intersections, urban and rural minor-road stop controlled intersections

Right-Turn Lane on One Intersection Approach

- Exhibit 14-23 summarizes the crash effects of providing a right-turn lane on one intersection approach by setting and intersection type.

The base condition for the AMFs in Exhibit 14-23 (i.e., the condition in which the AMFs = 1.00) is an intersection without right-turn lanes on the major-road approaches.

The AMFs in Exhibit 14-23 apply to providing a right-turn lane on an uncontrolled approach to an unsignalized intersection or any approach to a signalized intersection.

Exhibit 14-23: Potential Crash Effects of Providing a Right-Turn Lane on One Approach to an Intersection[16]

Treatment	Setting (Intersection type)	Traffic Volume AADT (vpd)	Accident type (Severity)	AMF	Std. Error
Provide a right-turn lane on one major-road approach	Rural and urban (three- or four-leg minor-road stop-controlled intersection)	Major road 1,520 to 40,600 Minor road 25 to 26,000 vpd	All types (All severities)	**0.86**	**0.06**
			All types (Injury)	**0.77**	**0.08**
	Rural and urban (three- or four-leg signalized intersection)	Major road 7,200 to 55,100 Minor road 550 to 8,400	All types (All severities)	**0.96**	**0.02**
			All types (Injury)	**0.91**	**0.04**
Base Condition: Intersection without right-turn lanes on major road approaches					

NOTE: AMFs apply to installation of right-turn lanes for uncontrolled approaches at unsignalized intersections an for any approach at signalized intersections.

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

Right-turn Lane on Two Approaches to an Intersection

Exhibit 14-24 summarizes the crash effects of providing a right-turn lane on two approaches to a rural or urban intersection.

The AMFs in Exhibit 14-24 apply to providing a right-turn lane on an uncontrolled approach to an unsignalized intersection or any approach to a signalized intersection. The AMFs for providing right-turn lanes on approaches to an intersection in Exhibit 14-24 are equivalent to the AMF values for one approach, shown in Exhibit 14-23, squared. For signalized intersections where right-turn lanes are provided on three or four approaches, the AMF values for installing right-turn lanes is equal to the AMF value for installing a right-turn lane on one approach, shown in Exhibit 14-23, raised to the third or fourth power, respectively.

The base condition for the AMFs in Exhibit 14-24 (i.e., the condition in which the AMF = 1.00) is an intersection without right-turn lanes on the major-road approaches.

Exhibit 14-24: Potential Crash Effects of Providing a Right-Turn Lane on Two Approaches to an Intersection[16]

Treatment	Setting (Intersection type)	Traffic Volume AADT (Veh/Day)	Accident type (Severity)	AMF	Std. Error
Provide a right-turn lane on both major-road approaches	Rural and urban (minor-road stop-controlled intersection)	Major road 1,520 to 40,600 Minor road 25 to 26,000	All types (All severities)	**0.74**	**0.08**
	Rural and urban (Signalized intersection)	Major road 7,200 to 55,100 Minor road 550 to 8,400		**0.92**	**0.03**
	Rural and urban minor-road stop-controlled intersection[15]	Unspecified	All types Injury	0.59	N/A°
	Rural and urban Signalized intersection[15]			0.83	N/A °
Base Condition: Intersection without right-turn lanes on major-road approaches					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
° Standard error of AMF is unknown.

14.6.2.7. Increase Intersection Median Width

This section presents the crash effects related to median width. Medians are intended to perform several functions. Some of the main functions are:

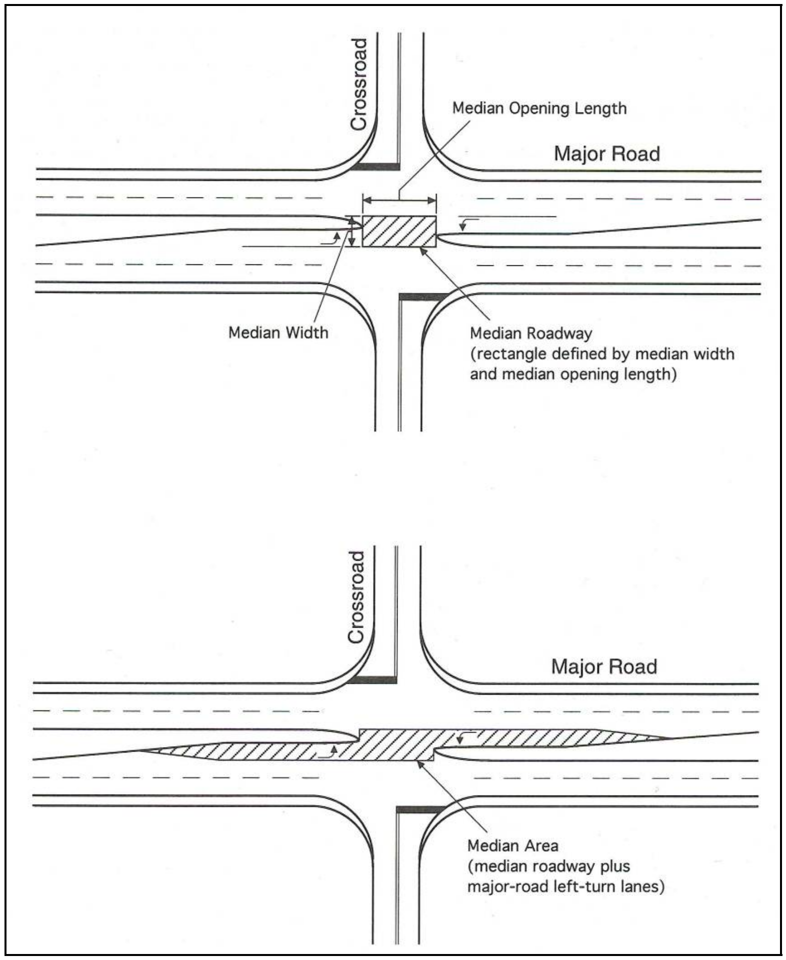
- To separate opposing traffic;
- To allow space for the storage of left-turning, U-turning vehicles;
- Minimize headlight glare; and
- Provide width for future lanes.[1,25]

At an intersection, the following definitions of the median apply.

- Median width is the total width between the edges of opposing through lanes, including the left shoulder and the left-turn lanes, if any.[18]
- Median opening length is the total length of break in the median provided for cross street and turning traffic.[18] The design of a median opening is generally based on traffic volumes, urban/rural area characteristics, and type of turning vehicles.[1]
- Median roadway is the paved area in the center of the divided highway at an intersection defined by the median width and the median opening length.[18]
- Median area is the median roadway plus the major-road left-turn lanes, if any.[18]

The median width, length, roadway, and area are illustrated in Exhibit 14-25.

Exhibit 14-25: Median Width, Median Roadway, Median Opening Length, and Median Area [18]



Urban, suburban, and rural 4-leg unsignalized intersections,
Urban and suburban 3-leg unsignalized intersections, and
Urban and suburban 4-leg signalized intersections

If increasing the median width by more than 3-ft, the AMF is calculated by raising the AMF to the power of the number of increments.

Exhibit 14-26 summarizes the crash effects of increasing intersection median width by a 3-ft increment at intersections, where existing medians are between 14 and 80-ft wide.[18]

The base condition for the AMFs summarized in Exhibit 14-26 (i.e., the condition in which the AMF = 1.00) is a median of 14-ft to 80-ft wide.

Exhibit 14-26: Potential Crash Effects of Increasing Intersection Median Width (18)

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Increase intersection median width by 3-ft increment	Rural (Four-leg unsignalized)	Unspecified	Multiple-vehicle (All severities)	**0.96^**	**0.02**
			Multiple-vehicle (Injury)	**0.96^**	**0.02**
	Urban and suburban (Four-leg unsignalized)		Multiple-vehicle (All severities)	**1.06**	**0.01**
			Multiple-vehicle (Injury)	**1.05**	**0.02**
	Urban and suburban (Three-leg unsignalized)		Multiple-vehicle (All severities)	**1.03**	**0.01**
	Urban and suburban (Four-leg signalized)		Multiple-vehicle (All severities)	**1.03**	**0.01**
			Multiple-vehicle (Injury)	**1.03**	**0.01**
Base Condition: A median 14-ft to 80-ft wide					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
These values are valid for median widths between 14 and 80-ft (4 to 24 m).
^ Observed variability suggests that this treatment could result in no effect on crashes. See Part D Applications Guidance.

14.6.2.8. Provide Intersection Lighting

Intersection lighting includes conventional forms of installing luminaires to illuminate the intersection proper and approach to the intersection.

All intersections

The base condition for the AMFs shown in Exhibit 14-27 (i.e., the condition in which the AMF = 1.00) is an intersection without illumination (i.e. artificial lighting).

Exhibit 14-27: Potential Crash Effects of Providing Intersection Illumination (9,12,10,26)

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Provide intersection illumination	All settings (All types)	Unspecified	All types Nighttime (Injury)	**0.62**	**0.1**
			Pedestrian Nighttime (Injury)	*0.58*	*0.2*
Base Condition: An intersection without lighting					

NOTE: Based on U.S. studies: Griffith 1994, Preston 1999 and International studies: Wanvik 2004; Elvik and Vaa 2004

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

Non-injury accidents may also be reduced by installing illumination. Intersection illumination appears to have the greatest effect on fatal pedestrian nighttime crashes. However, the magnitude of the crash effect is not certain at this time.

14.7. CRASH EFFECTS OF INTERSECTION TRAFFIC CONTROL AND OPERATIONAL ELEMENTS

14.7.1. Background and Availability of AMFs

The following sections provide information on the crash effects of treatments related to intersection traffic control and operational elements. Traffic control devices at an intersection include signs, signals, warning beacons, and pavement markings. Operational elements of an intersection include the type of traffic control, traffic signal operations, speed limits, traffic calming, and on-street parking.

The treatments discussed in this section and the corresponding AMFs available are summarized in Exhibit 14-28.

Exhibit 14-28: Treatments Related to Intersection Traffic Control and Operational Elements

HSM Section	Treatment	Urban				Suburban				Rural			
		Stop		Signal		Stop		Signal		Stop		Signal	
		Minor Road	All-Way	3-Leg	4-Leg	Minor Road	All-Way	3-Leg	4-Leg	Minor Road	All-Way	3-Leg	4-Leg
14.7.2.1	Prohibit left-turns and/or U-turns with "No Left Turn", "No U-Turn" signs	✓	-	✓	✓	✓	-	✓	✓	-	-	-	-
14.7.2.2	Provide Stop Ahead pavement markings	-	-	-	-	-	-	-	-	✓	✓	-	-
14.7.2.3	Provide flashing beacons at stop-controlled intersections	✓	✓	N/A	N/A	✓	✓	N/A	N/A	✓	✓	N/A	N/A
14.7.2.4	Modify left-turn phase	-	-	-	✓	-	-	-	-	-	-	-	-
14.7.2.5	Replace direct left-turns with right-turn/U-turn combination	✓	-	-	-	✓	-	-	-	✓	-	-	-
14.7.2.6	Permit right-turn on red	-	-	✓	✓	-	-	✓	✓	-	-	✓	✓
14.7.2.7	Modify change and clearance interval	-	-	-	✓	-	-	-	✓	-	-	-	✓
14.7.2.8	Install red-light cameras	-	-	✓	✓	-	-	-	-	-	-	-	-
Appendix	Place transverse markings on roundabout approaches	T											
Appendix	Install pedestrian signal heads at signalized intersections	N/A	N/A	T	T	N/A	N/A	-	-	N/A	N/A	-	-
Appendix	Modify pedestrian signal heads	N/A	N/A	T	T	N/A	N/A	-	-	N/A	N/A	-	-
Appendix	Install pedestrian countdown signals	N/A	N/A	T	T	N/A	N/A	T	T	N/A	N/A	T	T
Appendix	Install automated pedestrian detectors	N/A	N/A	T	T	N/A	N/A	T	T	N/A	N/A	T	T
Appendix	Install stop lines and other crosswalk enhancements	T	T	T	T	T	T	T	T	T	T	T	T
Appendix	Provide exclusive pedestrian signal timing pattern	-	-	T	T	-	-	-	-	-	-	-	-
Appendix	Provide leading pedestrian interval signal timing pattern	N/A	N/A	T	T	N/A	N/A	T	T	N/A	N/A	T	T
Appendix	Provide actuated control	N/A	N/A	T	T	N/A	N/A	T	T	N/A	N/A	T	T
Appendix	Operate signals in "night-flash" mode	N/A	N/A	T	T	N/A	N/A	T	T	N/A	N/A	T	T
Appendix	Provide advance static warning signs and beacons	T	T	T	T	T	T	T	T	T	T	T	T
Appendix	Provide advance warning flashers and warning beacons	N/A	N/A	T	T	N/A	N/A	T	T	N/A	N/A	T	T
Appendix	Provide advance overhead guide signs	T	T	T	T	T	T	T	T	T	T	T	T
Appendix	Install additional pedestrian signs	T	T	T	T	T	T	T	T	T	T	T	T
Appendix	Modify pavement color for bicycle crossings	T	T	-	-	T	T	-	-	T	T	-	-
Appendix	Place "slalom" profiled pavement markings at bicycle lanes	T	T	T	T	T	T	T	T	T	T	T	T
Appendix	Install rumble strips on intersection approaches	T	T	T	T	-	-	-	-	-	-	-	-

NOTE: ✓ = Indicates that an AMF is available for this treatment.

T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.

\- = Indicates that an AMF is not available and a trend is not known.

N/A = Indicates that the treatment is not applicable to the corresponding setting.

Crash effects of intersection traffic control and operational elements are summarized in 14.7.2.

14.7.2. Intersection Traffic Control and Operational Element Treatments with Accident Modification Factors

14.7.2.1. Prohibit Left-Turns and/or U-Turns by Installing "No Left Turn" and "No U-Turn" Signs

Prohibiting left-turns and/or U-turns at an intersection is one means to increase an intersection's capacity and reduce the number of vehicle conflict points at the intersection. The crash effects of prohibiting these movements via signing are discussed in this section.

Urban, suburban minor-road stop-controlled and signalized intersections

Exhibit 14-29 summarizes the crash effects of prohibiting left-turns and U-turns at intersections through the use of "No Left-Turn" and/or "No U-Turn" for urban and suburban three- and four-leg intersections and median crossovers.

Accident migration is a possible result of prohibiting left-turns and U-turns at intersections and median crossovers since drivers may use different streets or take different routes to reach a destination.

The base condition for the AMFs summarized in Exhibit 14-29 (i.e., the condition in which the AMF = 1.00) is not clear and was not specified in the original compilation of the material.

Exhibit 14-29: Potential Crash Effects of Prohibiting Left-Turns and/or U-Turns by Installing "No Left Turn" and "No U-Turn" Signs [6]

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Prohibit left-turns with "No Left Turn" sign	Urban and suburban (Arterial three- and four-leg, and median crossovers)	Entering AADT 19,435 to 42,000 vpd	Left-turn (All severities)	*0.36*	*0.20*
			All intersection crashes (All severities)	**0.32**	**0.10**
Prohibit left-turns and U-turns with "No Left Turn" and "No U-Turn" signs			Left-turn and U-Turn crashes (All severities)	*0.23*	*0.20*
			All intersection crashes (All severities)	*0.28*	*0.20*
Base Condition: Unspecified.					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

Prohibiting U-Turns by only installing "No U-Turn" signs appears to reduce U-turn crashes of all severities and all intersection crashes of all severities.[6] However, the magnitude of the crash effect is not certain at this time.

14.7.2.2. Provide "Stop Ahead" Pavement Markings

Providing "Stop Ahead" pavement markings can alert drivers to the presence of an intersection. These markings can be especially useful in rural areas at unsignalized intersections with patterns of crashes which suggest that drivers may not be aware of the presence of the intersection.

Rural stop-controlled intersections

Exhibit 14-30 summarizes the crash effects of providing "stop ahead" pavement markings on approaches to stop controlled intersections in rural areas. The base condition for the AMFs summarized in Exhibit 14-30 (i.e., the condition in which the AMF = 1.00) is a stop controlled intersection in a rural area without a "stop ahead" pavement marking.

Exhibit 14-30: Potential Crash Effects of Providing Stop Ahead Pavement Markings [13]

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Provide "stop ahead" pavement markings	Rural (Stop-controlled)	Unspecified	Right angle (All severities)	*1.04**	*0.3*
			Rear-end (All severities)	*0.71*	*0.3*
			All types (Injury)	*0.78*	*0.2*
			All types (All severities)	**0.69**	**0.1**
	Rural (Stop-controlled three-leg)		All types (Injury)	*0.45*	*0.3*
			All types (All severities)	*0.40*	*0.2*
	Rural (Stop-controlled four-leg)		All types (Injury)	*0.88*	*0.3*
			All types (All severities)	*0.77*	*0.2*
	Rural (All-way stop-controlled)		All types (Injury)	*0.58*	*0.3*
			All types (All severities)	*0.44*	*0.2*
	Rural (Minor-road stop-controlled)		All types (Injury)	*0.92**	*0.3*
			All types (All severities)	*0.87*	*0.2*
Base condition: Stop controlled intersection in a rural area without a "stop ahead" pavement marking					

Notes: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.
* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

14.7.2.3. Provide Flashing Beacons at Stop-Controlled Intersections

Flashing beacons can help alert drivers to the presence of unsignalized intersections that may be unexpected or may not be visible. Flashing beacons may be particularly appropriate for intersections with patterns of angle collisions related to lack of driver awareness of the intersection. Flashing beacons could be installed overhead or mounted on the stop sign. There are two major types of beacons: (1) standard beacons that flash all the time, and (2) actuated beacons that are triggered by an approaching vehicle. The AMFs presented in this section apply to standard beacons that flash all the time.

Urban, suburban, and rural stop controlled intersections

Exhibit 14-30 summarizes the effects on crash frequency of providing flashing beacons at stop-controlled four-leg intersections on two-lane roads.

The base condition for the AMFs summarized in

Exhibit 14-31 (i.e., the condition in which the AMF = 1.00) is a stop- controlled four-leg intersection without flashing beacons on a two-lane road.

Traffic Volume Setting AADT Accident type

Treatment	Setting (Intersection type)	Traffic Volume AADT (veh/day)	Accident type (Severity)	AMF	Std. Error
Provide flashing beacons at stop controlled intersections	All settings (Stop- controlled)	Major road volume: 250 to 42,520 Minor road volume: 90 to 13,270	All types (All severities)	**0.95***	**0.04**
			All types (Injury)	**0.90***	**0.06**
			Rear end (All severities)	**0.92***	**0.1**
			Angle (All severities)	**0.87**	**0.06**
	Rural (Stop-controlled)		Angle (All severities)	**0.84**	**0.06**
	Suburban (Stop-controlled)		Angle (All severities)	**0.88**	**0.1**
	Urban (Stop-controlled)		Angle (All severities)	*1.12*	*0.3*
	All settings (Minor-road stop-controlled)		Angle (All severities)	**0.87**	**0.06**
	All settings (All-way stop-controlled)		Angle (All severities)	*0.72*	*0.2*
	All settings (Standard overhead beacons)		Angle (All severities)	**0.88**	**0.06**
	All settings (Standard stop mounted beacons)		Angle (All severities)	*0.42*	*0.2*
	All settings (Standard overhead and stop mounted beacons)		Angle (All severities)	**0.87**	**0.06**
	All settings (Actuated beacons)		Angle (All severities)	**0.86**	**0.1**
Base condition: Stop-controlled four-leg intersection on a two-lane road without flashing beacons					

Exhibit 14-31: Potential Crash Effects of Providing Flashing Beacons at Stop-Controlled Intersections on Two-Lane Roads (31)

Notes: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

* Observed variability suggests that this treatment could result in a increase, decrease, or no change in crashes. See Part D Applications Guidance.

14.7.2.4. Modify Left-Turn Phase

Left-turn phasing at a traffic signal is generally determined by considering traffic flows at the intersection and the intersection design. The following types of left-turn signal phases may be used:

- Permissive;
- Protected/permissive;
- Permissive/protected;
- Protected leading (protected left phase before through phase);
- Protected lagging (through phase before protected left phase); or
- Split phasing (left turns operate independently of each other and concurrently with the through movements).

Alternatively, under certain conditions, left-turns at intersections can be replaced with a combined right-turn/U-turn maneuver. This subsection addresses the effects on crash frequency of replacing permissive, permissive/protected, or protected/permissive with protected left-turn phase, and replacing permissive phasing with permissive/protected or protected/permissive phasing.

Urban 4-leg signalized intersections

Exhibit 14-32 summarizes the crash effects of modifying the left-turn phase at one or more approaches to a four-legged intersection.

The base condition for the AMFs summarized in Exhibit 14-32 (i.e., the condition in which the AMF = 1.00) for changing to protected phasing is permissive, permissive/protected or protected/permissive phasing. The base condition for changing to permissive/protected or protected/permissive phasing is permitted phasing.

Exhibit 14-32: Potential Crash Effects of Modifying Left-Turn phase at Urban Signalized Intersections [8,15,22]

Treatment	Setting (Intersection type)	Traffic Volume AADT (veh/day)	Accident type (Severity)	AMF	Std. Error
Change to protected phasing [8,15]	Urban (Four- and three-leg signalized)	Unspecified	Left-turn crashes on treated approach (All severities)	**0.01**+	**0.01**
			All types (All severities)	**0.94***+	**0.1**
Change from permissive to protected/permissive or permissive/protected phasing [15,22]	Urban (Four-leg signalized)	Major road 3,000 to 77,000 and Minor road 1 to 45,500	Left-turn (Injury)	**0.84**	**0.02**
Change from permissive to protected/permissive or permissive/protected phasing [15]	Urban (Four-leg signalized)	Unspecified	All types (All severities)	0.99	N/A°
Base Condition: For changing to protected phasing, base condition is permissive, permissive/protected or protected/permissive phasing. For changing to permissive/protected or protected/permissive phasing, base condition is permitted phasing.					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

° Standard error of AMF is unknown.

+ Combined AMF, see Part D Applications Guidance.

The AMFs in Exhibit 14-32 are difficult to apply in practice because the number of approaches for which left-turn phasing is provided is not specified. Exhibit 14-33 shows the AMF for left-turn phasing developed by an expert panel from an extensive literature review.[17,19] Where left-turn phasing is provided on two, three, or four approaches to an intersection, the AMF values shown in Exhibit 14-33 may be multiplied together. For example, where protected left-turn phasing is provided on two approaches to a signalized intersection, the applicable AMF would be the AMF shown in Exhibit 14-33 squared. The base condition for the AMFs summarized in Exhibit 14-33 (i.e., the condition in which the AMF = 1.00) is the use of permissive left-turn signal phasing.

Exhibit 14-33: Potential Crash Effects of Modifying Left-Turn Phase on One Intersection Approach [17,19]

Treatment	Setting (Intersection type)	Traffic Volume AADT (veh/day)	Accident type (Severity)	AMF	Std. Error
Change from permissive to protected/permissive or permissive/protected phasing	Unspecified (Unspecified)	Unspecified	Unspecified (All severities)	0.99	N/A°
Change from permissive to protected	Unspecified (Unspecified)	Unspecified	Unspecified (All severities)	0.94	N/A°
Base Condition: Permissive left-turn phase.					

NOTE: Use AMF = 1.00 for all unsignalized intersections. If several approaches to a signalized intersection have left-turn phasing, the values of the AMF for each approach should be multiplied together.

The gray box below illustrates how to apply the information in Exhibit 14-33 to assess the crash effects of providing protected leading left-turn phasing.

Effectiveness of Modifying Left-Turn Phasing

Question:

An urban signalized intersection has permissive/protected east-west left-turn phases and permissive north/south left-turn phases. As part of a signal retiming project, the governing jurisdiction looked into providing only leading protected left-turn phases on the east-west approaches and maintaining the permissive north/south left-turn phasing. What will be the likely change in expected average crash frequency?

Given Information:

- Existing intersection control = urban four-leg traffic signal
- Existing left-turn signal phasing = permissive/protected on the east/ west approaches, permissive on the north/south approaches.
- Intersection expected average crash frequency with the existing treatment (See Part C Predictive Method) = 14 crashes/year

Find:

- Expected average crash frequency with implementation of leading protected left-turn phases at the east and west approaches
- Change in expected average crash frequency

Answer:

1) Calculate the existing conditions AMF

AMF = 0.99 for each permissive/protected left-turn approach (Exhibit 14-33)

AMF = 1.00 for each permissive left-turn approach (Exhibit 14-33)

$AMF_{Existing}$ = 0.99 x 0.99 x 1.00 x 1.00 = 0.98

The intersection-wide AMF for existing conditions is computed by multiplying the individual AMFs at each approach to account for the combined effect of left-turn phasing treatments. Each approach is assigned an AMF from Exhibit 14-33 which corresponds to individual left-turn phasing treatments at each approach.

2) Calculate the Future Conditions AMF

AMF = 0.94 per protected left-turn approach

AMF_{Future} = 0.94 x 0.94 x 1.00 x 1.00 = 0.88

Calculations for future conditions are similar to the calculations for existing conditions.

3) Calculate the treatment AMF ($AMF_{Treatment}$)

$AMF_{Treatment} = AMF_{Future} / AMF_{Existing}$ = 0.88/0.98 = 0.90

The AMF corresponding to the treatment condition is divided by the AMF corresponding to the existing condition yielding the treatment AMF ($AMF_{Treatment}$). The division is conducted to quantify the difference between the existing condition and the treatment condition. See *Part D Introduction and Applications Guidance*.

4) Apply the treatment AMF ($AMF_{Treatment}$) to the expected average crash frequency at the intersection with the existing treatment.

= 0.90 x (14 crashes/year) = 12.6 crashes/year

5) Calculate the difference between the expected average crash frequency with the existing treatment and with the future treatment.

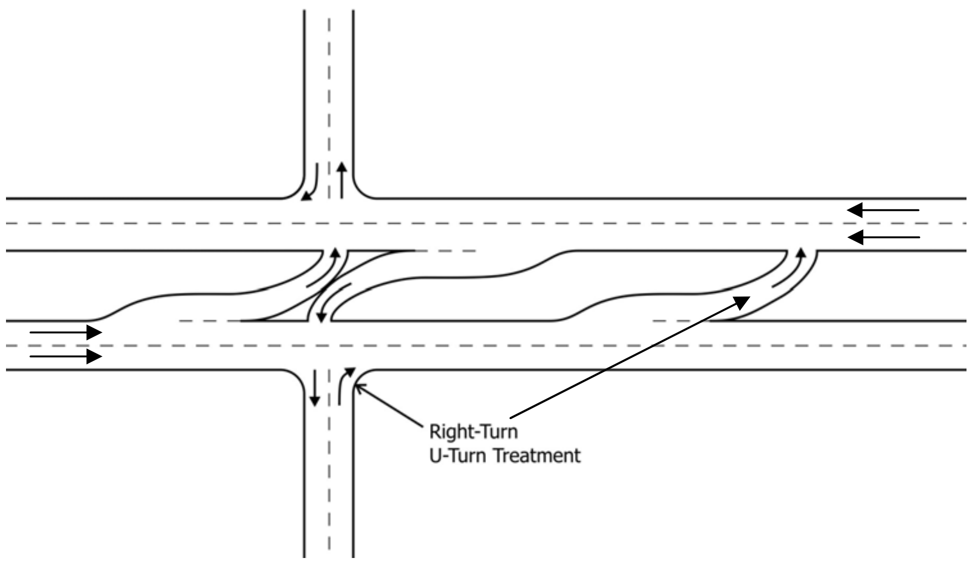
Change in Expected Average Crash Frequency Variation:
14.0 – 12.6 = 1.4 crashes/year reduction

6) Discussion: This example shows that expected average crash frequency may potentially be reduced by 1.4 crashes/year with the implementation of protected left-turn phasing on the east and west approaches. A standard error was not available for this AMF, therefore a confidence interval for the reduction cannot be calculated.

14.7.2.5. Replace Direct Left-Turns with Right-Turn/U-turn Combination

Replacing direct left-turns with right-turn/u-turn combination is applied to minor streets and driveways intersecting with divided arterials. A directional median is typically used to eliminate left-turns off of the minor street. Closing the side-street left-turn using directional median openings effectively forms a T-intersection with a closed median, eliminating direct left-turns at unsignalized intersections and driveways on to divided arterials. Drivers must turn right and then perform a U-turn on the divided arterial at a downstream location to access the desired side street or access point.[32] Exhibit 14-34 illustrates a conceptual example of closing a side street left-turn and serving the left-turn movement through a right-turn and U-turn movement.

Exhibit 14-34: Right-Turn/U-Turn Combination



Urban, suburban, and rural stop-controlled intersections

The crash affects of this treatment on four-, six-, and eight-lane divided arterials with AADT greater than 34,000 vehicles/day are shown in Exhibit 14-35.[32] Exhibit 14-35 also summarizes the effects on non-injury, injury, rear-end and angle crashes. The information in Exhibit 14-35 is based on arterials with the following characteristics:

- Posted speed limits between 40 and 55 mph,
- No on-street parking, and
- Segments of 0.1 to 0.25 miles in length.

Additional information regarding the setting of the intersections, median width, and the minor street volume are not specified in the original studies.

The base condition for the AMFs summarized in Exhibit 14-35 (i.e., the condition in which the AMF = 1.00) consists of an unsignalized intersection that provides for direct left-turns.

Exhibit 14-35: Potential Crash Effects of Replacing Direct Left-Turns with Right-Turn/U-turn Combination (32)

Treatment	Setting (Intersection type)	Traffic Volume AADT (veh/day)	Accident type (Severity)	AMF	Std. Error
Replace direct left-turn with right-turn/U-turn	Unspecified (Unsignalized intersections-access points on 4-, 6-, and 8-lane divided arterial)	Arterial AADT > 34,000 Minor road/ access point volume unspecified	All types (All severities)	**0.80**	**0.1**
			All types (Non-injury)	*0.89*	*0.2*
			All types (Injury)	*0.64*	*0.2*
			Rear-end (All severities)	*0.84*	*0.2*
			Angle (All severities)	*0.64*	*0.2*
	Unspecified (Unsignalized intersections-access points on 4-lane divided arterial)		All types (All severities)	*0.49*	*0.3*
	Unspecified (Unsignalized intersections-access points on 6-lane divided arterial)		All types (All severities)	*0.86*	*0.2*
			All types (Non-injury)	*0.95**	*0.2*
			All types (Injury)	*0.69*	*0.2*
			Rear-end (All severities)	*0.91**	*0.3*
			Angle (All severities)	*0.67*	*0.3*
Base Condition: An unsignalized intersection at which direct left-turns can be made					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.
* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

14.7.2.6. Permit Right-Turn-on-Red Operation

Right-turn operations are generally determined by considering traffic flows at the intersection and the intersection design. Right-turn operations at traffic signals may include restricted, permitted, or right-turn-on-red phasing.

Urban, suburban, and rural signalized intersections

Permitting right-turn-on-red operation at signalized intersections:

- Increases pedestrian and bicyclist crashes;(27)
- Increases injury and non-injury crashes involving right-turning vehicles; and(9)

- Increases the total number of accidents of all types and severities.[7]

The effects on crash frequency of permitting right-turn-on-red operations at signalized intersections are presented in Exhibit 14-36.

Alternatively, right-turn operations can be considered from the perspective of prohibiting right-turn-on-red operations, rather than permitting right-turn-on-red. The AMF for prohibiting right-turn-on-red on one or more approaches to a signalized intersection is determined as:

$$AMF = (0.98)^{nprohib} \tag{14-7}$$

Where,

AMF = accident modification factor for the effect of prohibiting right-turn on-red on total crashes (not including vehicle-pedestrian and vehicle-bicycle collision); and

nprohib = number of signalized intersection approaches for which right-turn on-red is prohibited.

Both forms of the AMFs are consistent with one another.

Care should be taken to recognize the base conditions for this treatment (i.e., the condition in which the AMF = 1.00). When considering the crash effects of permitting right-turn-on-red operations, the base condition for the AMFs above is a signalized intersection prohibiting right-turns-on-red. Alternatively, when considering the AMF for prohibiting right-turn-on-red operations at one or more approaches to a signalized intersection, the base condition is permitting right-turn-on-red at all approaches to a signalized intersection.

Exhibit 14-36: Potential Crash Effects of Permitting Right-Turn-On-Red Operation [7,27]

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Permit right-turn-on-red	Unspecified (Signalized)	Unspecified	Pedestrian and Bicyclist (All severities) [27]	**1.69+**	**0.1**
			Pedestrian (All severities) [27]	*1.57*	*0.2*
			Bicyclist (All severities) [27]	*1.80*	*0.2*
			Right-turn (Injury) [9]	**1.60**	**0.09**
			Right-turn (Non-injury) [9]	**1.10**	**0.01**
			All types (All severities) [7]	**1.07**	**0.01**
Base Condition: A signalized intersection with prohibited right-turn-on-red operation					

NOTE: (6) Based on U.S. studies: McGee and Warren 1976; McGee 1977; Preusser, Leaf, DeBartolo, Blomberg and Levy 1982; Zador, Moshman and Marcus 1982; Hauer 1991

Bold text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

\+ Combined AMF, see Part D Applications Guidance.

14.7.2.7. Modify Change plus Clearance Interval

Intersection signal operational characteristics, such as cycle lengths and change plus clearance intervals, are typically based on the established practices and standards of the jurisdiction. Intersection-specific characteristics, such as traffic flows and intersection design, influence certain signal operational changes. Signal timings, clearance intervals, and cycle lengths at intersections can vary greatly. This section addresses modifications to the change plus clearance interval of an intersection and the corresponding effects on crash frequency.

Urban, suburban, and rural 4-leg intersections

The ITE “Proposed Recommended Practice for Determining Vehicle Change Intervals” suggests determining the change plus clearance interval based on:

- Driver perception/reaction time;
- Velocity of approaching vehicles;
- Deceleration rate;
- Grade of the approach;
- Intersection width;
- Vehicle length;
- Velocity of approaching vehicle; and
- Pedestrian presence.[28]

Exhibit 14-37 summarizes the specific AMFs related to modifying the change plus clearance interval. The base condition for the AMFs summarized in Exhibit 14-37 (i.e., the condition in which the AMF = 1.00) was unspecified.

Exhibit 14-37: Potential Crash Effects of Modifying Change Plus Clearance Interval (28)

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Modify change plus clearance interval to ITE 1985 Proposed Recommended Practice	Unspecified (Four-leg signalized)	Unspecified	All types (All severities)	**0.92***	**0.07**
			All types (Injury)	**0.88**	**0.08**
			Multiple-vehicle (All severities)	**0.95***	**0.07**
			Multiple-vehicle (Injury)	**0.91***	**0.09**
			Rear-end (All severities)	*1.12*[?]	*0.2*
			Rear-end (Injury)	*1.08**[?]	*0.2*
			Right angle (All severities)	*0.96**[?]	*0.2*
			Right angle (Injury)	*1.06*[?]	*0.2*
			Pedestrian and Bicyclist (All severities)	*0.63*	*0.3*
			Pedestrian and Bicyclist (Injury)	*0.63*	*0.3*

Base Condition: Unspecified

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

? Treatment results in an increase in rear-end crashes and right-angle injury crashes and a decrease in other crash types and severities. See Chapter 3.

Change plus clearance interval is the yellow-plus-all-red interval.

14.7.2.8. Install Red-Light Cameras at Intersections

Various Intelligent Transportation System (ITS) treatments are available for at-grade intersections. Treatments include signal coordination, red-light hold systems, queue detection systems, automated enforcement, and red-light cameras. At the time of this edition of the HSM, red-light cameras were the only treatment for which the crash effects were better understood. This section discusses the effects on crash frequency of installing red-light cameras.

Red-light cameras are positioned along the approaches to intersections with traffic signals to detect and record the occurrence of red-light violations. Installing red-light cameras and the associated enforcement program is generally accompanied by signage and public information programs.

Urban signalized intersections

The crash effects of installing red-light cameras at urban signalized intersections are shown in Exhibit 14-38. The base condition for the AMFs shown in Exhibit 14-38 (i.e., the condition in which the AMF = 1.00) is a signalized intersection without red-light cameras.

Exhibit 14-38: Potential Crash Effects of Installing Red-Light Cameras at Intersections*(23,30)*

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Install red light cameras	Urban (Unspecified)	Unspecified	Right-angle and left-turn opposite direction (All severities)*(23,30)*	**0.74**?+	**0.03**
			Right-angle and left-turn opposite direction (Injury) *(23)*	**0.84**?	**0.07**
			Rear-end (All severities)*(23,30)*	**1.18**?+	**0.03**
			Rear-end (Injury)*(23)*	**1.24**?	**0.1**
Base Condition: A signalized intersection without red-light cameras					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.
"vpd" = vehicles per day
\+ Combined AMF, see Part D Applications Guidance.
? Treatment results in a decrease in right-angle crashes and an increase in rear-end crashes. See Chapter 3.

It is possible that installing red-light cameras at intersections will result either in a positive spillover effect or in accident migration at nearby intersections or throughout a jurisdiction. A positive spillover effect is the reduction of crashes at adjacent intersections without red-light cameras due to drivers' sensitivity to the possibility of a red-light camera being present. Accident migration is a reduction in crash occurrence at the intersections with red-light cameras and an increase in crashes at adjacent intersections without red light cameras as travel patterns shift to avoid red-light camera locations. However, the existence and/or magnitude of the crash effects are not certain at this time.

14.8. CONCLUSION

The treatments discussed in this chapter focus on the crash effects of characteristics, design elements, traffic control elements, and operational elements related to intersections. The information presented is the AMFs known to a degree of statistical stability and reliability for inclusion in this edition of the HSM. Additional qualitative information regarding potential intersection treatments is contained in Appendix A.

The remaining chapters in *Part D* present treatments related to other site types such as roadway segments and interchanges. The material in this chapter can be used in conjunction with activities in *Chapter 6 Select Countermeasures*, and *Chapter 7 Economic Appraisal*. Some *Part D* AMFs are included in *Part C* for use in the predictive method. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7 of the *Part C Introduction and Applications Guidance*.

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APPENDIX A—TREATMENTS WITHOUT AMFS

A.1 INTRODUCTION

The appendix presents general information, trends in crashes and/or user-behavior as a result of the treatments, and a list of related treatments for which information is not currently available. Where AMFs are available, a more detailed discussion can be found within the chapter body. The absence of an AMF indicates that at the time this edition of the HSM was developed, completed research had not developed statistically reliable and/or stable AMFs that passed the screening test for inclusion in the HSM. Trends in crashes and user behavior that are either known or appear to be present are summarized in this appendix.

This appendix is organized into the following sections:

- Intersection Types (Section A.2)
- Access Management (Section A.3)
- Intersection Design Elements (Section A.4)
- Traffic Control and Operational Elements (Section A.5)
- Treatments with Unknown Crash Effects (Section A.6)

A.2 INTERSECTION TYPES

A.2.1 Intersection Type Elements with No AMFs - Trends in Crashes or User Behavior

A.2.1.1 Convert a Signalized Intersection to a Modern Roundabout

European experience suggests that single-lane modern roundabouts appear to increase safety for pedestrians and bicyclists.[13,37] ADA requirements to serve pedestrians with disabilities can be incorporated through roundabout planning and design.

There are some specific concerns related to visually impaired pedestrians and the accessibility of roundabout crossings. Concerns are related to the ability to detect audible cues that may not be as distinct as those detected at rectangular intersections; these concerns are similar to the challenges visually impaired pedestrians also encounter at channelized, continuous flowing right-turn lanes and unsignalized midblock crossings. At the time of this Edition of the HSM, specific safety information related to this topic was not available.

A.2.1.2 Convert a Stop- Control Intersection to a Modern Roundabout

See text above in section A.2.1.1.

A.3 ACCESS MANAGEMENT

A.3.1 Access Management Elements with No AMFs - Trends in Crashes or User Behavior

A.3.1.1 Close or Relocate Access Points in Intersection Functional Area

Access points are considered minor-street, side-street, and private driveways intersecting with a major roadway. The intersection functional area (Exhibit 14-1 and Exhibit 14-2) is defined as the area extending upstream and downstream from the physical intersection area and includes auxiliary lanes and their associated channelization. [1]

It is intuitive and generally accepted that reducing the number of access points within the functional areas of intersections reduces the potential for crashes.[5,34] Restricting access to commercial properties near intersections by closing private driveways on major roads or moving them to a minor road approach reduces conflicts between through and turning traffic. This reduction in conflicts may lead to reductions in rear-end crashes related to speed changes near the driveways, and angle crashes related to vehicles turning into and out of driveways.[5]

In addition to the reduction in conflicts, it is possible that locating driveways outside of the intersection functional area also provides more time and space for vehicles to turn or merge across lanes.[21] It is generally accepted that access points located within 250-ft upstream or downstream of an intersection are undesirable.[34]

A.3.1.2 Provide Corner Clearance

Corner clearances are the minimum distances required between intersections and driveways along arterials and collector streets. *"Driveways should not be situated within the functional boundary of at-grade intersections."*[1] Corner clearances vary greatly, from 16-ft to 350-ft, depending on the jurisdiction.

It is generally accepted that driveways that are located too close to intersections result in an increase in accidents, and as many as one half of accidents within the functional area of an intersection may be driveway-related.[17]

A.4 INTERSECTION DESIGN ELEMENTS

A.4.1 General Information

The material below provides an overview of considerations related to shoulders/sidewalks and roadside elements at intersections. These two categories of intersection design elements are integral parts of intersection design; however, crash effects are not known to a statistically reliable and/or stable level to include as AMFs, or to identify trends within this edition of the HSM.

Shoulders and Sidewalks

Shoulders are intended to perform several functions. Some of the main functions are: to provide a recovery area for out-of-control vehicles, to provide an emergency stopping area, and to improve the structural integrity of the pavement surface.[23]

The main purposes of paving shoulders are: to protect the physical road structure from water damage, to protect the shoulder from erosion by stray vehicles, and to enhance the controllability of stray vehicles.

Motorized Vehicle Perspective and Considerations

Some concerns when increasing shoulder width include:

- Wider shoulders on the approach to an intersection may result in higher operating speeds through the intersection which, in turn, may impact accident severity;
- Steeper side or backslopes may result from wider roadway width and limited right-of-way; and,
- Drivers may choose to use the wider shoulder as a turn lane.

Geometric design standards for shoulders are generally based on the intersection setting, amount of traffic, and right-of-way constraints.[23]

Shoulders at mid-block or along roadway segments are discussed in *Chapter 13*.

Roadside Elements

The roadside is defined as the "area between the outside shoulder edge and the right-of-way limits. The area between roadways of a divided highway may also be considered roadside".[4] The AASHTO Roadside Design Guide is an invaluable resource for roadside design including clear zones, geometry, features and barriers.[4]

The following sections discuss the general characteristics and considerations related to:

- Roadside geometry, and
- Roadside features.

Roadside Geometry

Roadside geometry refers to the physical layout of the roadside, such as curbs, foreslopes, backslopes, and transverse slopes.

AASHTO's Policy on Geometric Design states that a "a curb, by definition, incorporates some raised or vertical element."[1] Curbs are used primarily on low-speed urban highways, generally with a design speed of 45 mph or less.[1]

Designing a roadside environment to be clear of fixed objects with stable flattened slopes is intended to increase the opportunity for errant vehicles to regain the roadway safely, or to come to a stop on the roadside. This type of roadside environment, called a "forgiving roadside", is also designed to reduce the chance of serious consequences if a vehicle leaves the roadway. The concept of a "forgiving roadside" is explained in AASHTO's Roadside Design Guide.[4]

Chapter 13 includes information on clear zones, forgiving roadsides, and roadside geometry for roadway segments.

Roadside Elements - Roadside Features

Roadside features include signs, signals, luminaire supports, utility poles, trees, driver aid call boxes, railroad crossing warning devices, fire hydrants, mailboxes, bus shelters, and other similar roadside features.

The AASHTO Roadside Design Guide contains information about the placement of roadside features, criteria for breakaway supports, base designs, etc.[4] It is generally accepted that the best treatment for all roadside objects is to remove them from the clear zone.[35] Since removal is not always possible, the objects may be relocated farther from the traffic flow, shielded with roadside barriers, or replaced with breakaway devices.[35]

Roadside features on roadway segments are discussed in *Chapter 13*.

A.4.2 Intersection Design Elements with No AMFs - Trends in Crashes and/or User Behavior

A.4.2.1 Provide bicycle lanes or wide curb lanes at intersections

Bicycle lane is defined as a part of the roadway that is designated for bicycle traffic and separated by pavement markings from motor vehicles in adjacent lanes. Most often, bicycle lanes are installed near the right edge or curb of the road although they are sometimes placed to the left of right-turn lanes or on-street parking. [3] An alternative to providing a dedicated bicycle lane is to provide a wide curb lane. A wide curb lane is defined as a shared-use curb lane that is wider than a standard lane and can accommodate both vehicles and bicyclists.

Exhibit 14-39 below summarizes the crash effects and other observations known, at this time, related to bicycle lanes and wide curb lanes.

Exhibit 14-39: Summary of Bicycle Lanes and Wide Curb Lanes Crash Effects

Application	Crash Effect	Other Comments
Bicycle Lanes at Signalized Intersections	Appears to have no crash effect on bicycle-motor vehicle crashes or overall crashes. [29]	None
Bicycle Lanes at Minor-Road Stop Controlled Intersections	May be an increase in bicycle-motor vehicle crashes. [29]	Magnitude of increase is uncertain.
Wide curb lane greater than 12-ft (3.67 m)	Appears to improve the interaction between bicycles and motor vehicles in the shared lane.[33]	There is likely a lane width beyond which safety may decrease due to misunderstanding of shared space. [33]
Bicycle Lane verses Wide Curb Lane	No trends indicating which may be better than the other in terms of safety.	Bicyclists appear to ride further from the curb in bike lanes that are 5.2-ft wide or greater compared to wide curb lanes under the same traffic volume. [28]
		Bicyclist compliance at traffic signals does not appear to differ between bicycle lanes and wide lanes. [33]
		More bicyclists may comply at stop signs with bike lanes compared to wide curb lanes. [33]
		At wide curb lane locations bicyclists may perform more pedestrian style left- and right-turns (i.e. dismounting and use crosswalk) compared to bike lanes. [33] At this time, it is not clear which turning maneuver (as a car or a pedestrian) is safer.

A.4.2.2 Narrow Roadway at Pedestrian Crossing

Narrowing the roadway width using curb extensions, sometimes called chokers, curb bulbs, neckdowns, or nubs, extends the curb line or sidewalk out into the parking lane, and thus reduces the street width for pedestrians crossing the road. Curb extensions can also be used to mark the start and end of on-street parking lanes.

Reducing the street width at intersections appears to reduce vehicle speeds, improve visibility between pedestrians and oncoming motorists, and reduce the crossing distance for pedestrians.[24]

A.4.2.3 Install Raised Pedestrian Crosswalk

Common locations of crosswalks are at intersections on public streets and highways where there is a sidewalk on at least one side of the road. Marked crosswalks are typically installed at signalized intersections, school zones, and stop-controlled intersections.[14] The specific application of raised pedestrian crosswalks most often occurs on local urban two-lane streets in residential or commercial areas. They may be applied at intersections or midblock.

Raised pedestrian crosswalks are often considered as a traffic calming treatment to reduce vehicle speeds at locations where vehicle and pedestrian movements' conflict with each other.

On urban and suburban two-lane roads, this treatment appears to reduce injury accidents.[13] It is reasonable to conclude that raised pedestrian crosswalks have an overall positive effect on crash frequency since they are designed to reduce vehicle operating speed.[13] However, the magnitude of the crash effect is not certain at this time. The manner in which the crosswalks were raised is not provided in the original study from which the above information was gathered.

A.4.2.4 Install Raised Bicycle Crossing

Installing a raised bicycle crossing can be considered a form of traffic calming as a means to slow vehicle speeds and create a defined physical separation of a bicycle crossing relative to the travel way provided for motor vehicles.

Installing raised bicycle crossings at signalized intersections appears to reduce bicycle-motor vehicle crashes.[29] However, the magnitude of the crash effect is not certain at this time.

A.4.2.5 Mark Crosswalks at Uncontrolled Locations, Intersection or Midblock

Common locations of crosswalks are at intersections on public streets and highways where there is a sidewalk on at least one side of the road. Marked crosswalks are typically installed at signalized intersections, school zones, and stop-controlled intersections.[14] This section discusses the crash effects of providing marked crosswalks at uncontrolled locations – the uncontrolled approaches of stop-controlled intersection or uncontrolled midblock locations.

Exhibit 14-40 summarizes the effects on crash frequency and other observations known related to marking crosswalks at uncontrolled locations.

Exhibit 14-40: Potential Crash Effects of Marked Crosswalks at Uncontrolled Locations, Intersections or Midblock

Application	Crash Effect	Other Comments
Two-lane roads and multilane roads with < 12,000 AADT	A marked crosswalk alone, compared to an unmarked crosswalk, appears to have no statistically significant effect on pedestrian crash rate (pedestrian crashes per million crossings). [45]	The magnitude of the crash effect is not certain at this time.
Approaches with a 35mph speed limit on recently resurfaced roads	No specific crash effects apparent or are known.	Marking pedestrian crosswalks appears to slightly reduce vehicle approach speeds. [10,31] Drivers at lower speeds are generally more likely to stop and yield to pedestrians than higher-speed motorists. [10]
Two- or three-lane roads with speed limits from 35 to 40mph and < 12,000 AADT	Marking pedestrian crosswalks appears to have no measurable negative crash effect on either pedestrians or motorists. [32]	Crosswalk usage appears to increase after markings are installed.[32] Pedestrians walking alone appear to stay within marked crosswalk lines. [32] Pedestrians walking in groups appear to take less notice of markings. [32] There is no evidence that pedestrians are less vigilant or more assertive in the crosswalk after markings are installed. [32]
Multilane roads with AADT > 12,000 veh/day	A marked crosswalk alone appears to result in a statistically significant increase in pedestrian crash rates compared to uncontrolled sites with unmarked crosswalks. [45]	None.

When deciding whether to mark or not mark crosswalks, the results summarized in Exhibit 14-40 indicate the need to consider the full range of elements related to pedestrian needs when crossing the roadway.[45]

A.4.2.6 Provide a Raised Median or Refuge Island at Marked and Unmarked Crosswalks

Exhibit 14-41 summarizes the crash effects known related to the crash effects of providing a raised median or refuge island at marked or unmarked crosswalks.

Exhibit 14-41: Potential Crash Effects of Providing a Raised Median or Refuge Island at Marked

Application	Crash Effect	Other Comments
Multilane roads marked or unmarked intersection and midblock locations	Treatment appears to reduce pedestrian crashes.[45]	None.
Urban or suburban multilane roads (4 to 8 lanes) with marked crosswalks and an AADT of 15,000 veh/day or greater	Pedestrian crash rate is lower with a raised median than without a raised median.[45]	The magnitude of the crash effect is not certain at this time.
Unsignalized four-leg intersections across streets that are two-lane with parking on both sides and use zebra crosswalk markings	No specific crash effect known.	Refuge islands appears to increase the percentage of pedestrians who cross in the crosswalk and the percentage of motorists who yield to pedestrians.[24]

A.5 TRAFFIC CONTROL AND OPERATIONAL ELEMENTS

A.5.1 Traffic Control and Operational Elements with No AMFs - Trends in Crashes or User Behavior

A.5.1.1 Place Transverse Markings on Roundabout Approaches

Transverse pavement markings are sometimes placed on the approach to roundabouts that are preceded by long stretches of highway.[18] One purpose of transverse markings is to capture the motorists attention of the need to slow down on approach to the intersection. In this sense, transverse markings can be considered a form of traffic calming. Transverse pavement markings are one potential calming measure; in this section, the crash effect of its application to roundabout approaches is discussed.

This treatment appears to reduce all speed-related injury crashes, during wet or dry conditions, daytime and nighttime.[18] However, the magnitude of the crash effect is not certain at this time.

A.5.1.2 Install Pedestrian Signal Heads at Signalized Intersections

Pedestrian signal heads are generally desirable at certain types of locations including school crossings, on wide streets, or places where the vehicular traffic signals are not visible to pedestrians.[14]

Providing pedestrian signal heads, with a concurrent or standard pedestrian signal timing pattern, at urban signalized intersections with marked crosswalks appears to have no effect on pedestrian crashes compared to traffic signals without pedestrian signal heads for those locations where vehicular traffic signals are visible to pedestrians. [43,44]

A.5.1.3 Modify Pedestrian Signal Heads

Pedestrian signal heads may be modified by adding a third pedestrian signal head with the message DON'T START, or by changing the signal displays to be steady or flashing during the pedestrian "don't walk" phase. Exhibit 14-42 summarizes the crash effects known regarding modifying pedestrian signal heads.

Exhibit 14-42: Potential Crash Effects of Modifying Pedestrian Signal Heads

Application	Specific Modification to Pedestrian Signal Heads	Crash Effect and/or Resulting User Behavior
Urban signalized intersections with moderate to high pedestrian volumes	Add a third pedestrian signal head – a steady yellow DON'T START to the standard WALK and flashing DON'T WALK signal heads.	Treatment appears to reduce pedestrian violations and conflicts.[43]
Signalized intersections	Use a steady or flashing DON'T WALK signal display during the clearance and pedestrian prohibition intervals.	No difference in pedestrian behavior. [43] Pedestrians may not readily understand the word messages.
Signalized intersections	Use a steady or a flashing WALK signal display during the pedestrian WALK phase.	No difference in pedestrian behavior. [4] Pedestrians may not readily understand the word messages.
Signalized intersections	Use of symbols on pedestrian signal heads, such as a walking person or upheld hand.	Shown to be more readily comprehended by pedestrians than word messages. [10]

A.5.1.4 Install Pedestrian Countdown Signals

Pedestrian countdown signals are a form of pedestrian signal heads that displays the number of seconds pedestrians have to complete the crossing of street; this information is provided in addition to displaying WALK and DON'T WALK information in the form of either word messages or symbols.

Installing pedestrian countdown signals appears to reduce pedestrian-motor vehicle conflicts at intersections.[12] There appears to be no effect on vehicle approach speeds during the pedestrian clearance interval, i.e., the flashing DON'T WALK, with the countdown signals.[12]

A.5.1.5 Install Automated Pedestrian Detectors

Automated pedestrian detection systems can sense the presence of people standing at the curb waiting to cross the street. The system activates the WALK signal without any action from the pedestrian. The detectors in some systems can be aimed to monitor slower walking pedestrians in the crossing, so clearance intervals can be extended until the pedestrians reach the curb. Infrared and microwave sensors appear to provide similar results. Fine tuning of the detection equipment at the location is required to achieve an appropriate detection level and zone.

Installing automated pedestrian detectors at signalized intersections appears to reduce pedestrian-vehicle conflicts as well as the percent of pedestrian crossings initiated during the DON'T WALK phase.[26]

A.5.1.6 Install Stop Lines and Other Crosswalk Enhancements

Installing pedestrian crossing ahead signs, a stop line, and yellow lights activated by pedestrians at marked intersection crosswalks appears to reduce the number of conflicts between motorists and pedestrians. This treatment also appears to increase the percentage of motorists that yield to pedestrians.[11]

At marked intersection crosswalks, other treatments such as installing additional roadway markings and signs, providing feedback to pedestrians regarding compliance, and police enforcement, appear to increase the percentage of motorists who yield to pedestrians.[11]

A.5.1.7 Provide Exclusive Pedestrian Signal Timing Pattern

An exclusive pedestrian signal timing pattern provides a signal phase in which pedestrians are permitted to cross while motorists on the intersection approaches are prohibited from entering or traveling through the intersection.

At urban signalized intersections with marked crosswalks and pedestrian volumes of at least 1,200 people per day, this treatment appears to reduce pedestrian crashes when compared to concurrent timing or traffic signals with no pedestrian signals.[43,44] However, the magnitude of the crash effect is not certain at this time.

A.5.1.8 Provide Leading Pedestrian Interval Signal Timing Pattern

A leading pedestrian interval (LPI) is a pre-timed allocation to allow pedestrians to begin crossing the street in advance of the next cycle of vehicle movements. For example, pedestrians crossing the western leg of an intersection are traditionally permitted to cross during the north-south vehicle green phase. Implementing an LPI would provide pedestrians crossing the western leg of the intersection a given amount of time to start crossing the western leg after the east-west vehicle movements and before the north-south vehicle movements. The LPI provides pedestrians an opportunity to begin a crossing without concern for turning vehicles (assuming right-on-red is permitted).

Providing a pre-timed three-second LPI at signalized intersections with pedestrian signal heads and a one-second all-red interval appears to reduce conflicts between pedestrians and turning vehicles.[40] In addition, a three-second LPI appears to reduce the incidence of pedestrians yielding the right-of-way to turning vehicles, making it easier for pedestrians to cross the street by allowing them to occupy the crosswalk before turning vehicles are permitted to enter the intersection.[40]

A.5.1.9 Provide Actuated Control

The choice between actuated or pre-timed operations is influenced by the practices and standards of the jurisdiction. Intersection-specific characteristics such as traffic flows and intersection design also influence the use of actuated or pre-timed phases.

For the same traffic flow conditions at an actuated signal and pre-timed signal, actuated control appears to reduce some types of crashes compared to pre-timed traffic signals.[7] However, the magnitude of the crash effect is not certain at this time.

A.5.1.10 Operate Signals in "Night-Flash" Mode

Night-flash operation or mode is the use of flashing signals during low-volume periods to minimize delay at a signalized intersection.

Research indicates that replacing night-flash with regular phasing operation may reduce nighttime and nighttime right-angle crashes[19]. However, the results are not sufficiently conclusive to determine an AMF for this edition of the HSM.

The crash effect of providing "night-flash" operations appears to be related to the number of approaches to the intersection.[8]

A.5.1.11 Provide Advance Static Warning Signs and Beacons

Traffic signs are typically classified into three categories: regulatory signs, warning signs, and guide signs. As defined in the Manual on Uniform Traffic Control Devices (MUTCD),[14] regulatory signs provide notice of traffic laws or regulations, warning signs give notice of a situation that might not be readily apparent, and guide signs show route designations, destinations, directions, distances, services, points of interest, and other geographical, recreational or cultural information. The MUTCD provides standards and guidance for signing within the right-of-way of all types of highways open to public travel. Many agencies supplement the MUTCD with their own guidelines and standards. This section discusses the crash effects of providing advance static warning signs with beacons.

Providing advance static warning signs with beacons prior to an intersection appears to reduce accidents.[9] This treatment may have a larger crash effect when drivers do not expect an intersection or have limited visibility to the intersection ahead.[5] However, the magnitude of the crash effect is not certain at this time.

A.5.1.12 Provide Advance Warning Flashers and Warning Beacons

An advance warning flasher (AWF) is a traffic control device that provides drivers with advance information on the status of a downstream traffic signal.

Advance warning flashers may be responsive, i.e., linked to the signal timing mechanism, or continuous. Continuous AWFs are also called warning beacons.

The crash effects of responsive AWFs appear to be related to entering traffic flows from minor and major road approaches.(38)

A.5.1.13 Provide Advance Overhead Guide Signs

The crash effect of advance overhead directional or guide signs appears to be positive (i.e. reduces crash occurrences). However, the magnitude of the crash effect is not certain at this time.(9)

A.5.1.14 Install Additional Pedestrian Signs

Additional pedestrian signs include YIELD TO PEDESTRIAN WHEN TURNING signs for motorists and PEDESTRIANS WATCH FOR TURNING VEHICLES signs for pedestrians.

In general, additional signs may reduce conflicts between pedestrians and vehicles. However, it is generally accepted that signage alone does not have a substantial effect on motorist or pedestrian behavior without education and enforcement. (25)

Exhibit 14-43 summarizes the known and/or apparent crash effects or changes in user behavior as the result of installing additional pedestrian signs.

Exhibit 14-43: Potential Crash Effects of Installing Additional Pedestrian Signs

Application	Specific Pedestrian Signs	Crash Effect and/or Resulting User Behavior
Intersections permitting pedestrians crossings	Install a red and white triangle YIELD TO PEDESTRIAN WHEN TURNING sign (36” x 36” x 36”)	Reduces conflicts between pedestrians and turning vehicles. (44)
Intersections permitting pedestrians crossings	Provide a black-on-yellow PEDESTRIANS WATCH FOR TURNING VEHICLES sign	Decreases conflicts between turning vehicles and pedestrians. (44)
Intersections with a history of pedestrian violations such as crossing against the signal	Install a sign explaining the operation of pedestrian signal	Appears to increase pedestrian compliance and reduce conflicts with turning vehicles. (44)
Signalized intersections permitting pedestrian crossings	Provide a three-section signal that displays the message WALK WITH CARE during the crossing interval to warn pedestrians about turning vehicles or potential red-light running vehicles	Reduces pedestrian signal violations and reduces conflicts with turning vehicles. (44)
Marked crosswalks at unsignalized locations	Provide an overhead CROSSWALK sign	Increases the percentage of drivers that stop for pedestrians. (25)
Narrow low-speed roadways, unsignalized intersections	Install overhead illuminated CROSSWALK sign with high-visibility ladder crosswalk markings	Increases the percentage of motorists who yield to pedestrians. (36)
		Increases the percentage of pedestrians who use the crosswalk. (36)
Marked crosswalks at unsignalized locations	Install pedestrian safety cones reading STATE LAW – YIELD TO PEDESTRIANS IN CROSSWALK IN YOUR HALF OF ROAD	Increases the percentage of drives that stop for pedestrians. (25)

A.5.1.15 Modify Pavement Color for Bicycle Crossings

Modifying the pavement color at locations where bicycle lanes cross through an intersection is intended to increase the bicycle lanes conspicuity to motorized vehicles turning through or across the bicycle lane that is passing through the intersection. Increasing the conspicuity of the bicycle lane is intended to translate to an increase awareness of the presence of bicyclists thereby reducing the number of motorized vehicle-bicycle crashes.

Modifying the pavement color of bicycle path crossing points at unsignalized intersections, e.g., blue pavement, increases bicyclist compliance with stop signs and crossing within the designated area.(28) In addition, there is a reduction in vehicle-cyclist conflicts.(27)

Modifying the pavement color of bicycle lanes at exit ramps, right-turn lanes, and entrance ramps has the following effects:

- Increases the proportion of motorists yielding to cyclists;
- Increases cyclist use of the designated area;
- Increases the incidence of motorists slowing or stopping on the approach to conflict areas;
- Decreases the incidence of cyclists slowing on the approach to conflict areas;
- Decreases motorist use of turn signals; and,
- Decreases hand signaling and head turning by cyclists. (27)

A.5.1.16 Place "slalom" Profiled Pavement Markings at Bicycle Lanes

Placing profiled pavement markings on the pavement between bicycle lanes and motor vehicles lanes is intended to increase the lateral distance between bicyclists and drivers on intersection approaches, and to increase the attentiveness of both types of road users.(27) Profiled pavement markings can be applied to create a "slalom" effect, first directing bicyclists closer to the vehicle lane and then diverting bicyclists away from the vehicle lanes close to the stop bar.

Placing "slalom" profiled pavement markings at four-leg and T-intersections appears to regulate motorist speed to that of the bicyclists.(27) These markings also result in more motorists staying behind the stop line at the intersection, and reduces the number of motorists who turn right in front of a bicyclist.(27)

A.5.1.17 Install Rumble Strips on Intersection Approaches

Transverse rumble strips (also called "in-lane" rumble strips or "rumble strips in the traveled way") are installed across the travel lane perpendicular to the direction of travel to warn drivers of an upcoming change in the roadway. They are designed so that each vehicle will encounter them. Transverse rumble strips have been used as part of traffic calming or speed management programs, in work zones, and in advance of toll plazas, intersections, railroad-highway grade crossings, bridges and tunnels. They are also considered a form of traffic calming that can be used with intent of capturing motorists' attention and slowing speeds sufficient enough to provide drivers additional time for decision making tasks.

There are currently no national guidelines for the application of transverse rumble strips. There are concerns that drivers will cross into opposing lanes of traffic in order to avoid transverse rumble strips. As in the case of other rumble strips, there are concerns about noise, motorcyclists, bicyclists, and maintenance.

On the approach to intersections of urban roads with unspecified traffic volumes, this treatment appears to reduce all accidents of all severities.[13] However, the magnitude of the crash effect is not certain at this time.

A.6 TREATMENTS WITH UNKNOWN CRASH EFFECTS

A.6.1 Treatments Related to Intersection Types

- Convert stop-control intersection to yield-control intersection (not a roundabout);
- Convert uncontrolled intersection to yield, minor road or all-way stop control;
- Remove unwarranted signals on two-way streets;
- Close one or more intersection legs;
- Convert two three-leg intersections to one four-leg intersection;
- Right-left or left-right staggering of two three-leg intersections; and
- Convert intersection approaches from urban two-way streets to a couplet or vice versa.

A.6.2 Treatments Related to Intersection Design Elements

Approach Roadway Elements

- Eliminate through vehicle path deflection
- Increase shoulder width
- Provide a sidewalk or shoulder at an intersection;
- Increase pedestrian storage at intersection via sidewalks, shoulders, and/or pedestrian refuges
- Modify sidewalk width or walkway width
- Provide separation between the walkway and the roadway (i.e. buffer zone)
- Change the type of walking surface provided for pedestrians on sidewalks and/or crosswalks
- Modify sidewalk cross-slope, grade, curb ramp design
- Provide a left-turn bypass lane or combined bypass right-turn lane
- Modify lane width
- Provide positive offset for left-turn lanes
- Provide double or triple left-turn lanes

- Provide median left-turn acceleration lane
- Provide right-turn acceleration lanes
- Change length of left-turn and right-turn lanes
- Change right-turn curb radii
- Provide double right-turn lanes
- Provide positive offset for right turn lanes
- Provide shoulders or improve continuity at intersections
- Provide sidewalks or increase sidewalk width at intersections
- Provide a median, or change median shape or change length of median opening
- Provide a flush median at marked and unmarked crosswalks
- Modify pedestrian refuge island design (e.g. curb extensions, refuge island width)
- Presence of utility poles and vegetation on medians
- Provide grade separation for cyclists
- Improve continuity of bike lanes

Roadside Elements

- Increase intersection sight triangle distance
- Flatten sideslopes
- Modify backslopes
- Modify transverse slopes
- Increase clear roadside recovery distance
- Provide a curb
- Change curb offset from the traveled way
- Change curb type
- Change curb material
- Increase the distance to the utility poles and decrease utility pole density
- Increase the distance to/or remove roadside features
- Change the location of tress, poles, posts, news racks and other roadside features – crash effect from pedestrian and/or bicyclist perspective
- Increase sight-distance for left-turning vehicles

- Delineate roadside features
- Modify drainage structures or features
- Modify location and support types of signs, signals, and luminaries
- Install breakaway devices
- Modify location and type of driver-aid call boxes, mailboxes, newspaper boxes, fire hydrants

A.6.3 Treatments Related to Intersection Traffic Control and Operational Elements

- Provide signage for pedestrian and bicyclist information
- Provide illuminated pedestrian push buttons
- Provide late-release pedestrian signal timing pattern
- Install in-pavement lights at crosswalks
- Place advanced stop line or bike box pavement markings at bicycle lanes on intersection approaches
- Provide near-side pedestrian signal heads
- Adjust pedestrian signal timing for various pedestrian crossing speeds
- Install bicycle signal heads at signalized intersections
- Modify signalized intersection spacing
- Restrict turning movement at access points
- Install pedestrian half-signals at minor road stop controlled intersections
- Convert pre-timed phases to actuated phases
- Convert protected/permitted to permitted/protected left-turn operations
- Convert leading protected to lagging protected left-turn operations
- Provide protected or protected-permitted left-turn phasing with the addition of a left-turn lane
- Reduce left-turn conflicts with pedestrians
- Install all-red clearance interval
- Modify cycle length
- Modify phase durations
- Implement split phases
- Install more conspicuous pavement markings

- Extend edgelines and centerlines through median openings and unsignalized intersections
- Place lane assignment markings
- Place stop bars at previously unmarked intersections
- Increase stop bar width at marked intersections
- Install post-mounted delineators at intersections
- Install markers and/or markings on curbs at intersections
- Install raised median
- Install speed humps or speed tables on intersection approaches
- Close the intersection or one leg of the intersection (e.g. diagonal diverters, half closures, full closures, median barriers)
- Implement or improve signal coordination
- Implement or improve queue detection system
- Implement automated speed enforcement

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PART D— ACCIDENT MODIFICATION FACTORS

CHAPTER 15— INTERCHANGES

15.1. Introduction 15-1

15.2. Definition, Application, and Organization of AMFs..15-1

15.3. Definition of an Interchange and Ramp Terminal 15-2

15.4. Crash Effects of Interchange Design Elements 15-3

15.4.1. Background and Availability of AMFs 15-3

15.4.2. Interchange Design Element Treatments with AMFs 15-5

15.4.2.1. Convert Intersection to Grade-Separated Interchange 15-5

15.4.2.2. Design Interchange with Crossroad Above Freeway 15-5

15.4.2.3. Modify Speed Change Lane Design..15-6

15.4.2.4. Modify Two-Lane-Change Merge/Diverge Area to One-Lane-Change 15-9

15.5. Conclusions 15-9

15.6. References 15-10

EXHIBITS

Exhibit 15-1: Interchange Configurations(1) 15-3

Exhibit 15-2: Treatments Related to Interchange Design 15-4

Exhibit 15-3: Potential Crash Effects of Converting an At-Grade Intersection To a Grade-Separated Interchange(3) 15-5

Exhibit 15-4: Potential Crash Effects of Designing an Interchange with Crossroad Above Freeway(4) 15-6

Exhibit 15-5: Potential Crash Effects of Extending Deceleration Lanes(4) 15-7

Exhibit 15-6: Two-Lane-Change and One-Lane-Change Merge/Diverge Area 15-9

Exhibit 15-7: Potential Crash Effects of Modifying Two-Lane-Change Merge/Diverge Area to One-Lane-Change (3) 15-9

APPENDIX A

A.1 Introduction 15-12

A.2 Interchange Design Elements 15-12

A.2.1 General Information 15-12

A.2.2 Trends in Crashes or User Behavior for Treatments without AMFs 15-13

A.2.2.1 Redesign Interchange to Modify Interchange Configuration 15-13

A.2.2.2 Modify Interchange Spacing 15-13

A.2.2.3 Provide Right-Hand Exit and Entrance Ramps 15-13

A.2.2.4 Increase Horizontal Curve Radius of Ramp Roadway 15-13

A.2.2.5 Increase Lane Width of Ramp Roadway 15-13

A.2.2.6 Increase Length of Weaving Areas Between Adjacent Entrance and Exit Ramps 15-14

A.2.2.7 Redesign Interchange to Provide Collector-Distributor Roads 15-14

A.2.2.8 Provide Bicycle Facilities at Interchange Ramp Terminals 15-14

A.3 Treatments with Unknown Crash Effects 15-14

A.3.1 Treatments Related to Interchange Design 15-14

A.3.2 Treatments Related to Interchange Traffic Control and Operational Elements 15-15

A.4 Appendix References 15-16

CHAPTER 15 INTERCHANGES

15.1. INTRODUCTION

Chapter 15 presents Accident Modification Factors (AMFs) for design, traffic control, and operational elements at interchanges and interchange ramp terminals. Roadway, roadside and human factors elements related to pedestrian and bicycle crashes are also discussed. The information is used to identify effects on expected average crash frequency resulting from treatments applied at interchanges and interchange ramp terminals.

Chapter 15 presents design, traffic control and operational elements at interchanges and ramps with AMFs.

The *Part D Introduction and Applications Guidance* section provides more information about the processes used to determine the information presented in this chapter.

Chapter 15 is organized into the following sections:

- Definition, Application and Organization of AMFs (Section 15.2);
- Definition of an Interchange and Ramp Terminal (Section 15.3);
- Crash Effects of Interchange Design Elements (Section 15.4); and,
- Conclusion (Section 15.5).

Appendix A presents the crash effects of treatments for which AMFs are not currently known.

15.2. DEFINITION, APPLICATION, AND ORGANIZATION OF AMFS

Chapter 3 provides a thorough definition and explanation of AMFs.

AMFs quantify the change in expected average crash frequency (crash effect) at a site caused by implementing a particular treatment (also known as a countermeasure, intervention, action, or alternative), design modification, or change in operations. AMFs are used to estimate the potential change in expected crash frequency or crash severity plus or minus a standard error due to implementing a particular action. The application of AMFs involves evaluating the expected average crash frequency with or without a particular treatment, or estimating it with one treatment versus a different treatment.

Specifically, the AMFs presented in this chapter can be used in conjunction with activities in *Chapter 6 Select Countermeasures*, and *Chapter 7 Economic Appraisal*. Some *Part D* AMFs are included in *Part C* for use in the predictive method. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7 of the *Part C Introduction and Applications Guidance*. *Chapter 3 Fundamentals*, Section 3.5.3 Accident Modification Factors provides a comprehensive discussion of AMFs including: an introduction to AMFs, how to interpret and apply AMFs, and applying the standard error associated with AMFs.

In all Part D chapters, the AMFs of researched treatments are organized into one of the following categories:

The treatments are organized into 3 categories: treatments with AMFs; treatments with trend information; and, no trend or AMF information.

1. AMF is available;
2. Sufficient information is available to present a potential trend in crashes or user behavior, but not to provide an AMF;
3. Quantitative information is not available.

Treatments with AMFs (Category 1 above) are typically estimated for three accident severities: fatal, injury, and non-injury. In the HSM, fatal and injury are generally combined and noted as injury. Where distinct AMFs are available for fatal and injury severities, they are presented separately. Non-injury severity is also known as property-damage-only severity.

Treatments for which AMFs are not presented (Categories 2 and 3 above) indicate that quantitative information currently available did not pass the AMF screening test established for inclusion in the HSM. The absence of an AMF indicates additional research is needed to reach a level of statistical reliability and stability to meet the criteria set forth within the HSM. Treatments for which AMFs are not presented are discussed in Appendix A.

15.3. DEFINITION OF AN INTERCHANGE AND RAMP TERMINAL

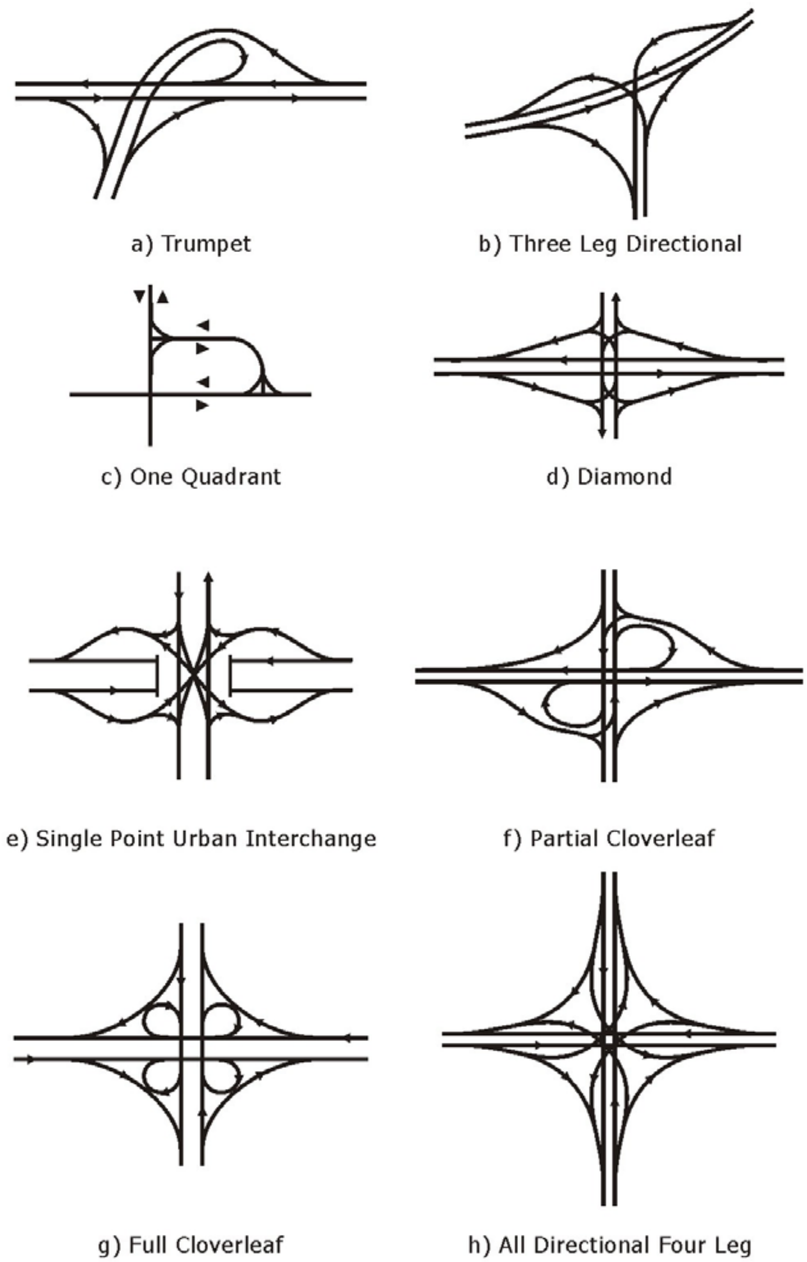
Section 15.3 provides a definition of facilities under consideration in this chapter.

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." Interchanges vary from single ramps connecting local streets to complex and comprehensive layouts involving two or more highways. (1)

An interchange ramp terminal is defined as an at-grade intersection where a freeway interchange ramp intersects with a non-freeway cross-street.

Exhibit 15-1 illustrates typical interchange configurations. (1)

Exhibit 15-1: Interchange Configurations[(1)]



15.4. CRASH EFFECTS OF INTERCHANGE DESIGN ELEMENTS

15.4.1. Background and Availability of AMFs

Exhibit 15-2 lists common treatments related to interchange design and the AMFs available in this edition of the HSM. Exhibit 15-2 also contains the section number where each AMF can be found.

Exhibit 15-2: Treatments Related to Interchange Design

HSM Section	Treatment	Trumpet	One Quadrant	Diamond	Single Point Urban	Partial Cloverleaf	Full Cloverleaf	Directional
15.4.2.1	Convert intersection to grade-separated interchange	✓	✓	✓	✓	✓	✓	✓
15.4.2.2	Design interchange with crossroad above freeway	✓	-	✓	-	✓	✓	-
15.4.2.3	Modify speed change lane design	✓	✓	✓	✓	✓	✓	✓
15.4.2.4	Modify two-lane-change merge/diverge area to one-lane-change	✓	✓	✓	✓	✓	✓	✓
Appendix A	Redesign interchange to modify interchange configuration	T	T	T	T	T	T	T
Appendix A	Modify interchange spacing	T	T	T	T	T	T	T
Appendix A	Modify ramp type or configuration	T	T	T	T	T	T	T
Appendix A	Provide right-hand exit and entrance ramps	T	T	T	T	T	T	T
Appendix A	Increase horizontal curve radius of ramp roadway	T	T	T	T	T	T	T
Appendix A	Increase lane width of ramp roadway	T	T	T	T	T	T	T
Appendix A	Increase length of weaving areas between adjacent entrance and exit ramps	T	T	T	T	T	T	T
Appendix A	Redesign interchange to provide collector-distributor roads	T	T	T	T	T	T	T
Appendix A	Provide bicycle facilities at interchange ramp terminals	T	T	T	T	T	T	T
Appendix A	Provide pedestrian facilities on ramp terminals	T	T	T	T	T	T	T

NOTE: ✓ = Indicates that an AMF is available for this treatment.

T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.

\- = Indicates that an AMF is not available and a crash trend is not known.

15.4.2. Interchange Design Element Treatments with AMFs

15.4.2.1. Convert Intersection to Grade-Separated Interchange

The potential crash effects of converting a three-leg or four-leg at-grade intersection to a grade-separated interchange is shown in Exhibit 15-3.[(3)] The base condition for the AMFs summarized in Exhibit 15-3 (i.e. the condition in which the AMF = 1.00) is maintaining the subject intersection at-grade.

Exhibit 15-3: Potential Crash Effects of Converting an At-Grade Intersection To a Grade-Separated Interchange[(3)]

Treatment	Setting (Intersection type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Convert at-grade intersection to grade-separated interchange	Setting unspecified (Four-leg intersection, traffic control unspecified)	Unspecified	All accidents in the area of the intersection (All severities)	**0.58**	**0.1**
			All accidents in the area of the intersection (Injury)	**0.43**	**0.05**
			All accidents in the area of the intersection (Non-injury)	**0.64**	**0.1**
	Setting unspecified (Three-leg intersection, traffic control unspecified)		All accidents in the area of the intersection (All severities)	*0.84*	*0.2*
	Setting unspecified (Three-leg or Four-leg, signalized intersection)		All accidents in the area of the intersection (All severities)	**0.73**	**0.08**
			All accidents in the area of the intersection (Injury)	**0.72**	**0.1**
Base Condition: At-grade intersection.					

NOTE: **Bold** text is used for the more statistically reliable AMFs. These AMFs have a standard error of 0.1 or less.
Italic text is used for less reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

15.4.2.2. Design Interchange with Crossroad Above Freeway

The potential crash effects of designing a diamond, trumpet or cloverleaf interchange with the crossroad above the freeway is shown in Exhibit 15-4.[(4)]

The base condition of the AMFs summarized in Exhibit 15-4 (i.e. the condition in which the AMF = 1.00) consists of designing a diamond, trumpet, or cloverleaf interchange with the crossroad below the freeway.

Exhibit 15-4: Potential Crash Effects of Designing an Interchange with Crossroad Above Freeway[4]

Treatment	Setting (Interchange type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Design diamond, trumpet or cloverleaf interchange with crossroad above freeway	Unspecified (Unspecified)	Unspecified	All accidents in the area of the interchange (All severities)	**0.96***	**0.1**
Base Condition: Design diamond, trumpet, or cloverleaf interchange with crossroad below freeway.					

NOTE: **Bold** text is used for the more statistically reliable AMFs. These AMFs have a standard error of 0.1 or less.

* Observed variability suggests that this treatment could result in fewer crashes, more crashes, or the same frequency of crashes. See Part D Introduction and Applications Guidance.

15.4.2.3. Modify Speed Change Lane Design

A speed change lane typically connects two facilities with differing speed limits. Speed change lanes include acceleration and deceleration lanes at on-ramps and off-ramps respectively. Speed change lanes include several design elements, such as lane width, shoulder width, length, and taper design.

AMF functions for acceleration lane length are incorporated in the FHWA Interchange Safety Analysis Tool (ISAT) software tool as follows: [2,6]

For total accidents (all severity levels combined):

$$AMF = 1.296 \times e^{(-2.59 \times L_{accel})} \qquad (15\text{-}1)$$

For fatal-and-injury accidents:

$$AMF = 1.576 \times e^{(-4.55 \times L_{accel})} \qquad (15\text{-}2)$$

Where,

L_{accel} = length of acceleration lane (mi)

L_{accel} is measured from the nose of the gore area to the end of the lane drop taper. The base condition for the AMFs in Equations 15-1 and 15-2 is an acceleration lane length of 0.1 mi (528 ft). The variability of these AMFs is unknown.

If an acceleration lane with an existing length other than 0.1 mi (528 ft) is lengthened, an AMF for that change in length can be computed as a ratio of two values computed with Equations 15-1 and 15-2. For example, if an acceleration lane with a length of 0.12 mi (634 ft) were lengthened to 0.20 mi (1,056 ft), the applicable AMF for total accidents would be the ratio of the AMF determined with Equation 15-1 for the existing length of 0.20 mi (1,056 ft) to the AMF determined with Equation 15-1 for the proposed length of 0.12 mi (634 ft), this calculation is illustrated in Equation 15-3.

$$AMF = \frac{1.576 \times e^{(-4.55 \times 0.12)}}{1.576 \times e^{(-4.55 \times 0.20)}} = 0.69 \qquad (15\text{-}3)$$

The crash effects and standard error associated with increasing the length of a deceleration lane that is currently 690-ft or less in length by about 100-ft is shown in Exhibit 15-5.(4)

The base condition of the AMFs in Exhibit 15-5 (i.e. the condition in which the AMF = 1.00) is maintaining the existing deceleration lane length of less than 690-ft. The AMF in Exhibit 15-5 may be extrapolated in proportion to the change in lane length for increases in length of less than or more than 100-ft as long as the resulting deceleration lane length does not exceed 790-ft.

Exhibit 15-5: Potential Crash Effects of Extending Deceleration Lanes*(4)*

Treatment	Setting (Interchange type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Extend deceleration lane by approx. 100-ft	Unspecified (Unspecified)	Unspecified	All types (All severities)	**0.93***	**0.06**
Base Condition: Maintain existing acceleration/deceleration lane that is less than 690 ft in length.					

NOTE: **Bold** text is used for the more statistically reliable AMFs. These AMFs have a standard error of 0.1 or less.
* Observed variability suggests that this treatment could result in an increase, decrease or no change in crashes. See Part D Introduction and Applications Guidance.

No quantitative information about the crash effect of increasing the length of existing deceleration lanes that are already greater than 690-ft in length was found for this edition of the HSM.

The gray box below illustrates how to apply the information in Exhibit 15-5 to calculate the crash effects of extending speed change lanes.

Effectiveness of Extending Speed Change Lanes

Question:

An urban grade-separated interchange has an off-ramp with a 650 ft deceleration lane. The governing jurisdiction is considering lengthening the ramp by 100-feet as part of a roadway rehabilitation project. What is the likely change in expected average crash frequency?

Given Information:

- Existing 650-foot long deceleration lane
- Expected average crash frequency without treatments on the ramp (See Part C Predictive Method) = 15 crashes/year

Find:

- Crash frequency with the longer deceleration lane
- Change in crash frequency

Answer:

1) Identify the applicable AMFs

   $AMF_{deceleration}$ = 0.93 (Exhibit 15-5)

2) Calculate the 95^{th} percentile confidence interval estimation of crashes with the treatment

   Expected crashes with treatment: = [0.93 ± (2 x 0.06)] x (15 crashes/year) = 12.2 or 15.8 crashes/year

   The multiplication of the standard error by 2 yields a 95% probability that the true value is between 12.2 and 15.8 crashes/year. See Section 3.5.3 in *Chapter 3 Fundamentals* for a detailed explanation of standard error application.

   This range of values (12.2 to 15.8) contains the original 15.0 crashes/year suggesting a possible increase, decrease, or no change in crashes. An asterisk next to the AMF in Exhibit 15-5 indicates this possibility. See the *Part D Introduction and Applications Guidance* for additional information on the standard error and notation accompanying AMFs.

3) Calculate the difference between the number of crashes without the treatment and the number of crashes with the treatment.

   Change in expected average crash frequency:

   Low Estimate = 15.8 – 15.0 = 0.8 crashes/year increment

   High Estimate = 15.0 – 12.2 = 2.8 crashes/year reduction

4) **Discussion: This example illustrates that increasing the deceleration lane length by 100 ft in the vicinity of the subject interchange may potentially increase, decrease, or cause no change in expected average crash frequency.**

15.4.2.4. Modify Two-Lane-Change Merge/Diverge Area to One-Lane-Change

Merge/diverge areas are defined as those portions of the freeway at an interchange where vehicles entering and exiting must change lanes to continue traveling in their chosen direction. The terms "ramp-freeway junction" or "weaving sections" may be used to describe merge/diverge areas.(7) Exhibit 15-6 illustrates a one-lane-change and a two-lane-change merge/diverge area. The crash effects of modifying two-lane change merge/diverge area to a one-lane-change are shown in Exhibit 15-7.(3)

The base condition of the AMFs above (i.e. the condition in which the AMF = 1.00) consists of a merge/diverge area requiring two lane changes.

Exhibit 15-6: Two-Lane-Change and One-Lane-Change Merge/Diverge Area

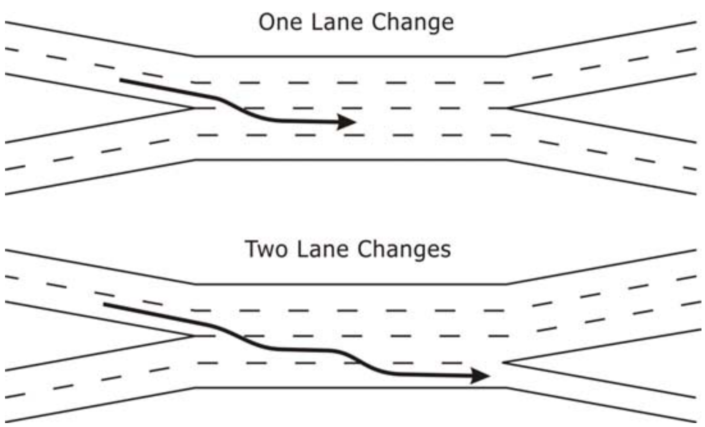


Exhibit 15-7: Potential Crash Effects of Modifying Two-Lane-Change Merge/Diverge Area to One-Lane-Change (3)

Treatment	Setting (Interchange type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Modify two-lane to one-lane merge/diverge area	Unspecified (Unspecified)	Unspecified	Accidents in the merging lane (All severities)	**0.68**	**0.04**
Base Condition: Merge/diverge area requiring two lane changes.					

NOTE: **Bold** text is used for the more statistically reliable AMFs. These AMFs have a standard error of 0.1 or less.

15.5. CONCLUSIONS

The treatments discussed in this chapter focus on the AMFs of design elements related to interchanges. The material presented consists of the AMFs known to a degree of statistical stability and reliability for inclusion in this edition of the HSM. Potential treatments for which quantitative information was not sufficient to determine an AMF or trend in crashes, in accordance with HSM criteria, are listed in Appendix A. The material in this chapter can be used in conjunction with activities in *Chapter 6 Select Countermeasures*, and *Chapter 7 Economic Appraisal*. Some *Part D* AMFs are included in *Part C* for use in the predictive method. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7 of the *Part C Introduction and Applications Guidance*.

Appendix A presents the treatments that have an identified trend or no known quantitative information.

15.6. REFERENCES

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APPENDIX A

A.1 INTRODUCTION

The material included in this appendix contains information regarding treatments for which AMFs are not available.

The appendix presents general information, trends in crashes and/or user-behavior as a result of the treatments, and a list of related treatments for which information is not currently available. Where AMFs are available, a more detailed discussion can be found within the chapter body. The absence of an AMF indicates that at the time this edition of the HSM was developed, completed research had not developed statistically reliable and/or stable AMFs that passed the screening test for inclusion in the HSM. Trends in crashes and user behavior that are either known or appear to be present are summarized in this appendix.

This appendix is organized into the following sections:

- Interchange Design Elements (Section A.2)
- **Error! Reference source not found.** (Section **Error! Reference source not found.**)
- Treatments with Unknown Crash Effects (Section A.3)

A.2 INTERCHANGE DESIGN ELEMENTS

A.2.1 General Information

The material provided below provides an overview of considerations related to bicyclists and pedestrians at interchanges and freeways.

Bicyclist Considerations

Some agencies permit bicyclist travel on freeway shoulders, toll bridges and tunnels in the absence of a suitable alternate route.(5) Agencies may require cyclists who use high-speed roadways to wear a helmet and to have a driver's license.(5) In addition, drain inlets can be modified to bicycle-friendly designs that reduce challenges for cyclists. At locations not intended for bicycles, agencies may choose to install prohibitory signs and alternate route information. (5)

Pedestrian Considerations

Most agencies do not permit pedestrian travel on freeways. Pedestrians using the cross-street at interchanges may, however, cross the ramp or the interchange ramp terminal. Grade-separated crossings may be an option. (12) Providing these crossings depends on the benefits, costs, and likelihood of pedestrian use. At locations not intended for pedestrian use, agencies may choose to install prohibitory signs and alternate route information. (5)

A.2.2 Trends in Crashes or User Behavior for Treatments without AMFs

A.2.2.1 Redesign Interchange to Modify Interchange Configuration

The designers of new freeway systems have an opportunity to choose the most appropriate configuration for each interchange. The configuration of an interchange may also be changed as part of a freeway reconstruction project. Examples of typical interchange configurations are shown in Exhibit 15-1. Guidance on the selection of interchange configurations can be found in the AASHTO Policy on Geometric Design of Highways and Streets(2) and the ITE Freeway and Interchange Geometric Design Handbook.(8) Both new construction and reconstruction of interchanges represent major highway agency investment decisions that must consider many factors including safety, traffic operations, air quality, noise, effects on existing development, cost, and a variety of other factors.

Further information on the differences between specific intersection types can be found in the work of Elvik and Vaa(4) and Elvik and Erke.(3) FHWA has developed an Interchange Safety Analysis Tool (ISAT) for assessing the crash effect of changing interchange configurations.(10) ISAT was assembled from existing models developed in previous research and should be considered as a preliminary tool until more comprehensive analysis tools can be developed.

A.2.2.2 Modify Interchange Spacing

Interchange spacing refers to the distance from one interchange influence area to the next.

Decreasing interchange spacing appears to increase accidents.(11) However, the magnitude of the crash effect is not certain at this time.

A.2.2.3 Provide Right-Hand Exit and Entrance Ramps

The configuration of ramps and the consistency of design along a corridor (e.g., all exit ramps are found in the right side) have key safety implications when considering driver expectation.(2) Drivers expect exit and entrance ramps on freeways to be on the right-hand side of the freeway.(6) Providing left-hand exit or entrance ramps contradicts driver expectations. In general, ramp design is directly related to the type of interchange.

A.2.2.4 Increase Horizontal Curve Radius of Ramp Roadway

Many ramps at freeway interchanges incorporate horizontal curves. Increasing a ramp roadway's curve radius from that which is currently less then 650-ft appears to decrease all accidents on the ramp roadway. However, the magnitude of the crash effect is not certain at this time.(3)

A.2.2.5 Increase Lane Width of Ramp Roadway

The roadway and lane widths for ramps at freeway interchanges are generally greater than for conventional roads and streets.

Increasing lane width on off-ramps appears to decrease accidents.(2) However, the magnitude of the crash effect is not certain at this time.

A.2.2.6 Increase Length of Weaving Areas Between Adjacent Entrance and Exit Ramps

A weaving area between adjacent entrance and exit ramps is essentially a combined acceleration and deceleration area, usually with a combined acceleration and deceleration lanes running from one ramp to the next. Such weaving areas are inherent in the design of full cloverleaf interchanges, but can occur in or between other interchange types. Short weaving areas between adjacent entrance and exit ramps have been found to be associated with increased accident frequencies. Research indicates that providing longer weaving areas will reduce accidents.[1] However, the available research is not sufficient to develop a quantitative AMF.

A.2.2.7 Redesign Interchange to Provide Collector-Distributor Roads

Accidents associated with weaving areas within an interchange or between adjacent interchanges can be reduced by redesigning the interchange(s) to provide collector-distributor roads. This design moves weaving from the mainline freeway to an auxiliary roadway, typically reducing both the volumes and the traffic speeds in the weaving area. The addition of collector-distributor roads has been shown to reduce accidents.[7,9] However, the available research is not sufficient to develop a quantitative AMF.

A.2.2.8 Provide Bicycle Facilities at Interchange Ramp Terminals

Continuity of bicyclist facilities can be provided at interchange ramp terminals. Bicyclists are considered vulnerable road users as they are more susceptible to injury when involved in a traffic crash than vehicle occupants. Vehicle occupants are usually protected by the vehicle.

Bicyclists must sometimes cross interchange ramps at uncontrolled locations. Encouraging bicyclists to cross interchange ramps at right angles appears to increase driver sight distance, and reduce the bicyclists' risk of a crash.[5]

A.3 TREATMENTS WITH UNKNOWN CRASH EFFECTS

A.3.1 Treatments Related to Interchange Design

Merge/Diverge Areas

- Modify merge/diverge design (e.g., parallel versus taper, left-hand versus right-hand);
- Modify roadside design or elements at merge/diverge areas;
- Modify horizontal and vertical alignment of the merge or diverge area; and,
- Modify gore area design.

Ramp Roadways

- Increase shoulder width of ramp roadway;
- Modify shoulder type of ramp roadway;
- Provide additional lanes on the ramp;

- Modify roadside design or elements on ramp roadways;
- Modify vertical alignment of the ramp roadway;
- Modify superelevation of ramp roadway;
- Provide two-way ramps;
- Provide directional ramps;
- Modify ramp design speed; and,
- Provide high occupancy vehicle lanes on ramp roadways.

Ramp Terminals

- Modify ramp terminal intersection type;
- Modify ramp terminal approach cross-section;
- Modify ramp terminal roadside elements;
- Modify ramp terminal alignment elements;
- Provide direct connection or access to commercial or private sites from ramp terminal; and,
- Provide physically channelized right-turn lanes.

Bicyclists and Pedestrian

- Provide pedestrian and/or cyclist traffic control devices at ramp terminals;
- Provide refuge islands; and,
- Develop policies related to pedestrian and bicyclist activity at interchanges.

A.3.2 Treatments Related to Interchange Traffic Control and Operational Elements

Traffic Control at Ramp Terminals

- Provide traffic signals at ramp terminal intersection; and,
- Provide stop-control or yield-control signs at ramp terminal intersections.

A.4 APPENDIX REFERENCES

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PART D— ACCIDENT MODIFICATION FACTORS

CHAPTER 16— SPECIAL FACILITIES AND GEOMETRIC SITUATIONS

16.1. Introduction 16-1
16.2. Definition, Application and Organization of AMFs 16-1
16.3. Crash Effects of Railroad-Highway Grade Crossings, Traffic Control, and Operational Elements 16-2
16.3.1. Background and Availability of AMFs 16-2
16.3.2. Railroad-Highway Grade Crossing, Traffic Control and Operational Treatments with AMFs 16-4
16.3.2.1. Install Flashing Lights and Sound Signals 16-4
16.3.2.2. Install Automatic Gates 16-5
16.4. Crash Effects of Work Zone Design Elements 16-6
16.4.1. Background and Availability of AMFs 16-6
16.4.2. Work Zone Design Treatments with AMFs 16-7
16.4.2.1. Modify Work Zone Duration and Length 16-7
16.5. Crash Effects of Two-Way Left-Turn Lane Elements 16-11
16.5.1. Background and Availability of AMFs 16-11
16.5.2. Two-Way Left-Turn Lane Treatments with AMFs 16-12
16.5.2.1. Provide Two-Way Left-Turn Lane 16-12
16.6. Crash Effects Of Passing And Climbing Lanes 16-13
16.6.1. Background and Availability of AMFs 16-13
16.6.2. Passing and Climbing Lane Treatments with AMFs 16-14
16.6.2.1. Provide a Passing Lane/Climbing Lane or a Short Four-Lane Section 16-14
16.7. Conclusion 16-15
16.8. References 16-16

EXHIBITS

Exhibit 16-1: Treatments Related to Railroad-Highway Grade Crossing Traffic Control and Operational Elements 16-4
Exhibit 16-2: Potential Crash Effects of Installing Flashing Lights and Sound Signals (2) 16-5
Exhibit 16-3: Potential Crash Effects of Installing Automatic Gates(2) 16-5
Exhibit 16-4: Treatments Related to Work Zone Design Elements 16-7
Exhibit 16-5: Expected Average Crash Frequency Effects of Increasing Work Zone Duration 16-8
Exhibit 16-6: Expected Average Crash Frequency Effects of Increasing Work Zone Length (miles) 16-9
Exhibit 16-7: Treatments Related to Two-Way Left-Turn Lanes 16-11
Exhibit 16-8: Potential Crash Effects of Providing a TWLTL on Rural Two-lane Roads with Driveways 16-13
Exhibit 16-9: Treatments Related to Passing and Climbing Lanes 16-14
Exhibit 16-10: Potential Crash Effects of Providing a Passing Lane/Climbing Lane or Short Four-Lane Section on Rural Two-Lane Roads (7) 16-14

APPENDIX A

A.1 Introduction 16-17
A.2 Railroad-Highway Grade Crossings, Traffic Control, and Operational Elements 16-17
A.2.1 Trends in Crashes or User Behavior for Treatments with no AMFs 16-17
A.2.1.1 Install Crossbucks 16-17
A.2.1.2 Install Vehicle-Activated Strobe Light and Supplemental Signs 16-17
A.2.1.3 Install Four-Quadrant Automatic Gates 16-18
A.2.1.4 Install Four-Quadrant Flashing Light Signals 16-18
A.2.1.5 Install Pre-Signals 16-18
A.2.1.6 Provide Constant Warning Time Devices 16-18
A.3 Work Zone Design Elements 16-18
A.3.1.1 Operate Work Zones in the Daytime or Nighttime 16-18
A.3.1.2 Use Roadway Closure with Two-Lane Two-Way Operation or Single-Lane Closure 16-19
A.3.1.3 Use Indiana Lane Merge System (ILMS) 16-19
A.4 Work Zone Traffic Control and Operational Elements 16-20
A.4.1 General Information 16-20
A.4.2 Trends in Crashes or User Behavior for Treatments with no AMFs 16-20
A.4.2.1 Install Changeable Speed Warning Signs 16-20
A.4.2.2 Install Temporary Speed Limit Signs and Speed Zones 16-21
A.4.2.3 Use Innovative Flagging Procedures 16-22
A.4.2.4 Install Changeable Message Signs 16-22
A.4.2.5 Install Radar Drones 16-22
A.4.2.6 Police Enforcement of Speeds 16-23
A.5 Two-Way Left-Turn Lane Elements 16-23
A.5.1.1 Provide Two-Way Left-Turn Lane 16-23
A.6 Treatments with Unknown Crash Effects 16-23
A.6.1 Railroad-Highway Grade Crossing, Traffic Control, and Operational Elements 16-23
A.6.2 Work Zone Design Elements 16-24
A.6.3 Work Zone Traffic Control and Operational Elements 16-24
A.6.4 Two-Way Left-Turn Elements 16-25
A.6.5 Passing and Climbing Lane Elements 16-26
A.7 Appendix References 16-27

CHAPTER 16 SPECIAL FACILITIES AND GEOMETRIC SITUATIONS

16.1. INTRODUCTION

This chapter presents AMFs for traffic control and operational element treatments at various facilities.

Chapter 16 presents Accident Modification Factors (AMFs) for design, traffic control, and operational elements at various special facilities and geometric situations. Special facilities include railroad-highway grade crossings, work zones, two-way left-turn lanes, and passing and climbing lanes. The information is used to identify effects on expected average crash frequency resulting from treatments applied at interchanges and interchange ramp terminals.

The *Part D Introduction and Applications Guidance* section provides more information about the processes used to determine the AMFs presented in this chapter.

Chapter 16 is organized into the following sections:

- Definition, Application and Organization of AMFs (Section 16.2)
- Crash Effects of Railroad-Highway Grade Crossings, Traffic Control, and Operational Elements (Section 16.3)
- Crash Effects of Work Zone Design Elements (Section 16.4)
- Crash Effects of Two-Way Left-Turn Lane Elements (Section 16.5)
- Crash Effects Of Passing And Climbing Lanes (Section 16.6)
- Conclusion (Section 16.7)

Appendix A presents the crash effects of treatments for which AMFs are not currently known.

16.2. DEFINITION, APPLICATION AND ORGANIZATION OF AMFS

Chapter 3 provides a thorough definition and explanation of AMFs.

AMFs quantify the change in expected average crash frequency (crash effect) at a site caused by implementing a particular treatment (also known as a countermeasure, intervention, action, or alternative), design modification, or change in operations. AMFs are used to estimate the potential change in expected crash frequency or crash severity plus or minus a standard error due to implementing a particular action. The application of AMFs involves evaluating the expected average crash frequency with or without a particular treatment, or estimating it with one treatment versus a different treatment.

Specifically, the AMFs presented in this chapter can be used in conjunction with activities in *Chapter 6 Select Countermeasures*, and *Chapter 7 Economic Appraisal*. Some *Part D* AMFs are included in *Part C* for use in the predictive method. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7 of the *Part C Introduction and Applications Guidance*. *Chapter 3 Fundamentals*, Section 3.5.3 Accident Modification Factors provides a comprehensive discussion of AMFs including: an introduction to AMFs, how to interpret and apply AMFs, and applying the standard error associated with AMFs.

The treatments are organized into 3 categories: treatments with AMFs; treatments with trend information; and, no trend or AMF information.

In all *Part D* chapters, the treatments are organized into one of the following categories:

1. AMF is available;
2. Sufficient information is available to present a potential trend in crashes or user behavior, but not to provide an AMF; and
3. Quantitative information is not available.

Unless otherwise specified, fatal and injury AMFs are generally combined and categorized as injury crashes.

Treatments with AMFs (Category 1 above) are typically estimated for three accident severities: fatal, injury, and non-injury. In *Part D*, fatal and injury are generally combined and noted as injury. Where distinct AMFs are available for fatal and injury severities, they are presented separately. Non-injury severity is also known as property-damage-only severity.

Treatments for which AMFs are not presented (Categories 2 and 3 above) indicate that quantitative information currently available did not meet the criteria for inclusion in the HSM. The absence of an AMF indicates additional research is needed to reach a level of statistical reliability and stability to meet the criteria set forth within the HSM. Treatments for which AMFs are not presented are discussed in Appendix A.

16.3. CRASH EFFECTS OF RAILROAD-HIGHWAY GRADE CROSSINGS, TRAFFIC CONTROL, AND OPERATIONAL ELEMENTS

16.3.1. Background and Availability of AMFs

There are two main types of railroad-highway crossings: at grade and grade-separated. A grade-separated railroad-highway crossing eliminates the conflict points between rail and road and removes the potential for crossing accidents.[13] The HSM focuses on railroad-highway at-grade crossings. Grade-separated crossings are not discussed.

In general, the discussion focuses on crossings with heavy freight rail. Where distinct information on light passenger rail and heavy freight rail is available, these modes are noted separately. Private crossings are not addressed separately.

Signs and Markings

Advance traffic control and warning devices for railroad-highway grade crossings typically consist of signs and pavement markings. Other advance control and warning devices include flashing light signals, vehicle activated signals, and transverse rumble strips. The advance traffic control and warning devices used vary with the crossing design.[1]

Signals and Gates

Section 16.3 provides AMFs for common treatments related to railroad highway grade crossing, traffic control, and operational elements.

Traffic control at railroad-highway grade crossings includes traffic signal preemption, traffic signal interconnection, pre-signals in the vicinity of railroad-highway grade crossings, and gates. The type of traffic control at a railroad-highway grade crossing depends on a number of factors including daily train volumes, vehicle volumes, and sight distances.

Traffic control devices used to warn road users that a train is approaching a railroad-highway grade can be passive or active:[4]

- Passive traffic control systems typically consist of signs and pavement markings that identify and direct motorists' and pedestrians' attention to a grade crossing. Stand-alone passive devices provide no information to motorists on whether a train is approaching. [9] These devices provide static messages; the message conveyed by the advanced warning signs and markings remain constant regardless of the presence or absence of a train.[3,6,10,11,14]

- Active traffic control systems are inactive until a train approaches. An approaching train activates some combination of automatic gates, bells or flashing lights. Active devices provide crossing users with an auditory or visual clue that a train is approaching the crossing in question. In some cases, for example when gates are lowered, the traffic control device physically separates crossing users from the railroad right-of-way.

Illumination

Artificial illumination is occasionally provided at railroad-highway grade crossings. No quantitative information about the crash effects of illumination at railroad-highway grade crossings was found for this edition of the HSM. *Chapter 14*, presents reference material for potential crash effects of illumination.

Exhibit 16-1 summarizes the treatments related to railroad-highway grade crossing, traffic control, and operational elements and the corresponding AMFs available.

Exhibit 16-1: Treatments Related to Railroad-Highway Grade Crossing Traffic Control and Operational Elements

HSM Section	Treatment	Rural Two-Lane road	Rural Multi-Lane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
16.3.2.1	Install flashing lights and sound signals	✓	✓	N/A	N/A	✓	✓
16.3.2.2	Install automatic gates	✓	✓	N/A	N/A	✓	✓
Appendix A	Install crossbucks	T	T	N/A	N/A	T	T
Appendix A	Install vehicle-activated strobe light and supplemental signs	T	T	N/A	N/A	T	T
Appendix A	Install four-quadrant automatic gates	T	T	N/A	N/A	T	T
Appendix A	Install four-quadrant flashing light signals	T	T	N/A	N/A	T	T
Appendix A	Install pre-signals	T	T	N/A	N/A	T	T
Appendix A	Provide constant warning time devices	T	T	N/A	N/A	T	T

NOTE: ✓= Indicates that an AMF is available for the treatment.

T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.

N/A = Indicates that the treatment is not applicable to the corresponding setting.

16.3.2. Railroad-Highway Grade Crossing, Traffic Control and Operational Treatments with AMFs

16.3.2.1. Install Flashing Lights and Sound Signals

Active traffic control systems are inactive until a train approaches. An approaching train activates some combination of automatic gates, bells, or flashing lights. Active devices provide crossing users with an auditory or visual clue that a train is approaching the crossing in question.

Rural two-lane road, rural multi-lane highways, urban, and suburban arterials

The crash effects of installing flashing lights and sound signals at railroad-highway grade crossings that previously had only signs are shown in Exhibit 16-2.

The base condition for this AMF (i.e., the condition in which the AMF = 1.00) is the absence of flashing lights and sound signals at railroad-highway crossings (passive control).

Exhibit 16-2: Potential Crash Effects of Installing Flashing Lights and Sound Signals (2)

Treatment	Setting (Crossing Type)	Traffic Volume	Accident Type (Severity)	AMF	Std. Error
Install flashing lights and sound signals	Unspecified (Unspecified)	Unspecified	Grade crossing (all severities)	**0.50**	**0.05**
Base Condition: Passive control at railroad-highway crossing.					

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

16.3.2.2. Install Automatic Gates

Automatic gates are active control devices that physically separate crossing users (cars, pedestrians, bicycles) from the railroad right-of-way.

Rural two-lane road, rural multi-lane highways, urban, and suburban arterials

The crash effects of installing automatic gates at railroad-highway grade crossings that previously had passive traffic control are shown in Exhibit 16-3.*(Error! Reference source not found.)*

The crash effects of installing automatic gates at railroad-highway grade crossings that previously had flashing lights and sound signals are shown in Exhibit 16-3.*(Error! Reference source not found.)*

The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) consists of crossings with passive traffic control or crossings with flashing lights and sound signals, in either case with an absence of automatic gates.

Exhibit 16-3: Potential Crash Effects of Installing Automatic Gates*(2)*

<table>
<tr><th>Treatment</th><th>Setting (Crossing type)</th><th>Traffic Volume</th><th>Accident type (Severity)</th><th>AMF</th><th>Std. Error</th></tr>
<tr><td>Install automatic gates at crossings that previously had passive traffic control</td><td rowspan="2">Unspecified (Unspecified)</td><td rowspan="2">Unspecified</td><td rowspan="2">Grade crossing (All severities)</td><td>0.33</td><td>0.09</td></tr>
<tr><td>Install automatic gates at crossings that previously had flashing lights and sound signals</td><td>0.55</td><td>0.09</td></tr>
<tr><td colspan="6">Base Condition: Crossings with passive traffic control or crossings with flashing lights and sound signals, in either case with an absence of automatic gates.</td></tr>
</table>

NOTE: **Bold** text is used for the most reliable AMFs. These AMFs have a standard error of 0.1 or less.

The gray box below presents an example of how to apply the preceding AMFs to assess the change in expected average crash frequency when installing automatic gates on a rural two-lane road grade rail crossing.

Effectiveness of Installing Automatic Gates

Question:

As part of a roadway improvement project, a rail crossing with flashing lights and sound signals is now being considered for the installation of automatic gates. What will be the likely reduction in the expected average crash frequency?

Given Information:

- Existing roadway = rural two-lane road
- Crossing type = at-grade crossing
- Existing traffic control = flashing lights and sound signals
- Expected average crash frequency with existing treatment = 0.25 crashes/year

Find:

- Expected average crash frequency with installation of automatic gates
- Change in expected average crash frequency

Answer:

1) Identify the applicable treatment AMF

   $AMF_{Treatment}$ = 0.55 (Exhibit 16-3)

2) Calculate the 95th Percentile Confidence Interval Estimation of Crashes with the Treatment

   Expected Crashes with Treatment: = (0.55 ± 2 x 0.09) x (0.25 crashes/year) = 0.09 or 0.18 crashes/year

   The multiplication of the standard error by 2 yields a 95% probability that the true value is between 0.09 and 0.18 crashes/year. See Section 3.5.3 in *Chapter 3 Fundamentals* for a detailed explanation.

3) Calculate the difference between the expected average crash frequency without the treatment and with the treatment.

   Change in Expected Average Crash Frequency:
   Low Estimate = 0.25 – 0.09 = 0.16 crashes/year reduction
   High Estimate = 0.25 – 0.18 = 0.07 crashes/year reduction

4) **Discussion: The implementation of automatic gates at the rail crossing may potentially produce a reduction of between 0.16 and 0.07 crashes/year.**

16.4. CRASH EFFECTS OF WORK ZONE DESIGN ELEMENTS

16.4.1. Background and Availability of AMFs

Work zones can result in disruptions in driving speed, trip routes, and driver expectancy. Accidents in work zones can cause additional delays and congestion.

Exhibit 16-4 summarizes treatments related to work zone design elements and the corresponding AMF availability.

Exhibit 16-4: Treatments Related to Work Zone Design Elements

HSM Section	Treatment	Rural Two-Lane road	Rural Multi-Lane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
16.4.2.1	Modify work zone duration and length	-	-	✓	-	-	-
Appendix A	Use crossover closure or single lane closure	-	T	T	T	-	-
Appendix A	Use Indiana Lane Merge System (ILMS)	-	-	T	-	-	-

NOTE: ✓ = Indicates that an AMF is available for the treatment.

T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.

\- = Indicates that an AMF is not available and a crash trend is not known.

16.4.2. Work Zone Design Treatments with AMFs

16.4.2.1. Modify Work Zone Duration and Length

Section 16.4 provides crash effects information for work zone design elements.

Freeways

Work zone design elements include duration in number of days, and length in miles. Equation 16-1 and Exhibit 16-5 present an AMF for the potential crash effects of modifying work zone duration. Equation 16-2 and Exhibit 16-6 present an AMF for the potential crash effects of modifying work zone length. These AMFs are based on research that considered work zone durations from 16 to 714 days, work zone lengths from 0.5 to 12.2 mi, and freeway AADTs from 4,000 to 237,000 veh/day.[8]

The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is a work zone duration of 16 days and/or work zone length of 0.51 miles. The standard errors of the AMFs below are unknown.

Expected average crash frequency effects of increasing work zone duration [8]

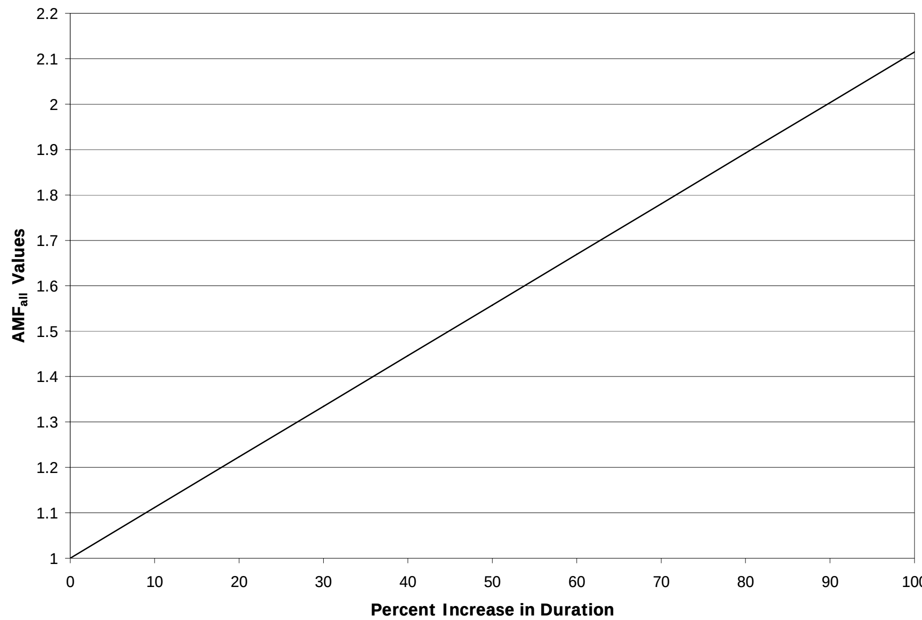
$$AMF_{all} = 1.0 + \frac{(\%\ increase\ in\ duration \times 1.11)}{100} \qquad (16\text{-}1)$$

Where,

AMF_{all} = accident modification factor for all crash types and all severities in the work zone

% increase in duration = the percentage change in duration (days) of the work zone

Exhibit 16-5: Expected Average Crash Frequency Effects of Increasing Work Zone Duration



Expected average crash frequency effects of increasing work zone length (miles)[8]

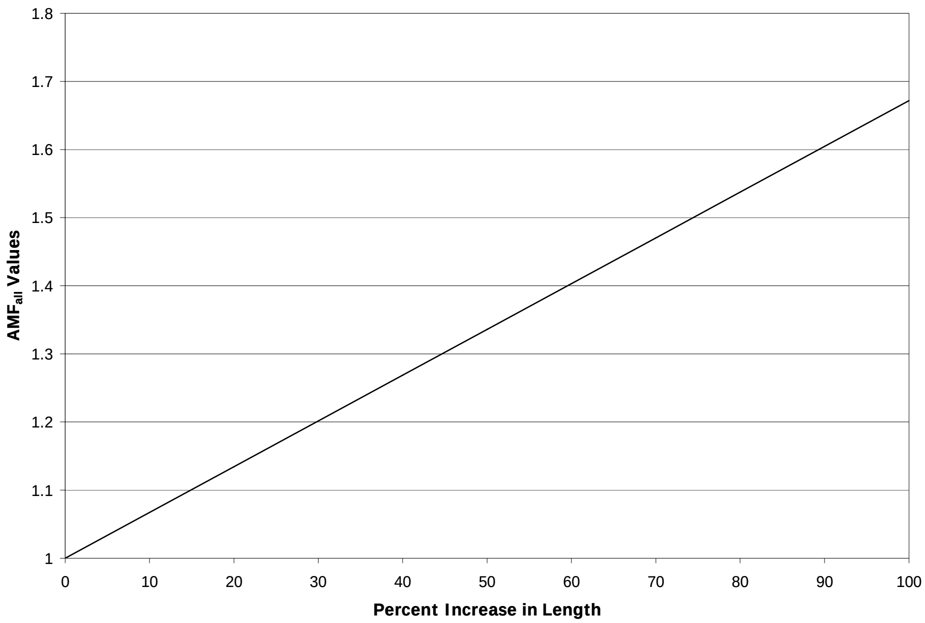
$$AMF_{all} = 1.0 + \frac{(\%\ increase\ in\ length \times 0.67)}{100} \qquad (16\text{-}2)$$

Where,

AMF_{all} = the accident modification factor for all crash types and all severities in the work zone

% increase in length = the percentage change in length (mi) of the work zone

Exhibit 16-6: Expected Average Crash Frequency Effects of Increasing Work Zone Length (miles)



The gray box below presents an example of how to apply Equation 16-2 and Exhibit 16-5; and Equation 16-3 and Exhibit 16-6 to assess the crash effects of modifying the work zone duration and length concurrently.

Effectiveness of Modifying the Work Zone Duration

Question:

A 5 mile stretch of highway is being rehabilitated. The design engineer has identified a construction period of 9 months with a full project length work zone. What will be the likely change in the expected average crash frequency?

Given Information:

- Base Condition for AMFs
  - Project Work Zone length = 0.51 miles
  - Project Work Zone duration = 16 days
- Proposed Work Zone Length = 1 miles
- Proposed Work Zone Duration = 32 days
- Expected Average Crash Frequency under the Base Scenario (See Part C Predictive Methods) = 6 crashes/year

Find:

- Expected Average Crash Frequency under Proposed Scenario
- Change in Expected Average Crash Frequency

Answer:

1) Calculate the Work Zone Length AMF_{length}

$$AMF_{length} = 1.0 + \frac{(\%\ increase\ in\ length \times 0.67)}{100} \quad \text{(Equation 16-2)}$$

$$AMF_{length} = 1.0 + \frac{(96 \times 0.67)}{100} =$$

$$AMF_{length} = 1.64$$

2) Calculate the Work Zone Duration $AMF_{duration}$

$$AMF_{duration} = 1.0 + \frac{(\%\ increase\ in\ duration \times 1.11)}{100} \quad \text{(Equation 16-1)}$$

$$AMF_{duration} = 1.0 + \frac{(100 \times 1.11)}{100} =$$

$$AMF_{duration} = 2.11$$

3) Calculate the Combined AMF_{total} Work Zone Condition

$AMF_{total} = AMF_{length} \times AMF_{duration} = 1.64 \times 2.11 = 3.46$

Both AMFs are multiplied to account for the combined effect of work zone length and duration.

4) Calculate the expected number of crashes under the proposed work zone scenario.

Expected Crashes under the Proposed Work Zone Scenario =
= 3.46 x (6 crashes/year) = 20.8 crashes/year

5) Calculate the difference between the expected average crash frequency under the base condition and with the treatment.

Change in Expected Average Crash Frequency

20.8 - 6.0 = 14.8 crashes/year increment

6) Discussion: The proposed work zone length and duration may potentially cause an increment of 14.8 crashes/year, when compared to a base scenario work zone length and duration.

16.5. CRASH EFFECTS OF TWO-WAY LEFT-TURN LANE ELEMENTS

Section 16.6 provides crash effects information for two-way left turn lane elements.

16.5.1. Background and Availability of AMFs

Two-way left turn-lanes (TWLTL) are intended to reduce potential conflicts with turning traffic and to provide a refuge from through vehicles for drivers waiting to turn left. Potential offsetting challenges may, however, arise:

- Where drivers increase their speed on the through lanes due to the left-turning traffic being removed;
- In urban areas where the TWLTL increases the width that pedestrians have to walk across the road;
- In urban areas where pedestrians may treat the TWLTL as a refuge area;
- Where traffic volumes back up into the TWLTL, blocking the TWLTL for the opposing direction;
- Where the driveway entrance is poorly designed and cannot readily accommodate the turning traffic which may then slow down or even stop as it crosses the through lanes;
- Where driveways and access points are not clearly marked and conspicuous, drivers may not be able to see where to turn resulting in slowing or quick stopping;
- Where drivers use the TWLTL for passing. A TWLTL that leads to the loss of a passing lane requires careful evaluation(5);
- Where seven-lane urban arterials (six through lanes/one TWLTL) are constructed, turning and crossing traffic have longer crossing times. Increased driver risk taking may occur; and,
- Where a curb lane is an HOV lane with low traffic volumes, encouraging drivers turning from a TWLTL to risk crossing the HOV lane even when their view is blocked, since they do not expect a vehicle to be in that lane.

Exhibit 16-7 summarizes treatments related to two-way left-turn lanes and the corresponding AMF and trend availability.

Exhibit 16-7: Treatments Related to Two-Way Left-Turn Lanes

HSM Section	Treatment	Rural Two-Lane road	Rural Multilane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
16.5.2.1	Provide Two-Way Left-Turn Lane	✓	-	-	-	T	T

NOTE: ✓= Indicates that an AMF is available for the treatment.

T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.

- = Indicates that an AMF is not available and a crash trend is not known.

16.5.2. Two-Way Left-Turn Lane Treatments with AMFs

16.5.2.1. Provide Two-Way Left-Turn Lane

A TWLTL, or continuous center left-turn lane, is a special lane in the center of the highway. The lane is reserved for vehicles making mid-block left-turns, i.e., turns into or out of access points between intersections. A TWLTL is a common treatment on urban and suburban arterials with many access points.

Rural two-lane roads

The potential crash effects of providing a TWLTL on rural two-lane roads where driveway density is known and consists of at least five driveways per mile is shown in Equation 16-3 and Exhibit 16-8, for driveway-related left-turn accidents.[7] The potential crash effect for non-driveway-related accidents or non-left-turn driveway accidents is not certain at this time.

The base condition for this AMF (i.e., condition in which AMF = 1.0) is the absence of TWLTL or a driveway density less than five driveways per mile. The standard error of this AMF is unknown.

$$AMF = 1.0 - \left(0.7 \times p_{dwy} \times p_{LT/D}\right) \tag{16-3}$$

$$p_{dwy} = \frac{\left(0.0047 \times DD\right) + \left(0.0024 \times DD^2\right)}{1.199 + \left(0.0047 \times DD\right) + \left(0.0024 \times DD^2\right)} \tag{16-3A}$$

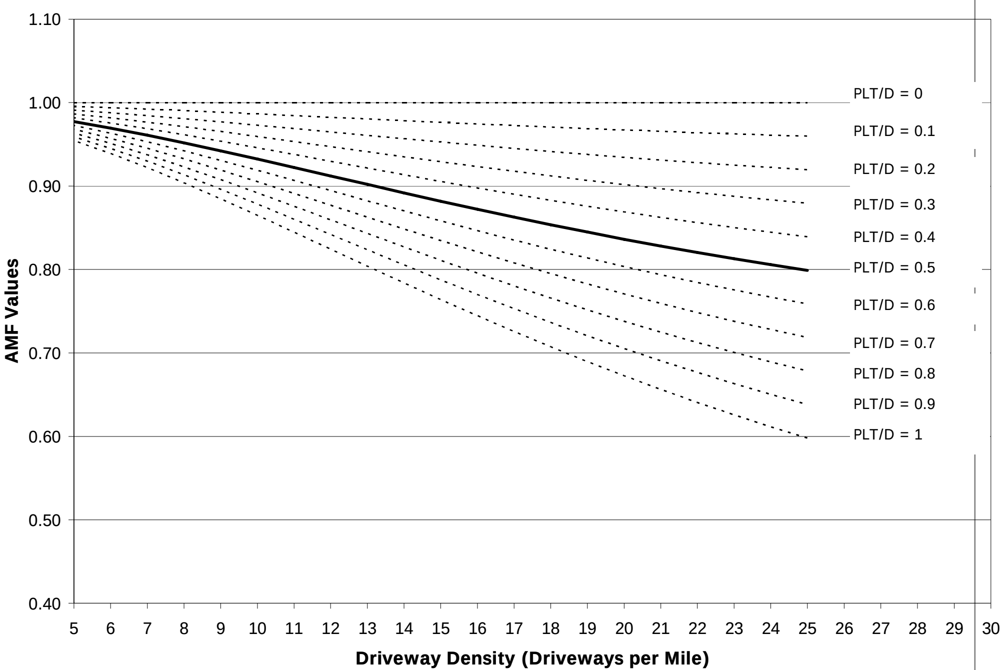
Where,

P_{dwy} = driveway-related accidents as a proportion of total accidents

DD = driveway density (driveways per mile)

$P_{LT/D}$ = left-turn accidents subject to correction by a TWLTL as a proportion of driveway-related accidents (can be estimated to be 0.5)

Exhibit 16-8: Potential Crash Effects of Providing a TWLTL on Rural Two-lane Roads with Driveways



16.6. CRASH EFFECTS OF PASSING AND CLIMBING LANES

16.6.1. Background and Availability of AMFs

A passing lane may be provided in one direction on two-lane two-way rural roads to increase overtaking opportunities and reduce delays. A climbing lane may be provided to overcome delays caused by slow-moving vehicles on steep upgrades. Other similar treatments include:

- Short four-lane sections. Short four-lane sections are created where passing lanes are provided in both travel directions.
- Turnouts. A turnout is a widened, unobstructed shoulder area that allows slow-moving vehicles to pull out of the through lane to give passing opportunities to following vehicles.(1)
- Shoulder use sections. Driving on shoulders is usually illegal; however, shoulders may be used by slow-moving vehicles in certain areas to allow other vehicles to pass. Some shoulders are signed where shoulder use is allowed.

Exhibit 16-9 summarizes treatments related to passing and climbing lanes and the level of information presented in the HSM.

Exhibit 16-9: Treatments Related to Passing and Climbing Lanes

HSM Section	Treatment	Rural Two-Lane road	Rural Multilane Highway	Freeway	Expressway	Urban Arterial	Suburban Arterial
16.6.2.1	Provide a Passing/Climbing Lane or a Short Four-Lane Section	✓	N/A	N/A	N/A	N/A	N/A

NOTE: ✓= Indicates that an AMF is available for the treatment.

N/A = Indicates that the treatment is not applicable to the corresponding setting.

16.6.2. Passing and Climbing Lane Treatments with AMFs

16.6.2.1. Provide a Passing Lane/Climbing Lane or a Short Four-Lane Section

Passing lanes allow vehicles to pass, and may have the potential to reduce crashes such as head-on, same-direction sideswipe, and opposite-direction sideswipe crashes at some locations. Passing-related head-on crashes are a relatively low percentage of all head-on crashes.[12] Passing lanes may affect traffic operations 3 to 8 mi downstream of the passing lane due to the segregation they permit between faster and slower vehicles.[7,12]

Climbing lanes allow vehicles to pass on grades, and may have the potential to reduce rear-end and same-direction sideswipe crashes at some locations that may result from speed differentials and conflicts between slow-moving and passing vehicles. Climbing lanes allow traffic platoons which have formed behind slower vehicles to dissipate without using an oncoming traffic lane to complete a passing maneuver.

Rural two-lane roads

The potential crash effects of providing a passing lane or climbing lane in one direction on a rural two-lane road is shown in Exhibit 16-10. [7] The potential crash effects of providing a short four-lane section on a rural two-lane road is also shown in Exhibit 16-10. [7]

The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is a two-lane rural road.

Exhibit 16-10: Potential Crash Effects of Providing a Passing Lane/Climbing Lane or Short Four-Lane Section on Rural Two-Lane Roads [7]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Provide passing lane or climbing lane	Rural (Two-lane)	Unspecified	All types (All severities)	0.75	N/A°
Provide short four-lane section				0.65	N/A°
Base Condition: Two-lane rural road.					

NOTE: ° Standard error of AMF is unknown.

16.7. CONCLUSION

The treatments discussed in this chapter focus on the potential crash effects of treatments that are applicable to roadway specific facilities and geometric situations. The material presented represents the AMFs known to a degree of statistical stability and reliability for inclusion in this edition of the HSM. Additional qualitative information regarding potential treatments is contained in Appendix A.

Appendix A presents the treatments which have an identified trend or no known information.

Other chapters in *Part D* present treatments related to specific site types such as roadway segments and intersections. The material in this chapter can be used in conjunction with activities in *Chapter 6 Select Countermeasures*, and *Chapter 7 Economic Appraisal*. Some *Part D* AMFs are included in *Part C* for use in the predictive method. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7 of the *Part C Introduction and Applications Guidance*.

16.8. REFERENCES

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APPENDIX A

A.1 INTRODUCTION

The appendix presents general information, trends in crashes and/or user-behavior as a result of the treatments, and a list of related treatments for which information is not currently available. Where AMFs are available, a more detailed discussion can be found within the chapter body. The absence of an AMF indicates that at the time this edition of the HSM was developed, completed research had not developed statistically reliable and/or stable AMFs that passed the screening test for inclusion in the HSM. Trends in crashes and user behavior that are either known or appear to be present are summarized in this appendix.

This appendix is organized into the following sections:

- Railroad-Highway Grade Crossings, Traffic Control, and Operational Elements (Section A.2)
- Work Zone Design Elements (Section A.3)
- Work Zone Traffic Control and Operational Elements (Section A.4)
- Two-Way Left-Turn Lane Elements (Section A.5)
- Treatments with Unknown Crash Effects (Section A.6)

A.2 RAILROAD-HIGHWAY GRADE CROSSINGS, TRAFFIC CONTROL, AND OPERATIONAL ELEMENTS

A.2.1 Trends in Crashes or User Behavior for Treatments with no AMFs

A.2.1.1 Install Crossbucks

Rural two-lane road, rural multi-lane highway, urban and suburban arterial

Installing crossbucks at railroad-highway grade crossings that previously had no signs appears to have the potential to reduce all grade crossing crashes.[2] However, the magnitude of the potential crash effects is not certain at this time.

A.2.1.2 Install Vehicle-Activated Strobe Light and Supplemental Signs

Rural two-lane road, rural multi-lane highway, urban and suburban arterial

Research has evaluated supplementary traffic control devices at passive railroad-highway grade crossings. The existing MUTCD W10-1 sign was supplemented with a "LOOK FOR TRAIN AT CROSSING" sign in conjunction with a strobe-light activated by approaching vehicles.[3]

Research results indicate that installing a vehicle-activated strobe light and supplemental sign, in addition to the MUTCD W10-1 sign at passive railroad-highway grade crossings, appears to have the potential to reduce average vehicle speeds near the crossing.[3]

A.2.1.3 Install Four-Quadrant Automatic Gates

Rural two-lane road, rural multi-lane highway, urban and suburban arterial

Installing four-quadrant automatic gates (one gate on each quadrant of the railroad/roadway intersection) appears to significantly reduce drivers violating crossing signals, and appears to have the potential to reduce the average number of vehicles crossing while the gate arms are being lowered.[13] No conclusive results about the potential crash effects of installing four-quadrant automatic gates were available for this edition of the HSM.

A.2.1.4 Install Four-Quadrant Flashing Light Signals

Rural two-lane road, rural multi-lane highway, urban and suburban arterial

Installing four-quadrant flashing light signals with overhead strobe lights appears to have no substantial affect on driver behavior compared to standard two-quadrant flashing light signals.[4] No conclusive results about the potential crash effects of installing four-quadrant flashing light signals were available for this HSM.

A.2.1.5 Install Pre-Signals

Rural two-lane road, rural multi-lane highway, urban and suburban arterial

Installing pre-signals to control traffic entering the railroad-highway grade crossing appears to have the potential to reduce risky driver behavior in the vicinity of the crossing. For instance, within 10 seconds of a train's arrival and while the flashing light signals are activated, both the number of crossings per signal activation and the number of vehicles crossing have been shown to decrease.[4] No conclusive results about the potential crash effects of installing pre-signals were available for this HSM.

A.2.1.6 Provide Constant Warning Time Devices

Rural two-lane road, rural multi-lane highway, urban and suburban arterial

Train predictors can be used to provide constant warning times to road users. Providing a constant warning time appears to have the potential to reduce the number of vehicles crossing the tracks between activation of the warning device and the train's arrival at the crossing.[18] Installing train predictors and the resulting constant warning times generally lead to fewer long warning times at crossings, and a potential reduction in incidences of risky driver behavior.[18] No conclusive results about the potential crash effects of providing constant warning time devices were available for this HSM.

A.3 WORK ZONE DESIGN ELEMENTS

A.3.1.1 Operate Work Zones in the Daytime or Nighttime

Rural two-lane roads; rural multilane highways; urban and suburban arterials; expressways

Time of day operations are considered a work zone design element. Compared to the no work zone condition, accidents appear to increase at work zones during nighttime more than during daytime.[10,21] Recent research has quantified the daytime and nighttime increases in accidents at work zones, in comparison to the pre-work-zone condition.[21] Work zone illumination appears to affect the safety of a work zone.[2] However, the magnitude of the crash effect is not certain at this time.

A.3.1.2 Use Roadway Closure with Two-Lane Two-Way Operation or Single-Lane Closure

Rural multilane highways, freeways, and expressways

There are two main types of lane closure design for work zones on freeways, rural multilane roadways, and urban and suburban arterials:

1. Roadway closure with a median crossover and two-lane two-way operations (TLTWO): All the lanes in one travel direction of a divided or undivided multilane highway are closed. Vehicles must cross over to use a lane that is normally dedicated to opposing traffic. The two main categories for median crossover design are flat diagonal designs and reverse curve designs.[9] Temporary centerlines, concrete median barriers, or other dividers may be used to separate the traffic. Concrete median barriers may be installed temporarily to separate traffic traveling in opposite directions in the TLTWO section. With this design, work crews may perform work on the closed roadway without having traffic near them. However, heavy traffic volumes, loaded trucks, nighttime, and bad weather can create safety concerns in the TLTWO.

2. Single (or partial) lane closure: One or more lanes in one travel direction are closed. The number of lanes closed depends on the total number of lanes on the roadway and the construction circumstances. A single lane closure does not directly affect traffic on the non-construction side of the roadway. Traffic on the construction side passes close to or adjacent to the work zone and work crew.

Work zones with crossover closures appear to have the potential to increase all accident types and severities compared to the non-work zone condition. [1,9,16] Roadway closures with a TLTWO section also appear to result in a potential increase in severe accidents and head-on crashes in the TLTWO section compared to the non-work zone condition.[9] Pavement surface and shoulder conditions may be important elements for crossover closures, particularly in the TLTWO section.[9]

Work zones with single lane closures appear to result in a potential increase in all accident types and severities compared to the non-work zone condition.[1,9,16] Single lane closures appear to have the potential to increase fixed object crashes compared to the non-work zone condition.[9]

There is some evidence that there may be a greater chance of a higher severity crash in a roadway closure with a TLTWO section than in a partial closure.[16] However, the magnitude of the potential crash effects is not certain at this time.

A.3.1.3 Use Indiana Lane Merge System (ILMS)

Freeways

The ILMS is an advanced dynamic traffic control system designed to encourage drivers to switch lanes well in advance of the work zone lane drop and entry taper.[20]

At many work zones, it is necessary to close one or more lanes. Vehicles must then merge into the lanes available. The transition area at the beginning of a work zone requires drivers to adapt their driving behavior to the new, and possibly unexpected, conditions ahead. Speed changes, lane positioning, and interacting with other drivers may be required.

The ILMS appears to have the potential to reduce the number of merging conflicts and to reduce vehicle delay on divided rural four-lane freeways with AADT of 42,000 veh/day or more.[20] No conclusive results about the potential crash effects of using the Indiana Lane Merge System (ILMS) were available for this HSM.

A.4 WORK ZONE TRAFFIC CONTROL AND OPERATIONAL ELEMENTS

A.4.1 General Information

Signs and Signals

The MUTCD classifies signs into three categories: regulatory, warning, and guide.[5] The MUTCD provides standards, guidance, and options for providing signs within the right-of-way for all highway types. Many agencies supplement the MUTCD information with their own guidelines and standards.

The type of signs and signals used in work zones generally depends on the road class and setting, the work zone layout, the work zone duration, the cost, whether the work zone is static or moving, and institutional constraints (e.g., whether trained flaggers are available). Combinations of signs and signals are commonly used, including speed signs and flashing arrows.

Delineation

Delineation includes all methods of defining the roadway operating area for drivers, and has long been considered a key element to guide drivers. Delineation is likely to have added impact in work zones where the conditions are unfamiliar or have changed substantially from the non-work zone condition. In work zones, temporary delineation methods may be used.

Methods of delineation include devices such as pavement markings (made from a variety of materials), raised pavement markers (RPMs), chevron signs, object markers, and post-mounted delineators (PMDs).[15] Delineation may be used alone to convey regulations, guidance, or warnings.[5] Delineation may also be used to supplement other traffic control devices such as signs and signals. The MUTCD provides guidelines for retroreflectivity, color, placement, material types, and other delineation issues.[5]

Pavement markings can be obscured by snow, debris, and water on the road surface. Visibility and retroreflectivity can be reduced over time by weather, vehicle tire wear, and location.[5]

Rumble Strips

Rumble strips warn drivers by creating vibration and noise when driven over. The objective of rumble strips is to reduce crashes caused by drowsy or inattentive drivers. In general, rumble strips are used in areas where the noise generated is unlikely to disturb adjacent residents; that is, in non-residential areas. Temporary rumble strips may be used in work zones as a traffic control device.

A.4.2 Trends in Crashes or User Behavior for Treatments with no AMFs

A.4.2.1 Install Changeable Speed Warning Signs

Changeable speed warning signs can provide individual or collective information to drivers. Individual changeable speed warning signs give individual drivers real-time feedback regarding each driver's speed. The signs can be an alternative to having law enforcement officers stationed at work zones. Collective changeable speed warning signs give information such as the percentage of road users exceeding the speed limit.[2]

Freeways

Installing individual changeable speed warning signs, that display the license plate and speed of a speeding vehicle in a freeway work zone, appears to have the potential to reduce injury and non-injury accidents.[22] However, the magnitude of the potential crash effects is not certain at this time.

Installing individual changeable speed warning signs that display personalized messages to high-speed drivers at work zones on Interstate highways appears to reduce vehicle speeds more than static MUTCD signs.[8] This treatment appears to be effective in work zone projects of long duration, from 7 days to 7 weeks. For work zones longer than 3,500 feet, a second changeable speed warning sign may reduce the tendency of drivers to speed up as they approach the end of a work zone.[8]

Installing individual changeable speed warning signs in advance of a single lane closure work zone on a freeway appears to have the potential to reduce the speed of traffic approaching the work zone.[14]

Rural two-lane roads

Installing individual changeable speed warning signs appears to have the potential to reduce average vehicle speed and the percentage of speeding vehicles at rural, short-term (typically a single day) work zones.[6]

A.4.2.2 Install Temporary Speed Limit Signs and Speed Zones

All road types

It is generally accepted that speed selection by drivers is a key factor in work zone crashes.[22]

Conventional practice for speed limits or speed zones in work zones follows the static signing procedures, using regulatory or advisory speed signs found in the MUTCD.[5] The procedure depends on the road type and setting, the work zone layout, the work zone duration, whether the work zone is static or moving, the cost of the speed control, and institutional constraints, such as the availability of a police presence or trained flaggers. Combinations of speed controls are commonly used.

Changing the posted speed limit generally has little effect on operating speeds.[17] Drivers select their speed using perceptual and "road message" cues. *Chapter 2* contains more information on driver speed choice.

It is generally accepted that installing temporary speed limit signs and speed zones in work zones, whether advisory or regulatory, has little to no effect on vehicle speeds.[22] It is also generally accepted that drivers adjust their vehicle speed and lane position according to the environment, the geometry of the roadway and work zone, the lateral clearance, and other factors, rather than on signing.[10] If speed limits are dramatically reduced, the limit may not match the perception of safe driving speed for the majority of drivers which may result in instability in the traffic flow through the speed zone.[23] Conclusive results about the potential crash effects of temporary speed limit signs and speed zones were not available for this HSM.

A.4.2.3 Use Innovative Flagging Procedures

All road types

Innovative flagging procedures include having a flagger with a speed sign paddle in one hand and motioning to traffic with the other hand, or a flagger motioning to traffic to slow down with one hand and pointing to a posted speed sign. Difficulties with flagging procedures include flagger fatigue and boredom, and ensuring that flaggers follow the procedures consistently.(14)

A flagger positioned in advance of a single lane closure on a freeway and holding a 45-mph sign paddle in one hand while motioning traffic to slow down with the other appears to have the potential to reduce average traffic speeds compared to having no flaggers present in advance of the work zone. (14) An alternative to this procedure is a flagger wearing bright coveralls and using a larger speed paddle sign.

On rural two-lane roads, rural freeways, urban freeways, and undivided urban arterials, a flagger motioning traffic to slow down with one hand and then pointing to the nearby posted speed sign appears to have the potential to reduce average traffic speeds more than standard MUTCD flagging procedures. (19) The average speed reduction appears to be greater on rural two-lane roads and urban arterials than on urban or rural freeways. Conclusive results about the potential crash effects of using innovative flagging procedures were not available for this HSM.

Using flaggers on both sides of the travel lanes of a freeway appears to result in greater speed reductions compared with using a flagger on one side only. (19)

The MUTCD provides guidance on the safety of workers in work zones.

A.4.2.4 Install Changeable Message Signs

All road types

Active speed control devices include changeable message signs, flaggers, and law enforcement. Passive measures (e.g., static signing) are generally thought to be less effective on traffic operations than active measures, but the difference in effectiveness is not certain at this time. (8)

Installing changeable message signs in advance of the work zone or within a work zone with the alternating messages "WORKERS AHEAD" and "SPEED LIMIT 45 MPH" appears to have the potential to reduce vehicle speeds, but only among vehicles close to the changeable message signs. (22) No quantitative information about the potential crash effects of installing changeable message signs with other speed limits in work zones is currently available.

A.4.2.5 Install Radar Drones

Radar drones emit a signal equivalent to that of a speed radar gun. These devices are used to communicate to drivers with radar detectors of possible hazards on the road ahead including dangerous curves, accidents, etc. The devices may be temporarily or permanently installed.

Rural two-lane roads

Installing radar drones at short-term (typically a single day) work zones on rural two-lane roads appears to have the potential to reduce vehicle speeds and the percentage of drivers who were speeding before the taper approaching the work zone and in the work zone.(6)

Rural multilane highways, urban and suburban arterials

Installing radar drones in short and long-term work zones on urban and rural interstate highways and on urban and rural roadways with AADTs ranging from 20,000 veh/day to 70,000 veh/day appears to have the potential to reduce mean speeds and the number of vehicles exceeding the speed limit by more than 10 mph.[7]

A.4.2.6 Police Enforcement of Speeds

All road types

Police enforcement methods include a police traffic controller, a stationary patrol car, a stationary patrol car with emergency lights or radar, and a circulating patrol car. [19]

Speed enforcement by police in work zones on rural two-lane roads, rural freeways, urban freeways, and undivided urban arterials appears to have the potential to reduce average vehicle speeds. [19] Police enforcement appears to be most effective over the length of highway receiving the treatment. [10]

A.5 TWO-WAY LEFT-TURN LANE ELEMENTS

A.5.1.1 Provide Two-Way Left-Turn Lane

Urban and suburban arterials

The potential crash effects of providing a TWLTL on urban and suburban arterials appears to be similar for rural two-lane roads.[11,12] However, the magnitude of the potential crash effects is not certain at this time. See Section 16.5.2.1 in the body of Chapter 16 Special Facilities for additional information.

A.6 TREATMENTS WITH UNKNOWN CRASH EFFECTS

A.6.1 Railroad-Highway Grade Crossing, Traffic Control, and Operational Elements

- Install stop or yield signs
- Install retroreflective advance warning signs
- Install transverse rumble strips on the approach to railroad-highway grade crossings
- Install advance warning flashers or beacons on the approach to railroad-highway grade crossings
- Place enhanced pavement markings on the approach to railroad-highway grade crossings
- Provide warning bells or flag persons on the approach to railroad-highway grade crossings
- Use train whistles
- Implement traffic signal preemption

A.6.2 Work Zone Design Elements

Lane Closure Design

- Modify crossover closure design
- Modify median crossover design for crossover closures
- Modify centerline treatment of TLTWO zone
- Modify single lane closure design

Lane Closure/Merge Design

- Use late merge control strategy;
- Use early merge control strategy;
- Position work zone on right-side or left-side of roadway;
- Modify merge design, including taper lengths and lane widths;
- Modify diverge design at the end of a work zone;
- Use the shoulder as a travel lane;
- Temporarily realign lanes; and,
- Modify location of the work zone relative to interchange ramps and roadway intersections.

A.6.3 Work Zone Traffic Control and Operational Elements

Signs and Signals

- Place signs in advance of work zone
- Use diverging lights or flashing arrows display
- Use temporary traffic signals, manual traffic direction, flaggers, or remote-control flags
- Improve visibility and clarity of signs
- Install active or passive warning signs or flashing arrows
- Use temporary diversions
- Install ITS applications

Delineation

- Install post-mounted delineators (PMDs)
- Place temporary centerline and/or edgeline markings
- Install raised pavement markers (RPMs)
- Install chevron signs on horizontal curves

- Install flashing beacons to supplement signage
- Mount reflectors on guardrails, curbs, and other barriers
- Place temporary transverse pavement markings

Rumble Strips

- Install continuous shoulder rumble strips
- Install continuous shoulder rumble strips and wider shoulders
- Install centerline rumble strips
- Install transverse rumble strips
- Install rumble strips with different dimensions and patterns
- Install edgeline rumble strips
- Install mid-lane rumble strips

Speed Limits and Speed Zones

- Use standard MUTCD flagging procedures
- Install real-time portable Variable Speed Limit systems
- Use radar activated horn system
- Reduce lane width
- Broadcast Citizens Band (CB) messages
- Provide automated speed enforcement

A.6.4 Two-Way Left-Turn Elements

- Number of through lanes on the road
- Width of the TWLTL
- How the TWLTL was incorporated (e.g., re-striping existing roadway width or widening the road)
- Volume of turning vehicles and opposing vehicles
- Capacity of storage for turning vehicles
- Driveway design
- Treatment at intersections
- Posted speed limit
- Markings
- Signage

- Land use (urban, rural, suburban)
- Presence of pedestrians
- Presence or prohibition of parallel street parking

A.6.5 Passing and Climbing Lane Elements

- Use three-lane alternate passing lane design
- Modify design elements, e.g., length, spacing, horizontal and vertical alignment, sight distance, tapers, merges, shoulders
- Modify posted speed limits and operating speed
- Install signage and pavement markings
- Modify density of intersections and/or access points along the auxiliary lane.
- Inclusion of passing and climbing lanes on the roadway as a whole (corridor approach)
- Provide a turnout
- Provide shoulder use sections

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PART D— ACCIDENT MODIFICATION FACTORS

CHAPTER 17—ROAD NETWORKS

17.1. Introduction 17-1
17.2. Definition, Application, and Organization of AMFs..17-1
17.3. Crash Effects of Network Planning and Design Approaches/Elements 17-2
17.3.1. Background and Availability of AMFs 17-2
17.4. Crash Effects of Network Traffic Control and Operational Elements 17-3
17.4.1. Background and Availability of AMFs 17-3
17.4.2. Network Traffic Control and Operations Treatments with AMFs............17-4
17.4.2.1. Implement Area-Wide Traffic Calming 17-4
17.5. Crash Effects of Elements of Road-Use Culture Network Considerations 17-5
17.5.1. Background and Availability of AMFs 17-5
17.5.2. Road Use Culture Network Consideration Treatments with AMFs...........17-6
17.5.2.1. Install Automated Speed Enforcement..17-6
17.5.2.2. Install Changeable Speed Warning Signs 17-8
17.6. Conclusion..17-9
17.7. References 17-10

EXHIBITS

Exhibit 17-1: Treatments Related to Network Planning and Design Approaches/Elements 17-3

Exhibit 17-2: Treatments Related to Network Traffic Control and Operational Elements 17-3

Exhibit 17-3: Potential Crash Effects of Applying Area-Wide or Corridor-Specific Traffic Calming to Urban Local Roads while Adjacent Collector Roads Remain Untreated (2,4,6) (injury excludes fatal crashes in this exhibit) 17-4

Exhibit 17-4: Road-Use Culture Network Considerations and Treatments 17-6

Exhibit 17-5: Potential Crash Effects of Automated Speed Enforcement (1,3,5,7,9,12) 17-7

Exhibit 17-6: Potential Crash Effects of Installing Changeable Speed Warning Signs for Individual Drivers(7) 17-9

Exhibit 17-7: Relationship between Crash Speed and the Probability of a Pedestrian Fatality(17) 17-13

APPENDIX A

A.1 Introduction 17-11
A.2 Network Planning and Design Approaches/Elements 17-11
A.2.1 General Information 17-11
A.2.2 Trends in Crashes or User Behavior for Treatments with no AMFs 17-11
A.2.2.1 Apply Elements of Self-Explaining Roadway Design 17-11
A.2.2.2 Apply Elements of Transportation Safety Planning in Transportation Network Design 17-13
A.3 Network Traffic Control and Operational Elements 17-14
A.3.1 Trends in Crashes or User Behavior for Treatments with no AMFs 17-14
A.3.1.1 Convert Two-Way Streets to One-Way Streets 17-14
A.3.1.2 Convert One-Way Streets to Two-Lane, Two-Way Streets 17-14
A.3.1.3 Modify the Level of Access Control 17-15
A.4 Elements of Road-Use Culture Network Considerations 17-15
A.4.1 Trends in Crashes or User Behavior for Treatments with no AMFs 17-17
A.4.1.1 Deploy Mobile Patrol Vehicles 17-17
A.4.1.2 Deploy Stationary Patrol Vehicles 17-17
A.4.1.3 Deploy Aerial Enforcement 17-17
A.4.1.4 Deploy Radar and Laser Speed Monitoring Equipment 17-17
A.4.1.5 Install Drone Radar 17-17
A.4.1.6 Modify Posted Speed Limit 17-17
A.4.1.7 Conduct Enforcement to Reduce Impaired Driving 17-18
A.4.1.8 Conduct Enforcement to Increase Seat Belt and Helmet Use 17-18
A.4.1.9 Implement Network-Wide Engineering Consistency 17-19
A.4.1.10 Conduct Public Education Campaigns 17-19
A.4.1.11 Implement Young Driver and Graduated Driver Licensing Programs 17-20
A.5 Treatments with Unknown Crash Effects 17-20
A.5.1 Network Traffic Control and Operational Elements 17-20
A.5.2 Road-Use Culture Network Considerations 17-20
A.6 Appendix References 17-22

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CHAPTER 17 ROAD NETWORKS

17.1. INTRODUCTION

Chapter 17 presents Accident Modification Factors (AMFs) applicable to planning, design, operations, education, and enforcement-related decisions that are applied holistically to a road network. From the federal level to the state and local levels, planning, engineering, and policy decisions affect the physical road network. This in turn has an impact on the mode, route, and trip choices that users make. As the pattern of trips on the network changes, the collective safety effects on the network will change. The information presented in this chapter is used to identify effects on expected average crash frequency resulting from treatments applied to road networks.

Chapter 17 presents AMFs applicable to planning, design, operations, education, and enforcement-related decisions that are applied holistically to a road network.

The *Part D Introduction and Applications Guidance* section provides more information about the processes used to determine the information presented in this chapter.

Chapter 17 is organized into the following sections:

- Definition, Application, and Organization of AMFs (Section 17.2);
- Crash Effects of Network Planning and Design Approaches/Elements (Section 17.3);
- Crash Effects of Network Traffic Control and Operational Elements (Section 17.4);
- Crash Effects of Road-Use Culture Network Considerations and Treatments (Section 17.5); and
- Conclusion (Section 17.6).

Appendix A presents the crash effects of treatments for which AMFs are not currently known.

17.2. DEFINITION, APPLICATION, AND ORGANIZATION OF AMFS

AMFs quantify the change in expected average crash frequency (crash effect) at a site caused by implementing a particular treatment (also known as a countermeasure, intervention, action, or alternative), design modification, or change in operations. AMFs are used to estimate the potential change in expected crash frequency or crash severity plus or minus a standard error due to implementing a particular action. The application of AMFs involves evaluating the expected average crash frequency with or without a particular treatment, or estimating it with one treatment versus a different treatment.

Chapter 3 Fundamentals, Section 3.5.3 Accident Modification Factors provides a comprehensive discussion of AMFs.

Specifically, the AMFs presented in this chapter can be used in conjunction with activities in *Chapter 6 Select Countermeasures*, and *Chapter 7 Economic Appraisal*. Some *Part D* AMFs are included in *Part C* for use in the predictive method. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7 of the *Part C Introduction and Applications Guidance*. *Chapter 3 Fundamentals*, Section 3.5.3 Accident Modification Factors provides a comprehensive discussion of AMFs including: an introduction to AMFs, how to interpret and apply AMFs, and applying the standard error associated with AMFs.

There are three categories of treatments: an AMF is available; a trend is available but not AMF; no trend and no AMF information is available.

In all *Part D* chapters, the treatments are organized into one of the following categories:

1. AMF is available;
2. Sufficient information is available to present a potential trend in crashes or user behavior, but not to provide an AMF; and
3. Quantitative information is not available.

Treatments with AMFs (Category 1 above) are typically estimated for three accident severities: fatal, injury, and non-injury. In *Part D*, fatal and injury are generally combined and noted as injury. Where distinct AMFs are available for fatal and injury severities, they are presented separately. Non-injury severity is also known as property-damage-only severity.

Treatments for which AMFs are not presented (Categories 2 and 3 above) indicate that quantitative information currently available did not meet the criteria for inclusion in the HSM. The absence of an AMF indicates additional research is needed to reach a level of statistical reliability and stability to meet the criteria set forth within the HSM. Treatments for which AMFs are not presented are discussed in Appendix A.

17.3. CRASH EFFECTS OF NETWORK PLANNING AND DESIGN APPROACHES/ELEMENTS

17.3.1. Background and Availability of AMFs

This section presents general background information about the crash effects of network planning and design approaches/elements. Planning decisions include a range of issues that may affect the expected average crash frequency on the road network. Examples of planning decisions that affect network safety include:

- The travel frequencies and travel distances in the course of people's daily activities;
- The travel mode used (train, subway, bus, car, bicycle or walking);
- The period of greatest travel demand (throughout the day, week, and year);
- The facility type used (whether people travel on a freeway or an arterial road);
- The number of high-traffic volume or low-traffic volume intersections that road-users must pass through;
- The distance between access points;
- The need for children to cross roads on their way to school; and,
- The operating speeds implied by the local residential road network (e.g., straight wide roadways, narrow curved roads, or cul-de-sacs).

Similar to planning decisions, design and operational decisions vary in their impact on the network. Decisions to widen a shoulder or to provide a turn lane may have little effect on travel patterns over the network as a whole. Other design and operational decisions may affect a wider part of the network. For example, one-way street systems appear to affect a relatively limited area, but may have crash implications for other streets in the road network due to changes in traffic patterns.

Network design elements include treatments and broader design concepts intended to achieve uniformity and similarities across a roadway network. Self-explaining roads and transportation safety planning (TSP) are two examples of design principles that are applied across a network to achieve geometric and operational characteristics aimed at reducing crashes. Self-explaining roads are designed to make the function and role of a road immediately clear, recognizable, and self-enforcing. Design stimulates drivers to adapt and reduce speed. Transportation safety planning involves explicitly, proactively, and comprehensively implementing measures known to reduce expected average crash frequency.

Exhibit 17-1 summarizes the treatments related to network planning and design approaches and elements. There are currently no AMFs for these treatments. Appendix A presents general information and potential trends in crashes and user behavior for these treatments.

There are no treatments related to network planning and design with AMFs.

Exhibit 17-1: Treatments Related to Network Planning and Design Approaches/Elements

HSM Section	Treatment	Urban	Suburban	Rural
Appendix A	Apply elements of self-explaining roadway design	T	T	T
Appendix A	Apply elements of transportation safety planning in transportation network design	T	T	T

NOTE: T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.

17.4. CRASH EFFECTS OF NETWORK TRAFFIC CONTROL AND OPERATIONAL ELEMENTS

17.4.1. Background and Availability of AMFs

The material presented in this section focuses on treatments related to traffic control and operational elements that are applied across a network or sub-area. Network traffic control and operational elements include treatments such as area-wide traffic calming, creating a network of one-way couplets, or implementing a specific level of access management across a set of facility types within a network.

Exhibit 17-2 summarizes treatments related to network traffic control and operational elements and the corresponding AMFs available.

Exhibit 17-2: Treatments Related to Network Traffic Control and Operational Elements

HSM Section	Treatment	Urban	Suburban	Rural
17.4.2.1	Implement Area-Wide Traffic Calming	✓	-	-
Appendix A	Convert two-way streets to one-way streets	T	T	T
Appendix A	Convert one-way streets to two-lane, two-way streets	T	T	T
Appendix A	Modify the level of access control on transportation network	T	-	-

NOTE: ✓= Indicates that an AMF is available for the treatment.

T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.

- = Indicates that an AMF is not available and a trends is not known.

17.4.2. Network Traffic Control and Operations Treatments with AMFs

17.4.2.1. Implement Area-Wide Traffic Calming

The main purpose of traffic calming is to reduce traffic volumes and operating speeds on residential local roads. The traditional approach to traffic calming is known as Level I Traffic Calming.[11] In Level I Traffic Calming, various site-specific calming techniques are applied to a local street network, usually a residential area.

AMFs related to traffic calming are summarized in Section 17.4.2.

Numerous traffic calming measures can be used to reduce traffic volume and driving speed on an area-wide basis. Most measures focus on managing vehicles through physical or operational devices such as: vehicle restrictions, lane narrowing, traffic circles, speed humps, raised crosswalks, chicanes, rumble strips, pavement treatments, etc. Traffic calming is one application of the "self-explaining road" approach. The measures that are implemented are designed to lead drivers to reduce speed and to adapt their driving appropriately. Before implementing traffic calming, the effects on pedestrians (including those with disabilities who may rely on paratransit), cyclists, emergency services vehicles, and transit may be considered.

The potential crash effects of applying area-wide or corridor-specific traffic calming measures to urban local roads, while adjacent collector roads remain untreated are shown in Exhibit 17-3.[2,4,6] These AMFs are not applicable to fatal accidents. The potential crash effects to non-injury crash frequency are also shown in Exhibit 17-3. The base condition of the AMFs (i.e., the condition in which the AMF = 1.00) is the absence of area-wide traffic calming.

The potential crash effects of specific traffic calming measures are provided in *Chapters 13* and *14*.

Exhibit 17-3: Potential Crash Effects of Applying Area-Wide or Corridor-Specific Traffic Calming to Urban Local Roads while Adjacent Collector Roads Remain Untreated [2,4,6] (injury excludes fatal crashes in this exhibit)

Treatment	Setting (Road type)	Traffic Volume AADT (veh/day)	Accident type (Severity)	AMF	Std. Error
Area-wide or corridor-specific traffic calming	Urban (All area-wide roads)	< 2,000 to 30,000	All types (Injury)	**0.89**	**0.1**
			All types (Non-injury)	*0.95**	*0.2*
	Urban (Two-lane Local roads)	< 2,000	All types (Injury)	**0.82**	**0.1**
			All types (Non-injury)	**0.94***	**0.1**
	Urban (Two-lane or Multilane Collector roads)	5,000 to 30,000	All types (Injury)	**0.94***	**0.1**
			All types (Non-injury)	*0.97**	*0.2*
Base Condition: Absence of area-wide traffic calming.					

NOTE: Injury excludes fatal accidents in this exhibit

Bold text is used for the most statistically reliable AMFs. These AMFs have a standard error of 0.1 or less.

Italic text is used for less statistically reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

* Observed variability suggests that this treatment could result in an increase, decrease or no change in expected average crash frequency. See *Part D Introduction and Applications Guidance*

17.5. CRASH EFFECTS OF ELEMENTS OF ROAD-USE CULTURE NETWORK CONSIDERATIONS

17.5.1. Background and Availability of AMFs

National policy leads transportation authorities to work to improve safety by going beyond engineering-based strategies. Transportation authorities, in partnership with related organizations, seek ways to incorporate education, enforcement, and emergency services strategies into their goal for a safer transportation network. These strategies can potentially influence road-use culture and may be designed to create a safer road-use culture. Engineering and planning decisions create and shape the transportation network, and clearly affect the safety of the transportation network. The road-use culture of the people using the network also affects the safety of the transportation network.

This HSM section discusses road-use culture and how expected average crash frequency may be reduced by understanding how road-use culture responds to engineering, enforcement, and education.

Road-use culture involves each individual road user's choices, and the attitudes of society as a whole towards transportation safety. The choices made by each individual road user flow from the beliefs, values, and ideas that each road user brings to the road. The attitudes of society as a whole towards transportation safety flow from the social norms regarding acceptable behaviors on the road, and from society's decisions regarding acceptable regulation, legislation, and enforcement levels. Road-use culture evolves as individuals influence society, and society influences individuals. Additional information regarding road-use culture can be found in Appendix A.

Exhibit 17-4 summarizes treatments related to road use culture and the corresponding AMFs available. The treatments summarized below encompass engineering, enforcement, and education.

AMFs and trends related to road use culture considerations are summarized in section 17.5.2 and Appendix A.

Exhibit 17-4: Road-Use Culture Network Considerations and Treatments

HSM Section	Treatment	Urban	Suburban	Rural
17.5.2.1	Install automated speed enforcement	✓	-	✓
17.5.2.2	Install changeable speed warning signs	✓	✓	✓
Appendix	Deploy mobile patrol vehicles	T	T	T
Appendix	Deploy stationary patrol vehicles	T	T	T
Appendix	Deploy aerial enforcement	T	T	T
Appendix	Deploy radar and laser speed monitoring equipment	T	T	T
Appendix	Install drone radar	T	T	T
Appendix	Modify posted speed limit	T	T	T
Appendix	Conduct enforcement to reduce red-light running	T	T	T
Appendix	Conduct enforcement to reduce impaired driving	T	T	T
Appendix	Conduct enforcement to increase seat belt and helmet use	T	T	T
Appendix	Implement network-wide engineering consistency	T	T	T
Appendix	Mitigate aggressive driving through engineering	T	T	T
Appendix	Conduct public education campaigns	T	T	T
Appendix	Implement young drivers and graduated driver licensing programs	T	T	T
Appendix	Implement older driver education and retesting programs	T	T	T

NOTE: ✓= Indicates that an AMF is available for the treatment.

T = Indicates that an AMF is not available but a trend regarding the potential change in crashes or user behavior is known and presented in Appendix A.

- = Indicates that an AMF is not available and a trends is not known.

17.5.2. Road Use Culture Network Consideration Treatments with AMFs

17.5.2.1. Install Automated Speed Enforcement

Automated enforcement systems use video or photographic identification in conjunction with radar or lasers to detect speeding drivers. The systems automatically record vehicle registrations without having to have police officers at the scene.

The crash effects of installing automated speed enforcement in urban or rural areas on all road types are shown in Exhibit 17-5.[1,3,5,7,9,12] The base condition for this AMF (i.e., the condition in which the AMF = 1.00) is the absence of automated speed enforcement.

Exhibit 17-5: Potential Crash Effects of Automated Speed Enforcement [1,3,5,7,9,12]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Install automated speed enforcement	All settings (All types)	Unspecified	All types (Injury)	**0.83+**	**0.01**

Base Condition: No automated speed enforcement.

NOTE: **Bold** text is used for the most statistically reliable AMFs. These AMFs have a standard error of 0.1 or less.
+ Combined AMF, see *Part D Applications Guidance*.

Multiyear programs indicate operating speeds dropped substantially at sites with fixed cameras compared to sites with mobile cameras.[8] However, the magnitude of the crash effect of mobile versus fixed camera sites is not certain at this time.

Some speed enforcement approaches are known to have spillover effects across the network. For example, speed cameras may affect behavior at locations not equipped with the cameras. The publicity and public interest accompanying installation of the cameras may lead to a generalized change in driver behavior at locations with and without cameras.[10] Some enforcement approaches may also have "time halo" effects. For example, the effect of operating speeds being enforced for a specific period may remain after the enforcement is withdrawn.

The gray box below illustrates how to apply the information in Exhibit 17-5 to calculate the crash effects of installing automated speed enforcement.

Effectiveness of Installing Automated Speed Enforcement

Question:

As part of an overall change to speed enforcement policy and an evolving safety culture, a local jurisdiction is proposing the implementation of automated speed enforcement on an urban arterial. What will be the likely reduction in the expected average crash frequency?

Given Information:

- Existing roadway = urban arterial
- Expected average crash frequency without treatment (See *Part C* Predictive Method) = 10 crashes/year

Find:

- Expected average crash frequency with installation of automated speed enforcement
- Change in expected average crash frequency

Answer:

1) Identify the Applicable AMF

   AMF = 0.83 (Exhibit 17-5)

2) Calculate the 95th percentile confidence interval estimation of crashes with the treatment

   = (0.83 ± 2 x 0.01) x (10 crashes/year) = 8.1 or 8.5 crashes/year

   The multiplication of the standard error by 2 yields a 95% probability that the true value is between 8.1 and 8.5 crashes/year. See Section 3.5.3 in Chapter 3 Fundamentals for a detailed explanation.

3) Calculate the difference between the expected number of crashes without the treatment and the expected number of crashes with the treatment.

   Change in Expected Average Crash Frequency:

   Low Estimate = 10 - 8.5 = 1.5 crashes/year reduction

   High Estimate = 10 - 8.1 = 1.9 crashes/year reduction

4) **Discussion: The implementation of automated speed enforcement may potentially cause a reduction or 1.5 to 1.9 crashes/year.**

17.5.2.2. Install Changeable Speed Warning Signs

Individual changeable speed warning signs give individual drivers real-time feedback regarding their speed.(7) The potential crash effects of installing these warning signs are shown in Exhibit 17-6. The base condition for this AMF (i.e., the condition in which the AMF = 1.00) is the absence of changeable speed warning signs.

Exhibit 17-6: Potential Crash Effects of Installing Changeable Speed Warning Signs for Individual Drivers[7]

Treatment	Setting (Road type)	Traffic Volume	Accident type (Severity)	AMF	Std. Error
Install changeable speed warning signs for individual drivers	Unspecified (Unspecified)	Unspecified	All types (All severities)	*0.54*	*0.2*
Base Condition: Absence of changeable speed warning signs					

NOTE: Based on international study: Van Houten and Nau 1981

Italic text is used for less statistically reliable AMFs. These AMFs have standard errors between 0.2 to 0.3.

Collective changeable speed warning signs give information such as the percentage of road users exceeding the speed limit.

17.6. CONCLUSION

The material in this chapter focuses on the potential crash effects of treatments that are applicable on a network-wide basis. The information presented is the AMFs known to a degree of statistical stability and reliability for inclusion in this edition of the HSM. Additional qualitative information regarding potential network wide treatments is contained in Appendix A.

Other chapters in *Part D* present treatments related to specific site types such as roadway segments and intersections. The material in this chapter can be used in conjunction with activities in *Chapter 6 Select Countermeasures*, and *Chapter 7 Economic Appraisal*. Some *Part D* AMFs are included in *Part C* for use in the predictive method. Other *Part D* AMFs are not presented in *Part C* but can be used in the methods to estimate change in crash frequency described in Section C.7 of the *Part C Introduction and Applications Guidance*.

17.7. REFERENCES

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APPENDIX A

A.1 INTRODUCTION

The appendix presents general information, trends in crashes and/or user-behavior as a result of the treatments, and a list of related treatments for which information is not currently available. Where AMFs are available, a more detailed discussion can be found within the chapter body. The absence of an AMF indicates that at the time this edition of the HSM was developed, completed research had not developed statistically reliable and/or stable AMFs that passed the screening test for inclusion in the HSM. Trends in crashes and user behavior that are either known or appear to be present are summarized in this appendix.

This appendix is organized into the following sections:

- Network Planning and Design Approaches/Elements (Section A.2)
- Network Traffic Control and Operational Elements (Section A.3)
- Road-Use Culture Network Considerations and Treatments (Section A.4)
- Catalogue of Treatments with Unknown Crash Effects (Section A.5)

A.2 NETWORK PLANNING AND DESIGN APPROACHES/ELEMENTS

A.2.1 General Information

Practitioners have opportunities to consider safety at every stage and level of transportation planning and the corresponding early stages of design. By striving to construct roadways that are as safe as possible, and by explicitly incorporating safety considerations into the planning and design stages, practitioners can minimize the need for crash mitigation after construction.

A.2.2 Trends in Crashes or User Behavior for Treatments with no AMFs

A.2.2.1 Apply Elements of Self-Explaining Roadway Design

Self-explaining roads convey a clear, simple and consistent message about the road's function and role. The message is embedded in the design and appearance of the road, using a limited number of design options and traffic control devices based on the road class. Self-explaining roads are designed to reduce driver errors and crashes. The first self-explaining roads were introduced in Holland in the 1990s.[21]

Drivers respond to the roadway design by adapting their driving and adjusting their speed. The cues may be physical and/or perceptual. For example, residential streets that are short and narrow create a sense of spatial enclosure which encourages drivers to slow down. Road surfaces that are color coded (e.g., to show bicycle lanes) convey information about how road users should use the space within the roadway. On self-explaining roads, drivers, pedestrians, and cyclists readily recognize and understand the relationship between the road, the adjacent land use, and environment, and the appropriate road-user response.

Classification of self-explaining roads

Different road functionality requires different self-explaining design techniques. Self-explaining roads are most relevant to local planning. Three levels of functionality classification are suggested for self-explaining roads:[25]

1. Roads with a through function;
2. Roads with a distributor function; and,
3. Roads with an access function (residential streets).

Each road category is designed to match the road's function and desired operating speed. For example, access to homes, schools, and offices is provided from residential and distributor roads. The self-explaining approach is intended to prevent through motorists from encroaching on residential streets. This approach appears to reduce traffic volumes and crash rates on residential streets.[3]

Self-explaining roads in residential areas

The design of self-explaining roads in residential areas stimulates drivers to be aware that they have left the network of arterials and collectors and must reduce their speed. The design also leads drivers to expect to encounter children, pedestrians, and bicyclists. The low speeds of self-explaining roads are particularly important for pedestrian and child safety. Children are highly vulnerable to speeding traffic because they are often impulsive and lack the experience and judgment necessary to assess traffic conditions.

Lower driving speeds and increased driver expectation potentially mitigate some of the factors that are known to contribute to pedestrian crashes. These factors include:[9,15]

- Improper crossing of the roadway or intersection;
- Walking or playing in the roadway;
- Restricted sight lines;
- Limited time for drivers to respond to unanticipated pedestrian movements;
- Inadequate searching and checking by pedestrians and drivers, especially when the vehicle is turning;
- Speeding; and,
- Pedestrians assuming that they are more visible than they actually are.

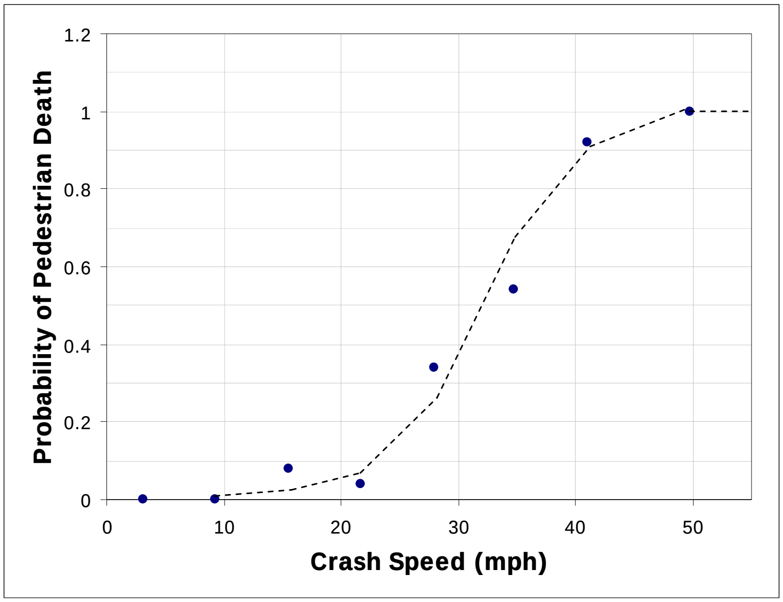
Self-explaining roads are generally designed to reduce operating speeds to about 18 mph in the zones where the roads are introduced. The roads are also designed to minimize the speed differential among different road users.

A study of the crash effects of self-explaining roads in Holland found that:[25]

- The number of fatalities declined; and,
- The vast majority of local residents were satisfied with the creation of an 18-mph zone.

Exhibit 17-7 shows how the relationship between crash speed and the probability of a pedestrian fatality rises rapidly when the crash speed exceeds about 18 mph.[17]

Exhibit 17-7: Relationship between Crash Speed and the Probability of a Pedestrian Fatality[17]



Self-explaining roads appear to reduce crashes when applied in planning and design. However, the magnitude of the crash effect is not certain at this time. More specifically, it appears that crashes are reduced in residential areas planned with self-explaining roads principles compared to other residential areas planned with more traditional principles.[11] Streets with no exit, such as cul-du-sacs, appear to be substantially safer for pedestrians, especially children when compared to other street layouts.[11] However, the magnitude of the crash effect is not certain at this time.

A.2.2.2 Apply Elements of Transportation Safety Planning in Transportation Network Design

Transportation Safety Planning (TSP) is a comprehensive, system-wide, proactive process that integrates safety into transportation decision making. TSP applies to all transportation modes and all network levels (i.e., local, regional, and state). TSP aims to create safety planning procedures that are explicit and measurable. TSP also aims to reduce accidents by establishing inherently safe transportation networks. On an inherently safe transportation network, a driver is less likely to be involved in a crash.[26]

The following websites provide information on the latest TSP strategies and tools:

http://www.fhwa.dot.gov/planning/SCP; and,

http://tsp.trb.org/.

TSP elements appear to improve safety when applied in planning and design. However, the magnitude of the crash effect is not certain at this time. More specifically, it appears that crashes are reduced in residential areas planned with TSP principles compared to other residential areas planned with more traditional principles.[11] Streets with no exit, such as cul-de-sacs, appear to be substantially safer for pedestrians, especially children when compared to other street layouts.[11] However, the magnitude of the crash effect is not certain at this time.

A.3 NETWORK TRAFFIC CONTROL AND OPERATIONAL ELEMENTS

A.3.1 Trends in Crashes or User Behavior for Treatments with no AMFs

A.3.1.1 Convert Two-Way Streets to One-Way Streets

One-way operations may apply to a whole area or to only a few streets, and may be found in both downtown and residential areas. One-way streets, usually implemented to increase traffic capacity, appear to reduce crashes under certain conditions.[11]

Implementing or removing one-way systems require careful thought and attention in their planning, design, and implementation. Detailed design considerations include the geometrics in the transition to and from one-way and two-way segments, appropriate regulatory signs, pavement markings, and providing suitable accommodation for turning movements at the beginning and end of one-way segments.[11] A consideration is the effect the one-way operations may have on the surrounding road network with the intent of avoiding the transfer of crashes to a neighboring area.

One-way systems have potential operational benefits which appear to reduce crashes. The potential benefits include:

- Elimination of two-way traffic conflicts;
- Reduction in the large number of potential conflicts at intersections in a two-way system, including the elimination of left turns by opposing traffic;
- Possible reduction in waiting times for pedestrians at signals;
- Simplification of intersection traffic control; and,
- Improved traffic signal synchronization. Platoons of traffic moving at the appropriate speed may travel the length of the street with few or no stops.

Converting two-way streets to one-way streets appears to reduce head-on and left-turn accidents.[11,19] However, the magnitude of the crash effect is not certain at this time.

Potential operational and safety concerns with one-way systems include increased vehicle speed and longer trips for drivers who travel one or more blocks out of their way to reach their destinations. Constraints to emergency vehicle operations are an additional consideration for one-way street systems.

A.3.1.2 Convert One-Way Streets to Two-Lane, Two-Way Streets

One-way operations may apply to a whole area or to only a few streets, and may be found in both downtown and residential areas. One-way streets, usually implemented to increase traffic capacity, appear to reduce crashes under certain conditions.[11]

In a study focusing on a pair of one-way streets that passed through a business district and a residential area, the design for converting the one-way streets to two-lane, two-way streets included bicycle lanes, all-day parallel parking, wider sidewalks, and new trees and benches in the business district. “Zebra” crosswalk markings with pedestrian warning signs were added to the two intersections closest to the school.[2] The study results showed that average speeds changed from 35 mph to about 25 mph. Travel times for car commuters increased slightly, and the number of bicyclists and pedestrians increased. Some vehicular traffic was diverted to alternate routes. [2]

A.3.1.3 Modify the Level of Access Control

The safety of an access point is influenced by broad characteristics such as road class and environment, the average density of access points, and median presence on the roadway. The safety of an access point is also influenced by specific characteristics related to detailed design and traffic control devices. These characteristics include alignment with opposite driveways, proximity to intersections, permitted entry/exit movements, storage, sight triangles, etc. Changing an access and incorporating that decision into a broader access management plan or policy means the change in one access is considered in an area-wide context. The purpose of this network perspective is to minimize the likelihood that a safety concern is transferred from one location to another.[12]

The following levels of access may be used on urban roadways:[5]

- Minimal access control: high density of intersecting streets, driveways, and median openings;
- Moderate level of access control: frontage roads running parallel with the main roadway segment and fewer cross streets; and,
- High level of access control: few driveways, cross streets or median openings.

The high level of access control has the fewest access points. On urban roadways, a high level of access control appears to reduce injury and non-injury accidents, and may also reduce angle and sideswipe accidents at intersections and mid-block areas.[5] However, the magnitude of the crash effect is not certain at this time.

A.4 ELEMENTS OF ROAD-USE CULTURE NETWORK CONSIDERATIONS

General Information

Road-use culture affects every aspect of driving behavior. Examples include driving above the speed limit, responses to red-light cameras at intersections, behavior at all-way stops, and attitudes towards pedestrians and bicyclists. Pedestrians and bicyclists use the transportation network in accordance with their road-use culture and perception of how to respond to the network and to other road users.

While road users' choices may not be fully understood, it is likely that the general level of patience and politeness, or of impatience and aggression, may vary over time and from place to place. Road-use culture is also affected by familiarity with surroundings.

Factors such as enforcement level and the efficiency of the supporting judicial system play a role in defining road-use culture. If drivers know that speeding tickets are unlikely to be processed or that speed limits are rarely enforced, drivers will see little reason to reduce their speed.

Road-Use Culture Development

The way in which road-use culture develops is not well known. It appears that visible behaviors such as seat belt usage, speeding, stopping at stop signs, etc., whether desirable or undesirable, spread more quickly than invisible behaviors, such as impaired driving.[27]

It also appears that conspicuous behaviors associated with a negative driving culture spread very quickly. Examples of these behaviors include parking on the wrong side of the street, "cutting off" another driver, making threatening gestures, or not signaling.[27]

Studies suggest that it is particularly difficult to change road-use culture regarding driving speed and observing speed limits. Progress has been made in changing road-use culture regarding driving under the influence (DUI) and seat belt usage. Programs and procedures targeted at young drivers, such as Graduated Driver's License (GDL), and at older drivers aim to reduce the accident rates of these two vulnerable groups. Studies show that enforcement can change driver behavior, if only in the short term. Automated enforcement for speed and red-light-running, combined with appropriate enabling legislation, offers the potential to reduce crashes.

Road Use Culture and Traffic Enforcement

Acceptable driving speed is one of the most important "norms" that helps to define a driving culture. For example, driving 5 to 10 mph greater than the posted speed limit may be culturally acceptable and considered the norm. Being aware of the norm, a driver who notices that a driver ahead is slowing down to the speed limit or to below the speed limit will likely respond in an appropriate way.

Drivers who do not conform to the norm for driving behavior, or who are driving in unfamiliar surroundings where the prevailing road-use culture differs from their own, may be more likely to have an accident than drivers who are familiar with the local road-use culture and conform to it. Drivers often choose to exceed the posted speed limit. This choice is an important safety issue because the risk may increase as operating speeds increase.[20]

Most drivers underestimate their driving speed, especially when driving fast. After a high-speed period, drivers who slow down typically perceive their new speed as less than it actually is. In addition, perceptual limitations to geometric features such as curvature can lead to drivers failing to respond appropriately to curvature.[20]

As most enforcement interventions appear to have little effect on modifying road-use culture, it is generally accepted that speed limits need to be self-enforcing. If drivers believe that speed limits are unreasonable, inappropriate, or inconsistently applied to the network, it is very unlikely that temporary enforcement measures can reduce speeds permanently.

Summary

Design of treatments and interventions that change driver behavior and result in crash reductions can be more successful through a better understanding of driver culture. An improved understanding of driver culture will also help contribute to increasingly effective safety campaigns and enforcement procedures.

A.4.1 Trends in Crashes or User Behavior for Treatments with no AMFs

A.4.1.1 Deploy Mobile Patrol Vehicles

Mobile patrol vehicles act as a speeding deterrent, but compliance with speed limits has been shown to decline with distance from the patrol vehicles.[20] The visibility of the patrol vehicle is important. It has been shown that when overhead lights were removed from patrol cars, mobile patrols ticketed 25% more motorists than when the patrol cars retained their overhead lights.[20]

The time halo effect of mobile patrol vehicles has been found to last from an hour to 8 weeks depending on the length and frequency of the deployments.[20]

A.4.1.2 Deploy Stationary Patrol Vehicles

Stationary patrol vehicles have been shown to lead to "a pronounced decrease in average traffic speed."[20]

A.4.1.3 Deploy Aerial Enforcement

Aerial speed enforcement has reduced vehicle crashes in Australia.[20] In New York, aerial enforcement was used successfully to apprehend drivers who used radar detectors and CB radio to avoid being caught speeding.[20]

A.4.1.4 Deploy Radar and Laser Speed Monitoring Equipment

Laser speed monitoring equipment can be used to apprehend drivers whose cars have radar detectors. These drivers tend to travel at the most extreme speeds.[20]

A.4.1.5 Install Drone Radar

Drone radars, or unattended radar transmitters, have been shown to slightly reduce average vehicle speed, and to decrease by 30 to 50% the number of drivers who exceed the speed limit by more than 10 mph.[20]

A.4.1.6 Modify Posted Speed Limit

Drivers tend to drive at the speed that they find acceptable and safe, despite posted speed limits.

Little or no effect on operating speed has been found for low- and moderate-speed roads where posted speed limits were changed (raised or lowered).[20] On high-speed roads such as freeways, "studies in the USA and abroad generally show an increase in speeds when speed limits are raised."[20]

The net crash effect of speed limits and changes in speed limits across the transportation network is not fully known. More information is needed to understand how drivers respond to speed limits and how driver behavior can be modified. This information would help to improve how speed limits are set, and would help to maximize the results of speed enforcement efforts.

A.4.1.7 Conduct Enforcement to Reduce Impaired Driving

Although alcohol and drugs have a major effect on driver error, and although driving under the influence (DUI) of alcohol or other drugs is widely regarded as a major problem, attitudes towards drinking and driving are not fully understood.

Behavioral controls appear to provide the best results for reducing drunk driving among people with multiple DUI offenses.[8] Behavioral controls include internal behavior controls such as moral beliefs concerning alcohol-impaired driving, and external behavioral controls such as the offenders' perceptions of accidents and criminal punishment. Social controls or peer group pressure appear to be less effective.

Many approaches have been tried to reduce DUI, including:

1. Classes for juvenile DUI offenders;
2. Alcohol abuse treatment as an alternative to license suspensions;
3. Lowering the legal blood alcohol limit to 0.05;
4. Introducing random breath testing;
5. Bar staff training;
6. Highly publicized sobriety checkpoints;
7. Underage drinking controls;
8. Limits on alcohol availability;
9. Media advocacy; and,
10. Punishment, including ignition interlock devices or impounding vehicles for repeat offenders.

The first five approaches do not result in a clear pattern of driver response. Some drivers are frequent violators and appear to need special attention and policies.[16]

As an example of a more severe approach, DUI laws introduced in California in 1990 included a pre-conviction license suspension on arrested DUI offenders. The approach was "…highly effective in reducing subsequent accidents and recidivism among DUI offenders."[18]

On the other hand, some evidence shows a multipronged approach may be a more effective choice. "Drinking and driving prevention seems to be most successful when it engages a broad variety of programs and interventions."[23] Such a program in Salinas, California "…succeeded not only in mobilizing the community, but also in reducing traffic injuries and impaired driving over a sustained period of time. Traffic crashes, injuries, and drinking and driving rates all decreased as a result of the project."[23] Programs that concentrated only on sobriety checkpoints appear to reduce accident frequency and increase DUI arrests over the short-term, but are not successful over the longer term.[23]

These DUI approaches suggest that road-use culture can be modified, but that change requires concentrated legislation and enforcement efforts, as well as appropriate community programs, to achieve long-term and sustainable results.

A.4.1.8 Conduct Enforcement to Increase Seat Belt and Helmet Use

The effectiveness of enforcing seat belt and helmet use is directly related to whether or not the laws are primary or secondary laws. A primary seat belt law allows law enforcement officials to ticket anyone not wearing a seat belt. A secondary seat belt law means that a police officer can only write a ticket for a seat belt violation if the driver is also cited for some other violation. If a seat belt law is secondary, not wearing a seat belt is still against the law; however, enforcement of the law is not as effective.

The adoption of primary laws is likely to increase seat belt and helmet use and to modify road-use culture. Primary enforcement may also lead to an increase in seat belt and helmet use.

A change from secondary to primary seat belt use laws has been shown to increase seat belt usage and to decrease driver fatalities.[10] Most jurisdictions have supported a change in law with enforcement campaigns. It appears that people are more likely to wear seat belts after legislation.[22] "States in which motorists can be stopped solely for belt nonuse had a combined use rate of 85 percent in 2006, compared to 74 percent in other States."[7]

Similarly, universal helmet requirements for motorcyclists increase helmet use. In June 2006, 68% of motorcyclists wore helmets that complied with federal safety regulations in states with universal helmet laws, compared to 37% in states without a universal helmet law.[6]

A.4.1.9 Implement Network-Wide Engineering Consistency

Network-wide engineering consistency refers to the degree to which a jurisdiction implements transportation engineering solutions using consistent principles and criteria to design transportation infrastructure and to control traffic. Consistently and uniformly applying regulatory, warning, and informational signs is one example. Another example is applying consistent and uniform pavement markings.

The consistency of engineering measures at individual locations and across a jurisdiction's transportation network is likely to affect the driving habits and road-use culture of local users. Road users come to expect certain procedures and to act accordingly. Examples include all-red phases at traffic signals, right-turn-on-red, the use of left-turn arrows or flashing lights at traffic signals, and policies regarding yielding to other vehicles and non-motorized travelers at intersections and roundabouts.

When procedures are not consistent across the jurisdiction, safety may deteriorate. This effect is shown when drivers traveling in a foreign country encounter different rules of the road.

A.4.1.10 Conduct Public Education Campaigns

Public education campaigns include efforts to educate the public with regards to new traffic control devices, general rules of the road, and similar topics.

Enforcement efforts can include public information, warnings, or educational campaigns. Such campaigns "…contribute significantly to the effectiveness of the technology…" used in enforcement, "…result in safer driving habits…", and can improve the image of police enforcement activities.[20] Extensive pedestrian safety education programs directed at children in elementary schools and those ages 4 to 7 appear to reduce child pedestrian crashes.[4]

It is also recognized that not all public information and education (PI&E) programs are effective. A review of some PI&E programs found that the only programs that resulted in a substantial reduction in speed, speeding, crashes, or crash severity were those that were integrated with a law enforcement program.[20]

"General assessment of public information programs has shown [PI&E programs] to have limited effect on actual behavior except when they are paired with enforcement."(14)

Program effectiveness generally depends on the use of multimedia, careful planning, and professional production. The impact, however, is difficult to measure and extremely difficult to separate from the effects of a campaign's enforcement component.(14)

A.4.1.11 Implement Young Driver and Graduated Driver Licensing Programs

Graduated driver licensing (GDL) programs developed for novice drivers have been implemented in many jurisdictions. GDL programs typically include restrictions such as zero blood alcohol, not driving on high-speed highways, not driving at night, and limitations on the number and age of passengers. The restrictions are designed to encourage new drivers to gain experience under conditions that minimize exposure to risk and to ensure drivers are exposed to more demanding driving situations only when they have enough experience.(13) The concern is new drivers are at risk while getting the experience they need.

Novice drivers are three times more likely to be involved in a fatal traffic crash than other drivers.(1,24) Evidence also indicates that the most dangerous times and situations for drivers aged 16 to 20 years are:(1)

- At night
- On freeways
- Driving with passengers

The level of risk for young drivers suggests that novice drivers need a learning period when they are subject to measures that "…minimize their exposure, especially in known risky circumstances like nighttime and on freeways."(1)

Although GDL programs and their results vary, it appears that there is a decrease in accident frequency with a GDL program.(13) There is also an indication that "increased driving experience is somewhat more important than increased age in reducing accidents among young novice" drivers.(13)

A.5 TREATMENTS WITH UNKNOWN CRASH EFFECTS

No information about the crash effects of the following treatments was available for this edition of the HSM.

A.5.1 Network Traffic Control and Operational Elements

- Implement network-wide or area-wide turn restrictions

A.5.2 Road-Use Culture Network Considerations

- Install enforcement notification signs
- Conduce enforcement to reduce red-light running
- Mitigate aggressive driving through engineering

- Implement older driver education and testing programs

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GLOSSARY

This chapter defines the terms used in the manual. The following terms are used interchangeably throughout the HSM:

- Accident and Crash

85th-percentile speed - the speed at or below which 85 percent of the motorists drive a given road. The speed is indicative of the speed that most motorists consider to be reasonably safe under normal conditions.

acceleration lane - a paved auxiliary lane, including tapered areas, allowing vehicles to accelerate when entering the through-traffic lane of the roadway.

acceptable gap - the distance to nearest vehicle in oncoming or cross traffic that a driver will accept to initiate a turning or crossing maneuver 50 percent of the time it is presented, typically measured in seconds.

access management - the systematic control of the location, spacing, design, and operation of driveways, median openings, interchanges, and street connections to a roadway, as well as roadway design applications that affect access, such as median treatments and auxiliary lanes and the appropriate separation of traffic signals.

accessible facilities - facilities where persons with disabilities have the same degree of convenience, connection, and safety afforded to the public in general. It includes, among others, access to sidewalks and streets, including crosswalks, curb ramps, street furnishings, parking, and other components of public rights-of-way.

accident/crash - a set of events not under human control that results in injury or property damage, due to the collision of at least one motorized vehicle and may involve collision with another motorized vehicle, a bicyclist, a pedestrian or an object. The terms accident and crash are used interchangeably in this manual.

accident modification factor (AMF) - an index of how much crash experience is expected to change following a modification in design or traffic control. AMF 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.

accident severity - the most severe injury sustained in an accident (e.g., in a fatal accident, two fatalities and three severe injuries were reported). Not to be confused with injury severity that refers to all the different injury levels sustained by drivers and passengers involved in an accident.

accommodation (visual) - the ability to change focus from instruments inside the vehicle to objects outside the vehicle.

all-way STOP-controlled - an intersection with stop signs at all approaches.

AADT - annual average daily traffic. *(See traffic, average annual daily).*

approach - a lane or set of lanes at an intersection that accommodates all left-turn, through, and right-turn movements from a given direction.

auxiliary lane - a lane marked for use, but not assigned for use by through traffic.

base model - a regression model for predicting the expected average crash frequency in each HSM prediction procedure given a set of site characteristics. The base model, like all regression models, predicts the value of a dependent variable as a function of a set of independent variables. The expected average crash frequency is adjusted for changes to set site characteristics with the use of an AMF.

Bayesian statistics - statistical method of analysis which bases statistical inference on a number of philosophical underpinnings that differ in principle from frequentist or classical statistical thought. First, this method incorporates knowledge from history or other sites. In other words, prior knowledge is formally incorporated to obtain the "best" estimation. Second, the method considers the likelihood of certain types of events as part of the analysis process. Third, it uses Bayes' theorem to translate probabilistic statements into degrees of belief (e.g., the belief that we are more certain about something than others), instead of the classical confidence interval interpretation.

before-after study - the evaluation of implemented safety treatments, accomplished by comparing frequency or severity of crashes before and after implementation. There are several different types of before-after studies. These studies often develop AMFs for a particular treatment or group of treatments. Also known as BA studies.

bicycle facility - a road, path, or way specifically designated for bicycle travel, whether exclusively or with other vehicles or pedestrians.

breakaway support - a design feature which allows a device such as a sign, luminary, or traffic signal support to yield or separate upon impact.

bus lane - a highway or street lane designed for bus use during specific periods.

calibration factor - a factor to adjust crash frequency estimates produced from an safety prediction procedure to approximate local conditions. The factor is computed by comparing existing accident data at the state, regional, or local level to estimates obtained from predictive models.

channelization - the separation of conflicting traffic movements into definite travel paths. Often part of access management strategies.

clear zone - the total roadside border area, starting at the edge of the traveled way, available for use by errant vehicles.

climbing lane - a passing lane added on an upgrade to allow traffic to pass heavy vehicles whose speeds are reduced.

closing speed - movement of objects based on their distance as observed from the driver.

coding - organization of information into larger units such as color and shape (e.g., warning signs are yellow, regulatory signs are white).

collision diagram - a schematic representation of the crashes that have occurred at a site within a given time period.

comparison group - a group of sites, used in before-and-after studies, which are untreated but are similar in nature to the treated sites. The comparison group is used to control for changes in crash frequency not influenced by the treatment.

comparison ratio - the ratio of expected number of "after" to the expected number of "before" target accidents on the comparison group.

condition diagram - a plan view drawing of relevant site characteristics.

conflict-to-crash ratio - number of conflicts divided by the number of crashes observed during a given period.

conspicuity - relates to the ability of a given object or condition to attract the attention of the road user.

context sensitive design (CSD) - a collaborative, interdisciplinary approach that involves all stakeholders to develop a transportation facility that fits its physical setting and preserves scenic, aesthetic, historic, and environmental resources, while maintaining safety and mobility.

continuous variable - a variable that is measured either on the interval or ratio scale. A continuous variable can theoretically take on an infinite number of values within an interval. Examples of continuous variables include measurements in distance, time, and mass. A special case of a continuous variable is a data set consisting of counts (e.g. crashes), which consist of non-negative integer values.

contrast sensitivity - the ability to distinguish between low contrast features. Ability to detect slight differences in luminance (level of light) between an object and its background (e.g. worn lane lines, concrete curbs).

control group - a set of sites randomly selected to not receive safety improvements.

control task - a major subtask of the driving task model consisting of keeping the vehicle at a desired speed and heading within the lane. Drivers exercise control through the steering wheel, accelerator or brake.

cost-effectiveness - a type of economic criteria for assessing a potential implementation of a countermeasure or design to reduce accidents. This term is generally expressed in terms of the dollars spent per reduction of accident frequency or accident severity.

cost-effectiveness index - ratio of the present value cost to the total estimated accident reduction.

count data - data that are non-negative integers.

countermeasure - a roadway based strategy intended to reduce the crash frequency or severity, or both at a site.

countermeasure, proven - countermeasures that are considered proven for given site characteristics because scientifically-rigorous evaluations have been conducted to validate the effectiveness of the proposed countermeasure for the given site characteristics.

countermeasure, tried and experimental - countermeasures for which a scientifically-rigorous evaluation has not been conducted or because an evaluation has not been performed to assess the effectiveness of such countermeasures.

corner clearance - minimum distance required between intersections and driveways along arterials and collector streets.

cost effectiveness - the annual cost per crash reduced.

crash - *(See accident).*

crash cushion (impact attenuator) - device that prevents an errant vehicle from impacting fixed objects by gradually decelerating the vehicle to a safe stop or by redirecting the vehicle away from the obstacle in a manner which reduces the likelihood of injury.

crash estimation - any methodology used to forecast or predict the crash frequency of an existing roadway for existing conditions during a past period or future period; an existing roadway for alternative conditions during a past or future period; a new roadway for given conditions for a future period.

crash evaluation - determining the effectiveness of a particular treatment or a treatment program after its implementation. The evaluation is based on comparing results obtained from crash estimation.

crash frequency – number of crashes occurring at a particular site, facility, or network in a one year period and is measure in number of crashes per year.

crash mapping - the visualization of crash locations and trends with computer software such as GIS.

crash prediction algorithm - procedure used to predict average crash frequency, consisting of three elements. It has two analytical components: baseline models and accident modification factors, as well as a third component: accident histories.

crash rate - the number of crashes per unit of exposure. For an intersection, this is typically the number of crashes divided by the total entering AADT; for road segments, this is typically the number of crashes per million vehicle-miles traveled on the segment. Also known as accident rate.

crash rate method - a method that normalizes the frequency of crashes against exposure (i.e. traffic volume for the study period for intersections, and traffic volume for the study period and segment length for roadway segments). Also known as accident rate method.

crash reduction factor (CRF) - the percentage crash reduction that might be expected after implementing a modification in design or traffic control. The CRF is equivalent to (1-AMF).

crash severity - the level of injury or property damage due to a crash, commonly divided into categories based on the KABCO scale.

Critical Rate Method (CRM) - a method in which the observed crash rate at each site is compared to a calculated critical crash rate that is unique to each site.

cross-sectional studies - studies comparing the crash frequency or severity of one group of entities having some common feature (e.g., STOP controlled intersections) to the crash frequency or severity of a different group of entities not having that feature (e.g., YIELD controlled intersections), in order to assess difference in crash experience between the two features (e.g., STOP versus YIELD sign).

cycle - a complete sequence of signal indications (phases).

cycle length - the total time for a traffic signal to complete one cycle.

dark adaptation (visual) - the ability to adjust light sensitivity on entering and exiting lighted or dark areas.

deceleration lane - a paved auxiliary lane, including tapered areas, allowing vehicles leaving the through-traffic lane of the roadway to decelerate.

decision sight distance (DSD) - the distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source, recognize the object, select an appropriate speed and path, and initiate and complete the maneuver efficiently and without a crash outcome.

delay - the additional travel time experienced by a driver, passenger, or pedestrian in comparison to free flow conditions.

delineation - methods of defining the roadway operating area for drivers.

dependent variable - in a function given as $Y = f(X_1, \ldots, X_n)$, it is customary to refer to $X_1, \ldots, X_n$ as independent or explanatory variables, and Y as the dependent or response variable. In each crash frequency prediction procedure, the dependent variable estimated in the base model is the annual accident frequency for a roadway segment or intersection.

descriptive analysis - methods such as frequency, crash rate, and equivalent property damage only (EPDO), which summarize in different forms the history of crash occurrence, type and/or severity at a site. These methods do not include any statistical analysis or inference.

design consistency - (1) the degree to which highway systems are designed and constructed to avoid critical driving maneuvers that may increase crash risk; (2) the ability of the highway geometry to conform to driver expectancy; (3) ensures that successive geometric elements are coordinated in a manner to produce harmonious driver performance without surprising events.

design speed - a selected speed used to determine the various geometric design features of the roadway. The assumed design speed should be a logical one with respect to the topography, anticipated operating speed, the adjacent land use, and the functional classification of highway. The design speed is not necessarily equal to the posted speed or operational speed of the facility.

diagnosis - the identification of factors that may contribute to a crash.

diamond interchange - an interchange that results in two or more closely spaced surface intersections, so that one connection is made to each freeway entry and exit, with one connection per quadrant.

discount rate - an interest rate that is chosen to reflect the time value of money.

dispersion parameter - *(See overdispersion parameter).*

distribution (data analysis and modeling related) - the set of frequencies or probabilities assigned to various outcomes of a particular event or trail. Densities (derived from continuous data) and distributions (derived from discrete data) are often used interchangeably.

driver expectancy - the likelihood that a driver will respond to common situations in predictable ways that the driver has found successful in the past. Expectancy affects how drivers perceive and handle information and affects the speed and nature of their responses.

driver workload – surrogate measure of the number of simultaneous tasks a driver performs while navigating a roadway.

driveway density - the number of driveways per mile on both sides of the roadway combined.

driving task model - the simultaneous and smooth integration of a number of sub-tasks required for a successful driving experience.

dynamic programming - a mathematical technique used to make a sequence of interrelated decisions to produce an optimal condition.

economically valid project - a project in which benefits are greater than the cost.

Empirical Bayes (EB) methodology - method used to combine *observed* crash frequency data for a given site with *predicted* crash frequency data from many similar sites to estimate its *expected* crash frequency.

entrance ramp - a ramp that allows traffic to enter a freeway.

equivalent property damage only (EPDO) method - assigns weighting factors to crashes by severity (fatal, injury, property damage only) to develop a combined frequency and severity score per site. The weighting factors are calculated relative to

Property Damage Only (PDO) crash costs. Crash costs include direct costs such as ambulance service, police and fire services, property damage, insurance and other costs directly related to the crashes. Crash costs also include indirect costs, which are the value society would place on pain and suffering or loss of life associated with the crash.

exit ramp - a ramp that allows traffic to depart a freeway.

expected average crash frequency - the estimate of long term expected average crash frequency of a site, facility or network under a given set of geometric conditions and traffic volumes (AADT) in a given period of years. In the EB methodology this frequency is calculated from observed accident frequency at the site, and predicted accident frequency at the site based on accident frequency estimates at other similar sites.

expected average crash frequency, change in - the difference between the expected average crash frequency in the absence of treatment and with the treatment in place.

expected crashes - an estimate of long range average number of crashes per year for a particular type of roadway or intersection.

expected excess crash method - method in which sites are ranked according to the difference between the adjusted observed crash frequency and the expected crash frequency for the reference population (e.g., two-lane rural segment, multilane undivided roadway, or urban stop-controlled intersection).

experimental studies - studies where sites are randomly assigned to a treatment or control group and the differences in accident experience can then be attributed to a treatment or control group.

explanatory variable (predictor) - a variable which is used to explain (predict) the change in the value of another variable. An explanatory variable is often defined as an independent variable; the variable which it affects is called the dependent variable.

facility - a length of highway that may consist of connected sections, segments, and intersections.

first harmful event - the first injury or damage-producing event that characterizes the crash.

freeway - a multilane, divided highway with a minimum of two lanes for the exclusive use of traffic in each direction and full control of access without traffic interruption.

frequency method - a method that produces a ranking of sites according to total crashes or crashes by type and/or severity.

frequentist statistics - statistical philosophy that results in hypothesis tests that provide an estimate of the probability of observing the sample data conditional on a true null hypothesis. This philosophy asserts that probabilities are obtained through long-run repeated observations of events.

gap - the time, in seconds, for the front bumper of the second of two successive vehicles to reach the starting point of the front bumper of the first vehicle. Also referred to as headway.

gap acceptance - the process by which a vehicle enters or crosses a vehicular stream by accepting an available gap to maneuver.

geometric condition - the spatial characteristics of a facility, including grade, horizontal curvature, the number and width of lanes, and lane use.

goodness-of-fit (GOF) statistics – the goodness of fit of a statistical model describes how well it fits a set of observations. Measures of goodness of fit typically summarize the discrepancy between observed values and the values expected under the model in question. There are numerous GOF measures, including the coefficient of determination R^2, the F test, and the chi-square test for frequency data, among others. Unlike F-ratio and likelihood-ratio tests, GOF measures are not statistical tests.

gore area - the area located immediately between the edge of the ramp pavement and the edge of the roadway pavement at a merge or diverge area.

guidance task - a major subtask of the driving task model consisting of interacting with other vehicles (following, passing, merging, etc.) through maintaining a safe following distance and through following markings, traffic control signs, and signals.

Haddon Matrix - a framework used for identifying possible contributing factors for crashes in which contributing factors (i.e. driver, vehicle, and roadway/environment) are cross-referenced against possible crash conditions before, during, and after a crash to identify possible reasons for the events.

headway - *(See gap).*

Heinrich Triangle - concept founded on the precedence relationship that “no injury accidents” precedes “minor injury accidents.” This concept is supported by two basic ideas: (1) events of lesser severity are more numerous than more severe events, and events closer to the base of the triangle precede events nearer the top; and (2) events near the base of the triangle occur more frequently than events near the triangle’s top, and their rate of occurrence can be more reliably estimated.

high-occupancy vehicle (HOV) - a vehicle with a defined minimum number of occupants (may consist of vehicles with more than one occupant).

high proportion of crashes method - the screening of sites based on the probability that their long term expected proportion of crashes is greater than the threshold proportion of crashes.

Highway Safety Improvement Program (HSIP) - SAFETEA-LU re-established the Highway Safety Improvement Program (HSIP) as a core program in conjunction with a Strategic Highway Safety Plan (SHSP). The purpose of the HSIP is to reduce the number of fatal and serious/life-changing crashes through state-level engineering measures.

holistic approach - a multidisciplinary approach to the reduction of crashes and injury severity.

homogeneous roadway segment - a portion of a roadway with similar average daily traffic volumes (veh/day), geometric design, and traffic control features.

human factors - the application of knowledge from human sciences such as human psychology, physiology, and kinesiology in the design of systems, tasks, and environments for effective and safe use.

incremental benefit-cost ratio - the incremental benefit/cost ratio is an extension of the benefit/cost ratio method. Projects with a benefit/cost ratio greater than one are arranged in increasing order based on their estimated cost.

Indiana Lane Merge System (ILMS) - advanced dynamic traffic control system designed to encourage drivers to switch lanes well in advance of the work zone lane drop and entry taper.

independent variables - a variable which is used to explain (predict) the change in the value of another variable.

indirect measures of safety - *(See surrogate measures).*

influence area (freeway) - an area that incurs operational impacts of merging (diverging) vehicles in Lanes 1 and 2 of the freeway and the acceleration (deceleration) lane for 1,500 ft from the merge (diverge) point downstream.

influence area (intersection) – functional area on each approach to an intersection consisting of three elements (1) perception-reaction distance, (2) maneuver distance, and (3) queue storage distance.

integer programming - a mathematical optimization technique involving a linear programming approach in which some or all of the decision variables are restricted to integer values.

interchange - intersections that consist of structures that provide for the cross-flow of traffic at different levels without interruption, thus reducing delay, particularly when volumes are high.

interchange ramp terminal - a junction with a surface street to serve vehicles entering or exiting a freeway.

intersection - general area where two or more roadways or highways meet, including the roadway, and roadside facilities for pedestrian and bicycle movements within the area.

intersection functional area - area extending upstream and downstream from the physical intersection area including any auxiliary lanes and their associated channelization.

intersection related accident - an accident that occurs at the intersection itself or an accident that occurs on an intersection approach within 250 ft (as defined in the HSM) of the intersection and is related to the presence of the intersection.

intersection sight distance - the distance needed at an intersection for drivers to perceive the presence of potentially conflicting vehicles in sufficient time to stop or adjust their speed to avoid colliding in the intersection.

KABCO - an injury scale developed by the National Safety Council to measure the observed injury severity for any person involved as determined by law enforcement at the scene of the crash. (Fatal injury (K), Incapacitating Injury (A), Non-Incapacitating Injury (B), Possible Injury (C), and No Injury (O).) The scale can also be applied to crashes: for example, a K crash would be a crash in which the most severe injury was a fatality, and so forth.

lateral clearance - lateral distance from edge of traveled way to a roadside object or feature.

level of service of safety (LOSS) method - the ranking of sites according to their observed and expected crash frequency for the entire population, where the degree of deviation is then labeled into four level of service classes.

median - the portion of a divided highway separating the traveled ways from traffic in opposite directions.

median refuge island - an island in the center of a road that physically separates the directional flow of traffic and that provides pedestrians with a place of refuge and reduces the crossing distance of a crosswalk.

meta analysis - a statistical technique that combines the independent estimates of crash reduction effectiveness from separate studies into one estimate by weighing each individual estimate according to its variance.

method of moments - method in which a site's observed accident frequency is adjusted based on the variance in the crash data and average crash counts for the site's reference population.

minor street - the lower volume street controlled by stop signs at a two-way, or four-way stop-controlled intersection; also referred to as a side street. The lower volume street at a signalized intersection.

Model Minimum Inventory of Roadway Elements (MMIRE) - set of guidelines outlining the roadway information that should be included in a roadway database to be used for safety analysis.

Model Minimum Uniform Crash Criteria (MMUCC) - set of guidelines outlining the minimum elements in crash, roadway, vehicle, and person data that should ideally be in an integrated crash database .

most harmful event - event that results in the most severe injury or greatest property damage for a crash event.

motor vehicle accident - any incident in which bodily injury or damage to property is sustained as a result of the movement of a motor vehicle, or of its load while the motor vehicle is in motion. Also referred to as a motor vehicle crash.

multilane highway - a highway with at least two lanes for the exclusive use of traffic in each direction, with no control, partial control, or full control of access, but that may have periodic interruptions to flow at signalized intersections.

multivariate statistical modeling - statistical procedure used for cross-sectional analysis which attempts to account for variables that affect crash frequency or severity, based on the premise that differences in the characteristics of features result in different crash outcomes.

navigation task – activities involved in planning and executing a trip from origin to destination.

net benefit - a type of economic criteria for assessing the benefits of a project. For a project in a safety program, it is assessed by determining the difference between the potential crash frequency or severity reductions (benefits) from the costs to develop and construct the project. Maintenance and operations costs may also be associated with a net benefit calculation.

net present value (NPV) or **net present worth (NPW)** - this method is used to express the difference between discounted costs and discounted benefits of an individual improvement project in a single amount. The term "discounted" indicates that the monetary costs and benefits are converted to a present-value using a discount rate.

network screening - network screening is a process for reviewing a transportation network to identify and rank sites from most likely to least likely to benefit from a safety improvement.

non-monetary factors - items that do not have an equivalent monetary value or that would be particularly difficult to quantify (i.e., public demand, livability impacts, redevelopment potential, etc.).

observational studies - often used to evaluate safety performance. There are two forms of observational studies: before-after studies and cross-sectional studies.

offset - lateral distance from edge of traveled way to a roadside object or feature. Also known as lateral clearance.

operating speed - the 85th percentile of the distribution of observed speeds operating during free-flow conditions.

overdispersion parameter - an estimated parameter from a statistical model that when the results of modeling are used to estimate accident frequencies, indicates how widely the accident counts are distributed around the estimated mean. This terms is used interchangeably with *dispersion parameter*.

p-value - the level of significance used to reject or accept the null hypothesis (whether a result is valid statistically or not).

passing lane - a lane added to improve passing opportunities in one or both directions of travel on a two-lane highway.

peak searching algorithm - a method to identify the segments that are most likely to benefit from a safety improvement within a homogeneous section.

pedestrian - a person traveling on foot or in a wheelchair.

pedestrian crosswalk - pedestrian roadway crossing facility that represents a legal crosswalk at a particular location.

pedestrian refuge - an at-grade opening within a median island that allows pedestrians to wait for an acceptable gap in traffic.

pedestrian traffic control - traffic control devices installed particularly for pedestrian movement control at intersections; it may include illuminated push buttons, pedestrian detectors, countdown signals, signage, pedestrian channelization devices, and pedestrian signal intervals.

perception-reaction time (PRT) - time required to detect a target, process the information, decide on a response, and initiate a response (it does not include the actual response element to the information). Also known as perception-response time.

perception-response time - *(See perception-reaction time).*

performance threshold - a numerical value that is used to establish a threshold of expected number of crashes (i.e. safety performance) for sites under consideration.

perspective, engineering - the engineering perspective considers crash data, site characteristics, and field conditions in the context of identifying potential engineering solutions that would address the potential safety concern. It may include consideration of human factors.

perspective, human factors – the human factors perspective considers the contributions of the human to the contributing factors of the crash in order to propose solutions that might break the chain of events leading to the crash.

peripheral vision - the ability of people to see objects beyond the cone of clearest vision.

permitted plus protected phase - compound left-turn protection that displays the permitted phase before the protected phase.

phase - the part of the signal cycle allocated to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals.

positive guidance – when information is provided to the driver in an clear manner and with sufficient conspicuity to allow the driver to detect an object in a roadway environment that may be visually cluttered, recognize the object and its potential impacts to the driver and vehicle, select an appropriate speed and path, and initiate and complete the required maneuver successfully.

potential for safety improvement (PSI) - estimates how much the long-term accident frequency could be reduced at a particular site.

predicted average crash frequency – the estimate of long-term average crash frequency which is forecast to occur at a site using a predictive model found in Part C of the HSM. The predictive models in the HSM involve the use of regression models, known as Safety Performance Functions, in combination with Accident Modification Factors and calibration factors to adjust the model to site specific and local conditions.

predictive method - the methodology in Part C of the manual used to estimate the 'expected average crash frequency' of a site, facility or roadway under given geometric conditions, traffic volumes and period of time.

primacy - placement of information on signs according to its importance to the driver. In situations where information competes for drivers' attention, unneeded and low priority information is removed. Errors can occur when drivers shred important information because of high workload (process less important information and miss more important information).

programming, linear - a method used to allocate limited resources (funds) to competing activities (safety improvement projects) in an optimal manner.

programming, integer - an instance of linear programming when at least one decision variable is restricted to an integer value.

programming, dynamic - a mathematical technique used to make a sequence of interrelated decisions to produce an optimal condition. Dynamic programming problems have a defined beginning and end. While there are multiple paths and options between the beginning and end, only one optimal set of decisions will move the problem from the beginning to the desired end.

project development process - typical stages of a project from planning to post-construction operations and maintenance activities.

project planning - part of the project development process in which project alternatives are developed and analyzed to enhance a specific performance measure or a set of performance measures, such as, capacity, multimodal amenities, transit service, and safety.

quantitative predictive analysis - methodology used to calculate an expected number of crashes based on the geometric and operational characteristics at the site for existing conditions, future conditions and/or roadway design alternatives.

queue - a line of vehicles, bicycles, or persons waiting to be served by the system in which the flow rate from the front of the queue determines the average speed within the queue.

randomized controlled trial - experiment deliberately designed to answer a research question. Roadways or facilities are randomly assigned to a treatment or control group.

ranking methods, individual - the evaluation of individual sites to determine the most cost-effective countermeasure or combination of countermeasures for the site.

ranking methods, systematic - the evaluation of multiple safety improvement projects to determine the combination of projects that will provide the greatest crash frequency or severity reduction benefit across a highway network given budget constraints.

rate - *(See crash rate).*

rate, critical - compares the observed crash rate at each site with a calculated critical crash rate unique to each site.

reaction time (RT) - the time from the onset of a stimulus to the beginning of a driver's (or pedestrian's) response to the stimulus by a simple movement of a limb or other body part.

redundancy - providing information in more than one way such as indicating a no passing zone with signs and pavement markings.

regression analysis - a collective name for statistical methods used to determine the interdependence of variables for the purpose of predicting expected average outcomes. These methods consist of values of a dependent variable and one or more independent variables (explanatory variables).

regression-to-the-mean (RTM) - the tendency for the occurrence of crashes at a particular site to fluctuate up or down, over the long term, and to converge to a long-term average. This tendency introduces regression-to-the-mean bias into crash estimation and analysis, which can make treatments at sites with extremely high crash frequency appear to be more effective than they truly are.

relative severity index (RSI) - a measure of jurisdiction-specific societal crash costs.

relative severity index (RSI) method - an average crash cost calculated based on the crash types at each site and then compared to an average crash cost for sites with similar characteristics to identify those sites that have a higher than average crash cost. The crash costs can include direct crash costs accounting for economic costs of the crashes only; or account for both direct and indirect costs.

road-use culture - each individual road user's choices, and the attitudes of society as a whole towards transportation safety.

roadside - the area between the outside shoulder edge and the right-of-way limits. The area between roadways of a divided highway may also be considered roadside.

roadside barrier - a longitudinal device used to shield drivers from natural or man-made objects located along either side of a traveled way. It may also be used to protect bystanders, pedestrians, and cyclists from vehicular traffic under special conditions.

roadside hazard rating - considers the clear zone in conjunction with the roadside slope, roadside surface roughness, recoverability of the roadside, and other elements beyond the clear zone such as barriers or trees. As the RHR increases from 1 to 7, the crash risk for frequency and/or severity increases.

roadway - the portion of a highway, including shoulders, for vehicular use.

roadway cross-section elements - roadway travel lanes, medians, shoulders, and sideslopes.

roadway environment - a system, where the driver, the vehicle, and the roadway interact with each other.

roadway, low speed - facility with traffic speeds or posted speed limits of 30 mph or less.

roadway, intermediate or high speed - facility with traffic speeds or posted speed limits greater than 45 mph.

roadway safety management process - a quantitative, systematic process for studying roadway crashes and characteristics of the roadway system and those who use the system, which includes identifying potential improvements, implementation, and the evaluation of the improvements.

roadway segment - a portion of a road that has a consistent roadway cross-section and is defined by two endpoints.

roundabout - an unsignalized intersection with a circulatory roadway around a central island with all entering vehicles yielding to the circulating traffic.

rumble strips - devices designed to give strong auditory and tactile feedback to errant vehicles leaving the travel way.

running speed - the distance a vehicle travels divided by running time, in miles per hour.

rural areas - places outside the boundaries of urban growth boundary where the population is less than 5,000 inhabitants.

Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU) - a federal legislature enacted in 2005. This legislature elevated the Highway Safety Improvement Program (HSIP) to a core FHWA program and created requirement for each state to develop a State Highway Safety Plan (SHSP).

safety - the number of accidents, by severity, expected to occur on the entity per unit of time. An entity may be a signalized intersection, a road segment, a driver, a fleet of trucks, etc.

safety management process - process for monitoring, improving, and maintaining safety on existing roadway networks.

safety performance function (SPF) - an equation used to estimate or predict the expected average crash frequency per year at a location as a function of traffic volume and in some cases roadway or intersection characteristics (e.g. number of lanes, traffic control, or median type).

segment - a portion of a facility on which a crash analysis is performed. A segment is defined by two endpoints.

selective attention - the ability, on an ongoing moment-to-moment basis while driving, to identify and allocate attention to the most relevant information, especially within a visually complex scene and in the presence of a number of distracters.

service life - number of years in which the countermeasure is expected to have a noticeable and quantifiable effect on the crash occurrence at the site.

severity index - a severity index (SI) is a number from zero to ten used to categorize accidents by the probability of their resulting in property damage, personal injury, or a fatality, or any combination of these possible outcomes. The resultant number can then be translated into an accident cost and the relative effectiveness of alternate treatments can be estimated.

shoulder - a portion of the roadway contiguous with the traveled way for accommodation of pedestrians, bicycles, stopped vehicles, emergency use, as well as lateral support of the sub base, base, and surface courses.

Strategic Highway Safety Plan (SHSP) - a comprehensive plan to substantially reduce vehicle-related fatalities and injuries on the nation's highways (AASHTO)

sight distance - the length of roadway ahead that is visible to the driver.

sight triangle - in plan view, the area defined by the point of intersection of two roadways, and by the driver's line of sight from the point of approach along one leg of the intersection, to the farthest unobstructed location on another leg of the intersection.

site - project location consisting of, but not limited to, intersections, ramps, interchanges, at-grade rail crossings, roadway segments, etc.

sites with potential for improvement - intersections and corridors with potential for safety improvements and identified as having possibility of responding to crash countermeasure installation.

skew angle, **intersection** - the deviation from an intersection angle of 90 degrees. Carries a positive or negative sign that indicates whether the minor road intersects the major road at an acute or obtuse angle, respectively.

slalom effect - dynamic illusion of direction and shape used to influence traffic behavior.

sliding-window approach - analysis method that can be applied when screening roadway segments. It consists of conceptually sliding a window of a specified length (e.g. 0.3 mile) along the road segment in increments of a specified size (e.g., 0.1 mile). The method chosen to screen the segment is applied to each position of the window and the results of the analysis are recorded for each window. The window that shows the most potential for safety improvement is used to represent the total performance of the segment.

slope - the relative steepness of the terrain expressed as a ratio or percentage. Slopes may be categorized as positive (backslopes) or negative (foreslopes) and as parallel or cross slopes in relation to the direction of traffic.

speed adaptation - phenomenon experienced by drivers leaving a freeway after a long period of driving, and having difficulty conforming to the speed limit on a different road or highway.

speed choice - speed chosen by a driver that is perceived to limit the risk and outcome of a crash.

spreading - where all the information required by the driver cannot be placed on one sign or on a number of signs at one location, spread the signage out along the road so that information is given in small amounts to reduce the information load on the driver.

stopping sight distance (SSD) - the sight distance required to permit drivers to see a stationary object soon enough to stop for it under a defined set of worst-case conditions, without the performance of any avoidance maneuver or change in travel path; the calculation of SSD depends upon speed, gradient, road surface and tire conditions, and assumptions about the perception-reaction time of the driver.

suburban environment - an area with a mixture of densities for housing and employment, where high-density nonresidential development is intended to serve the local community.

superelevation - the banking of a roadway in a curve to counteract lateral acceleration.

surrogate measure - an indirect safety measurement that provides the opportunity to assess safety performance when accident frequencies are not available because the roadway or facility is not yet in service or has only been in service for a short time, or when crash frequencies are low or have not been collected, or when a roadway or facility has significant unique features

system planning - the first stage of the project development process and it is the stage in which network priorities are identified and assessed.

systematic prioritization - the process used to produce an optimal project mix that will maximize crash frequency and severity reduction benefits while minimizing costs, or fitting a mixed budget or set of policies.

systematic reviews - process of assimilating knowledge from documented information.

taper area - an area characterized by a reduction or increase in pavement width typically located between mainline and ramp, or areas with lane reductions.

total million entering vehicles (TMEV) - measurement for total intersection traffic volume calculated from total entering vehicles (TEV) for each intersection approach.

total entering volume - Sum of total major and minor street volumes approaching an intersection.

traffic, annual average daily - the counted (or estimated) total traffic volume in one year divided by 365 days/year.

traffic barrier - a device used to prevent a vehicle from striking a more severe obstacle or feature located on the roadside or in the median or to prevent crossover median accidents. As defined herein, there are four classes of traffic barriers, namely, roadside barriers, median barriers, bridge railings, and crash cushions.

traffic calming - measures that are intended to prevent or restrict traffic movements, reduce speeds, or attract drivers' attention, typically used on lower speed roadways.

traffic conflict - an event involving two or more road users, in which the action of one user causes the other user to make an evasive maneuver to avoid a collision.

Transportation Safety Planning (TSP) - the comprehensive, system-wide, multimodal, proactive process that better integrates safety into surface transportation decision-making.

traveled way - lanes, excluding the shoulders.

urban environment - an area typified by high densities of development or concentrations of population, drawing people from several areas within a region.

useful field of view (UFOV) - a subset of the total field of view where stimuli can not only be detected, but can be recognized and understood sufficiently to permit a timely driver response. As such, this term represents an aspect of visual information processing, rather than a measure of visual sensitivity.

visual acuity - the ability to see details at a distance.

visual demand - aggregate input from traffic, the road, and other sources the driver must process to operate a motor vehicle. While drivers can compensate for increased visual demand to some degree, human factors experts generally agree that increasing visual demand towards overload will increase crash risk.

volume - the number of persons or vehicles passing a point on a lane, roadway, or other traffic-way during some time interval, often one hour, expressed in vehicles, bicycles, or persons per hour.

volume, annual average daily traffic - the average number of vehicles passing a point on a roadway in a day from both directions, for all days of the year, during a specified calendar year, expressed in vehicles per day.

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