Part B—Roadway Safety Management Process

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.

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 the following:

- A systematic and repeatable process for identifying opportunities to reduce crashes and for 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; and
- 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

Figure 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 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.

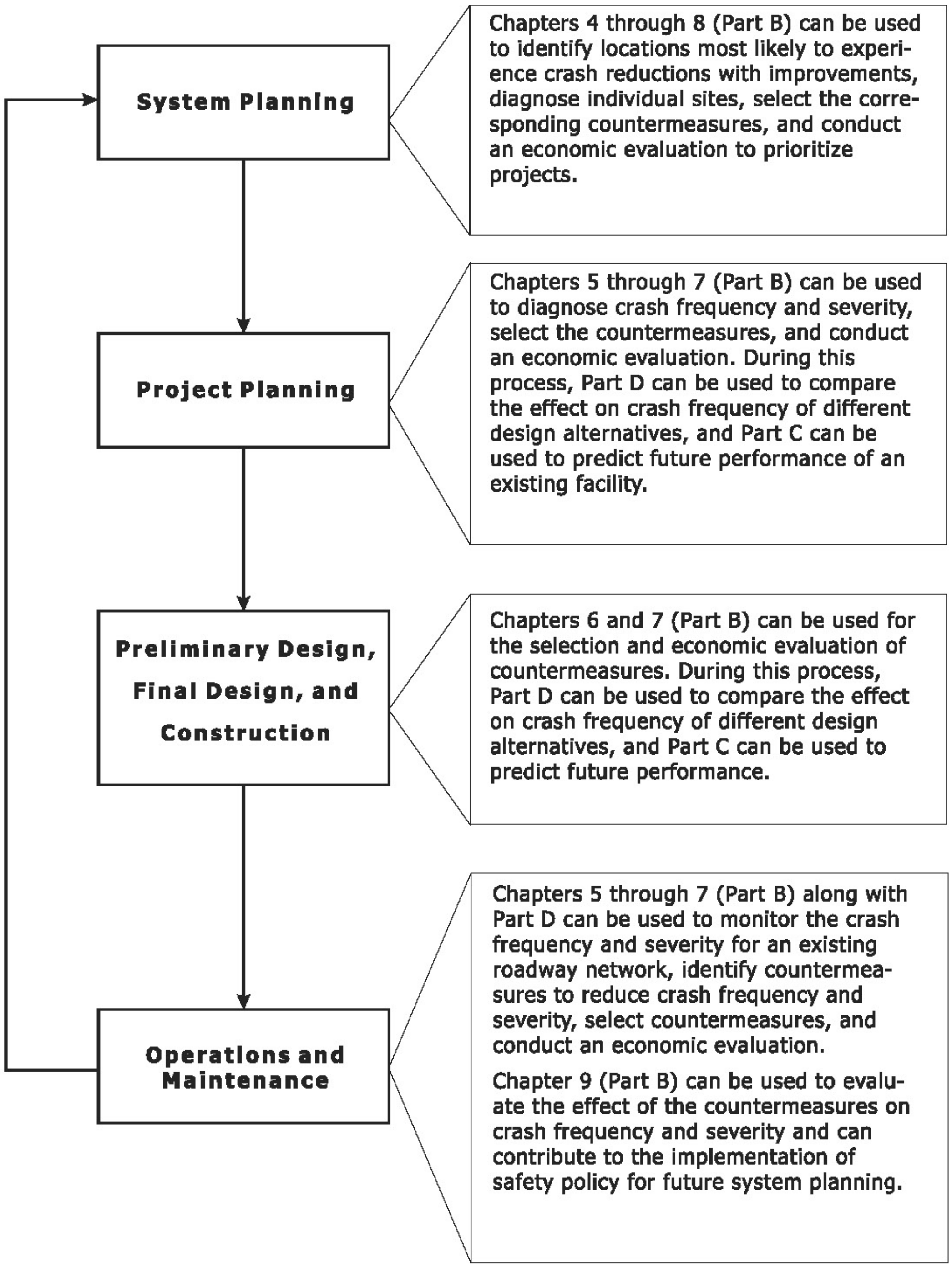


Figure B-1. The Project Development Process

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 or multiple screening methods, or both, 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 presents countermeasures and, when available, their corresponding Crash Modification Factors (CMFs). The CMFs presented in Part D have satisfied the screening criteria developed for the HSM, as described in 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., a CMF);
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 CMF; 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 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 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. he 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.

Finally, Part D consists of crash modification factors that can be applied in Chapters 4, 6, 7, and 8. The crash 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.

Chapter 4—Network Screening

4.1. INTRODUCTION

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

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

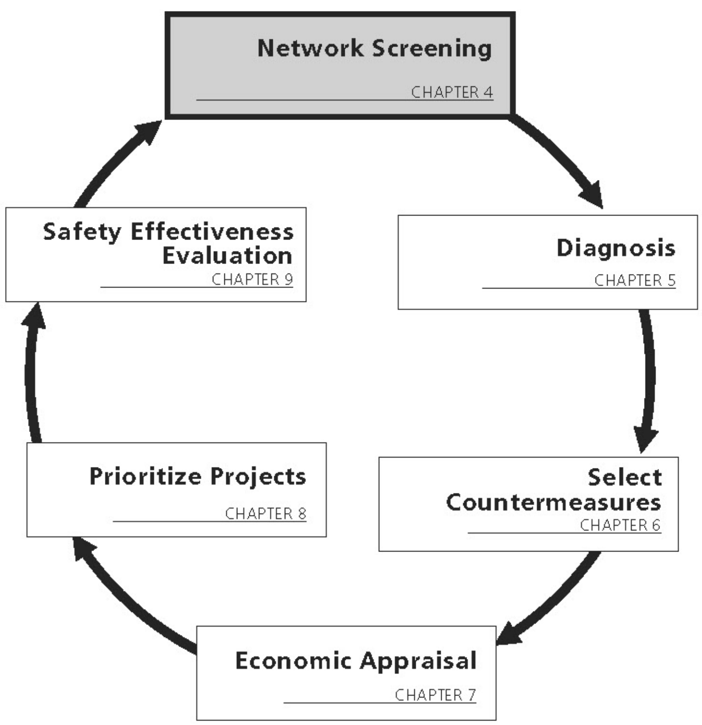


Figure 4-1. Roadway Safety Management Process

4.2. NETWORK SCREENING PROCESS

There are five major steps in network screening as shown in Figure 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 that 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 (Figure 4-2). Network screening can be conducted and focused on one or both of the following:

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

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

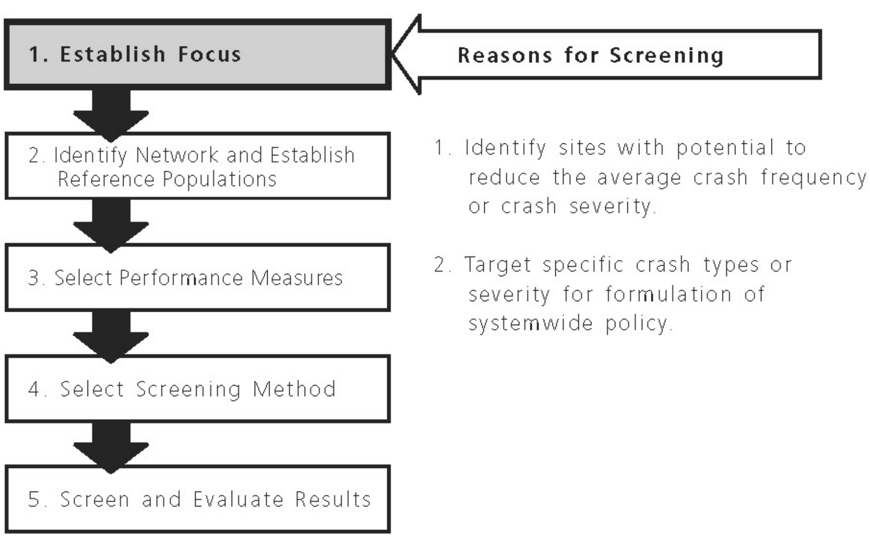


Figure 4-2. The Network Screening Process—Step 1, Establish Focus

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

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

Determining the Network Screening Focus

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

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

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

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

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

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 (Figure 4-3). Examples of roadway network elements that can be screened include intersections, roadway segments, facilities, ramps, ramp terminal intersections, and at-grade rail crossings.

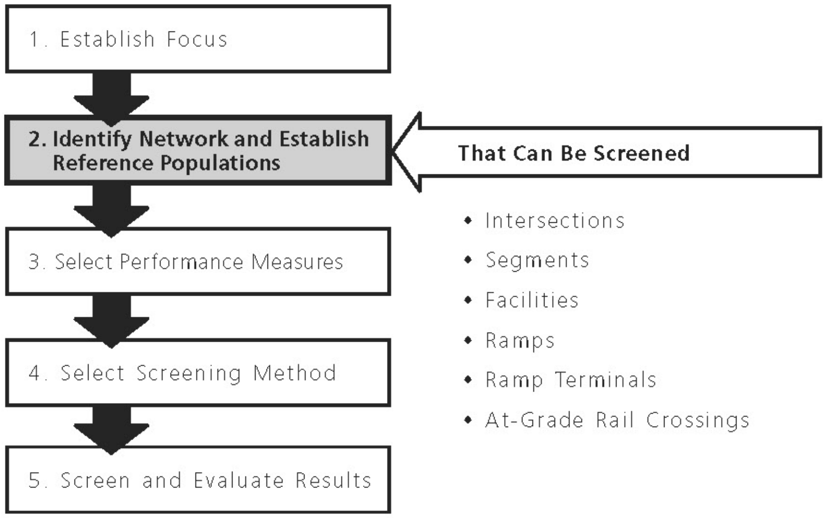


Figure 4-3. The Network Screening Process—Step 2, Identify Network and Establish Reference Populations

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

Intersection Reference Populations

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

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

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

Establishing Reference Populations for Intersection Screening

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

Example Intersection Reference Populations Defined by Functional Classification and Traffic Control

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 or width, or both;
- Operating speed or posted speed;
- Adjacent land use (e.g., urban, suburban, rural);
- Terrain (e.g., flat, rolling, mountainous); and
- Functional classification (e.g., arterial, collector, local).

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

Establishing Reference Populations for Segment Screening

Example:

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

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 (Figure 4-4). Just as intersection traffic operations analysis can be measured as a function of vehicle delay, queue length, or a volume-to-capacity ratio, intersection safety can be quantitatively measured in terms of average crash frequency, expected average crash frequency, a critical crash rate, or several other performance measures. In network screening, using multiple performance measures to evaluate each site may improve the level of confidence in the results.

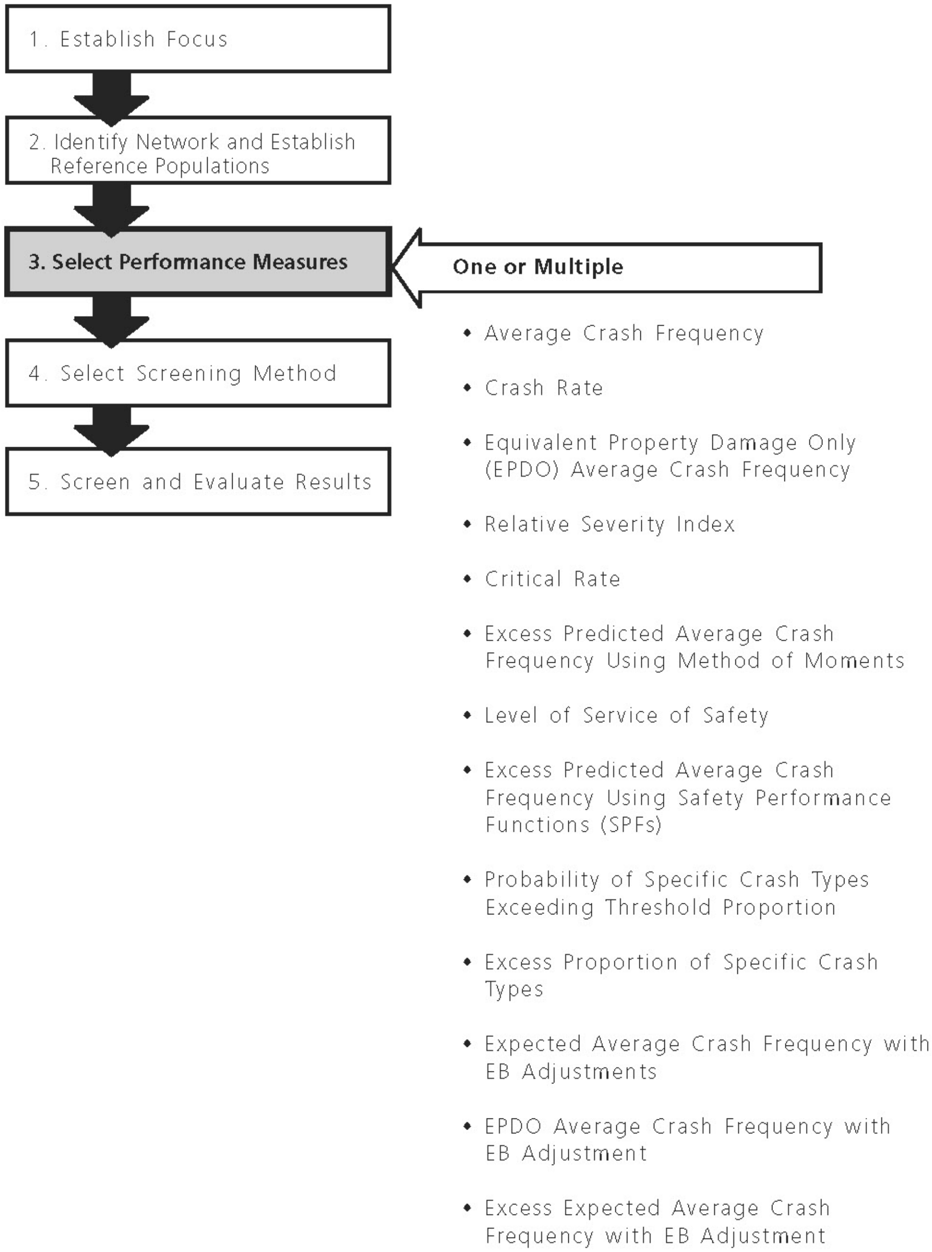


Figure 4-4. The Network Screening Process—Step 3, Select Performance Measures

Key Criteria for Selecting Performance Measures

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

Data and Input Availability

Typical data required for the screening analysis includes the facility information for establishing reference populations, crash data, traffic volume data, and, in some cases, safety performance functions. The amount of data and inputs that are available limits the number of performance measures that can be used. If traffic volume data is not available or cost prohibitive to collect, fewer performance measures are available for ranking sites. If traffic volumes are collected or made available, but calibrated safety performance functions and overdispersion parameters are not, the network could be prioritized using a different set of performance measures. Table 4-1 summarizes the data and inputs needed for each performance measure.

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

	Data and Inputs				
Performance Measure	**Crash Data**	**Roadway Information for Categorization**	**Traffic Volume[a]**	**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[b]	X	X	X		
Level of Service of Safety	X	X	X	X	
Excess Predicted Average Crash Frequency Using Safety Performance Functions (SPFs)	X	X	X	X	
Probability of Specific Crash Types Exceeding Threshold Proportion	X	X			
Excess Proportion of Specific Crash Types	X	X			
Expected Average Crash Frequency with EB Adjustment	X	X	X	X	
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment	X	X	X	X	EPDO Weighting Factors
Excess Expected Average Crash Frequency with EB Adjustment	X	X	X	X	

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

Regression-to-the-Mean Bias

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

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

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

Performance Threshold

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

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

Table 4-2. Stability of Performance Measures

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

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

Average Crash Frequency

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

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.

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

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 the HSM to demonstrate use of the performance measure.

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

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

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.

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

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.

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

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.

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

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.

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

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

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

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).

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

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

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

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

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

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

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.

The following summarizes the 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

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

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

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

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

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

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

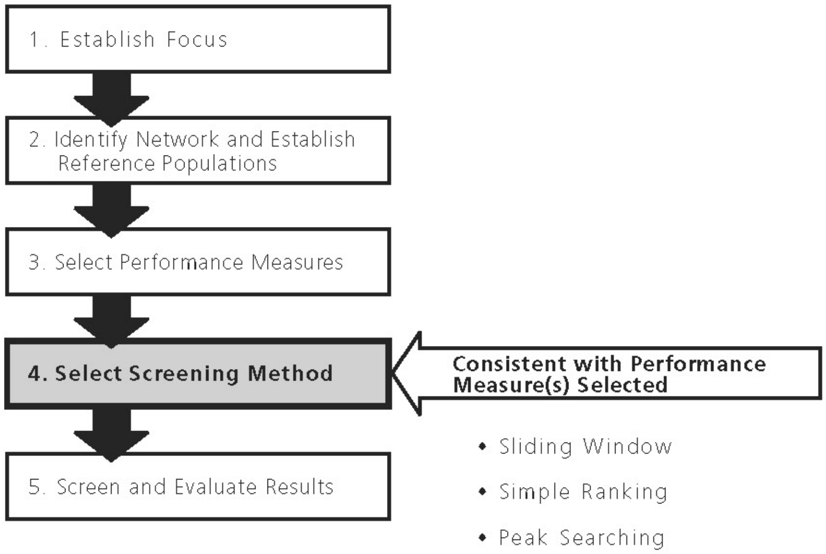


Figure 4-5. Network Screening Process—Step 4, Select Screening Method

Segment Screening Methods

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

Sliding Window Method

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

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

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

Sliding Window Method

Question

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

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

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

Example Application of Sliding Window Method

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

Answer

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

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

Peak Searching Method

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

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

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

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

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

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

Peak Searching Method

Question

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

Answer

Iteration #1

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

The variance is given as:

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

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

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

Example Application of Expected Average Crash Frequency with Empirical Bayes Adjustment (Iteration #1)

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

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

Iteration #2
The following shows the results of the second iteration. In the second iteration, the site is analyzed using 0.2-mi windows. For each window, the performance measure is calculated along with the CV.

Example Application of Expected Average Crash Frequency with Empirical Bayes Adjustment (Iteration #2)

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

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

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

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

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

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

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

Table 4-3 summarizes the performance measures that are consistent with the screening methods.

Table 4-3. Performance Measure Consistency with Screening Methods

	Segments			Nodes	Facilities
Performance Measure	**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 performance measure and the screening method are applied to one or more of the segments, nodes, or facilities according to the methods outlined in Steps 3 and 4. Conceptually, for each segment or node under consideration, the selected performance measure is calculated and recorded (see Figure 4-6). Results can be recorded in a table or on maps as appropriate or feasible.

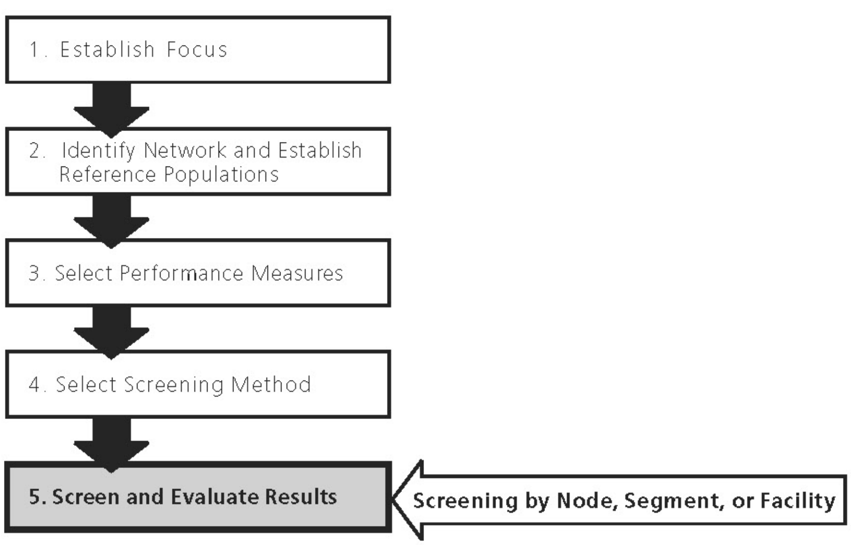


Figure 4-6. Optional Methods for Network Screening

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

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

4.3. SUMMARY

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

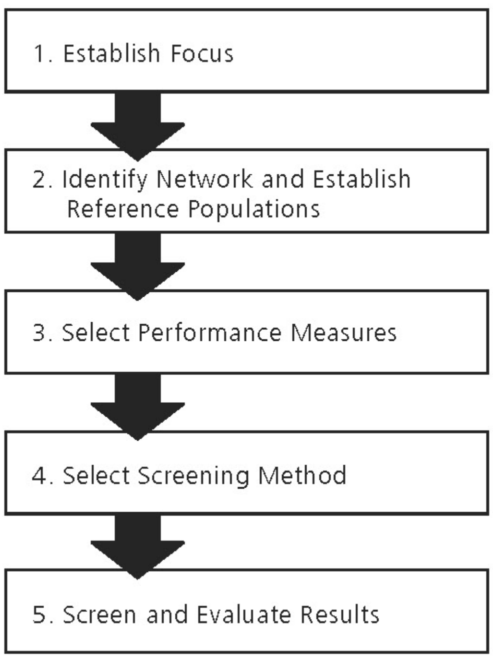


Figure 4-7. Network Screening Process

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

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

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

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;
- Thirteen are signalized intersections and 7 are unsignalized (two-way stop controlled) intersections;
- Major and Minor Street AADT volumes are provided in Table 4-4;
- A summary of crash data over the same three years as the traffic volumes is shown in Table 4-5; and
- Three years of detailed intersection crash data is shown in Table 4-6.

Assumptions

- The roadway agency has locally calibrated Safety Performance Functions (SPFs) and associated overdispersion parameters for the study intersections. Predicted average crash frequency from an SPF is provided in Table 4-6 for the sample intersections.
- The roadway agency supports use of FHWA crash costs by severity and type.

Intersection Characteristics and Crash Data

Tables 4-4 and 4-5 summarize the intersection characteristics and crash data.

Table 4-4. Intersection Traffic Volumes and Crash Data Summary

					Crash Data		
Intersections	Traffic Control	Number of Approaches	Major AADT	Minor AADT	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

Table 4-5. Intersection Detailed Crash Data Summary (3 Years)

		Crash Severity			Crash Type							
Intersections	Total	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

Table 4-6. 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	

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 or severity, or both. This method can be used to select an initial group of sites with high crash frequency for further analysis.

Data Needs

- Crash data by location

Strengths and Limitations

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

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 *Average Crash Frequency*

1 2

Count the number of crashes that occurred at each intersection.

STEP 2—Rank Locations *Average Crash Frequency*

1 2

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

Ranking of the 20 sample intersections is shown in the table. Column A shows the ranking by total crashes, Column B is the ranking by fatal and injury crashes, and Column C is the ranking by property damage-only crashes.

As shown in the table, ranking based on crash severity may lead to one intersection achieving a different rank depending on the ranking priority. The rank of Intersection 1 demonstrates this variation.

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

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

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 *Crash Rate*

1 2 3

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

$$MEV = \frac{TEV}{1{,}000{,}000} \times (n) \times (365) \tag{4-2}$$

Where:

MEV = Million entering vehicles

TEV = Total entering vehicles per day

n = Number of years of crash data

Total Entering Vehicles

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

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

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

Total Entering Vehicles

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 ***Crash Rate***

1 **2** 3

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.

$$R_i = \frac{N_{observed,\ i\ (total)}}{MEV_i} \tag{4-3}$$

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 Table 4-5.

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

Step 3—Rank Intersections ***Crash Rate***

1 2 **3**

Rank the intersections based on their crash rates.

This table summarizes the results from applying the crash rate method.

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 or PDO crashes, or both, above sites with no fatal crashes and a relatively high number of injury or PDO crashes, or both. In some applications, fatal and injury crashes are combined into one category of Fatal/Injury (FI) crashes to avoid overemphasizing fatal crashes. Fatal crashes are tragic events; however, the fact that they are fatal is often the outcome of factors (or a combination of factors) that is out of the control of the engineer and planner.

Data Needs

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

Strengths and Limitations

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

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 for Applying the EPDO Average Crash Frequency Performance Measure

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

The FHWA report *Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries*, prepared in October 2005, documented mean comprehensive societal costs by severity as listed in Table 4-7 (rounded to the nearest hundred dollars) (2). As of December 2008, this was the most recent FHWA crash cost information, although these costs represent 2001 values.

Appendix 4A includes a summary of crash costs and outlines a process to update monetary values to current year values.

Table 4-7. Societal Crash Cost Assumptions

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

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

The values in Table 4-7 were published in the FHWA study. A combined disabling (A), evident (B), and possible (C) injury crash cost was provided by FHWA to develop an average injury (A/B/C) cost. Injury crashes could also be subdivided into disabling injury, evident injury, and possible injury crashes depending on the amount of detail in the crash data and crash costs available for analysis.

STEP 1—Calculate EDPO Weights	***Equivalent Property Damage Only (EPDO) Average Crash Frequency***

1 2 **3**

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

The weighting factors are calculated as follows:

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

Where:

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

CC_y = Crash cost for crash severity, y

CC_{PDO} = Crash cost for PDO crash severity

As shown, a sample calculation for the injury (A/B/C) EPDO weight ($f_{inj(\text{weight})}$) is:

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

Therefore, the weighting factors for all crash severities are shown in the following table:

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	*Equivalent Property Damage Only (EPDO) Average Crash Frequency*

1 **2** 3

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

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

Where:

$f_{k(\text{weight})}$ = Fatal Crash Weight

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

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

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

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

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

STEP 3—Rank Locations ***Equivalent Property Damage Only (EPDO) Average Crash Frequency***

1 2 3

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

As shown, the calculation of EPDO Score for Intersection 7 is

Total EPDO $\text{Score}_7 = (542 \times 1) + (11 \times 17) + (1 \times 16) = 745$

The number of fatal, injury, and PDO crashes for each intersection were shown in the example box in Section 4.4.2.1. The table below summarizes the EPDO score.

The calculation is repeated for each intersection.

The ranking for the 20 intersections is based on EPDO method. The results of calculations for Intersection 7 are highlighted.

Sample EPDO Ranking

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

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

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 Table 4-8 are used to calculate the average RSI cost for each intersection and the average RSI cost for each population. The values shown represent 2001 dollar values and are rounded to the nearest hundred dollars. Appendix 4A provides a method for updating crash costs to current year values.

Table 4-8. Crash Cost Estimates by Crash Type

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 ***Relative Severity Index (RSI)***

1 2 3 4

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

The RSI crash cost per crash type is calculated for each location under consideration. The following example contains the detailed summary of the crashes by type at each intersection.

This table summarizes the number of crashes by crash type at Intersection 7 over the last three years and the corresponding RSI costs for each crash type.

Intersection 7 Relative Severity Index Costs

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 *Relative Severity Index (RSI)*

1 2 3 4

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.

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

Where:

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

RSI_j = RSI cost for each crash type, j

$N_{\text{observed},i}$ = Number of observed crashes at the site i

The RSI calculation for Intersection 7 is as follows:

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

STEP 3—Calculate the Average RSI Cost for Each Population *Relative Severity Index (RSI)*

1 2 3 4

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

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

Where:

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

RSI_i = Total RSI cost at site i

$N_{\text{observed},i}$ = 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 Table 4-8. The following table summarizes the information needed to calculate the average RSI cost for the 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 ***Relative Severity Index (RSI)***

1 2 3 4

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.

The following table shows the intersection ranking for all 20 intersections based on their average RSI costs. The RSI costs for Intersection 7 would be compared to the average RSI cost for the unsignalized intersection population. In this instance, the average RSI cost for Intersection 7 ($31,700) is less than the average RSI cost for all unsignalized intersections ($39,700 from calculations in Step 3).

Ranking Based on Average RSI Cost per Intersection

Intersection	Average RSI Cost[a]	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	

[a]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.

Data Needs

- Crashes by location
- Traffic Volume

Strengths and Limitations

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

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 percent confidence level. Other possible confidence levels, based on a Poisson distribution and one-tailed standard normal random variable, are shown in Table 4-9.

Table 4-9. Confidence Levels and P Values for Use in Critical Rate Method

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 *Critical Rate*

1 2 3 4 5

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.

$$MEV = \left(\frac{TEV}{1{,}000{,}000}\right) \times (n) \times (365) \tag{4-8}$$

Where:

MEV = Million entering vehicles

TEV = Total entering vehicles per day

n = Number of years of crash data

Shown below is the calculation for the MEV of Intersection 7. The TEV is found in Table 4-4.

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

STEP 2—Calculate the Crash Rate for Each Intersection ***Critical Rate***

1 2 3 4 5

Calculate the crash rate for each intersection by dividing the number of crashes by MEV, as shown in Equation 4-9.

$$R_i = \frac{N_{observed,i(total)}}{MEV_i} \quad (4\text{-}9)$$

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 Table 4-5, and the MEV is noted in Step 1.

$$R_i = \frac{34}{24.1} = 1.41 \text{ [crashes/MEV]}$$

STEP 3—Calculate Weighted Average Crash Rate per Population ***Critical Rate***

1 2 3 4 5

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.

$$R_a = \frac{\sum_{i=1}(TEV_i \times R_i)}{\sum_{i=1}(TEV_i)} \quad (4\text{-}10)$$

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 the following table:

Two-Way Stop Controlled	Crash Rate	Weighted Average Crash Rate
2	2.42	
3	1.12	
7	1.41	
10	0.94	1.03
15	0.59	
17	0.67	
19	0.56	
Signalized	**Crash Rate**	**Weighted Average Crash Rate**
1	0.58	
4	0.54	
5	0.28	
6	0.23	
8	0.18	
9	0.61	
11	0.79	0.42
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 ***Critical Rate***

1 2 3 4 5

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:

$$R_{c,7} = 1.03 + \left[1.645 \times \sqrt{\left(\frac{1.03}{24.1}\right)}\right] + \left[\frac{1}{(2 \times (24.1)}\right] = 1.40 \text{ [crashes/MEV]}$$

STEP 5—Compare Observed Crash Rate with Critical Crash Rate ***Critical Rate***

1 2 3 4 **5**

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.

The following table summarizes the results for all 20 intersections being screened by the roadway agency.

Critical Rate Method Results

Intersection	Observed Crash Rate (crashes/MEV)	Critical Crash Rate (crashes/MEV)	Identified for Further Review
1	0.58	0.60	
2	2.42	1.51	X
3	1.12	1.43	
4	0.54	0.66	
5	0.28	0.57	
6	0.23	0.60	
7	1.41	1.40	X
8	0.18	0.58	
9	0.61	0.56	X
10	0.94	1.45	
11	0.79	0.58	X
12	0.45	0.55	
13	0.24	0.65	
14	0.20	0.58	
15	0.59	1.36	
16	0.97	0.67	X
17	0.67	1.44	
18	0.79	0.66	X
19	0.56	1.44	
20	0.12	0.56	

4.4.2.6. Excess Predicted Average Crash Frequency Using Method of Moments

In the method of moments, a site's observed crash frequency is adjusted to partially account for regression to the mean. The adjusted observed average crash frequency is compared to the average crash frequency for the reference population to determine the potential for improvement (PI). The potential for improvement of all reference populations (e.g., signalized four-legged intersections, unsignalized three-legged intersections, urban, and rural, etc.) are combined into one ranking list as a basic multiple-facility network screening tool.

Data Needs

- Crashes by location
- Multiple reference populations

Strengths and Limitations

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

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 *Excess Predicted Average Crash Frequency Using Method of Moments*

1 2 3 4 5 6

Organize historical crash data of the study period based upon factors such as facility type, location, or other defining characteristics.

The intersections from Table 4-4 have been organized into two reference populations, as shown in the first table for two-way stop controlled intersections and in the second table for signalized intersections.

TWSC Reference Population

Intersection ID	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

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

Excess Predicted Average Crash Frequency Using Method of Moments

1 | **2** | 3 | 4 | 5 | 6

Sum the average annual observed crash frequency for each site in the reference population and divide by the number of sites.

$$N_{observed\ rp} = \frac{\sum_{i=1}^{n} N_{observed,i}}{n_{sites}} \tag{4-12}$$

Where:

$N_{observed\ rp}$ = Average crash frequency, per reference population

$N_{observed,i}$ = Observed crash frequency at site i

$n_{(sites)}$ = Number of sites per reference population

Calculate the 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
Excess Predicted Average Crash Frequency Using Method of Moments

1 2 **3** 4 5 6

Use Equation 4-13 to calculate variance. Alternatively, variance can be more easily calculated with common spreadsheet programs.

$$Var(N) = \frac{\sum_{i=1}^{n}\left(N_{\text{observed},i} - N_{\text{observed},rp}\right)^{(2)}}{n_{\text{sites}} - 1} \tag{4-13}$$

Where:

$Var(N)$ = Variance

$N_{observed,rp}$ = Average crash frequency, per reference population

$N_{\text{observed},i}$ = Observed crash frequency per year at site i

n_{sites} = Number of sites per reference population

Calculate the crash frequency variance calculation for the TWSC reference population:

$$S_{TWSC}{}^{2} = \frac{112.8}{6} = 18.8$$

The variance for signal and TWSC reference populations is shown in the following table:

	Crash Frequency	
Reference Population	**Average**	**Variance**
Signal	6.1	10.5
TWSC	7.1	18.8

STEP 4—Calculate Adjusted Observed Crash Frequency per Site
Excess Predicted Average Crash Frequency Using Method of Moments

1 2 3 **4** 5 6

Using the variance and average crash frequency for a reference population, find the adjusted observed crash frequency for each site using Equation 4-14.

$$N_{\text{observed},i(adj)} = N_{\text{observed},i} + \frac{N_{\text{observed},rp}}{Var(N)} \times \left(N_{\text{observed},rp} - N_{\text{observed},i}\right) \tag{4-14}$$

Where:

$N_{observed,i(adj)}$ = Adjusted observed number of crashes per year, per site

$Var(N)$ = Variance (equivalent to the square of the standard deviation, s^2)

$N_{observed,i}$ = Observed average crash frequency per year at site i

$N_{observed,rp}$ = Average crash frequency, per reference population

As shown, calculate the adjusted observed average crash frequency 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

Excess Predicted Average Crash Frequency Using Method of Moments

1 2 3 4 **5** 6

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

As shown below, calculate the potential for improvement for Intersection 7:

$PI_7 = 8.5 - 7.1 = 1.4$ [crashes per year]

STEP 6—Rank Sites According to PI ***Excess Predicted Average Crash Frequency Using Method of Moments***

1 2 3 4 5 **6**

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.

The PI rankings along with each site's adjusted observed crash frequency are as follows:

Intersections	Observed Average Crash Frequency	Adjusted Observed Crash Frequency	PI
11	12.7	9.8	3.6
9	12.3	9.6	3.4
12	10.7	8.6	2.5
2	11.7	8.6	1.4
7	11.3	8.5	1.4
1	7.3	6.8	0.7
16	7.0	6.6	0.5
3	7.7	7.3	0.2
18	6.3	6.2	0.1
10	5.7	6.7	–0.5
15	5.7	6.7	–0.5
5	5.0	5.5	–0.6
17	4.3	6.3	–0.9
4	4.3	5.1	–1.0
19	3.7	6.0	–1.1
14	3.3	4.6	–1.5
6	3.0	4.4	–1.7
8	3.0	4.4	–1.7
20	2.7	4.2	–1.9
13	2.0	3.8	–2.3

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

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

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 Table 4-6 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the base SPF model. It is also assumed that all CMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are to simplify this example and are rarely valid for application of the predictive method to actual field conditions.

STEP 1—Estimate Predicted Average Crash Frequency Using an SPF ***Level of Service of Safety (LOSS)***

1 2 3 4 5

Use the predictive method and SPFs outlined in Part C to estimate the average crash frequency. The predicted average crash frequency is summarized in Table 4-10:

Table 4-10. Estimated Predicted Average Crash Frequency from an SPF

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

STEP 2—Calculate Standard Deviation ***Level of Service of Safety (LOSS)***

1 2 3 4 5

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.

$$\sigma = \sqrt{k + N_{predicted}{}^2} \tag{4-16}$$

Where:

σ = Standard deviation

k = Overdispersion parameter of the SPF

$N_{predicted}$ = Predicted average crash frequency from the SPF

As shown, the standard deviation calculations for Intersection 7 are

$$\sigma = \sqrt{0.40 \times 2.6^2} = 1.6$$

The standard deviation calculation is performed for each intersection. The standard deviation for the TWSC intersections is summarized in the following table:

Intersection	Average Observed Crash Frequency	Predicted Average Crash Frequency from an SPF	Standard Deviation
2	11.7	1.7	1.1
3	7.7	2.2	1.4
7	11.3	2.6	1.6
10	5.7	2.2	1.4
15	5.7	2.3	1.5
17	4.3	2.6	1.6
19	3.7	2.5	1.6

STEP 3—Calculate Limits for LOSS Categories ***Level of Service of Safety (LOSS)***

1 2 **3** 4 5

Calculate the limits for the four LOSS categories for each intersection using the equations summarized in Table 4-11.

Table 4-11. LOSS Categories

LOSS	Condition	Description
I	$\sigma < N_{observed} < (N - 1.5 \times (\sigma))$	Indicates a low potential for crash reduction
II	$(N - 1.5 \times (\sigma)) \leq N_{observed} < N$	Indicates low to moderate potential for crash reduction
III	$N \leq N_{observed} (N + 1.5 \times (\sigma))$	Indicates moderate to high potential for crash reduction
IV	$N_{observed} \geq (N + 1.5 \times (\sigma))$	Indicates a high potential for crash reduction

This sample calculation for Intersection 7 demonstrates the upper limit calculation for LOSS III.

$N + 1.5 \times (\sigma) = 2.6 + 1.5 \times (1.6) = 5.0$

A similar pattern is followed for the other LOSS limits.

The values for this calculation are provided in the following table:

LOSS Limits for Intersection 7

Intersection	LOSS I Limits	LOSS II Limits	LOSS III Upper Limit	LOSS IV Limits
7	0 to 0.2	0.2 to 2.6	2.6 to 5.0	≥ 5.0

STEP 4—Compare Observed Crashes to LOSS Limits	*Level of Service of Safety (LOSS)*

1 2 3 **4** 5

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 5.0 crashes per year, Intersection 7 is categorized as Level IV.

STEP 5—Rank Intersections	*Level of Service of Safety (LOSS)*

1 2 3 4 **5**

List the intersections based on their LOSS for total crashes.

The following table summarizes the TWSC reference population intersection ranking based on LOSS:

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

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

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 Table 4-6 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the SPF. It is also assumed that all CMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the SPF. These assumptions are for theoretical application and are rarely valid for application of Part C predictive method to actual field conditions.

STEP 1—Summarize Crash History ***Excess Predicted Average Crash Frequency Using SPFs***

1 2 3 4

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 as an example in the following table:

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 *Excess Predicted Average Crash Frequency Using SPFs*

1 2 3 4

Using the predictive method in Part C, calculate the predicted average crash frequency, $N_{predicted,n}$, for each year, n, where $n = 1,2,\ldots,Y$. 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 the following table:

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
	1	1.7	0.6	1.1	
2	2	1.7	0.6	1.1	1.7
	3	1.8	0.7	1.1	
	1	2.1	0.8	1.3	
3	2	2.2	0.8	1.4	2.2
	3	2.2	0.9	1.4	
	1	2.5	1.0	1.6	
7	2	2.5	1.0	1.6	2.6
	3	2.7	1.1	1.7	
	1	2.1	0.8	1.3	
10	2	2.2	0.9	1.4	2.2
	3	2.2	0.9	1.4	
	1	2.5	1.0	1.6	
15	2	2.2	0.9	1.4	2.3
	3	2.1	0.8	1.3	
	1	2.5	1.0	1.5	
17	2	2.6	1.0	1.6	2.6
	3	2.6	1.0	1.6	
	1	2.4	1.0	1.5	
19	2	2.5	1.0	1.5	2.5
	3	2.6	1.0	1.6	

STEP 3—Calculate Excess Predicted Average Crash Frequency *Excess Predicted Average Crash Frequency Using SPFs*

1 2 **3** 4

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.

$$Excess(N) = \overline{N_{\text{observed},i}} - \overline{N_{\text{predicted},i}} \tag{4-17}$$

Where:

$\overline{N_{\text{observed},i}}$ = Observed average crash frequency for site i

$\overline{N_{\text{predicted},i}}$ = Predicted average crash frequency from SPF for site.

Shown below is the predicted excess crash frequency calculation for Intersection 7:

$Excess_{(TWSC)} = 11.3 - 2.6 = 8.7$ [crashes per year]

The following table shows the excess expected average crash frequency for the TWSC reference population:

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 *Excess Predicted Average Crash Frequency Using SPFs*

1 2 3 **4**

Rank all sites in each reference population according to the excess predicted average crash frequency.

The following table ranks the TWSC intersections according to the excess predicted average crash frequency:

Ranking of TWSC Population Based on Excess Predicted Average Crash Frequency from an SPF

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

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

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): (1) TWSC intersections and (2) signalized intersections.

STEP 1—Calculate Observed Proportions	*Probability of Specific Crash Types Exceeding Threshold Proportion*

1 2 3 4 5 6

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.

$$p_i = \frac{N_{observed,i}}{N_{observed,i(total)}} \tag{4-18}$$

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 Table 4-5.

$$p_i = \frac{5}{34} = 0.15$$

STEP 2—Estimate a Threshold Proportion	*Probability of Specific Crash Types Exceeding Threshold Proportion*

1 **2** 3 4 5 6

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.

$$p^*_i = \frac{\sum N_{observed,i}}{\sum N_{observed,i(total)}} \tag{4-19}$$

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.

$$p^*_i = \frac{33}{150} = 0.22$$

The following table summarizes the threshold proportions for the reference populations:

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 ***Probability of Specific Crash Types Exceeding Threshold Proportion***

1 2 3 4 5 6

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:

$$Var(N) = \left(\frac{1}{n_{sites}-1}\right) \times \left[\sum_{i=1}^{n}\left(\frac{N_{observed,i}^{2} - N_{observed,i}}{N_{observed,i(total)}^{2} - N_{observed,i(total)}}\right) - \left(\frac{1}{n_{sites}}\right) \times \left(\sum_{i=1}^{n}\frac{N_{observed,i}}{N_{observed,i(total)}}\right)^{2}\right] \tag{4-20}$$

for $N_{observed,i(total)} \geq 2$

Where:

n_{sites} = Total number of sites being analyzed

$N_{observed,i}$ = Observed target crashes for a site i

$N_{observed,i(total)}$ = Total number of crashes for a site i

The following table summarizes the calculations for the two-way stop-controlled subcategory. TWSC sites 15 and 19 were removed from the variance calculation because fewer than two angle crashes were reported over the study period.

Sample Variance Calculation

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	5	25	34	1156		
3	2	4	23	529	5	0.037
10	2	4	17	289		
17	2	4	13	169		

STEP 4—Calculate Alpha and Beta Parameters	*Probability of Specific Crash Types Exceeding Threshold Proportion*

1 2 3 **4** 5 6

Calculate the sample mean proportion of target crashes by type or severity for all sites under consideration using Equation 4-21.

$$\overline{p_i^*} = \frac{\sum p_i}{n_{\text{sites}}}, N_{\text{observed},i} \geq 2 \qquad (4\text{-}21)$$

Where:

n_{sites} = Total number of sites being analyzed

$\overline{p^*_i}$ = Mean proportion of target crash types

p_i = Observed proportion

Calculate Alpha (α) and Beta (ß) for each subcategory using Equations 4-22 and 4-23.

$$\alpha = \frac{\overline{p^*_i}^2 - \overline{p^*_i}^3 - s^2(\overline{p^*_i})}{Var(N)} \qquad (4\text{-}22)$$

$$\beta = \frac{\alpha}{\overline{p^*_i}} - \alpha \qquad (4\text{-}23)$$

Where:

$Var(N)$ = Variance (equivalent to the square of the standard deviation, s^2)

$\overline{p^*_i}$ = Mean proportion of target crash types

The calculation for the two-way stop-controlled subcategory is:

$$\alpha = \frac{0.22^2 - 0.22^3 - 0.037 \times 0.22}{0.037} = 0.80$$

$$\beta = \frac{0.80}{0.22} - 0.80 = 2.84$$

The following table shows the numerical values used in the equations and summarizes the alpha and beta calculations for the TWSC intersections:

Alpha and Beta Calculations

Subcategories	s^2	$\overline{p^*_i}$	α	ß
TWSC	0.034	0.22	0.91	3.2

STEP 5—Calculate the Probability *Probability of Specific Crash Types Exceeding Threshold Proportion*

1 2 3 4 **5** 6

Using a "betadist" spreadsheet function, calculate the probability for each intersection as shown in Equation 4-24.

$$p\left(\frac{p_i > p_i^*}{N_{\text{observed},i}, N_{\text{observed},i(\text{total})}}\right) = 1 - betadist(p_i^*, \alpha + N_{\text{observed},i}, \beta + N_{\text{observed},i(\text{total})} - N_{\text{observed},i}) \quad (4\text{-}24)$$

Where:

p^*_i = Threshold proportion

p_i = Observed proportion

$N_{\text{observed},i}$ = Observed target crashes for a site i

$N_{\text{observed},i(\text{total})}$ = Total number of crashes for a site i

The probability calculation for Intersection 7 is:

$$p\left(\frac{p_i > p_i^*}{N_{\text{observed},i}, N_{\text{observed},i(\text{total})}}\right) = 1 - betadist(0.22, 0.80 + 5, 2.84 + 34 - 5)$$

The following table summarizes the probability calculation for Intersection 7:

Probability Calculations

TWSC	Angle Crashes ($N_{\text{observed},i}$)	Total Crashes ($N_{\text{observed},i(\text{total})}$)	p_i	p^*_i	α	ß	Probability
7	5	34	0.15	0.22	0.80	2.84	0.13

For Intersection 7, the resulting probability is interpreted as "There is a 13 percent chance that the long-term expected proportion of angle crashes at Intersection 7 is actually greater than the long-term expected proportion for TWSC intersections." Therefore, in this case, with such a small probability, there is limited need of additional study of Intersection 7 with regards to angle crashes.

STEP 6—Rank Locations *Probability of Specific Crash Types Exceeding Threshold Proportion*

1 2 3 4 5 **6**

Rank the intersections based on the probability of angle crashes occurring at the intersection.

The TWSC intersection population is ranked based on the Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure as shown in the following table:

Ranking Based on Probability of Specific Crash Types Exceeding Threshold Proportion Performance Measure

Intersections	Probability
2	1.00
11	0.99
9	0.81
12	0.71
16	0.36
6	0.35
13	0.35
20	0.26
17	0.25
4	0.20
7	0.13
10	0.13
5	0.08
1	0.08
18	0.07
3	0.04

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

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

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

Procedure

Calculation of the excess proportion follows the same procedure outlined in Steps 1 through 5 of the Probability of Specific Crash Types Exceeding Threshold Proportions method. Therefore, the procedure outlined in this section builds on the previous method and applies results of sample calculations shown above in the example table of Step 6.

For the sample situation, the limiting probability is selected to be 60 percent. The selection of a limiting probability can vary depending on the probabilities of each specific crash types exceeding a threshold proportion. For example, if many sites have high probability, the limiting probability can be correspondingly higher in order to limit the number of sites to a reasonable study size. In this example, a 60 percent limiting probability results in four sites that will be evaluated based on the Excess Proportions performance measure.

STEP 6—Calculate the Excess Proportion *Excess Proportion of Specific Crash Types*

1 2 3 4 **6** 5 7

Calculate the difference between the true observed proportion and the threshold proportion for each site using Equation 4-25:

$$P_{diff} = p_i - p_i^* \quad (4\text{-}25)$$

Where:

p^*_i = Threshold proportion

p_i = Observed proportion

STEP 7—Rank Locations *Excess Proportion of Specific Crash Types*

1 2 3 4 5 6 **7**

Rank locations in descending order by the value of Pdiff. The greater the difference between the observed and threshold proportion, the greater the likelihood that the site will benefit from a countermeasure targeted at the collision type under consideration.

The four intersections that met the limiting probability of 60 percent are ranked in the following table:

Ranking Based on Excess Proportion

Intersections	Probability	Observed Proportion	Threshold Proportion	Excess Proportion
2	1.00	0.60	0.22	0.38
11	0.99	0.61	0.34	0.27
9	0.81	0.46	0.34	0.12
12	0.71	0.44	0.34	0.10

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 Part C, Appendix A. The version of the EB method implemented here uses yearly correction factors for consistency with network screening applications in the SafetyAnalyst software tools.

Data Needs

- Crash data by severity and location
- Traffic volume
- Basic site characteristics (i.e., roadway cross-section, intersection control, etc.)
- Calibrated Safety Performance Functions (SPFs) and overdispersion parameters

Strengths and Limitations

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

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

Procedure

The following sample problem outlines the assumptions and procedure for ranking intersections based on the expected average crash frequency with Empirical Bayes adjustments. The calculations for Intersection 7 are used throughout the sample problems to highlight how to apply each method.

Sample Problem Assumptions

The sample problems provided in this section are intended to demonstrate calculation of the performance measures, not predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using predictive method outlined in Part C and are provided in Table 4-6 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the SPF. It is also assumed that all CMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are for theoretical application and are rarely valid for application of the Part C predictive method to actual field conditions.

STEP 1—Calculate the Predicted Average Crash Frequency from an SPF

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

1 2 3 4 5 6 7

Using the predictive method in Part C calculate the predicted average crash frequency, $N_{predicted,n}$, for each year, n, where $n = 1,2,\ldots,Y$. 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

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

1	**2**	3	4	5	6	7

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)

$$C_{n(\text{total})} = \frac{N_{\text{predicted},n(\text{total})}}{N_{\text{predicted},1(\text{total})}} \quad \text{and} \quad C_{n(FI)} = \frac{N_{\text{predicted},n(FI)}}{N_{\text{predicted},1(FI)}} \tag{4-26}$$

Where:

$C_{n(\text{total})}$ = Annual correction factor for total crashes

$C_{n(FI)}$ = Annual correction factor for fatal or injury crashes, or both

$N_{\text{predicted},\,n(\text{total})}$ = Predicted number of total crashes for year *n*

$N_{\text{predicted},1(FI)}$ = Predicted number of fatal or injury crashes, or both, 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 the table that follows.

$$C_{3(\text{total})} = \frac{2.7}{2.5} = 1.1$$

$$C_{3(FI)} = \frac{1.1}{1.0} = 1.1$$

This calculation is repeated for each year and each intersection. The following table summarizes the annual correction factor calculations for the TWSC intersections:

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
	1	2.4	1.0	1.0	1.0
19	2	2.5	1.0	1.0	1.0
	3	2.6	1.0	1.1	1.0

STEP 3—Calculate Weighted Adjustment

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

1 2 **3** 4 5 6 7

Calculate the weighted adjustment, w, for each intersection and each severity (i.e., total and *FI*). The weighted adjustment accounts for the reliability of the safety performance function that is applied. Crash estimates produced using Safety Performance Functions with overdispersion parameters that are low (which indicates higher reliability) have a larger weighted adjustment. Larger weighting factors place a heavier reliance on the SPF estimate.

$$w_{\text{total}} = \frac{1}{1 + k_{\text{total}} \times \sum_{n=1}^{N} N_{\text{predicted},n(\text{total})}} \quad \text{and} \quad w_{FI} = \frac{1}{1 + k_{FI} \times \sum_{n=1}^{N} N_{\text{predicted},n(FI)}} \qquad (4\text{-}27)$$

Where:

w = Empirical Bayes weight

k = Overdispersion parameter of the SPF

$N_{\text{predicted, }n(\text{total})}$ = Predicted average total crash frequency from an SPF in year n

$N_{\text{predicted, }n(FI)}$ = 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 (7.7 and 3.1) is the result of summing the annual predicted crashes summarized in Step 2 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 the following table:

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

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

1 2 3 **4** 5 6 7

Calculate the base EB-adjusted expected average crash frequency for year 1, $N_{expected,1}$ using Equations 4-28 and 4-29.

This stage of the method integrates the observed crash frequency with the predicted average crash frequency from an SPF. The larger the weighting factor, the greater the reliance on the SPF to estimate the long-term predicted average crash frequency per year at the site. The observed crash frequency on the roadway segments is represented in the equations below as $N_{observed,n}$.

$$N_{expected,1(total)} = w_{total} \times N_{predicted,1(total)} + (1 - w_{total}) \times \left(\frac{\sum_{n=1}^{N} N_{observed,y(total)}}{\sum_{n=1}^{N} C_{n(total)}} \right) \quad (4\text{-}28)$$

and

$$N_{expected,1(FI)} = w_{FI} \times N_{predicted,1(FI)} + (1 - w_{FI}) \times \left(\frac{\sum_{n=1}^{N} N_{observed,y(FI)}}{\sum_{n=1}^{N} C_{n(FI)}} \right) \quad (4\text{-}29)$$

Where:

$N_{expected,1}$ = EB-adjusted estimated average crash frequency for year 1

w = Weight

$N_{predicted,i(total)}$ = 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 Steps 2 and 3.

$$N_{expected,1(total)} = 0.2 \times (2.5) + (1 - 0.2) \times \frac{34}{3.1} = 9.3$$

$$N_{expected,1(FI)} = 0.3 \times (1.0) + (1 - 0.3) \times \frac{18}{3.1} = 4.4$$

STEP 5—Calculate Final Year EB-adjusted Expected Average Crash Frequency

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

1 2 3 4 **5** 6 7

Calculate the EB-adjusted expected number of fatal and injury crashes and total crashes for the final year (in this example, the final year is year 3).

$$N_{expected,n(total)} = N_{expected,1(total)} \times C_{n(total)} \quad (4\text{-}30)$$

$$N_{expected,n(FI)} = N_{expected,1(FI)} \times C_{n(FI)} \quad (4\text{-}31)$$

Where:

$N_{expected,n}$ = EB-adjusted expected average crash frequency for final year

$N_{expected,1}$ = EB-adjusted expected average crash frequency for year 1

C_n = Annual correction factor for year, n

Shown below are the calculations for Intersection 7.

$$N_{expected,3(total)} = 9.3 \times (1.1) = 10.2$$

$$N_{expected,3(FI)} = 4.4 \times (1.1) = 4.8$$

$$N_{expected,3(PDO)} = N_{expected,3(total)} - N_{expected,3(FI)}$$

The following table summarizes the calculations for Intersection 7:

Year 3—EB-Adjusted Expected Average Crash Frequency[a]

	Fatal and/or Injury Crashes			Total Crashes			PDO Crashes
Intersection	$N_{E,1(FI)}$	$C_{3(FI)}$	$N_{E,3(FI)}$	$N_{E,1(total)}$	$C_{3(total)}$	$N_{E,3(total)}$	$N_{E,3(PDO)}$
7	4.4	1.1	4.8	9.3	1.1	10.2	5.4

[a] *E* = "expected" in the variables presented in this table

STEP 6—Calculate the Variance of the EB-Adjusted Average Crash Frequency (Optional)

Expected Average Crash Frequency with Empirical Bayes (EB) Adjustment

1 2 3 4 5 6 7

When using the peak searching method (or an equivalent method for intersections), calculate the variance of the EB-adjusted expected number of crashes for year n. Equation 4-32 is applicable to roadway segments and ramps, and Equation 4-33 is applicable to intersections.

$$Var(N_{\text{expected},n})_{\text{roadways}} = N_{\text{expected},n} \times \left(\frac{(1-w)}{L}\right) \times \frac{C_n}{\sum_{n=1}^{N} C_n} \quad (4\text{-}32)$$

$$Var\left(N_{\text{expected},n}\right)_{\text{intersections}} = N_{\text{expected},n} \times (1-w) \times \frac{C_n}{\sum_{n=1}^{n} C_n} \quad (4\text{-}33)$$

Shown below are the variation calculations for Year 3 at Intersection 7.

$$Var(N_{\text{expected},3(\text{total})})_{\text{intersections}} = 10.2 \times (1-0.2) \times \frac{1.1}{3.1} = 2.9$$

The following table summarizes the calculations for Year 3 at Intersection 7:

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

1 2 3 4 5 6 **7**

Rank the intersections based on the EB-adjusted expected average crash frequency for the final year in the analysis, as calculated in Step 5.

This table summarizes the ranking based on EB-Adjusted Crash Frequency for the TWSC Intersections.

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

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

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 Table 4-12 are used to calculate the EPDO weights.

Table 4-12. Societal Crash Cost Assumptions

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

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

Sample Problem Assumptions

The Sample problems provided in this section are intended to demonstrate calculation of the performance measures, not predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using predictive method outlined in Part C and are provided in Table 4-6 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the base SPF model. It is also assumed that all CMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are for theoretical application and are rarely valid for application of predictive method to actual field conditions.

STEP 1—Calculate Weighting Factors for Crash Severity

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

1	2	3	4	5	6	7	8	9	10

Calculate the EPDO weights for fatal, injury, and PDO crashes. The fatal and injury weights are calculated using Equation 4-34. The cost of a fatal or injury crash is divided by the cost of a PDO crash, respectively. Weighting factors developed from local crash cost data typically result in the most accurate results. If local information is not available, nationwide crash cost data is available from the Federal Highway Administration (FHWA). Appendix 4A provides information on the national data available and a method for updating crash costs to current dollar values.

The weighting factors are calculated as follows:

$$f_{y(\text{weight})} = \frac{CC_y}{CC_{PDO}} \tag{4-34}$$

Where:

$f_{y(\text{weight})}$ = 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):

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

Therefore, the EPDO weighting factors for all crash severities are shown in the following table:

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

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

1 2 3 4 5 6 7 8 9 10

Using the predictive method in Part C, calculate the predicted average crash frequency, $N_{predicted,n}$, for each year, n, where $n = 1, 2, \ldots, N$. Refer to Part C—Introduction and Applications Guidance for a detailed overview of the method to calculate the predicted average crash frequency. The example provided here is simplified to emphasize calculation of the performance measure, not the predictive method. The predicted average crash frequency from SPFs is summarized for the TWSC intersections for a three-year period in Table 4-13.

Calculations will have to be made for both total and Fatal/Injury crashes, or for Fatal/Injury and Property Damage Only crashes. This example calculates total and Fatal/Injury crashes, from which Property Damage Only crashes are derived.

Table 4-13. Estimated Predicted Average Crash Frequency from an SPF

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

STEP 3—Calculate Annual Correction Factors

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

1 2 3 4 5 6 7 8 9 10

Calculate the annual correction factors (C_n) at each intersection for each year and each severity using Equation 4-35.

The annual correction factor is predicted average crash frequency from an SPF for year y divided by the predicted average crash frequency from an SPF for year 1. This factor is intended to capture the effect that annual variations in traffic, weather, and vehicle mix have on crash occurrences (3).

$$C_{n(total)} = \frac{N_{predicted,n(total)}}{N_{predicted,n(total)}} \quad \text{and} \quad C_{n(FI)} = \frac{N_{predicted,n(FI)}}{N_{predicted,1(FI)}} \tag{4-35}$$

Where:

$C_{n(total)}$ = Annual correction factor for total crashes

$C_{n(FI)}$ = Annual correction factor for fatal and/or injury crashes

$N_{predicted,n(total)}$ = Predicted number of total crashes for year, n

$N_{predicted,1(total)}$ = 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 the following table:

Annual Correction Factors for all TWSC Intersections

Intersection	Year	Predicted Average Crash Frequency from an SPF (total)	Predicted Average Crash Frequency from an SPF (*FI*)	Correction Factor (total)	Correction Factor (*FI*)
	1	1.7	0.6	1.0	1.0
2	2	1.7	0.6	1.0	1.0
	3	1.8	0.7	1.1	1.2
	1	2.1	0.8	1.0	1.0
3	2	2.2	0.8	1.0	1.0
	3	2.2	0.9	1.0	1.1
	1	2.5	1.0	1.0	1.0
7	2	2.5	1.0	1.0	1.0
	3	2.7	1.1	1.1	1.1
	1	2.1	0.8	1.0	1.0
10	2	2.2	0.9	1.0	1.1
	3	2.2	0.9	1.0	1.1
	1	2.5	1.0	1.0	1.0
15	2	2.2	0.9	0.9	0.9
	3	2.1	0.8	0.8	0.8
	1	2.5	1.0	1.0	1.0
17	2	2.6	1.0	1.0	1.0
	3	2.6	1.0	1.0	1.0
	1	2.4	1.0	1.0	1.0
19	2	2.5	1.0	1.0	1.0
	3	2.6	1.0	1.1	1.0

STEP 4—Calculate Weighted Adjustment

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

1 2 3 **4** 5 6 7 8 9 10

Calculate the weighted adjustment, *w*, for each intersection and each severity. The weighted adjustment accounts for the reliability of the safety performance function that is applied. Crash estimates produced using safety performance functions with overdispersion parameters that are low (which indicates higher reliability) have a larger weighted adjustment. Larger weighting factors place a heavier reliance on the SPF to predict the long-term predicted average crash frequency per year at a site. The weighted adjustments are calculated using Equation 4-36.

$$w_{total} = \frac{1}{1 + k_{total} \times \sum_{n=1}^{N} N_{predicted,n(total)}} \quad \text{and} \quad w_{FI} = \frac{1}{1 + k_{FI} \times \sum_{n=1}^{N} N_{predicted,n(FI)}} \tag{4-36}$$

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 (7.7 and 3.1) is the result of summing the annual predicted crashes for Intersection 7 summarized in Step 3.

$$w_{total} = \frac{1}{1+(0.49 \times 7.7))} = 0.2$$

$$w_{FI} = \frac{1}{1+(0.74 \times 3.1))} = 0.3$$

The total and *FI* weights are summarized for the TWSC intersections in Step 5.

STEP 5—Calculate First Year EB-adjusted Expected Average Crash Frequency

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

1 2 3 4 **5** 6 7 8 9 10

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 4-37 and 4-38 below.

$$N_{expected,1(total)} = w_{total} \times N_{predicted,1(total)} + (1 - w_{total}) \times \left(\frac{\sum_{n=1}^{N} N_{observed,n(total)}}{\sum_{n=1}^{N} C_{n(total)}} \right) \tag{4-37}$$

and

$$N_{expected,1(FI)} = w_{FI} \times N_{predicted,1(FI)} + (1 - w_{FI}) \times \left(\frac{\sum_{n=1}^{N} N_{observed,n(FI)}}{\sum_{n=1}^{N} C_{n(FI)}} \right) \tag{4-38}$$

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.

$$N_{expected,1(total)} = 0.2 \times (2.5) + (1 - 0.2) \times \left(\frac{34}{3.1}\right) = 9.3$$

The following table summarizes the calculations for total crashes at Intersection 7.

Year 1—EB-Adjusted Number of Total Crashes

Intersection	$N_{predicted,1(total)}$	$w_{(total)}$	$N_{observed,n(total)}$ (All Years)	Sum of Total 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 Step 6.

STEP 6—Calculate Final Year EB-adjusted Average Crash Frequency
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

1	2	3	4	5	6	7	8	9	10

Calculate the EB-adjusted expected number of fatal and injury crashes and total crashes for the final year. Total and fatal and injury EB-adjusted expected average crash frequency for the final year is calculated using Equations 4-39 and 4-40, respectively.

$$N_{expected,n(total)} = N_{expected,1(total)} \times C_{n(total)} \quad (4\text{-}39)$$

$$N_{expected,n(FI)} = N_{expected,1(FI)} \times C_{n(FI)} \quad (4\text{-}40)$$

Where:

$N_{expected,,n}$ = 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 Step 3 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. The following table summarizes the results of Steps 4 through 6, including the EB-adjusted expected average crash frequency for all TWSC intersections:

EB-Adjusted Expected Average Crash Frequency for TWSC Intersections

Intersection	Year	Observed Number of Crashes (total)	Predicted Average Crash Frequency from an SPF (total)	Weight (total)	Weight (*FI*)	EB-Adjusted Expected Average Crash Frequency (total)	EB-Adjusted Expected Average Crash Frequency (*FI*)	EB-Adjusted Expected Average Crash Frequency (*PDO*)
	1	9.0	1.7			8.7	4.9	3.8
2	2	11.0	1.7	0.3	0.4	8.7	4.9	3.8
	3	15.0	1.8			9.6	5.8	3.8
	1	9.0	2.1			6.1	3.0	3.1
3	2	8.0	2.2	0.2	0.4	6.1	3.0	3.1
	3	6.0	2.2			6.1	3.3	2.8
	1	11.0	2.5			9.3	4.3	5.0
7	2	9.0	2.5	0.2	0.3	9.3	4.3	5.0
	3	14.0	2.7			10.2	4.8	5.4
	1	7.0	2.1			4.5	1.7	2.8
10	2	6.0	2.2	0.2	0.3	4.7	1.9	2.8
	3	4.0	2.2			4.5	1.9	2.6
	1	6.0	2.5			5.4	1.6	3.8
15	2	3.0	2.2	0.2	0.3	4.8	1.4	3.4
	3	8.0	2.1			4.3	1.3	3.0
	1	4.0	2.5			3.9	1.7	2.2
17	2	4.0	2.6	0.2	0.3	4.1	1.7	2.4
	3	5.0	2.6			3.9	1.7	2.2
	1	5.0	2.4			3.4	1.7	1.7
19	2	2.0	2.5	0.2	0.3	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
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

1	2	3	4	5	6	**7**	8	9	10

Equations 4-41 and 4-42 are used to identify the proportion of fatal crashes with respect to all non-PDO crashes in the reference population and injury crashes with respect to all non-PDO crashes in the reference population.

$$P_F = \frac{\sum N_{\text{observed},(F)}}{\sum N_{\text{observed},(FI)}} \tag{4-41}$$

$$P_I = \frac{\sum N_{\text{observed},(I)}}{\sum N_{\text{observed},(FI)}} \tag{4-42}$$

Where:

$N_{\text{observed},(F)}$ = Observed number of fatal crashes from the reference population;

$N_{\text{observed},(I)}$ = Observed number of injury crashes from the reference population;

$N_{\text{observed},(FI)}$ = 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
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

1	2	3	4	5	6	7	**8**	9	10

Compared to PDO crashes the relative EPDO weight of fatal and injury crashes is calculated using Equation 4-43.

$$w_{EPDO,FI} = P_F \times f_{K(\text{weight})} + P_I \times f_{inj(\text{weight})} \tag{4-43}$$

Where:

$f_{inj(\text{weight})}$ = EPDO injury weighting factor;

$f_{K(\text{weight})}$ = EPDO fatal 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 Step 1.

$w_{EPDO,FI} = (0.075 \times 542) + (0.925 \times 11) = 50.8$

STEP 9—Calculate the Final Year EPDO Expected Average Crash Frequency
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

1 2 3 4 5 6 7 8 **9** 10

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.

$$N_{\text{expected},3(EPDO)} = N_{\text{expected},n(PDO)} + w_{EPDO,FI} \times N_{\text{expected},n(FI)}$$

Shown below is the calculation for Intersection 7.

$N_{\text{expected},3(EPDO)} = 5.4 + 50.8 \times 4.8 = 249.2$

STEP 10—Rank Sites by EB-adjusted EPDO Score
Equivalent Property Damage Only (EPDO) Average Crash Frequency with EB Adjustment

1 2 3 4 5 6 7 8 9 **10**

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.

The following table summarizes the EB-Adjusted EPDO Ranking for the TWSC Intersections.

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. Part C Introduction and Applications Guidance, explains how to apply the EB Method. Intersections are ranked based on the difference between the predicted estimates and EB-adjusted estimates for each intersection, the excess expected average crash frequency per year.

Data Needs

- Crash data by severity and location
- Traffic volume
- Basic site characteristics (i.e., roadway cross-section, intersection control)
- Calibrated Safety Performance Functions (SPFs) and overdispersion parameters

Strengths and Limitations

The strengths and limitations of the Excess Expected Average Crash Frequency with EB Adjustments performance measure include the following:

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.

Table 4-14. Societal Crash Cost Assumptions

Crash Severity	Crash Cost
Combined Cost for Crashes with a Fatality or Injury, or Both (K/A/B/C)	$158,200
PDO (O)	$7,400

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

As shown in Table 4-14, the crash cost that can be used to weigh the expected number of FI crashes is $158,200. The crash cost that can be used to weigh the expected number of PDO crashes is $7,400. More information on crash costs, including updating crash cost values to current year of study values, is provided in Appendix 4A.

Sample Problem Assumptions

The sample problems provided in this section are intended to demonstrate calculation of the performance measures, not predictive method. Therefore, simplified predicted average crash frequency for the TWSC intersection population were developed using predictive method outlined in Part C and are provided in Table 4-6 for use in sample problems.

The simplified estimates assume a calibration factor of 1.0, meaning that there are assumed to be no differences between the local conditions and the base conditions of the jurisdictions used to develop the SPF. It is also assumed that all CMFs are 1.0, meaning there are no individual geometric design and traffic control features that vary from those conditions assumed in the base model. These assumptions are for theoretical application and are rarely valid for application of the Part C predictive method to actual field conditions.

Calculation of this performance measure follows Steps 1–5 outlined for the Expected Average Crash Frequency with EB Adjustments performance measure.

The results of Steps 1, 4, and 5 that are used in calculations of the excess expected average crash frequency are summarized in the following table:

Summary of Performance Measure Calculations for Steps 1, 4, and 5

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

Excess Expected Average Crash Frequency with EB Adjustments

1 2 3 4 5 **6** 7 8

The difference between the predicted estimates and EB-adjusted estimates for each intersection is the excess as calculated by Equation 4-45.

$$Excess_y = (N_{\text{expected},n(PDO)} - N_{\text{predicted},n(PDO)}) + (N_{\text{expected},n(FI)} - N_{\text{predicted},n(FI)}) \quad (4\text{-}45)$$

Where:

$Excess_y$ = Excess expected crashes for year, n

$N_{\text{expected},n}$ = EB-adjusted expected average crash frequency for year, n

$N_{\text{predicted},n}$ = 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 per year]

The calculations for all TWSC intersections are summarized in Step 8.

STEP 7—Calculate Severity Weighted Excess (Optional)
Excess Expected Average Crash Frequency with EB Adjustments

1 2 3 4 5 6 **7** 8

Calculate the severity weighted EB-adjusted excess expected crash value in dollars.

$$Excess_{(sw)} = (N_{\text{expected},n(PDO)} - N_{\text{predicted},n(PDO)}) \times CC_{(PDO)} + (N_{\text{expected},n(FI)} - N_{\text{predicted},n(FI)}) \times CC_{(FI)} \quad (4\text{-}46)$$

Where:

$Excess_{(sw)}$ = Severity weighted EB-adjusted expected excess crash value

$CC_{(Y)}$ = Crash cost for crash severity, Y

Shown below is the calculation for Intersection 7.

$Excess_{(sw)}$ = (5.4 –1.7) × \$7.400 + (4.8 – 1.1) × \$158,200 = \$612,720

The calculations for all TWSC intersections are summarized in Step 8.

STEP 8—Rank Locations ***Excess Expected Average Crash Frequency with EB Adjustments***

1 2 3 4 5 6 7 **8**

Rank the intersections based on either EB-adjusted expected excess crashes calculated in Step 6 or based on EB-adjusted severity weighted excess crashes calculated in Step 7. The first table shows the ranking of TWSC intersections based on the EB-adjusted expected excess crashes calculated in Step 6. The intersection ranking shown in the second table is based on the EB-adjusted severity weighted excess crashes calculated in Step 7.

Rankings according to calculations are as follows:

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

EB-Adjusted Severity Weighted Excess Crash Ranking

Intersection	$Excess_{(sw)}$ [a]
2	$826,800
7	$612,700
3	$390,000
10	$167,100
17	$115,200
19	$113,700
15	$91,700

[a] All $Excess_{(sw)}$ values rounded to the nearest hundred dollars.

4.4.3. Roadway Segments Performance Measure Sample Data

The Situation

A roadway agency is undertaking an effort to improve safety on their highway network. There are ten roadway segments from which the roadway agency wants to identify sites that will be studied in more detail because they show a potential for reducing the average crash frequency.

After reviewing the guidance in Section 4.2, the agency chooses to apply the sliding window method using the RSI performance measure to analyze each roadway segment. If desired, the agency could apply other performance measures or the peak searching method to compare results and confirm ranking.

The Facts

- The roadway segments are comprised of:
  - 1.2 mi of rural undivided two-lane roadway
  - 2.1 mi are undivided urban/suburban arterial with four lanes
  - 0.6 mi of divided urban/suburban two-lane roadway
- Segment characteristics and a three-year summary of crash data is in Table 4-15.
- Three years of detailed roadway segment crash data is shown in Table 4-16.

Assumptions

- The roadway agency has accepted the FHWA crash costs by severity and type as shown in Table 4-17.

Roadway Segment Characteristics and Crash Data

Tables 4-15 and 4-16 summarize the roadway segment characteristics and crash data.

Table 4-15. Roadway Segment Characteristics

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

Table 4-16. 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	Rollover	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

Table 4-17. 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
Rollover	$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

Sliding Window Procedure

The sliding window approach is one analysis method that can be applied when screening roadway segments. It consists of conceptually sliding a window of a specified length along the road segment in increments of a specified size. The method chosen to screen the segment is applied to each position of the window and the results of the analysis are recorded for each window. The window that shows the greatest potential for improvement is used to represent the total performance of the segment. After all segments are ranked according to the respective highest window value, those segments with the greatest potential for reduction in crash frequency or severity are studied in detail to identify potential countermeasures.

The following assumptions are used to apply the sliding window analysis technique in the roadway segment sample problems:

- Segment 1 extends from mile point 1.2 to 2.0
- The length of window in the sliding window analysis is 0.3 mi.
- The window slides in increments of 0.1 mi.

The name of the window subsegments and the limits of each subsegment are summarized in Table 4-18.

Table 4-18. Segment 1 Sliding Window Parameters

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 in Table 4-18 are the windows used to evaluate Segment 1 throughout the roadway segment sample problems. Therefore, whenever window subsegment 1a is referenced, it is the portion of Segment 1 that extends from mile point 1.2 to 1.5 and so forth.

Table 4-19 summarizes the crash data for each window subsegment within Segment 1. This data will be used throughout the roadway segment sample problems to illustrate how to apply each screening method.

Table 4-19. Segment 1 Crash Data per Sliding Window Subsegments

Window Subsegments	Total	Crash Severity			Crash Type			
		Fatal	Injury	PDO	Head-On	Sideswipe	Fixed Object	Rollover
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 ***Sliding Window Procedure***

1 **2** **3** **4**

For each window subsegment, multiply the average crash frequency for each crash type by their respective RSI crash type.

The following table summarizes the observed average crash frequency by crash type for each window subsegment over the last three years and the corresponding RSI crash costs for each crash type.

Crash Type Summary for Segment 1 Window Subsegments

Window Subsegments	Head-On	Sideswipe	Fixed Object	Rollover	Total[a]
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[b]					
1a	$0	$0	$284,100	$1,198,500	$1,482,600
1b	$375,100	$34,000	$284,100	$719,100	$1,412,300
1c	$1,125,300	$34,000	$0	$719,100	$1,878,400
1d	$375,100	$68,000	$473,500	$719,100	$1,635,700
1e	$0	$0	$94,700	$719,100	$813,800
1f	$375,100	$34,000	$284,100	$479,400	$1,172,600

[a] Crash types that were not reported to have occurred on Roadway Segment 1 were omitted from the table. The RSI costs for these crash types are zero.
[b] The values in this table are the result of multiplying the average crash frequency for each crash type by the corresponding RSI cost.

The calculation for Window Subsegment 1d is shown below.

Total RSI Cost = (1 × $375,100) + (2 × $34,000) + (5 × $94,700) + (3 × $239,700) = $1,635,700

STEP 2—Calculate Average RSI Cost per Subsegment *Sliding Window Procedure*

1 2 3 4

Sum the RSI costs for all crash types and divide by the total average crash frequency for the specific window subsegment as shown in Equation 4-47. The result is an Average RSI cost for each window subsegment.

$$\text{Average RSI Cost per Subsegment} = \frac{\text{Total RSI Cost}}{N_{observed,i(total)}} \tag{4-47}$$

Where:

$N_{observed,i(total)}$ = Total observed crashes at site, i

The calculation for Window Subsegment 1d is:

$$\textbf{Average RSI Cost} = \frac{\$1{,}635{,}700}{11} = \$148{,}700$$

The following table summarizes the Average RSI Crash Cost calculation for each window subsegment within Segment 1.

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 *Sliding Window Procedure*

1 2 3 4

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 ($\overline{RSI_p}$) is calculated using Equation 4-48.

$$\overline{RSI}_p = \frac{\sum_{i=1}^{n} RSI_i}{\sum_{i=1}^{n} N_{\text{observed},i}} \tag{4-48}$$

Where:

$\overline{RSI}_p$ = Average RSI cost for the population

RSI_i = RSI cost per site in the population

$N_{\text{observed},i}$ = Number of observed crashes in the population

The following example summarizes the information needed to calculate the average RSI cost for the population.

Average RSI Cost for Two-Lane Undivided Rural Highway Population

Roadway Segments	Angle	Head-On	Side-swipe	Pedestrian	Fixed Object	Rollover	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 subsegments within a segment.

$$\overline{RSI}_p = \frac{\sum_{i=1}^{n} RSI_i}{\sum_{i=1}^{n} N_{\text{observed},i}} = \frac{\$8,395,400 + \$5,979,900}{45 + 36} = \$177,500$$

STEP 4—Rank Locations and Compare ***Sliding Window Procedure***

1 2 3 **4**

Steps 1 and 2 are repeated for each roadway segment and Step 3 is repeated for each population. The roadway segments are ranked using the highest average RSI cost calculated for each roadway segment. For example, Segment 1 would be ranked using the highest average RSI cost shown in Step 2 from Window Subsegment 1c ($268,300). The highest average RSI cost for each roadway segment is also compared to the average RSI cost for the entire population. This comparison indicates whether or not the roadway segment's average RSI cost is above or below the average value for similar locations.

4.5. REFERENCES

(1) Allery, B., J. Kononov. Level of Service of Safety. *In Transportation Research Record 1840.* TRB, National Research Council, Washington, DC, 2003, pp. 57–66.

(2) Council, F., E. Zaloshnja, T. Miller, and B. Persaud. *Crash Cost Estimates by Maximum Police-Reported Injury Severity within Selected Crash Geometries.* FHWA-HRT-05-051. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, October 2005.

(3) Hauer, E. *Observational Before-After Studies in Road Safety.* Pergamon Press Inc., Oxford, UK, 1997.

(4) Kononov, J. *Use of Direct Diagnostics and Pattern Recognition Methodologies in Identifying Locations with Potential for Accident Reductions.* Transportation Research Board Annual Meeting CD-ROM. TRB, National Research Council, Washington, DC, 2002.

(5) Kononov, J. and B. Allery. Transportation Research Board Level of Service of Safety: Conceptual Blueprint and Analytical Framework. In *Transportation Research Record 1840.* TRB, National Research Council, Washington, DC, 2003, pp. 57–66.

(6) Midwest Research Institute. *White Paper for Module 1—Network Screening.* Federal Highway Administration, U.S. Department of Transportation, Washington, DC, 2002. Available from http://www.safetyanalyst.org/whitepapers.

(7) Ogden, K. W. *Safer Roads: A Guide to Road Safety Engineering.* Ashgate, Farnham, Surrey, UK, 1996.

APPENDIX 4A—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 U.S. DOT. 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. Tables 4A-1 and 4A-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.

Table 4A-1. Crash Cost Estimates by Crash Severity

Crash Type	Human Capital Crash Costs	Comprehensive Crash Costs
Fatal (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

Table 4A-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 Tables 4A-1 and 4A-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 U.S. Bureau of Labor Statistics (BLS) 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 (Tables 4A-1 and 4A-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 BLS. The recommended CPI can be found in the "all items" category of expenditures in the Average Annual Indexes tables of the BLS 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 follows.

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 *Crash Cost Annual Adjustment*

1 2 3 4

Multiply human capital costs by a ratio of the CPI for the year of interest divided by the CPI for 2001. Based on U.S. Bureau of Labor Statistics data, the CPI for year 2001 was 177.1 and in 2007 was 207.3 (1).

$$\textbf{CPI Ratio}_{(2001-2007)} = \frac{207.3}{177.1} = 1.2$$

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 the following table:

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 *Crash Cost Annual Adjustment*

1 2 3 4

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 Step 3.

STEP 3—Adjust the Difference Calculated in Step 2 Using the ECI *Crash Cost Annual Adjustment*

1 2 3 4

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 U.S. Bureau of Labor Statistics data the Employment Cost Index for year 2001 was 85.8 and in 2007 was 104.9 (2). 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]

The following table summarizes the 2007 ECT-adjusted crash costs:

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 *Crash Cost Annual Adjustment*

1 2 3 **4**

The 2007 CPI-adjusted costs (Step 2) and the 2007 ECI-adjusted cost differences (Step 3) are summed, as shown in the example below, 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]

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

4A.1. APPENDIX REFERENCES

(1) BLS. 2001 Consumer Price Index Detailed Report Tables. U.S. Bureau of Labor Statistics, Washington, DC, 20212. Available from http://www.bls.gov/cpi/cpi_dr.htm.

(2) BLS. *Employment Cost Index Historical Listing Current-Dollar March 2001–June 2008 (December 2005=100)*. U.S. Bureau of Labor Statistics, Office of Compensation Levels and Trends, Washington, DC, 20212-0001. Available from http://www.bls.gov/web/eci/echistrynaics.pdf.

(3) Council, F. M., E. Zaloshnja, T. Miller, and B. Persaud. *Crash Cost Estimates by Maximum Police Reported Injury Severity within Selected Crash Geometries.* FHWA-HRT-05-051. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, October 2005.

Chapter 5—Diagnosis

5.1. INTRODUCTION

Diagnosis is the second step in the roadway safety management process (Part B), as shown in Figure 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.

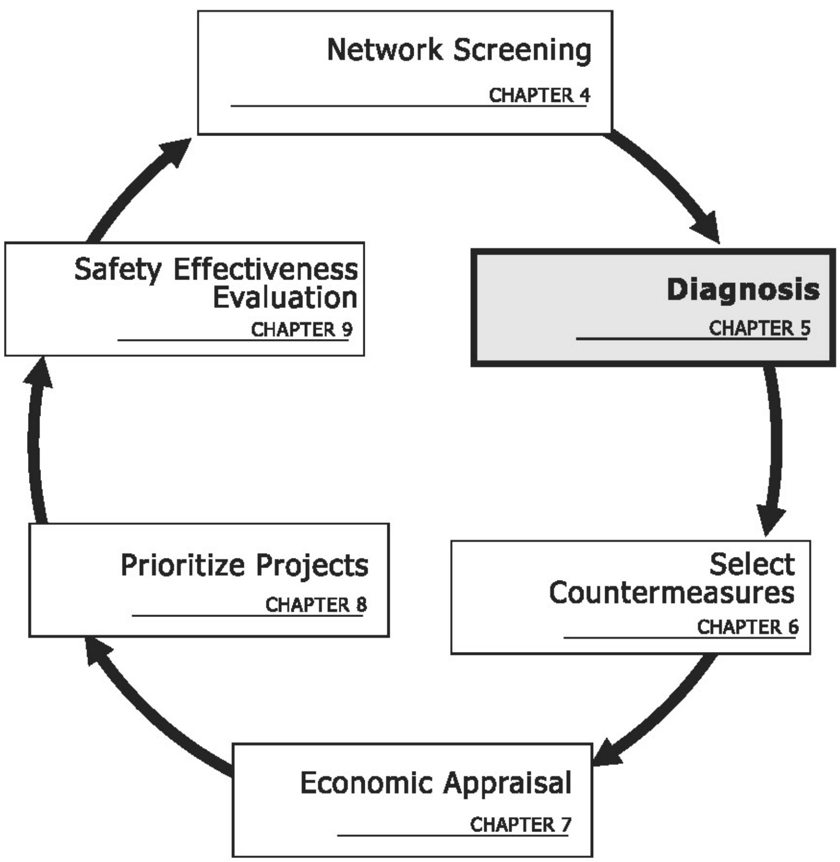


Figure 5-1. Roadway Safety Management Process Overview

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 although some steps may not apply to all projects:

- *Step 1*—Safety Data Review
  - 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 multimodal 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., one or more of the following: pavement, weather, 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, or roadway or environmental conditions); and
- Crash locations (i.e., collision diagrams, condition diagrams, and crash mapping using Geographic Information Systems (GIS) tools).

5.2.1. Descriptive Crash Statistics

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*—typically summarized according to the KABCO scale for defining crash severity (described in Chapter 3);
- 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 5A.

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. Figure 5-2 and Table 5-1 provide examples of graphical and tabular summaries of crash data.

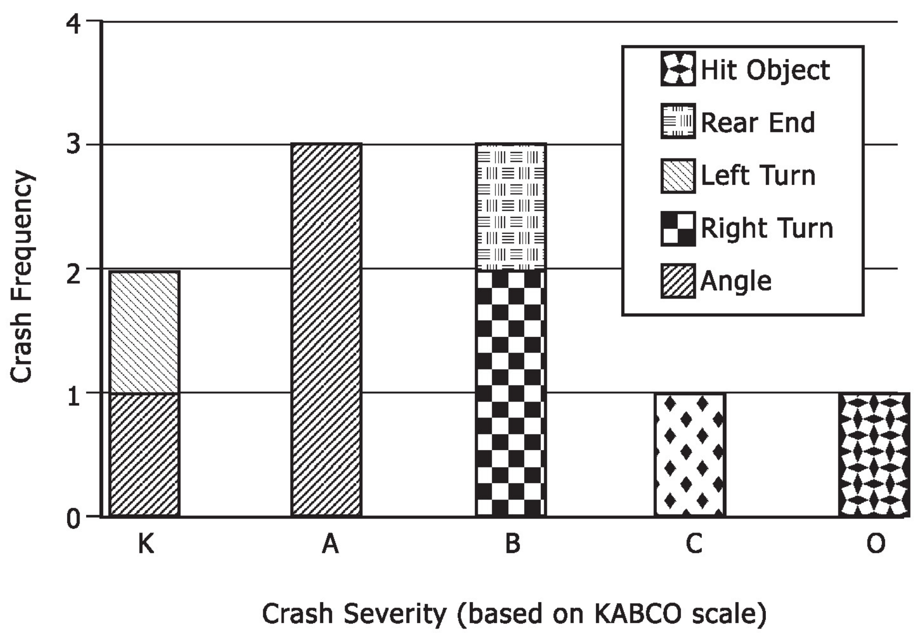


Figure 5-2. Example Graphical Summary

Table 5-1. Example Tabular Summary
(Adapted from Ogden (5))

Crash 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
Crash 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

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.

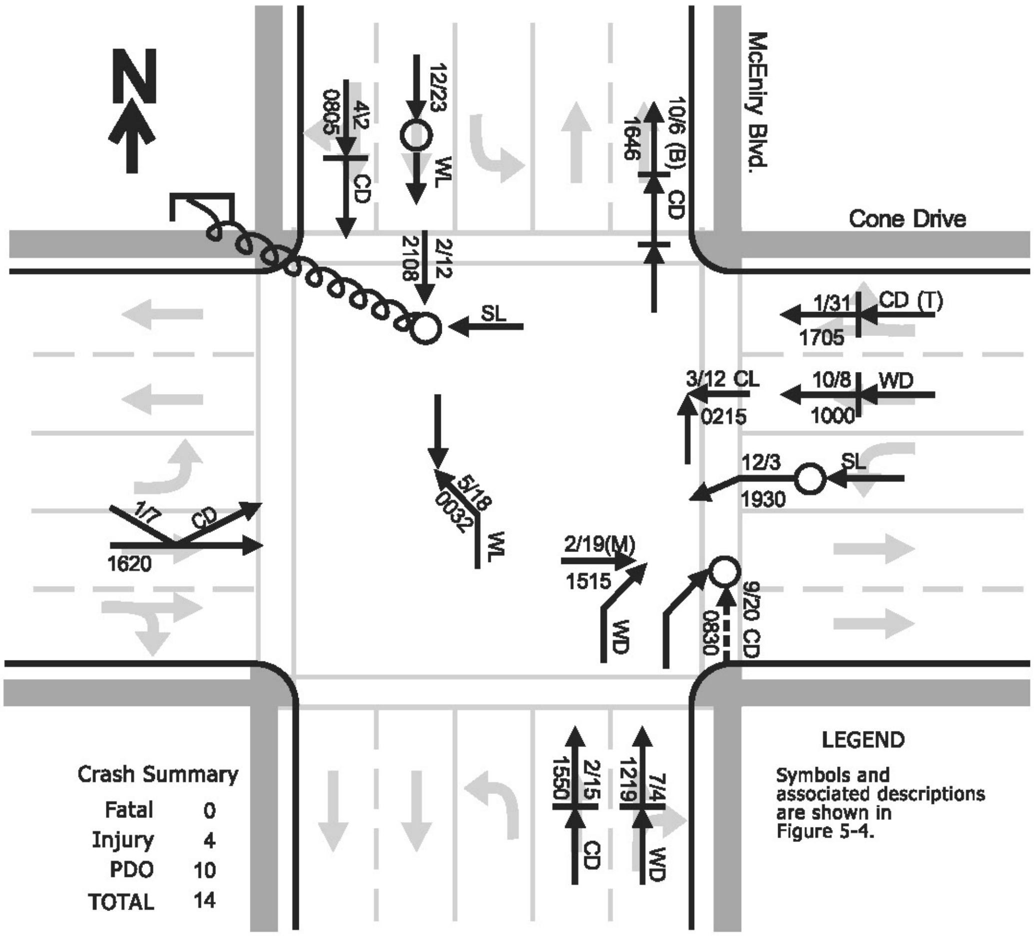
Collision Diagram

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.

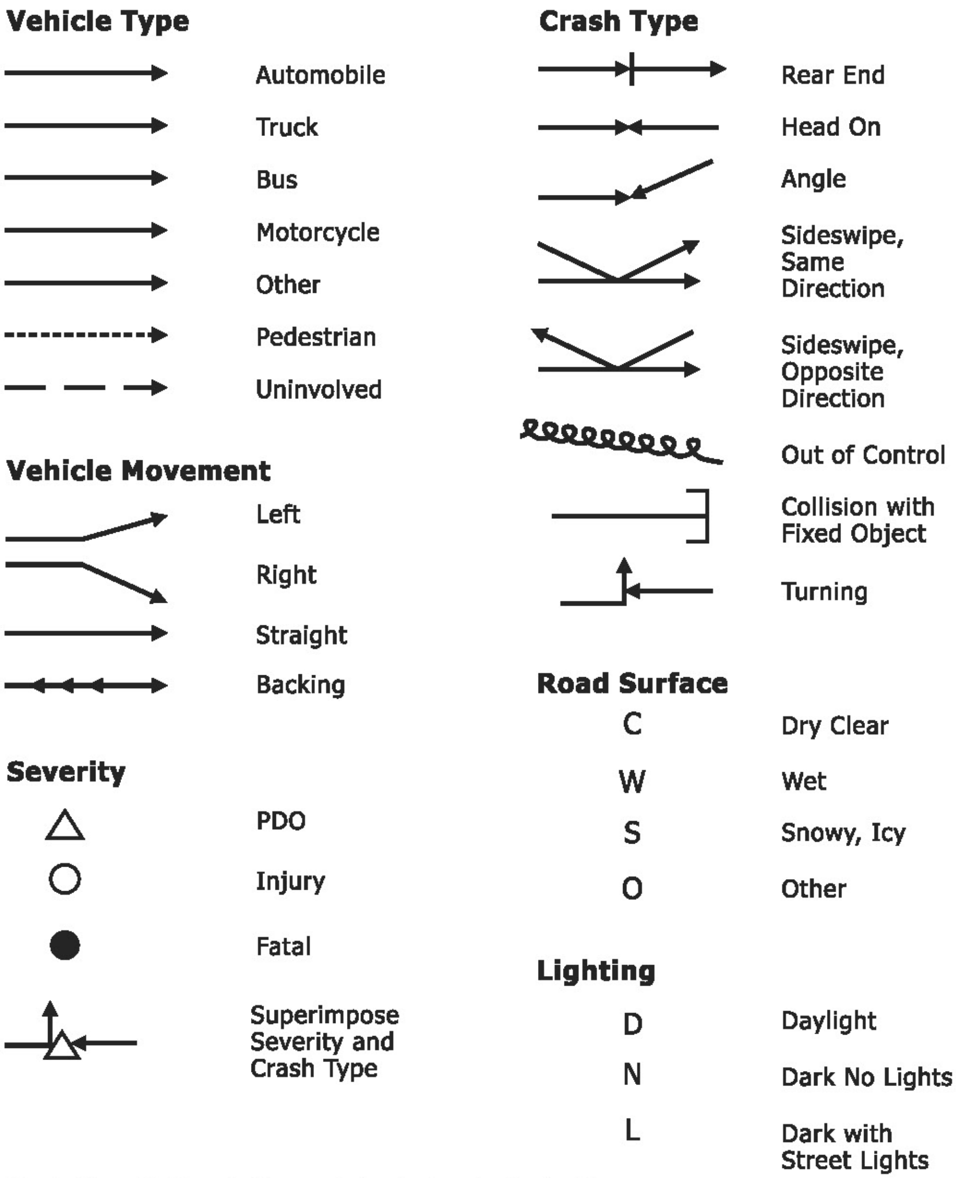
Figure 5-3 provides an example of a collision diagram. Crashes are represented on a collision diagram by arrows that indicate the type of crash and the direction of travel. Additional information associated with each crash is also provided next to each symbol. The additional information can be any of 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 Figure 5-4. These can be found in many safety textbooks and state transportation agency procedures.



Adapted from ITE *Manual of Transportation Engineering Studies* (4)

Figure 5-3. Example of an Intersection Collision Diagram



Adapted from ITE *Manual of Transportation Engineering Studies* (4)

Figure 5-4. Example Collision Diagram Symbols

Condition Diagram

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. Figure 5-5 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.

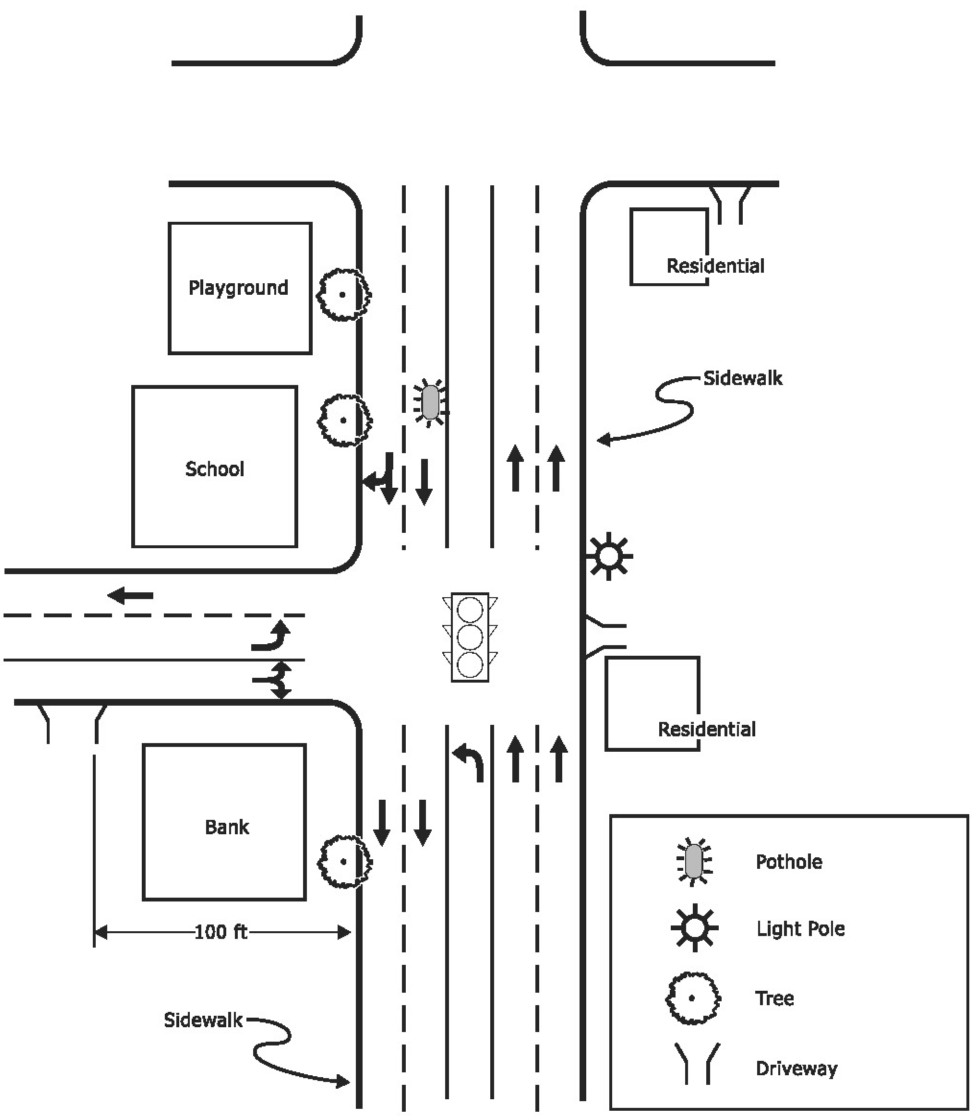


Figure 5-5. Example Condition Diagram

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 ft 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 or the increase in the number of lanes opposing a permitted left turn, or both.

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 or transportation studies, or both, 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 5B.

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 5C 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, although not exhaustive, provides several examples of useful considerations during a site review (1):

- Roadway and roadside characteristics:
  - Signing and striping
  - Posted speeds
  - Overhead lighting
  - Pavement condition
  - Landscape condition
  - 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, belted or unbelted;
  - *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, such as the following:
  - 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. Examples of prompt lists for different types of roadway environments are provided in Appendix 5D.

An assessment of field conditions is different from a road safety audit (RSA). An 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 an RSA are selected differently than those selected through the network screening process described in Chapter 4. An 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. Additional information and guidelines pertaining to RSAs are provided on the FHWA website (http://safety.fhwa.dot.gov/rsa/).

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 to 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 may 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:

- *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 5B 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 multimodal, multi-disciplinary perspective. Appendices 5C and 5D 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 Table 5-2.

Table 5-2. Sites Selected for Further Review

Intersection Number	Traffic Control	Number of Approaches	Major AADT	Minor AADT	Urban/Rural	Crash Totals Year 1	Crash Totals Year 2	Crash Totals 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 Number	**Cross-section (lanes)**	**Length (miles)**	**AADT**		**Undivided/Divided**	**Crash Totals Year 1**	**Crash Totals Year 2**	**Crash Totals 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 Table 5-3.
- 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 Table 5-2.
- The roadway cross-section and length is shown in Table 5-2.

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.

Table 5-3. Intersection Crash Data Summary

Intersection Number	Total	Crash Severity: Fatal	Injury	PDO	Crash Type: Rear-End	Sideswipe/ Overtaking	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

Table 5-4. Roadway Segment Crash Data Summary

Intersection Number	Total	Crash Severity: Fatal	Injury	PDO	Crash Type: Rear-End	Sideswipe/ Overtaking	Right Angle	Ped	Bike	Head-On	Fixed Object	Other
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

Figure 5-6 contains crash summary statistics for Intersection 2. Figure 5-7 illustrates the collision diagram for Intersection 2. Figure 5-8 is the condition diagram for Intersection 2. All three figures were generated and analyzed to diagnose Intersection 2.

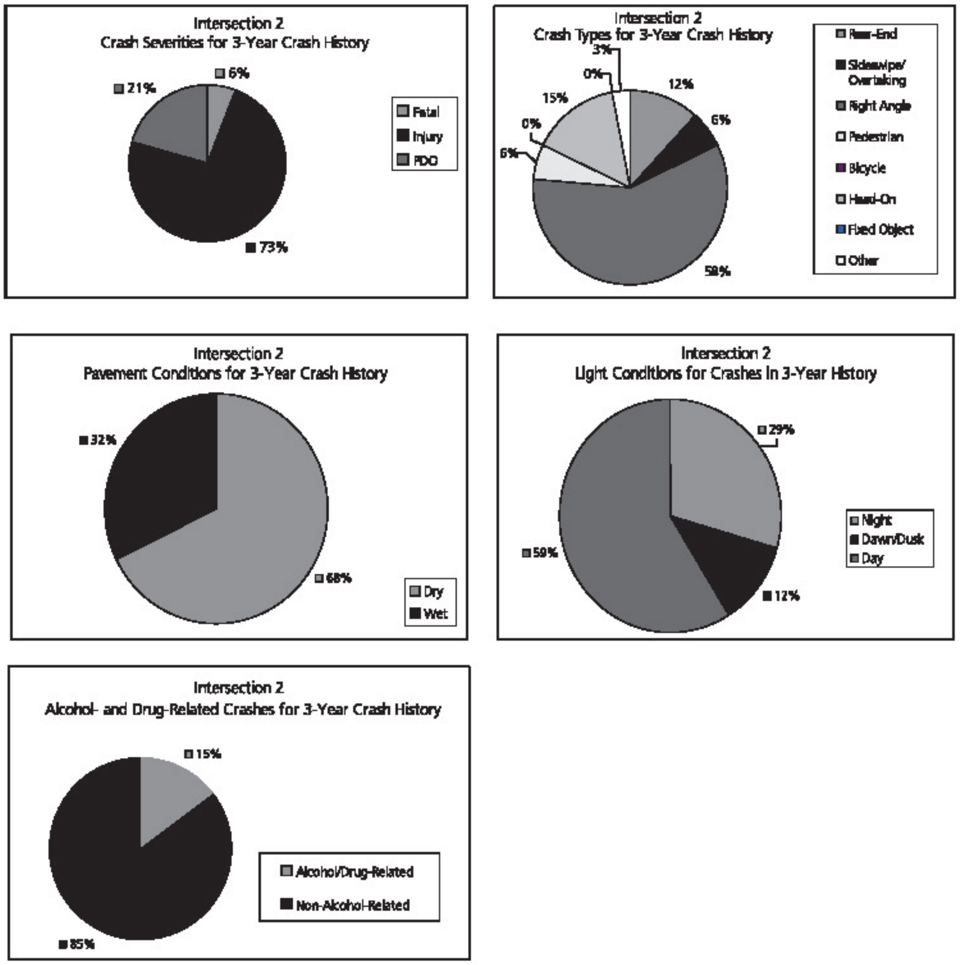


Figure 5-6. Crash Summary Statistics for Intersection 2

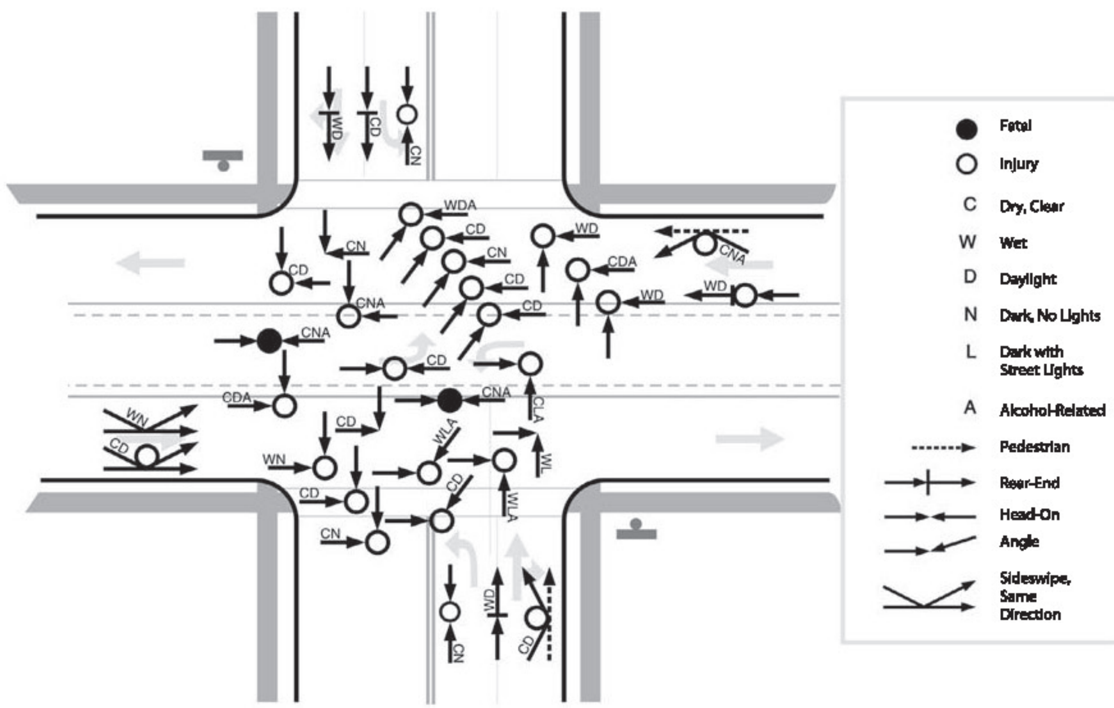


Figure 5-7. Collision Diagram for Intersection 2

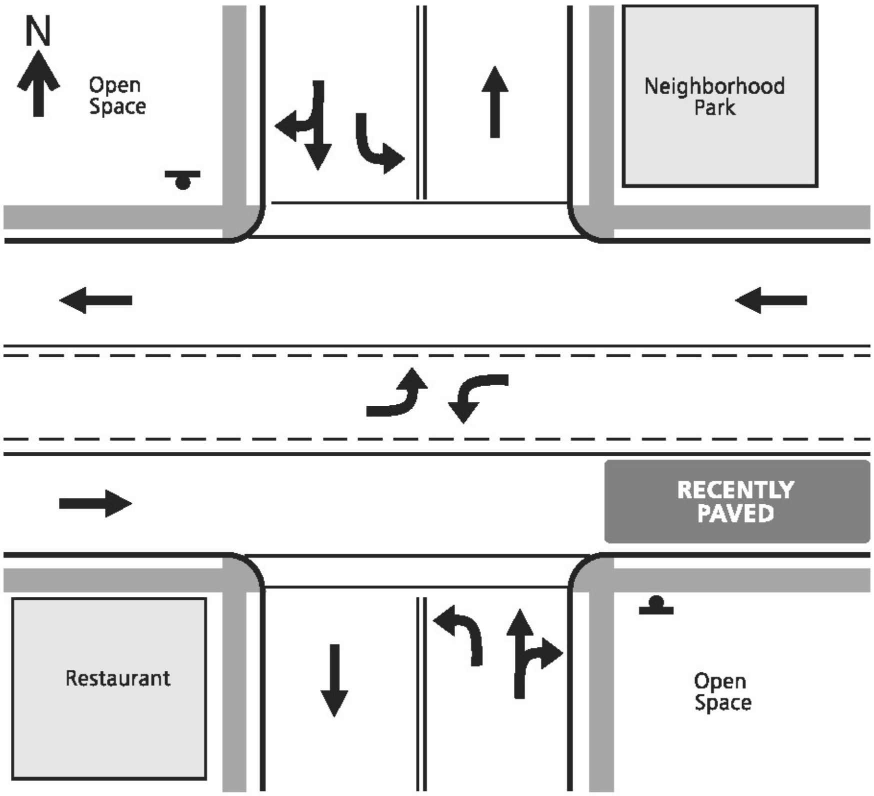


Figure 5-8. Condition Diagram for Intersection 2

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.

A field assessment of Intersection 2 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

Figure 5-9 contains crash summary characteristics for Intersection 9. Figure 5-10 illustrates the collision diagram for Intersection 9. Figure 5-11 is the condition diagram for Intersection 9. These figures were generated and analyzed to diagnose the safety concern at Intersection 9.

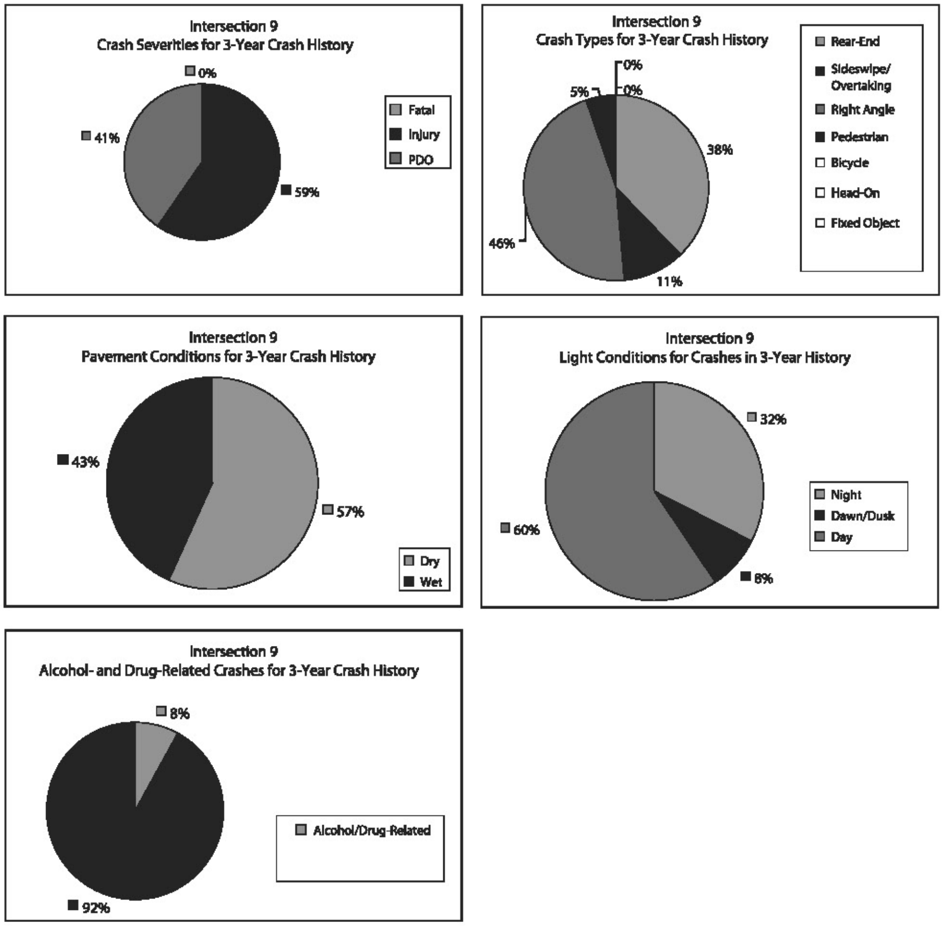


Figure 5-9. Crash Summary Statistics for Intersection 9

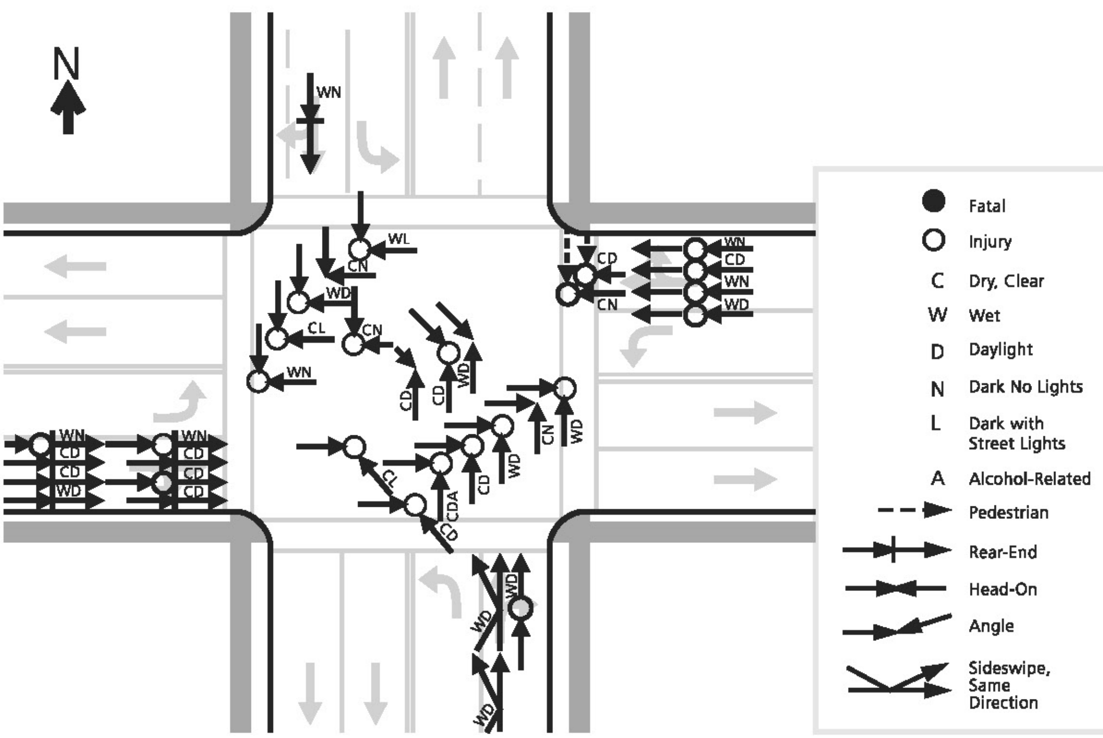


Figure 5-10. Collision Diagram for Intersection 9

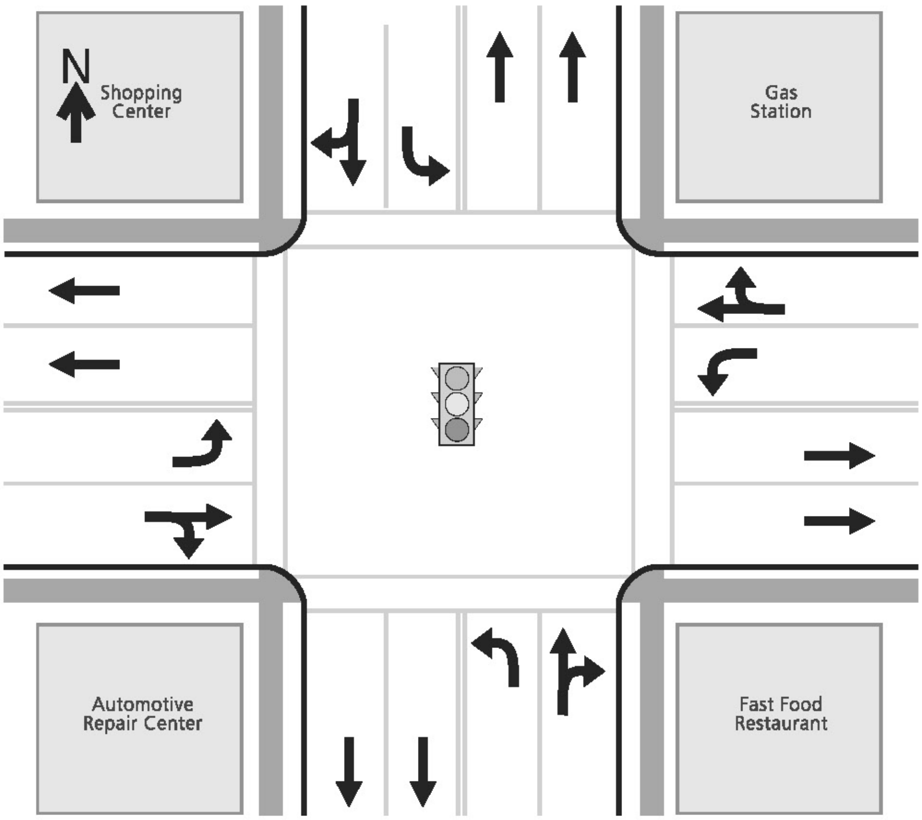


Figure 5-11. Condition Diagram of Intersection 9

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-in. 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

Figure 5-12 contains crash summary characteristics for Segment 1. Figures 5-13 and 5-14 illustrate the collision diagram and the condition diagram for Segment 1, respectively. All three of these figures were generated and analyzed to diagnose the safety concern at Segment 1.

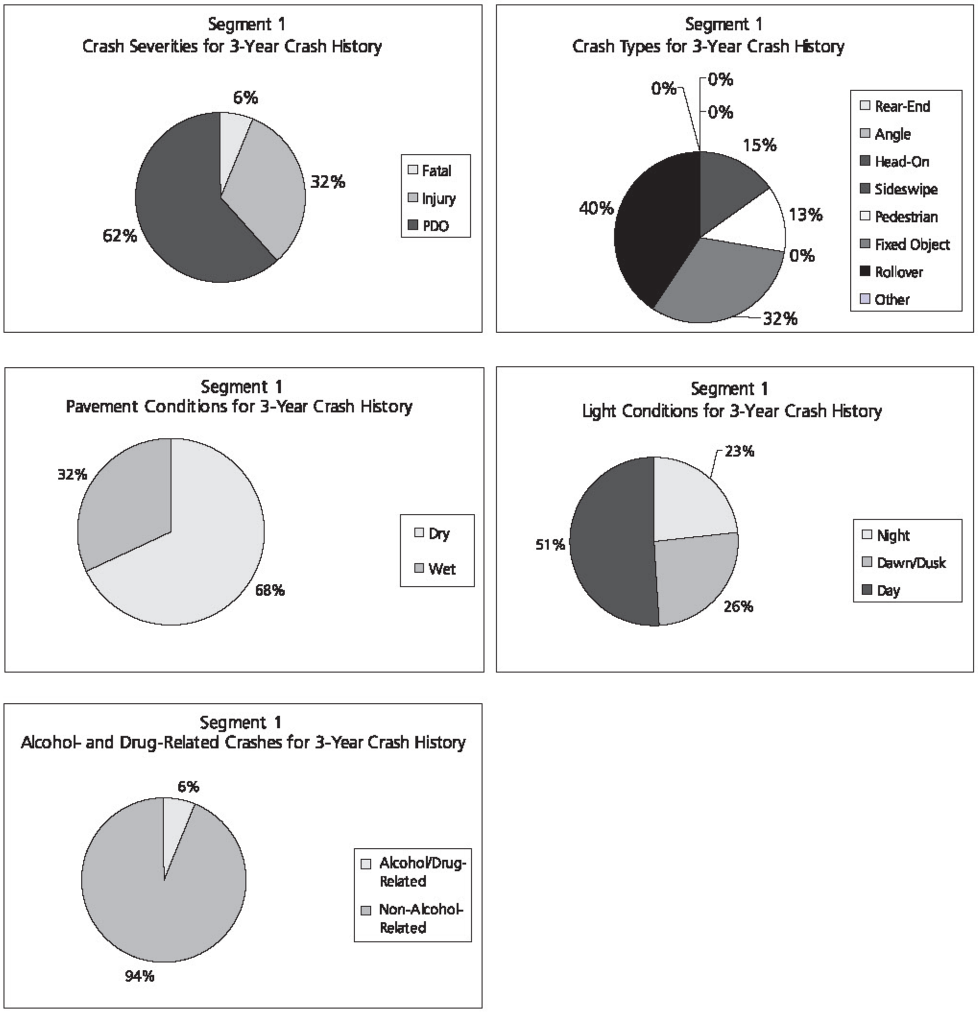


Figure 5-12. Crash Summary Statistics for Segment 1

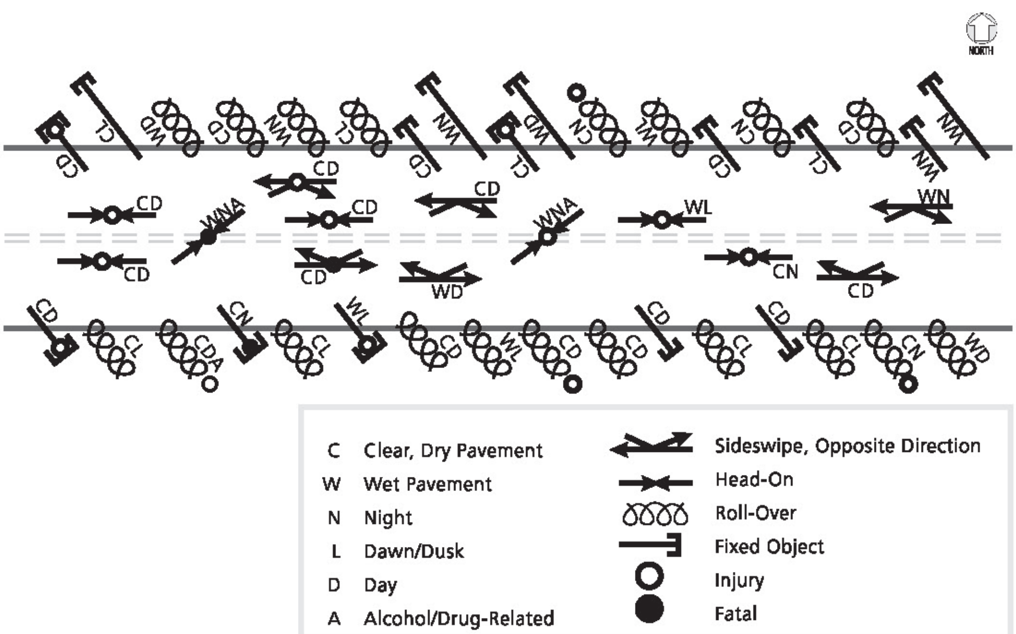


Figure 5-13. Collision Diagram for Segment 1

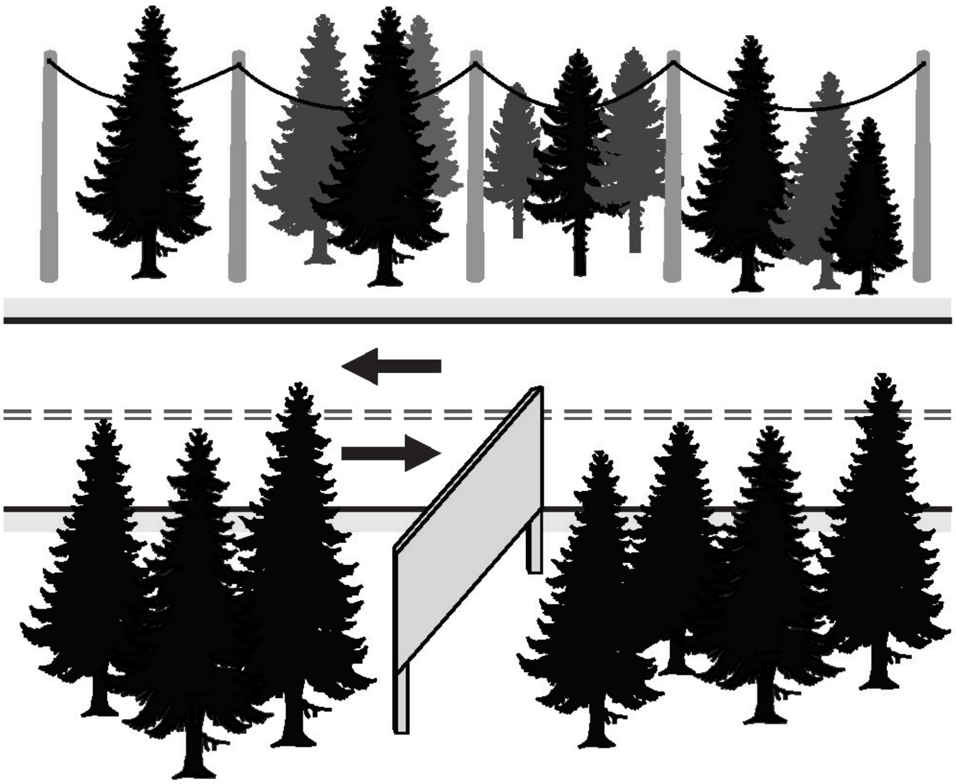


Figure 5-14. Condition Diagram for Segment 1

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., rollover 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

Figure 5-15 contains crash summary characteristics for Segment 5. Figure 5-16 illustrates the collision diagram for Segment 5. Figure 5-17 is the condition diagram for Segment 5. All three of these figures were generated and analyzed to diagnose Segment 5.

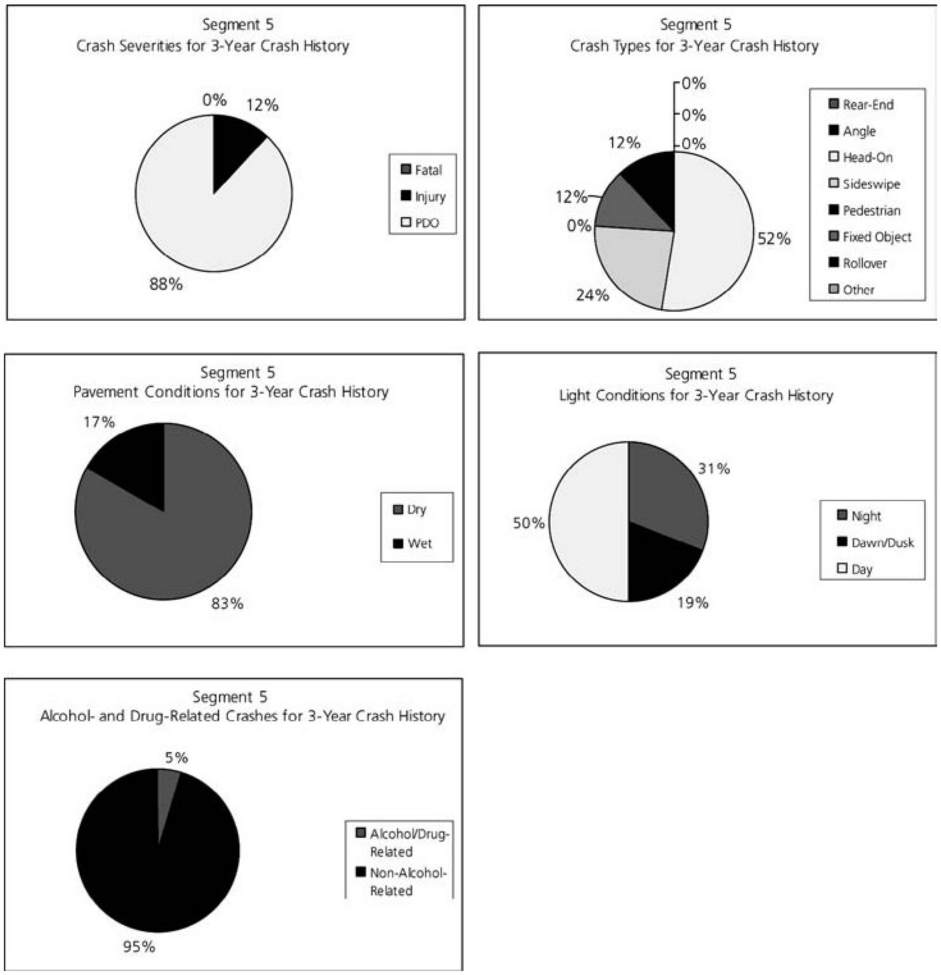


Figure 5-15. Crash Summary Statistics for Segment 5

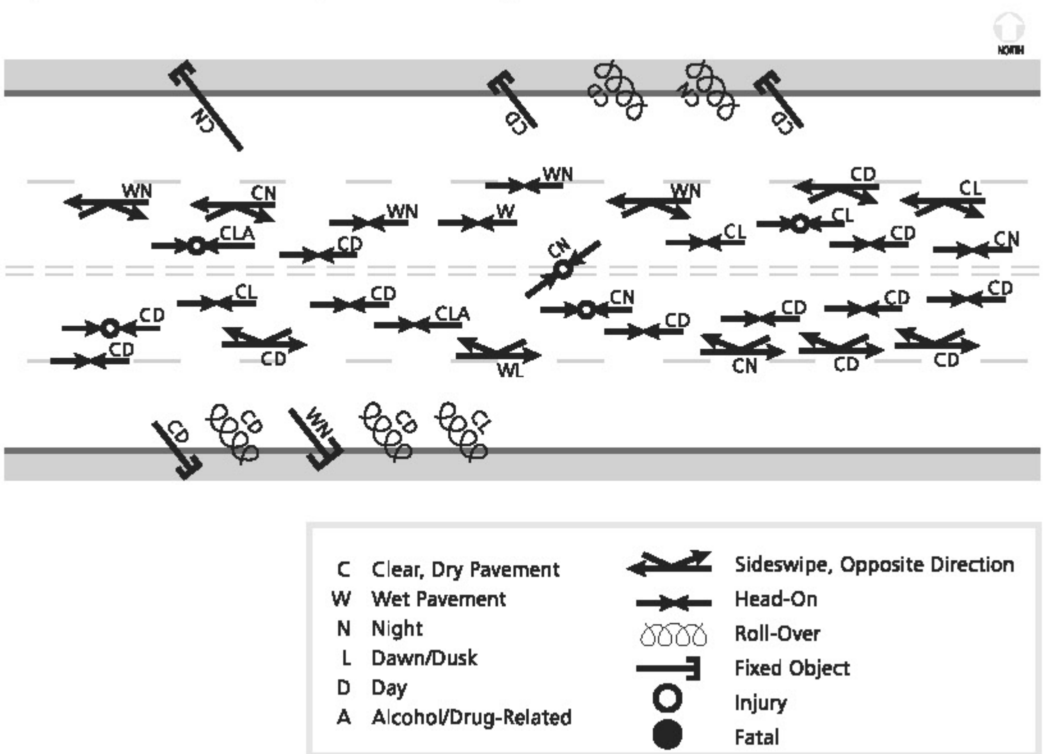


Figure 5-16. Collision Diagram for Segment 5

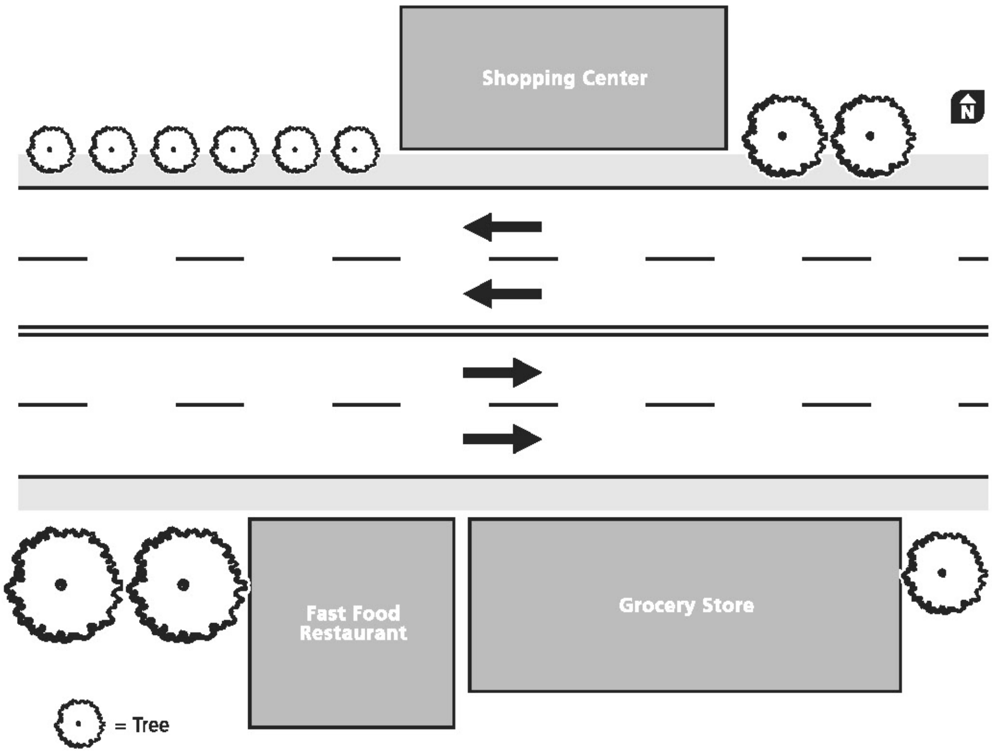


Figure 5-17. Condition Diagram for Segment 5

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 percent) of cross-over crashes (i.e., head-on and opposite direction sideswipe).

5.8. REFERENCES

(1) Austroads. *Guide to Road Safety—Part 6: Road Safety Audit.* 2nd Ed. Austroads, Sydney, Australia, 2002.

(2) FHWA. *Road Safety Fundamentals.* Federal Highway Administration Office of Safety by BMI-SG (draft), U. S. Department of Transportation, Washington, DC, 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, Surrey, UK, 1996.

(6) PIARC Technical Committee on Road Safety (C13). *Road Safety Manual.* World Road Association, Paris, France, 2003.

APPENDIX 5A—EXAMPLE OF POLICE CRASH REPORT

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 | LATITUDE | LONGITUDE | 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 ESTIMATE: ☐ UNDER $1500 ☐ OVER $1500 ☐ UNKNOWN ☐ HAZ. MATERIALS ☐ PHOTOS TAKEN ☐ TRAIN R/R ☐ TRUCK / BUS

UNIT # | NAME (LAST, FIRST, MIDDLE) | DRIVER LICENSE NUMBER | STATE | SEX | RACE | DOB

PED BIC PRK PRP | ADDRESS | PHONE: ☐ HOME ☐ WORK ☐ CELL ( )

VEHICLE OWNER ☐ SAME | PHONE: ☐ HOME ☐ WORK ☐ CELL ( )

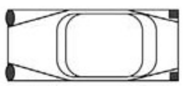
FIRE Y N | STD SPD | PST SPD | INSURANCE COMPANY ☐ NONE | INSURANCE POLICY NUMBER

EJECTED Y P N | EXTRCTD Y N | VEHICLE IDENTIFICATION NUMBER (VIN) | LICENSE PLATE NUMBER | STATE | YEAR | MAKE | MODEL | STYLE | COLOR

VEHICLE TOWED DUE TO VEHICLE DAMAGE Y N ☐ UNKNOWN BY: TO: | DRIVER TAKEN: Y N ☐ UNKNOWN BY: TO:

VEHICLE DAMAGE

FRONT



MARK ALL THAT APPLY: DAMAGE ESTIMATE ☐ NONE ☐ UNDER $1500 ☐ OVER $1500 ☐ ROLLOVER ☐ UNDERCAR ☐ TOTALED ☐ UNKNOWN

USE ARROW TO SHOW FIRST IMPACT (SHADE IN DAMAGED AREA)

INJURY: ☐ NONE ☐ COMPLAINT OF PAIN ☐ VISIBLE INJURY ☐ INCAPACITATED ☐ 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 | PHONE: ☐ HOME ☐ WORK ☐ CELL ( )

VEHICLE OWNER ☐ SAME | PHONE: ☐ HOME ☐ WORK ☐ CELL ( )

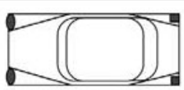
FIRE Y N | STD SPD | PST SPD | INSURANCE COMPANY ☐ NONE | INSURANCE POLICY NUMBER

EJECTED Y P N | EXTRCTD Y N | VEHICLE IDENTIFICATION NUMBER (VIN) | LICENSE PLATE NUMBER | STATE | YEAR | MAKE | MODEL | STYLE | COLOR

VEHICLE TOWED DUE TO VEHICLE DAMAGE Y N ☐ UNKNOWN BY: TO: | DRIVER TAKEN: Y N ☐ UNKNOWN BY: TO:

VEHICLE DAMAGE

FRONT



MARK ALL THAT APPLY: DAMAGE ESTIMATE ☐ NONE ☐ UNDER $1500 ☐ OVER $1500 ☐ ROLLOVER ☐ UNDERCAR ☐ TOTALED ☐ UNKNOWN

USE ARROW TO SHOW FIRST IMPACT (SHADE IN DAMAGED AREA)

INJURY: ☐ NONE ☐ COMPLAINT OF PAIN ☐ VISIBLE INJURY ☐ INCAPACITATED ☐ 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 | PHONE: ☐ HOME ☐ WORK ☐ CELL ( ) | INJURY ☐ COMPLAINT OF PAIN ☐ INCAPACITATED ☐ NONE ☐ VISIBLE INJURY ☐ FATAL | LOCATION ☐LF ☐CF ☐RF ☐LR ☐CR ☐RR | OTHER: | EJECTED Y P N | EXTRCTD Y N

PASSENGER TAKEN: Y N ☐ UNKNOWN BY: 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 | PHONE: ☐ HOME ☐ WORK ☐ CELL ( ) | INJURY ☐ COMPLAINT OF PAIN ☐ INCAPACITATED ☐ NONE ☐ VISIBLE INJURY ☐ FATAL | LOCATION ☐LF ☐CF ☐RF ☐LR ☐CR ☐RR | OTHER: | EJECTED Y P N | EXTRCTD Y N

PASSENGER TAKEN: Y N ☐ UNKNOWN BY: 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 | PHONE: ☐ HOME ☐ WORK ☐ CELL ( ) | INJURY ☐ COMPLAINT OF PAIN ☐ INCAPACITATED ☐ NONE ☐ VISIBLE INJURY ☐ FATAL | LOCATION ☐LF ☐CF ☐RF ☐LR ☐CR ☐RR | OTHER: | EJECTED Y P N | EXTRCTD Y N

PASSENGER TAKEN: Y N ☐ UNKNOWN BY: 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 (6-07) STK# 300017

Source: Oregon Department of Motor Vehicles

Figure 5A-1. Police Traffic Crash Form

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 (Explain)

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 (Explain)

★ 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 (Explain)

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 (Explain)

★ PASSENGER FACTORS
PASS **UNIT #1**
#1 #2
☐ ☐ NONE
☐ ☐ INTERFERED w/DRIVER
☐ ☐ UNDER INFL - DRUGS
☐ ☐ UNDER INFL - ALCOHOL
☐ ☐ UNKNOWN
☐ ☐ OTHER (Explain)

PASS **UNIT #2**
#1 #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)

SKETCH & NARRATIVE
North

(NOT TO SCALE)

	UNIT 1	2
SKID MARKS TO (FEET)	______	______
DISTANCE AFTER (FEET)	______	______

Figure 5A-1. Police Traffic Crash Form (continued)

APPENDIX 5B—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 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 5C—PREPARATION FOR CONDUCTING AN ASSESSMENT OF FIELD CONDITIONS

SELECT PARTICIPANTS

The field investigation is most successful when conducted from a multimodal, 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 multimodal, 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 multimodal 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.
- 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 or video camera, or both
- 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 5D—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 crash 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 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 provided below includes questions to consider when performing field investigations at unsignalized intersections (2):

- 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).

REFERENCES FOR CHAPTER 5 APPENDIXES

(1) Austroads. *Guide to Road Safety—Part 6: Road Safety Audit.* 2nd Ed. Austroads, Sydney, Australia, 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, Paris, France, 2003.

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 Figure 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.

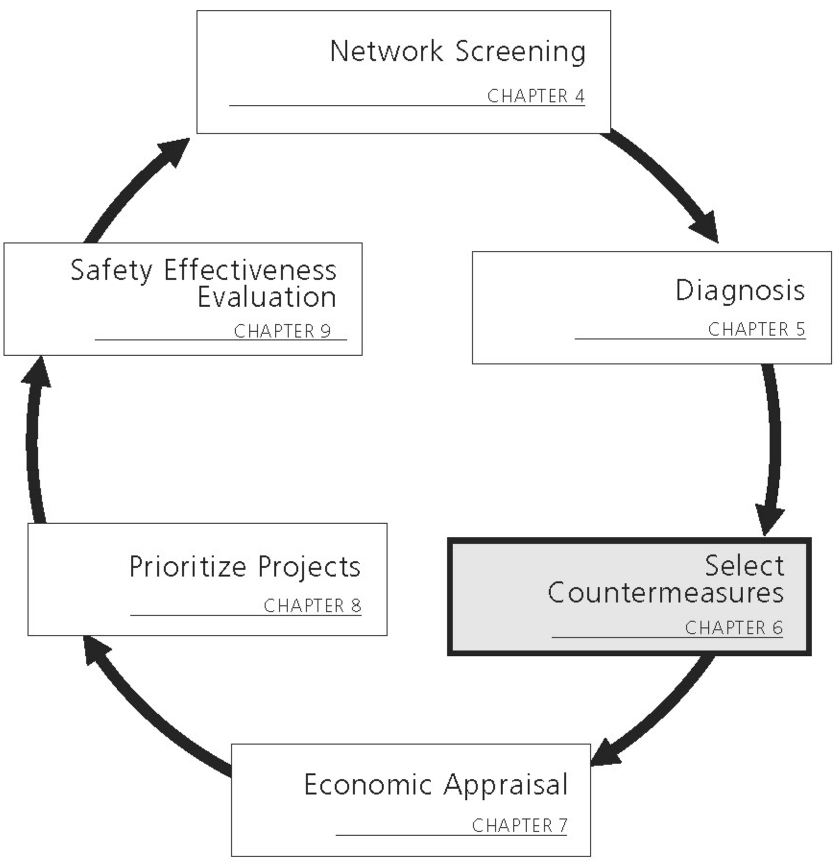


Figure 6–1. Roadway Safety Management Process Overview

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.

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 category of crash contributing factors to identify possible reasons for the crash. An example of a Haddon Matrix prepared for a rear-end crash is shown in Table 6-1. Additional details on the Haddon Matrix are provided in Chapter 3.

Table 6-1. 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 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 this Manual. Several fundamental principles essential to understanding the human factor 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 percent 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.

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).

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

Listed below are common types of crashes and multiple potential contributing factors for crashes on roadway segments. It is important to note that some of the possible contributing factor(s) shown for various crash types 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.

Possible contributing factors for the following types of crashes along roadway segments include:

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-the-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

Listed below are common types of crashes that occur at signalized intersections and possible contributing factor(s) 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.

Possible contributing factors for types of crashes at signalized intersections include the following:

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

Listed below are common types of crashes that occur at unsignalized intersections along with possible contributing factor(s) for each type. The types of crashes 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.

Possible contributing factors for types of crashes at unsignalized intersections include the following:

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

Listed below are common types of crashes that occur at highway-rail grade crossings and possible contributing factor(s) associated with each type. This is not intended to be a comprehensive list of all crash types and contributing factors.

Possible contributing factors for collisions at highway-rail grade crossings include the following:

- 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 types of crashes and possible contributing factor(s) in crashes involving pedestrians are listed below. These are not intended to be comprehensive lists of all crash types and contributing factors.

Possible contributing factor(s) to crashes involving pedestrians include the following:

- 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

Possible contributing factors for crashes involving bicyclists include the following:

- 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.

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 is a resource for treatments with quantitative crash modification factors (CMFs). The CMFs represent the estimated change in crash frequency with implementation of the treatment under consideration. A CMF 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 a CMF 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 a CMF 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 a CMF of 1.58 for rear-end crashes and a CMF of 0.40 for left-turn crashes. The CMFs suggest that an increase in rear-end crashes may occur while a reduction in left-turn crashes may occur.

If a CMF is not available, Part D 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, crash modification factors for treatments can be derived locally using procedures outlined in Chapter 9.

In some cases a specific contributing factor or associated treatment, or both, 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 a CMF is not available, Part D also provides information in some cases 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 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 Table 6-2.

Table 6-2. 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 Types of Crashes	Angle, Head-On	Rollover, 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 Table 5-2.
- All study intersections have four approaches and are located in urban environments.

Roadway Segments

- Three years of roadway segment crash data as shown in Table 5-2.
- The roadway cross-section and length as shown in Table 5-2.

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

Section 6.2.2 identifies possible crash contributing factors at unsignalized intersections by crash type. As shown, 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 analysis of existing traffic operations during the weekday afternoon/evening (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:

- Increase 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 CMFs 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 CMFs discussed 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 rollover crash. The statistics and crash reports do not show a strong correlation between the run-off-the-road crashes and lighting conditions.

Section 6.2.2 summarizes possible contributing factors for rollover and run-off-the-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 were also assessed to be within reason.

Potential countermeasures that the agency could implement were identified to include: increasing the lane or shoulder width, or both, 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 CMFs provided in Part D. Given that the roadway segment is located between two intersections and 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 8.

6.6. REFERENCES

(1) Antonucci, N. D, K. K. Hardy, K. L. Slack, R. Pfefer, and T. R. Neuman. *NCHRP Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan, Volume 12: A Guide for Reducing Collisions at Signalized Intersections.* TRB, National Research Council, Washington, DC, 2004.

(2) Haddon, W. A logical framework for categorizing highway safety phenomena and activity. *The Journal of Trauma*, Vol. 12. Lippincott Williams & Wilkins, Philadephia, PA, 1972, pp. 193–207.

(3) Hedlund, J. et al. *Countermeasures that Work: A Highway Safety Countermeasure Guide for State Highway Safety Offices*, Third Edition. Report No. DOT-HS-810-891. National Highway Traffic Safety Administration, Washington, DC, 2008.

(4) Hills, B. B. Visions, visibility and perception in driving. *Perception*, Vol. 9. 1980, pp. 183–216.

(5) Knipling , R. R., P. Waller, R. C. Peck, R. Pfefer, T. R. Neuman, K. L. Slack, and K. K. Hardy. *NCHRP Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan, Volume 13: A Guide for Addressing Collisions Involving Heavy Trucks.* TRB, National Research Council, Washington, DC, 2003.

(6) Lacy, K., R. Srinivasan, C. V. Zegeer, R. Pfefer, T. R. Neuman, K. L. Slack, and K. K. Hardy. *NCHRP Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan, Volume 8: A Guide for Addressing Collisions Involving Utility Poles.* NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2004.

(7) NCHRP. *National Cooperative Highway Research Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan.* NCHRP, Transportation Research Board, National Research Council, Washington, DC, 1998.

(8) Neuman, T. R., R. Pfefer. K. L Slack, K. K. Hardy, K. Lacy, and C. Zegeer. *NCHRP Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan, Volume 3: A Guide for Addressing Collisions with Trees in Hazardous Locations.* NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2003.

(9) Neuman, T. R., R. Pfefer, K. L. Slack, K. K. Hardy, H. McGee, L. Prothe, K. Eccles, and F. M. Council. *NCHRP Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan, Volume 4: A Guide for Addressing Head-On Collisions.* NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2003.

(10) Neuman, T. R., R. Pfefer, K. L. Slack, K. K. Hardy, D. W. Harwood, I. B. Potts, D. J. Torbic, and E. R. Rabbani. *NCHRP Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan, Volume 5: A Guide for Addressing Unsignalized Intersection Collisions.* NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2003.

(11) Neuman, T. R., et al. *National Cooperative Highway Research Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan, Volume 6: A Guide for Addressing Run-Off-Road Collisions.* NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2003.

(12) Potts, I., J. Stutts, R. Pfefer, T. R. Neuman, K. L. Slack, and K. K. Hardy. *National Cooperative Highway Research Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan, Volume 9: A Guide for Reducing Collisions With Older Drivers.* NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2004.

(13) Stutts, J., R. Knipling , R. Pfefer, T. Neuman, K. Slack, and K. Hardy. *NCHRP Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan, Volume 14: A Guide for Reducing Crashes Involving Drowsy and Distracted Drivers.* NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2005.

(14) Torbic, D. J., D.W. Harwood, R. Pfefer, T. R. Neuman, K. L. Slack, and K. K. Hardy. *NCHRP Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan, Volume 7: A Guide for Reducing Collisions on Horizontal Curves.* NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2004.

(15) Zegeer, C. V., J. Stutts, H. Huang, M. J. Cynecki, R. Van Houten, B. Alberson, R. Pfefer, T. R. Neuman, K. L. Slack, and K. K. Hardy. *National Cooperative Highway Research Report 500: Guidance for Implementation of the AASHTO Strategic Highway Safety Plan, Volume 10: A Guide for Reducing Collisions Involving Pedestrians.* NCHRP, Transportation Research Board, National Research Council, Washington, DC, 2004.

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). Figure 7-1 shows this step in the context of the overall roadway safety management process.

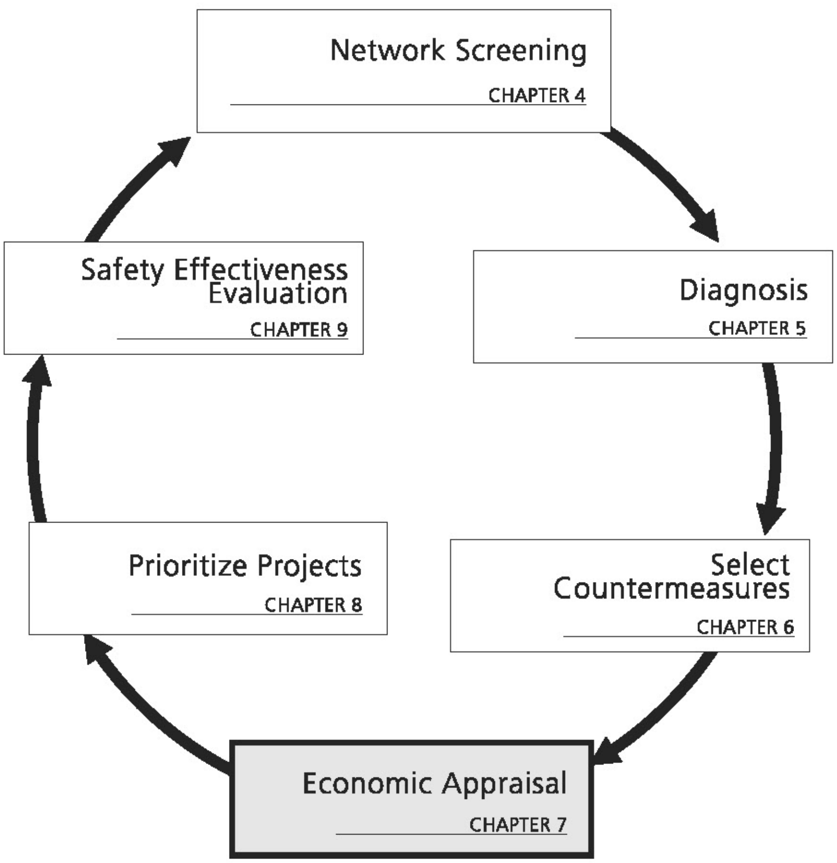


Figure 7-1. Roadway Safety Management Process Overview

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. Figure 7-2 provides a schematic of the economic appraisal process.

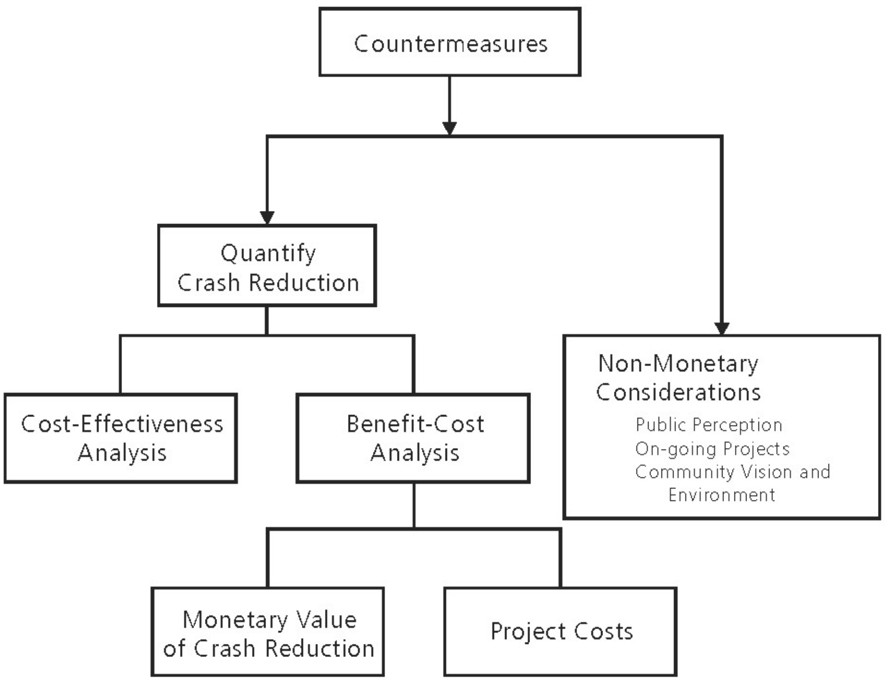


Figure 7-2. Economic Appraisal Process

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.

7.3. DATA NEEDS

The data needed to calculate the change in crash frequency and countermeasure implementation costs are summarized below. Appendix 7A includes a detailed explanation of the data needs.

Activity	Data Needed to Calculate Project Benefits
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)
	CMFs 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 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 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 CMF 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., an 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 CMF from Part D to estimate the expected average crash frequency of the proposed condition. A locally derived project CMF 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 CMF 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 a CMF from Part D is used in one of the four methods, the associated standard error of the CMF 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 CMF 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 CMFs, or with locally developed CMFs. When there is no applicable Part C predictive method, no applicable SPF, and no applicable CMF, the HSM procedures cannot provide an estimate of the expected project effectiveness.

In order to evaluate countermeasures when no valid CMF is available, an estimate of the applicable CMF may be chosen using engineering judgment. The results of such analysis are considered uncertain, and a sensitivity analysis based on a range of CMF 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 are 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

Annual benefits of a safety improvement can be calculated by multiplying the predicted reduction in crashes of a given severity by the applicable societal cost.

The Federal Highway Administration (FHWA) has completed research that establishes a basis for quantifying, in monetary terms, the human capital crash costs to society of fatalities and injuries from highway crashes. These estimates include the monetary losses associated with medical care, emergency services, property damage, lost productivity, and the like, to society as a whole. They are not to be confused with damages that may be awarded to a particular plaintiff in a personal injury or wrongful death lawsuit. Tort liability damages are based only on the particularized loss to the individual plaintiff and are not allowed to include any societal costs or burdens. Some agencies have developed their own values for societal costs of crashes, which can be used if desired.

State and local jurisdictions often have accepted societal crash costs by crash severity and collision type. When available, these locally-developed societal crash cost data are used with procedures in the HSM. 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 societal costs cited in this 2005 report are presented in 2001 dollars. Appendix 4A includes a summary of a procedure for updating annual monetary values to current year values. Table 7-1 summarizes the relevant information for use in the HSM (rounded to the nearest hundred dollars).

Table 7-1. Societal Crash Cost Estimates by Crash Severity

Collision Type	Comprehensive Societal Crash Costs
Fatal (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 CMFs do not always differentiate between 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 Table 7-1 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 × $158,200 = $791,000/year

PDO crashes: 11 × $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

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 in Section 7.4.3.1);
- 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.

$$PV_{benefits} = \text{Total Annual Monetary Benefits} \times (P/A,i,y) \tag{7-1}$$

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 percent, 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 × (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 percent, $i = 0.04$)

      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 costs 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, e.g., 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:

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).

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 Part C—Introduction and Applications Guidance).

2. Convert the change in estimated average crash frequency to an annual monetary value 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:

$$NPV = PV_{\text{benefits}} - PV_{\text{costs}} \quad (7\text{-}3)$$

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.

The strengths and limitations of NPV Analysis include the following:

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 7.4).

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.

$$BCR = \frac{PV_{\text{benefits}}}{PV_{\text{costs}}} \quad (7\text{-}4)$$

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.

The strengths and limitations of BCR Analysis include the following:

Strengths	Weaknesses
The magnitude of the benefit-cost ratio makes the relative desirability of a proposed project immediately evident to decision makers. 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.	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 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.

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 Sections 7.4 and 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:

$$\text{Cost-Effectiveness Index} = \frac{PV_{\text{costs}}}{N_{\text{predicted}} - N_{\text{observed}}} \tag{7-5}$$

Where:

PV_{costs} = Present Value of Project Cost

$N_{predicted}$ = Predicted crash frequency for year y

$N_{observed}$ = Observed crash frequency for year y

The strengths and limitations of NPV Analysis include the following:

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.	

7.7. NON-MONETARY CONSIDERATIONS

In most cases, the primary benefits of countermeasure implementation projects can be estimated in terms of the change in average crash frequency and injuries avoided or monetary values, or both. 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 focus only 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

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 CMFs, or locally developed CMFs 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 CMF.

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. Table 7-2 provides a summary of the crash conditions, contributory factors, and selected countermeasures.

Table 7-2. 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

- CMFs for installing a single-lane roundabout in place of a two-way stop-controlled intersection (see Chapter 14):
  - Total crashes = 0.56, and
  - 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 Table 7-3.

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. CMFs that are associated with the countermeasures implemented are provided. All other CMFs 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.

Table 7-3. Expected Average Crash Frequency at Intersection 2 WITHOUT Installing the Roundabout

Year in service life (y)	Major AADT	Minor AADT	$N_{expected(total)}$	$N_{expected(FI)}$
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 Table 7-4 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.

Table 7-4. Societal Crash Costs by Severity

Injury Severity	Estimated Cost
Fatal (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

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 include the following:

	Intersection 2
Countermeasure	Roundabout
Service Life	10 years
Annual Traffic Growth	2%
Discount Rate (i)	4.0%
Cost Estimate Method	$695,000

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 Table 7-5. The methods for conducting the appraisal are outlined in detail in the following sections.

Table 7-5. Economic Appraisal for Intersection 2

Roadway Segment Crash 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)	$N_{br} = N_{SPF\,rs} \times (CMF_{1r} \times CMF_{2r} \times \ldots \times CMF_{nr})$
EB-Adjusted Expected Average Crash Frequency	
Expected Crashes without Roundabout	See Table 7-3.
Expected Crashes with Roundabout	See Table 7-6 and Table 7-7.
Equations 7-6, 7-7	
Expected Change in Crashes	See Table 7-8
Equations 7-8, 7-9, 7-10	
Yearly Monetary Value of Change in Crashes	See Table 7-9
Equations 7-11, 7-12, 7-13	
Present Value of Change in Crashes	See Table 7-10
Equations 7-14, 7-15	
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. Table 7-3 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 (total) and fatal-and-injury (FI) crashes for each year of the service life (y) assuming the roundabout is installed.

Multiply the CMF for converting a stop-controlled intersection to a roundabout found in Chapter 14 (restated below in Table 7-6) by the expected average crash frequency calculated above in Section 7.6.1.2 using Equations 7-6 and 7-7.

$$N_{\text{expected roundabout (total)}} = N_{\text{expected (total)}} \times CMF_{\text{(total)}} \quad (7\text{-}6)$$

$$N_{\text{expected roundabout }(FI)} = N_{\text{expected }(FI)} \times CMF_{(FI)} \tag{7-7}$$

Where:

$N_{\text{expected roundabout (total)}}$ = EB-adjusted expected average crash frequency in year y WITH the roundabout installed;

$N_{\text{expected (total)}}$ = EB-adjusted expected average total crash frequency in year y WITHOUT the roundabout installed;

$CMF_{\text{(total)}}$ = Crash Modification Factor for total crashes;

$N_{\text{expected roundabout}(FI)}$ = EB-adjusted expected average fatal and injury crash frequency in year y WITH the roundabout installed;

$N_{\text{expected }(FI)}$ = EB-adjusted expected average fatal and injury crash frequency in year y WITHOUT the roundabout installed; and

$CMF_{(FI)}$ = Crash Modification Factor for fatal and injury crashes.

Table 7-6 summarizes the EB-adjusted average fatal and injury crash frequency for each year of the service life assuming the roundabout is installed.

Table 7-6. Expected Average FI Crash Frequency at Intersection 2 WITH the Roundabout

Year in Service Life (y)	$N_{\text{expected}(FI)}$	$CMF_{(FI)}$	$N_{\text{expected roundabout}(FI)}$
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

Table 7-7 summarizes the EB-adjusted average total crash frequency for each year of the service life assuming the roundabout is installed.

Table 7-7. Expected Average Total Crash Frequency at Intersection 2 WITH the Roundabout

Year in service life (y)	$N_{\text{expected(total)}}$	$CMF_{\text{(total)}}$	$N_{\text{expected roundabout(total)}}$
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.

$$\Delta N_{\text{expected}(FI)} = N_{\text{expected}(FI)} - N_{\text{expected roundabout }(FI)} \quad (7\text{-}8)$$

$$\Delta N_{\text{expected(total)}} = N_{\text{expected(total)}} - N_{\text{expected roundabout (total)}} \quad (7\text{-}9)$$

$$\Delta N_{\text{expected}(PDO)} = N_{\text{expected(total)}} - N_{\text{expected}(FI)} \quad (7\text{-}10)$$

Where:

$\Delta N_{\text{expected(total)}}$ = Expected change in average crash frequency due to implementing countermeasure;

$\Delta N_{\text{expected}(FI)}$ = Expected change in average fatal and injury crash frequency due to implementing countermeasure; and

$\Delta N_{\text{expected}(PDO)}$ = Expected change in average PDO crash frequency due to implementing countermeasure.

Table 7-8 summarizes the expected change in average crash frequency due to installing the roundabout.

Table 7-8. Change in Expected Average in Crash Frequency at Intersection 2 WITH the Roundabout

Year in service life, *y*	$\Delta N_{\text{expected(total)}}$	$\Delta N_{\text{expected}(FI)}$	$\Delta N_{\text{expected}(PDO)}$
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.

$$AM_{(PDO)} = \Delta N_{\text{expected}(PDO)} \times CC_{(PDO)} \quad (7\text{-}11)$$

$$AM_{(FI)} = \Delta N_{\text{expected}(FI)} \times CC_{(FI)} \quad (7\text{-}12)$$

$$AM_{\text{(total)}} = AM_{(PDO)} \times AM_{(FI)} \quad (7\text{-}13)$$

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;

$AM_{(FI)}$ = Monetary value of the estimated change in fatal and injury average crash frequency for year *y*;

$CC_{(FI)}$ = Crash cost for FI crash severity; and

$AM_{\text{(total)}}$ = Monetary value of the total estimated change in average crash frequency for year *y*.

Table 7-9 summarizes the monetary value calculations for each year of the service life.

Table 7-9. Annual Monetary Value of Change in Crashes

Year in service life, y	$\Delta N_{(FI)}$	FI Crash Cost	$AM_{(FI)}$	$\Delta N_{(PDO)}$	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 percent 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.

$$PV_{\text{benefits}} = \text{Total Annual Monetary Benefits} \times (P/F,i,y) \quad (7\text{-}14)$$

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 percent, $i = 0.04$); 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:

$$(P/A,i,y) = \frac{(1.0+i)^{(y)} - 1.0}{i \times (1.0+i)^{(y)}} \quad (7\text{-}15)$$

Where:

$(P/A,i,y)$ = factor that converts a series of uniform future values to a single present value.

Table 7-10 summarizes the results of converting the annual values to present values.

Table 7-10. Converting Annual Values to Present Values

Year in service life (y)	($P/A,i,y$)	$AM_{(total)}$	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 Table 7-10.

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 this project to other projects in order 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, Washington, DC, 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. Midwest Research Institute for FHWA. November 2003. More information available from http://www.safetyanalyst.org.

APPENDIX 7A—DATA NEEDS AND DEFINITIONS

7A.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 (total) 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 (total) and for fatal and injury crashes (FI). SPFs may be locally developed or calibrated to local conditions.
- Crash Modification Factors (CMFs) for the countermeasures under consideration. CMFs 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 as follows:

- Accepted monetary value of crashes by collision type or crash severity, or both.
- State and local jurisdictions often have accepted dollar value of crashes by collision type or crash severity, or both, 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).
- Fatal (K) = $4,008,900/fatal crash.
- Crashes that include fatalities or injuries, or both, (K/A/B/C) = $158,200/fatal or injury, or both, 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.

7A.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.

7A.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 titled *A Manual of User Benefit Analysis for Highways* (also known as the AASHTO Redbook) (1).

7A.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.

7A.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, U.S. Department of Transportation, Washington, DC, October 2005.

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. Figure 8-1 provides an overview of the complete Roadway Safety Management process presented in Part B of the manual.

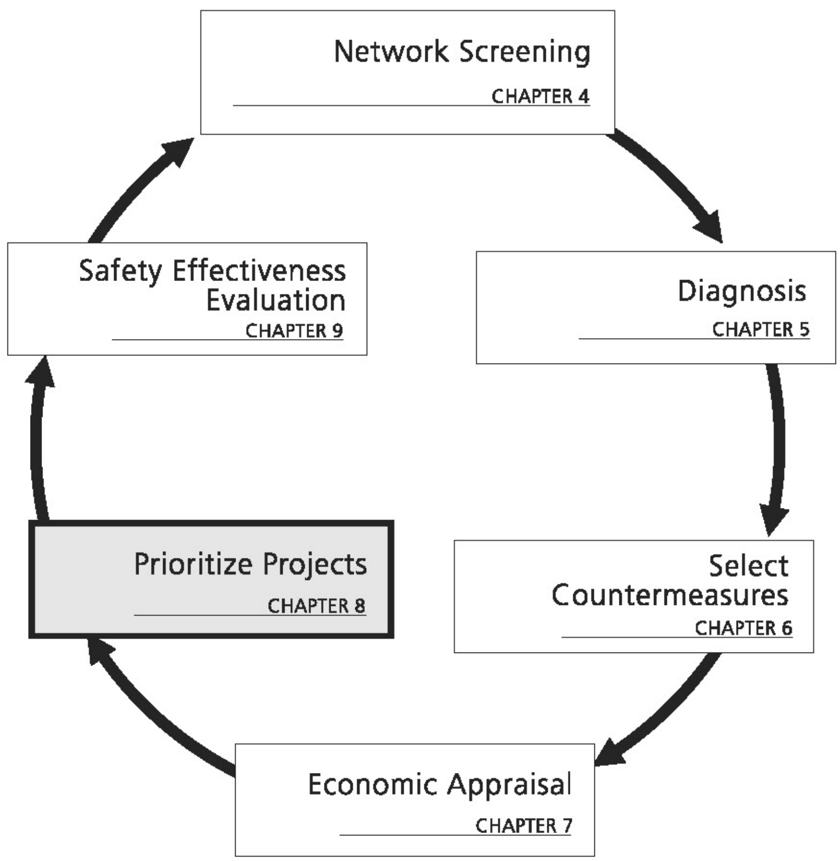


Figure 8-1. Roadway Safety Management Process Overview

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. Figure 8-2 provides an overview of the project prioritization process.

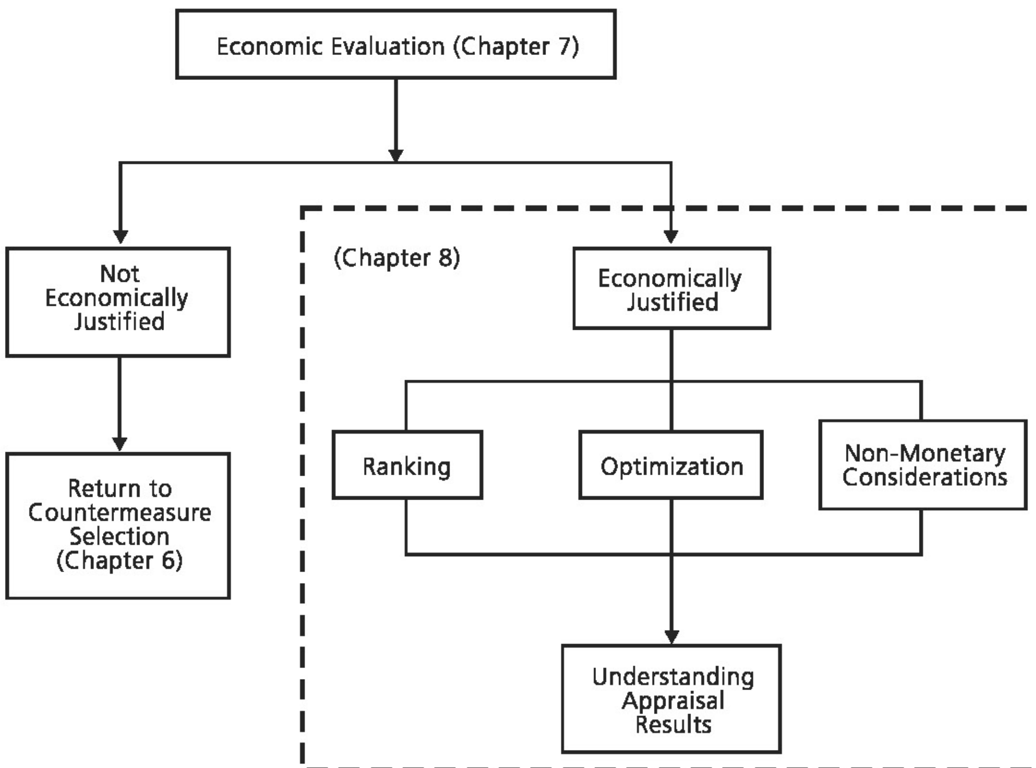


Figure 8-2. Project Prioritization Process

8.2. PROJECT PRIORITIZATION METHODS

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. Multi-objective 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).

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 or other constraints, or both, 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 8A 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).

Multi-Objective 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 multi-objective resource allocation can be used to address such decisions quantitatively. Multi-objective 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, multi-objective resource allocation provides a method to consider non-monetary factors, like those discussed in Chapter 7, in decision making.

All multi-objective 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 multi-objective 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 multi-objective resource allocation methods for highway engineering applications include Interactive Multi-objective Resource Allocation (IMRA) and Multicriteria Cost-Benefit Analysis (MCCBA) (1,4).

8.2.3. Summary of Prioritization Methods

Table 8-1 provides a summary of the prioritization methods described in Section 8.2.

Table 8-1. Summary of Project Prioritization Methods

Method	Input Needs	Outcomes	Considerations
Ranking by Safety-Related Measures	Various; inputs are readily available or derived using the methods presented in Chapter 7, or both.	A ranked list or lists of projects based on various cost or benefit factors, or both.	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 on 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 or a software program, or both.	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 or software program, or both.	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.
Multi-Objective Resource Allocation	Present value of monetary benefits and costs for economically justified projects. Software program to solve the multi-objective 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 multi-objective 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:

- 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 this chapter 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 Table 8-2.

Table 8-2. Intersections and Roadway Segments Selected for Further Review

						Crash Data		
Intersections	Traffic Control	Number of Approaches	Major AADT	Minor AADT	Urban/ Rural	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
						Crash Data (Total)		
Segments	Cross-Section (Number of Lanes)	Segment Length (miles)		AADT	Undivided/ Divided	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

Table 8-3 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.

Table 8-3. 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 Lane	$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

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?

The Facts

Table 8-4 summarizes the crash reduction, monetary benefits and costs for the safety improvement projects being considered.

Table 8-4. 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

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)

Table 8-5 summarizes the results of this method.

Table 8-5. 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. Table 8-6 shows the countermeasure implementation projects listed based on simple cost-effectiveness ranking.

Table 8-6. 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 Table 8-8.

Table 8-8. 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 Table 8-8, 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

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 Table 8-9.

Table 8-9. 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.

$$Incremental\ BCR = (PV_{\text{benefits 2}} - PV_{\text{benefits 1}}) / (PV_{\text{costs 2}} - PV_{\text{costs 1}}) \qquad (8\text{-}3)$$

Where:

$PV_{\text{benefits 1}}$ = Present value of benefits for lower-cost project

$PV_{\text{benefits 2}}$ = Present value of benefits for higher-cost project

$PV_{\text{costs 1}}$ = Present value of cost for lower-cost project

$PV_{\text{costs 2}}$ = Present value of cost for higher-cost project

Table 8-10 illustrates the sequence of incremental benefit-cost comparisons needed to assign priority to the projects.

Table 8-10. 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 Table 8-10, 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 Table 8-11.

Table 8-11. Ranking Results of Incremental BCR Analysis

Rank	Project
1	Intersection 2
2	Segment 5
3	Segment 7
4	Segment 6
5	Segment 1
6	Segment 2
7	Intersection 12
8	Intersection 11
9	Intersection 7

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.

8.5. REFERENCES

(1) Chowdhury, M. A., N. J. Garber, and D. Li. Multi-objective Methodology for Highway Safety Resource Allocation. *Journal of Infrastructure Systems*, Vol. 6, No. 4. American Society of Civil Engineers, Reston, VA, 2000.

(2) Harwood, D. W., E. R. Kohlman Rabbani, K. R. Richard, H. W. McGee, and G. L. Gittings. *National Cooperative Highway Research Program Report 486: Systemwide Impact of Safety and Traffic Operations Design Decisions for 3R Projects*. NCHRP, Transportation Research Board, Washington, DC, 2003.

(3) Mak, K. K., and D. L. Sicking. *National Cooperative Highway Research Program Report 492: Roadside Safety Analysis Program*. NCHRP, Transportation Research Board, Washington, DC, 2003.

(4) Roop, S. S., and S. K. Mathur. *Development of a Multimodal Framework for Freight Transportation Investment: Consideration of Rail and Highway Tradeoffs*. Final Report of NCHRP Project 20-29. Texas A&M University, College Station, TX, 1995.

APPENDIX 8A—BASIC OPTIMIZATION METHODS DISCUSSED IN CHAPTER 8

8A.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.

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.

8A.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).

8A.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).

8A.4. APPENDIX REFERENCES

(1) Agent, K. R., L. O'Connell, E. R. Green, D. Kreis, J. G. Pigman, N. Tollner, and E. Thompson. *Development of Procedures for Identifying High-Crash Locations and Prioritizing Safety Improvements*. Report No. KTC-03-15/SPR250-02-1F. University of Kentucky, Kentucky Transportation Center, Lexington, KY, 2003.

(2) Brown D. B., R. Buffin, and W. Deason. Allocating Highway Safety Funds. In *Transportation Research Record 1270*. TRB, National Research Council, Washington, DC, 1990.

(3) Harwood, D. W., E. R. Kohlman Rabbani, K. R. Richard, H. W. McGee, and G. L. Gittings. *National Cooperative Highway Research Program Report 486: Systemwide Impact of Safety and Traffic Operations Design Decisions for 3R Projects*. NCHRP, Transportation Research Board, Washington, DC, 2003.

(4) Kar, K., and T. K. Datta. Development of a Safety Resource Allocation Model in Michigan. In *Transportation Research Record 1865*. TRB, National Research Council, Washington, DC, 2004.

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 Figure 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 a crash modification factor (CMF) 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.

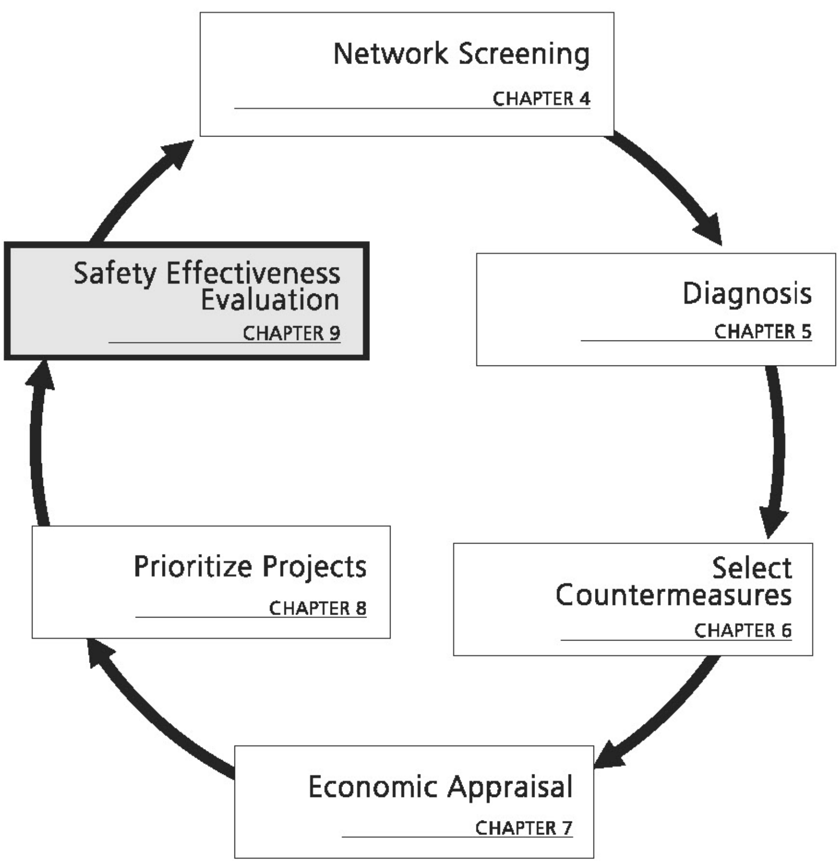


Figure 9-1. Roadway Safety Management Overview Process

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 EFFECTIVENESS 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 a CMF for a countermeasure, and
- Assessing the overall safety effectiveness of specific types of projects or countermeasures in comparison to their costs.

If a particular countermeasure has been installed on a systemwide 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, a CMF 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 Appendix 9A. 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.

Table 9-1 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.

Table 9-1. Generic Evaluation Study Design

Type of Site	Before Treatment	After Treatment
Treatment Sites		
Nontreatment Sites		

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 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. Table 9-2 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 provides more information about issues associated with regression-to-the-mean bias.

As shown in Table 9-2, 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.

Table 9-2. Observational Before/After Evaluation Study Design

Type of Site	Before Treatment	After Treatment
Treatment Sites	✓	✓
Non-treatment Sites (SPF or comparison group)	✓	✓

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.

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.

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 Appendix 9A.

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 Appendix 9A.

9.3.4. Observational Before/After Evaluation Studies to Evaluate Shifts in Collision Crash Type Proportions

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 Appendix 9A.

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 a CMF function rather than a single value for a CMF.

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.

As shown in Table 9-3, 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.

Table 9-3. Observational Cross-Sectional Evaluation Study Design

Type of Site	Before Treatment	After Treatment
Treatment Sites		✓
Nontreatment Sites	✓	

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

Table 9-4 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 table indicates which type(s) of observational before/after evaluation studies are feasible. On the other hand, based on data availability, the information provided in Table 9-4 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).

Table 9-4. Selection Guide for Observational Before/After Evaluation Methods

	Data availability					
	Treatment sites		Nontreatment sites			
Safety measure to be evaluated	Before period data	After period data	Before period data	After period data	SPF	Appropriate evaluation study method
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. Table 9-5 illustrates the study design for an experimental before/after study.

Table 9-5. 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. Table 9-6 provides a tabular overview of the data needs for each of the safety evaluation methods discussed in this chapter.

Table 9-6. Overview of Data Needs and Inputs for Safety Effectiveness Evaluations

	Safety Evaluation Method			
Data Needs and Inputs	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). Figure 9-2 provides a step-by-step overview of the EB before/after safety effectiveness evaluation method.

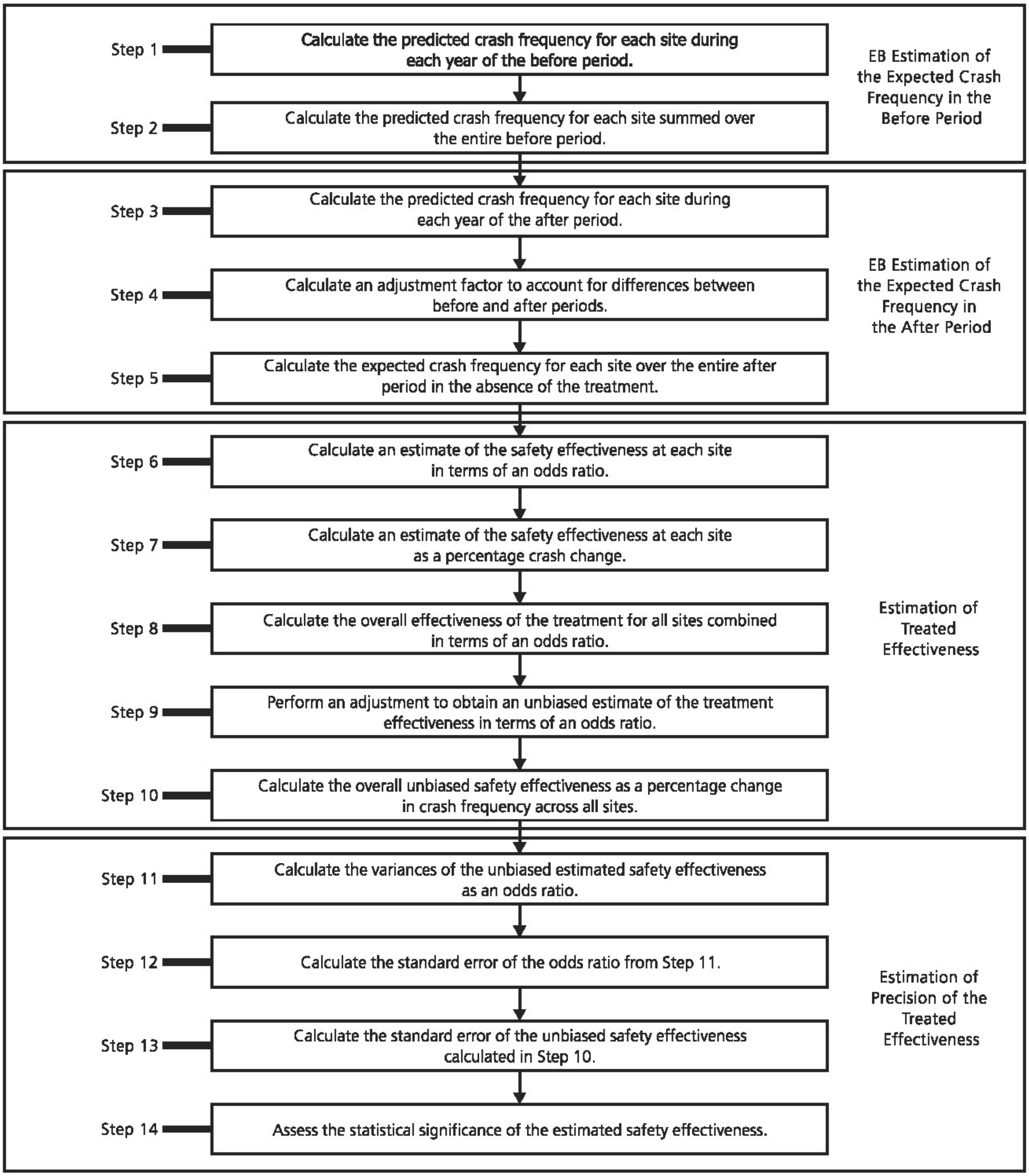


Figure 9-2. Overview of EB Before/After Safety Evaluation

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

An evaluation study can be performed with fewer sites or shorter time periods, or both, 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. 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 Appendix 9A.

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. Figure 9-3 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 crash data for the period before treatment implementation is recommended for both treatment and nontreatment sites.
- 3 to 5 years of crash data for the period after treatment implementation is recommended for both treatment and nontreatment sites.
- SPFs for treatment and nontreament sites.

An evaluation study can be performed with fewer sites or shorter time periods, or both, 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 or crash type, or both 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 or from SafetyAnalyst. 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 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.

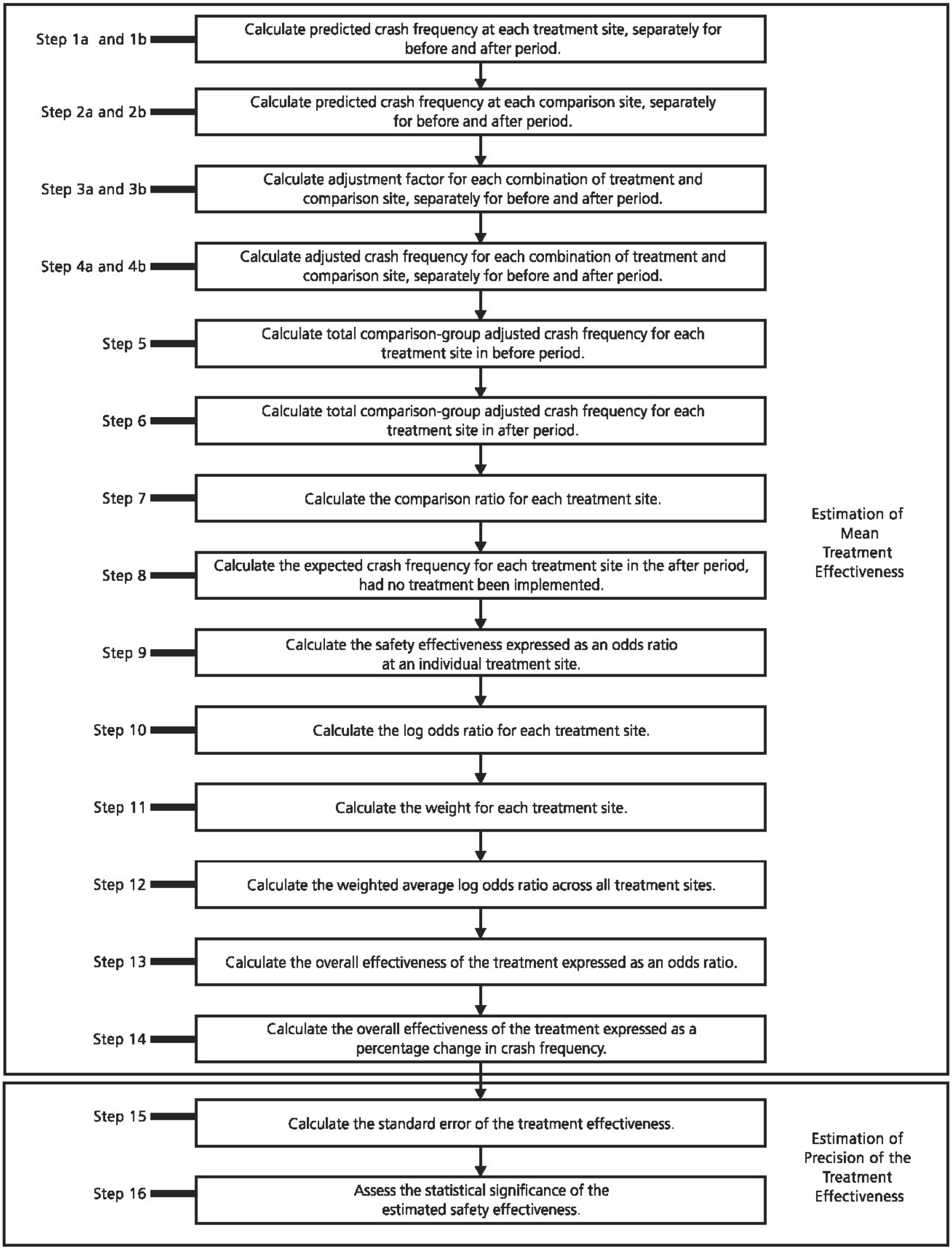


Figure 9-3. Overview of Before/After Comparison-Group Safety Evaluation

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 9A.

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-the-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). Figure 9-4 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 or shorter time periods, or both, but statistically significant results are less likely.

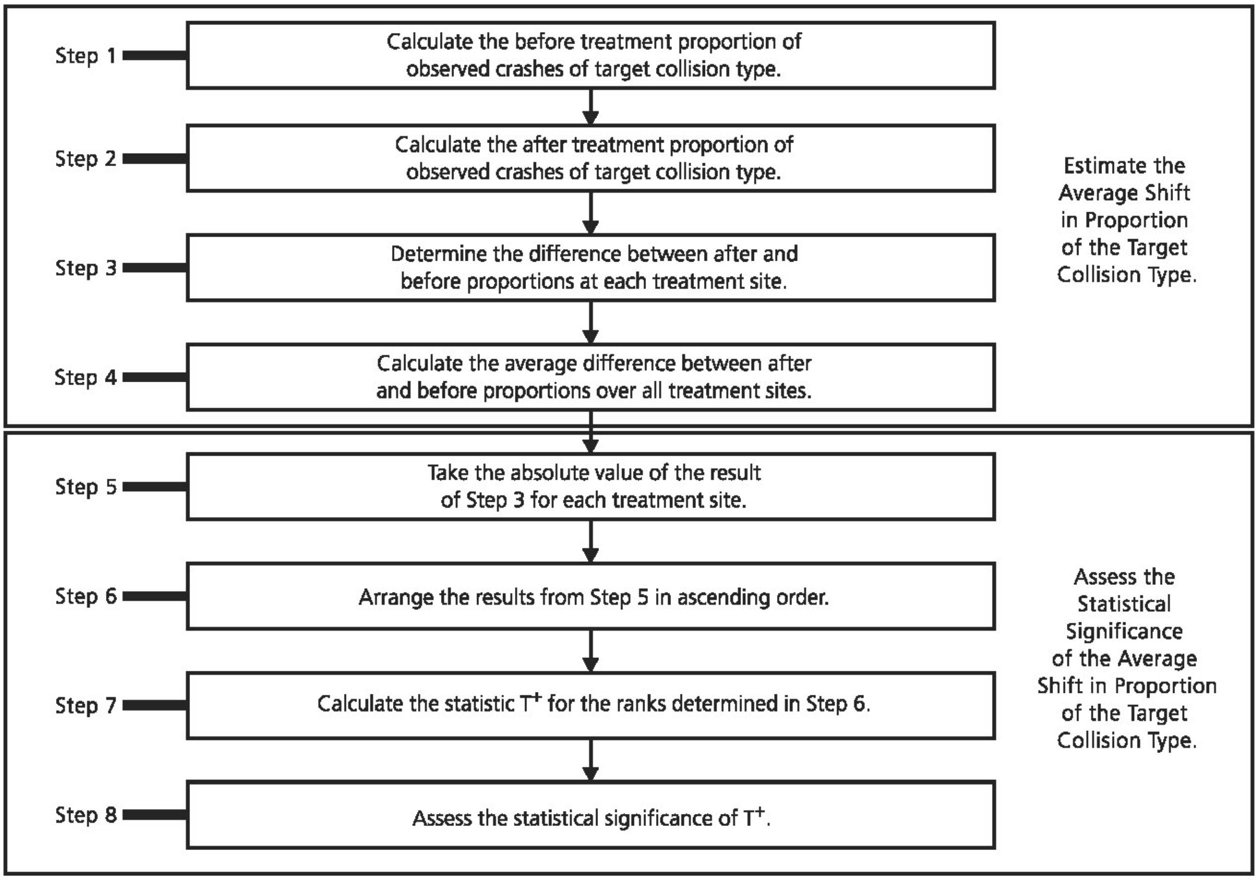


Figure 9-4. Overview Safety Evaluation for Before/After Shifts in Proportions

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 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.

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 Appendix 9A.

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 or crash type, or both, 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 example 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:

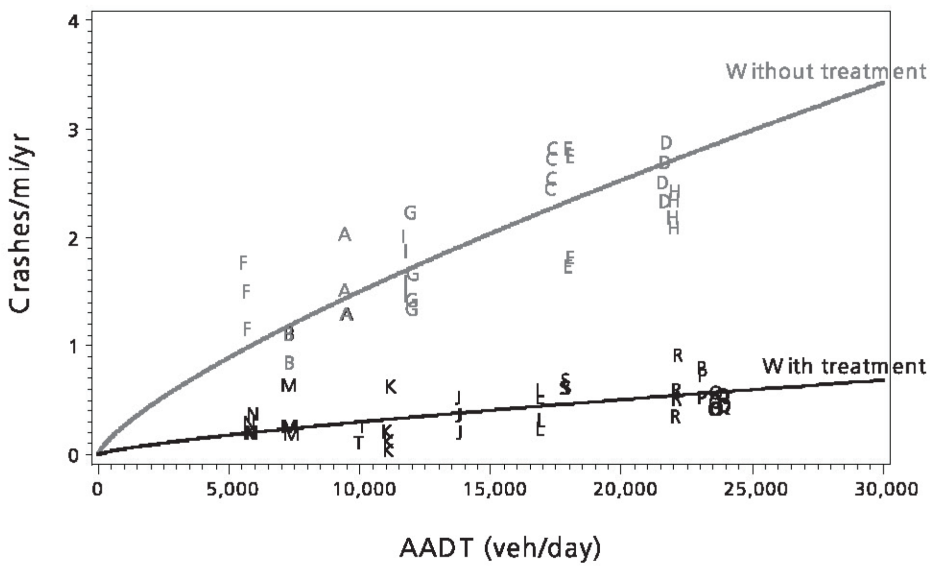
- 9 nontreatment sites (denoted A through I); 4 years of data at each site
- 11 treatment sites (denoted J through T); 4 years of data at each site

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.

Observed and Predicted Crash Frequencies at Treatment and Nontreatment Sites



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 CMFS AS A RESULT OF A SAFETY EFFECTIVENESS EVALUATION

A common application of safety effectiveness evaluation is to quantify the value of a CMF for a countermeasure by evaluating multiple sites where that countermeasure has been evaluated. The relationship between a CMF and safety effectiveness is given as CMF = (100 – Safety Effectiveness/100). Any of the study designs and evaluation methods presented in Sections 9.3 and 9.4 can be applied in quantifying a CMF 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 a CMF 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 a CMF for the project, as described above in Section 9.7. The economic analysis procedures presented in Chapter 7 are then applied to quantify the safety benefits of the projects in monetary terms, using the CMF, and to compare the safety benefits and costs of the implemented projects. Figure 9-5 provides a graphical overview of this comparison.

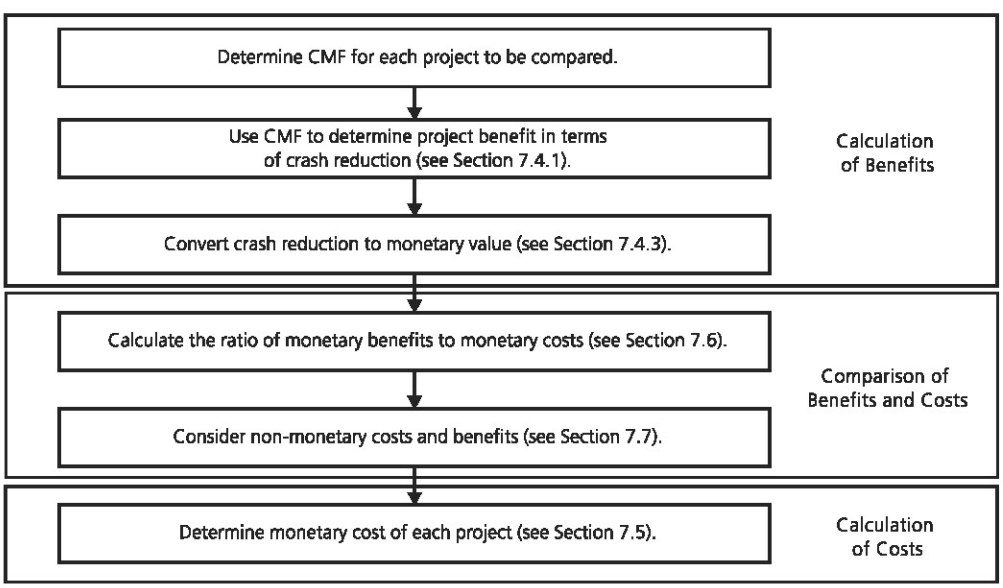


Figure 9-5. Overview of Safety Benefits and Costs Comparison of Implemented Projects

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 a CMF 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 9A 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 CMFs 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)
		AADT (veh/day)		Observed before total crash frequency by year (crashes/site/year)						Observed after total crash frequency by year (crashes/site/year)		
Site No.	Site length (L) (mi)	Before	After	Y1	Y2	Y3	Y4	Y5	Observed crash frequency in before period	Y1	Y2	Observed crash frequency in after period
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 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 as:

$$k = \frac{0.236}{L} \qquad (10\text{–}7)$$

Equation 10-1 presents the predicted average crash frequency for a specific site type x (roadway, rs, in this example). Note in this example all CMFs and the calibration factor are assumed to equal 1.0.

$$N_{predicted} = N_{spf\,x} \times (CMF_{1x} \times CMF_{2x} \times \ldots \times CMF_{yx}) \times C_x \qquad (10\text{-}1)$$

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*;

CMF_{yx} = Crash 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)
	Predicted before total crash frequency by year (crashes/year)					Predicted average crash frequency in before period
Site No.	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 9A.1-2, the calculated overdispersion parameter (shown in Column 20), and Column 19 (Step 1), calculate the weighted adjustment, *w*, for each site for the before period. The results appear in Column 21. Using Equation 9A.1-1, Columns 21, 19 (Step 1), and 10 (Basic Input Data), calculate the expected average crash frequency for each site, summed over the entire before period. The results appear in Column 22.

(1)	(20)	(21)	(22)
Site No.	Overdispersion parameter, *k*	Weighted adjustment, *w*	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)
	Predicted after total crash frequency (crashes/year)				
Site No.	Y1	Y2	Predicted average crash frequency in after period	Adjustment factor, *r*	Expected average crash frequency in after period without treatment
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 9A.1-3 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 9A.1-4 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 9A.1-5 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. 9A.1-11)
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 9A.1-6 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 9A.1-7 and the totals from Columns 13 and 27 (Step 6), calculate the overall effectiveness of the treatment for all sites combined, in the form of an odds ratio:

$$OR' = \frac{30}{42.88} = 0.700$$

Step 9—Calculate each Term of Equation 9A.1-9.
Using Columns 26 (Step 3), 22 (Step 2), and 21 (Step 2), calculate each term of Equation 9A.1-9. The results appear in Column 30 in the table presented in Step 6. Sum the terms in Column 30. Next, using Equations 9A.1-8 and 9A.1-9, the value for *OR'* from Step 8, and the sums in Column 30 and 27 in Step 6, 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 9A.1-10 and the above result, calculate the overall unbiased safety effectiveness as a percentage change in crash frequency across all sites:

Safety Effectiveness = 100 × (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 9A.1-11, the value for *OR'* from Step 8, and the sums from Columns 13, 30, and 27 in Step 6, 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 9A.1-12 and the result from Step 11, calculate the standard error of *OR*:

$$SE(OR) = \sqrt{0.019} = 0.138$$

Step 13—Calculate the Standard Error of the Safety Effectiveness.
Using Equation 9A.1-13 and the result from Step 12, calculate the standard error of the Safety Effectiveness:

SE(Safety Effectiveness) =100 × 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:

$$Abs\frac{\text{Safety Effectiveness}}{SE(\text{Safety Effectiveness})} = \frac{30.5}{13.85} = 2.20$$

Since *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] ≥ 2.0, conclude that the treatment effect is significant at the (approximate) 95 percent confidence level. The positive estimate of Safety Effectiveness, 30.5 percent, 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 percent 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 9A 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 9.10:

(1)	(2)	(3)	(4)	(5)	(6)
Treatment Sites					
		AADT (veh/day)		Observed crash frequency in before Period (5 years) ($N_{observed}$)	Observed crash frequency in after period (2 years) (*L*)
Site No.	Site length (*L*) (mi)	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					
		AADT (veh/day)			
Site No.	Site length (*L*) (mi)	Before	After	Observed crash frequency in before period (7 years)	Observed crash frequency in after period (3 years)
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 provides the applicable SPF for total crashes on rural two-lane roads:

$$N_{spf\,rs} = AADT \times L \times 365 \times 10^{-6} \times e^{(-0.312)} \qquad (10–6)$$

The overdispersion parameter for this SPF is not relevant to the comparison-group method.

Equation 10-1 presents the predicted average crash frequency for a specific site type *x* (roadway, *rs*, in this example). Note in this example all CMFs and the calibration factor are assumed to equal 1.0.

$$N_{\text{predicted}} = N_{spf\,x} \times (CMF_{1x} \times CMF_{2x} \times \ldots \times CMF_{yx}) \times C_x \qquad (10\text{-}1)$$

Where:

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

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

CMF_{yx} = Crash Modification Factors specific to site type *x* and 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 9A.2-1, 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 9A.2-1)												
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 9A.2-2, 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 sites. 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 9A.2-2)												
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.

(7)	(43)	(44)	(45)	(46)	(47)	(48)	(49)	(50)	(51)	(52)	(53)	(54)	(55)
	Comparison Group—Before Adjusted Crash Frequencies (Equation 9A.2-3)												
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

Using Equation 9A.2-3, 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.

Step 4b—Calculate the Expected Average Crash Frequencies in the after period for an individual comparison site. Similarly, using Equation 9A.2-4, 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 9A.2-4)												
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 9A.2-5, 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 9A.2-6, 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 9A.2-7, 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 9A.2-8, 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 9A.2-9, 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 9A.2-11 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	[a]	[a]	[a]	[a]
9	[a]	[a]	[a]	[a]
10	[a]	[a]	[a]	[a]
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

[a] 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 9A.2-13, 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 9A.2-12 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 9A.2-14 and the sums from Columns 76 and 77, calculate the weighted average log odds ratio (*R*) across all treatment sites:

$$R = \frac{5.86}{17.78} = 0.33$$

Step 13—Calculate the Overall Effectiveness of the Treatment expressed as an odds ratio.
Using Equation 9A.2-15 and the result from Step 12, calculate the overall effectiveness of the treatment, expressed as an odds ratio, *OR*, averaged across all sites:

$$OR = e^{(0.33)} = 1.391$$

Step 14—Calculate the Overall Safety Effectiveness, expressed as a percentage change in crash frequency, *CMF*, averaged across all sites.
Using Equation 9A.2-16 and the results from Step 13, calculate the overall safety effectiveness, expressed as a percentage change in crash frequency, Safety Effectiveness, averaged across all sites:

Safety Effectiveness = 100 × (1 – 1.391) = –39.1%

Note—The negative estimate of the Safety Effectiveness indicates a negative effectiveness, i.e., an increase in total crashes.

Step 15—Calculate the Precision of the Treatment Effectiveness.
Using Equation 9A.2-17 and the results from Step 13 and the sum from Column 76, calculate the precision of the treatment effectiveness:

$$SE(\text{Safety Effectiveness}) = 100\left(\frac{1.391}{\sqrt{17.78}}\right) = 33.0\%$$

Step 16—Assess the Statistical Significance of the Estimated Safety Effectiveness.
Assess the statistical significance of the estimated safety effectiveness by calculating the quantity:

$$Abs\left(\frac{\text{Safety Effectivness}}{SE(\text{Safety Effectiveness})}\right) = \frac{39.1}{33.0} = 1.18$$

Since *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] < 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 9A.3-1 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 9A.3-2 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 9A.3-3 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 9A.3-4, 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:

$$AvgP_{(FI)diff} = \frac{1.247}{13} = 0.10$$

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 9A.3-5, 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 9A.3-6:

$T^+ = 54$

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 9A.3-7 and Table 9A.3-1, obtain the upper and lower critical limits as:

- Upper limit—$t(\alpha_2,13) = 70$; this corresponds to an α_2 of 0.047, the closest value to 0.10/2
- Lower limit—$91 - t(\alpha_1,13) = 91 - 69 = 22$; here 69 corresponds to an α_1 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 percent) 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

(1) Griffin, L. I., and R. J. Flowers. *A Discussion of Six Procedures for Evaluating Highway Safety Projects*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, December 1997.

(2) Harwood, D. W., K. M. Bauer. I. B. Potts., D. J. Torbic. K. R. Richard, E. R. Kohlman Rabbani, E. Hauer, and L. Elefteriadou. *Safety Effectiveness of Intersection Left- and Right-Turn Lanes*. Report No. FHWA-RD-02-089. Federal Highway Administration, U.S. Department of Transportation, Washington, DC, April 2002.

(3) Harwood, D. W., et al. *SafetyAnalyst: Software Tools for Safety Management of Specific Highway Sites*. Federal Highway Administration, U.S. Department of Transportation, Washington, DC. More information available from http://www.safetyanalyst.org.

(4) Hauer, E. Cause and Effect in Observational Cross-Section Studies on Road Safety. *Transportation Research Board Annual Meeting CD-ROM*. TRB, National Research Council, Washington, DC, 2005.

(5) Hauer, E. *Observational Before-after Studies in Road Safety: Estimating the Effect of Highway and Traffic Engineering Measures on Road Safety*. Pergamon Press, Elsevier Science Ltd, Oxford, UK, 1997.

(6) Hauer, E., D. W. Harwood, F. M. Council., and M. S. Griffith. Estimating Safety by the Empirical Bayes Method: A Tutorial. In *Transportation Research Record 1784*. TRB, National Research Council, Washington, DC, 2002.

(7) Hollander, M., and D. A. Wolfe. *Nonparametric Statistical Methods*. John Wiley & Sons, Inc., Hoboken, NJ, 1973.

(8) Lord, D. and B. N. Persaud, 2000. Accident Prediction Models with and without Trend: Application of the Generalized Estimating Equation Procedure. In *Transportation Research Record 1717*. TRB, National Research Council, Washington, DC, pp. 102–108.

(9) Lyon, C., B. N. Persaud, N. X. Lefler, D. L. Carter, and K. A. Eccles. Safety Evaluation of Installing Center Two-Way Left-Turn Lanes on Two-Lane Roads. In *Transportation Research Record 2075*, TRB, National Research Council, Washington, DC, 2008, pp. 34–41.

(10) Persaud, B. N., R. A. Retting, P. E. Garder, and D. Lord. Safety Effect of Roundabout Conversions in the United States: Empirical Bayes Observational Before-After Studies. In *Transportation Research Record 1751*. TRB, National Research Council, Washington, DC, 2001.

APPENDIX 9A—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.

9A.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 as follows.

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:

$$N_{predicted} = N_{spf} \times (CMF_{1x} \times CMF_{2x} \times \ldots \times CMF_{yx}) \times C_x$$

However, for this level of evaluation, it may be assumed that all* CMFs *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.

$$N_{expected,B} = w_{i,B} N_{predicted} + (1 - w_{i,B}) N_{observed,B} \quad (9A.1\text{-}1)$$

Where the weight, $w_{i,B}$, for each site *i*, is determined as:

$$w_{i,B} = \frac{1}{1 + k \sum_{\substack{\text{Before}\\ \text{years}}} N_{predicted}} \quad (9A.1\text{-}2)$$

and:

$N_{expected}$ = Expected average crash frequency at site *i* for the entire before period

N_{spfx} = 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 an 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:

$$r_i = \frac{\sum_{\substack{\text{After}\\ \text{years}}} N_{predicted,A}}{\sum_{\substack{\text{Before}\\ \text{years}}} N_{predicted,B}} \quad (9A.1\text{-}3)$$

Step 5—Calculate the expected average crash frequency, $N_{expected}$, for each site *i*, over the entire after period in the absence of the treatment as:

$$N_{expected,A} = N_{expected,B} \times r_i \quad (9A.1\text{-}4)$$

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:

$$OR_i = \frac{N_{observed,A}}{N_{expected,A}} \quad (9A.1\text{-}5)$$

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* as:

$$\text{Safety Effectiveness}_i = 100 \times (1 - OR_i) \quad (9A.1\text{-}6)$$

Step 8—Calculate the overall effectiveness of the treatment for all sites combined, in the form of an odds ratio, *OR′*, as follows:

$$OR' = \frac{\sum_{\text{All sites}} N_{observed,A}}{\sum_{\text{All sites}} N_{expected,A}} \quad (9A.1\text{-}7)$$

Step 9—The odds ratio, *OR′*, calculated in Equation 9A.1-7 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:

$$OR = \frac{OR'}{1 + \dfrac{Var\left(\sum_{\text{All sites}} N_{expected,A}\right)}{\left(\sum_{\text{All sites}} N_{expected,A}\right)^2}} \quad (9A.1\text{-}8)$$

Where:

$$Var\left(\sum_{\text{All sites}} N_{expected,A}\right) = \sum_{\text{All sites}} \left[(r_i)^2 \times N_{expected,B} \times (1 - w_{i,B})\right] \quad (9A.1\text{-}9)$$

and $w_{i,B}$ is defined in Equation 9A.1-2 and r_i is defined in Equation 9A.1-3.

Step 10—Calculate the overall unbiased safety effectiveness as a percentage change in crash frequency across all sites as:

$$\text{Safety Effectiveness} = 100 \times (1 - OR) \qquad (9A.1\text{-}10)$$

Estimation of the Precision of the Treatment Effectiveness

To assess whether the estimated safety effectiveness of the treatment is statistically significant, one needs to determine its precision. This is done by first calculating the precision of the odds ratio, *OR*, in Equation 9A.1-8. 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:

$$Var(OR) = \frac{(OR')^2 \left[\frac{1}{N_{\text{observed},A}} + \frac{Var\left(\sum_{\text{All sites}} N_{\text{expected},A} \right)}{\left(\sum_{\text{All sites}} N_{\text{expected},A} \right)^2} \right]}{\left[1 + \frac{Var\left(\sum_{\text{All sites}} N_{\text{expected},A} \right)}{\left(\sum_{\text{All sites}} N_{\text{expected},A} \right)^2} \right]} \qquad (9A.1\text{-}11)$$

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:

$$SE(OR) = \sqrt{Var(OR)} \qquad (9A.1\text{-}12)$$

Step 13—Using the relationship between *OR* and Safety Effectiveness shown in Equation 9A.1-10, the standard error of Safety Effectiveness, *SE*(Safety Effectiveness), is calculated as:

$$SE(\text{Safety Effectiveness}) = 100 \times SE(OR) \qquad (9A.1\text{-}13)$$

Step 14—Assess the statistical significance of the estimated safety effectiveness by making comparisons with the measure *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] and drawing conclusions based on the following criteria:

- If *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] < 1.7, conclude that the treatment effect is not significant at the (approximate) 90 percent confidence level.
- If *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] ≥ 1.7, conclude that the treatment effect is significant at the (approximate) 90 percent confidence level.
- If *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] ≥ 2.0, conclude that the treatment effect is significant at the (approximate) 95 percent confidence level.

9A.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.

Note—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 AADT and number of before and after years. The notation is as follows:

- Subscript *i* denotes a treatment site, $i=1,\ldots,n$, where n denotes the total number of treatment sites
- Subscript *j* denotes a comparison site, $j=1,\ldots,m$, 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 observed 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 AADT, calculate $\sum N_{predicted,T,B}$, the sum of the predicted average crash frequencies at treatment site *i* in before period.

Step 1b—Using the applicable SPF and site-specific AADT, calculate $\sum N_{predicted,T,A}$, the sum of the predicted average crash frequencies at treatment site *i* in after period.

Step 2a—Using the applicable SPF and site-specific AADT, calculate $\sum N_{predicted,C,B}$, the sum of the predicted average crash frequencies at comparison site *j* in before period.

Step 2b—Using the applicable SPF and site-specific AADT, calculate $\sum N_{predicted,C,A}$, 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:

$$Adj_{i,j,B} = \frac{N_{predicted,T,B}}{N_{predicted,C,B}} \times \frac{Y_{BT}}{Y_{BC}} \tag{9A.2-1}$$

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:

$$Adj_{i,j,A} = \frac{N_{\text{predicted},T,A}}{N_{\text{predicted},C,A}} \times \frac{Y_{AT}}{Y_{AC}} \tag{9A.2-2}$$

Where:

$N_{\text{predicted},T,A}$ = Sum of predicted average crash frequencies at treatment site *i* in after period using the appropriate SPF and site-specific AADT;

$N_{\text{predicted},C,A}$ = 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 9A.2-1, calculate the expected average crash frequencies in the before period for each comparison site *j* and treatment site *i* combination, as follows:

$$N_{\text{expected},C,B} = \sum_{\text{All sites}} N_{\text{observed},C,B} \times Adj_{i,j,B} \tag{9A.2-3}$$

Where:

$\sum N_{\text{observed},C,B}$ = Sum of observed crash frequencies at comparison site *j* in the before period

Step 4b—Using the adjustment factor calculated in Equation 9A.2-2, calculate the expected average crash frequencies in the after period for each comparison site *j* and treatment site *i* combination, as follows:

$$N_{\text{expected},C,A} = \sum_{\text{All sites}} N_{\text{observed},C,A} \times Adj_{i,j,A} \tag{9A.2-4}$$

Where:

N_j = Sum of observed crash frequencies at comparison site *j* in the after period

Step 5—For each treatment site *i*, calculate the total comparison-group expected average crash frequency in the before period as follows:

$$N_{\text{expected},C,B,\text{total}} = \sum_{\text{All comparison sites}} N_{\text{expected},C,B} \tag{9A.2-5}$$

Step 6—For each treatment site *i*, calculate the total comparison-group expected average crash frequency in the after period as follows:

$$N_{\text{expected},C,A,\text{total}} = \sum_{\text{All comparison sites}} N_{\text{expected},C,A} \qquad (9A.2\text{-}6)$$

Step 7—For each treatment site *i*, calculate the comparison ratio, r_{iC}, as the ratio of the comparison-group expected average crash frequency after period to the comparison-group expected average crash frequency in the before period at the comparison sites as follows:

$$r_{i,C} = \frac{N_{\text{expected},C,A,\text{total}}}{N_{\text{expected},C,B,\text{total}}} \qquad (9A.2\text{-}7)$$

Step 8—Using the comparison ratio calculated in Equation 9A.2-7, calculate the expected average crash frequency for a treatment site *i* in the after period, had no treatment been implement as follows:

$$N_{\text{expected},T,A} = \sum_{\text{All sites}} N_{\text{observed},T,B} \times r_{iC} \qquad (9A.2\text{-}8)$$

Step 9—Using Equation 9A.2-9, 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:

$$OR_i = \sum_{\text{All sites}} \frac{N_{\text{observed},T,A}}{N_{\text{expected},T,A}} \qquad (9A.2\text{-}9)$$

or alternatively,

$$OR_i = \frac{N_{\text{observed},T,A,\text{total}}}{N_{\text{observed},T,B,\text{total}}} \times \frac{N_{\text{expected},C,B,\text{total}}}{N_{\text{expected},C,A,\text{total}}} \qquad (9A.2\text{-}10)$$

Where:

$N_{\text{observed},T,A}$,total and $N_{\text{observed},T,B,\text{total}}$ represent the total treatment group observed crash frequencies at treatment site *i* calculated as the sum of $N_{\text{observed},T,A}$ and $N_{\text{observed},T,B}$ 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:

$$R_i = \ln(OR_i) \qquad (9A.2\text{-}11)$$

Where the ln function represents the natural logarithm.

Step 11—For each treatment site *i*, calculate the weight w_i as follows:

$$w_i = \frac{1}{R^2_{i(se)}} \qquad (9A.2\text{-}12)$$

Where:

$$R^2_{i(SE)} = \frac{1}{N_{\text{observed},T,B,\text{total}}} + \frac{1}{N_{\text{observed},T,A,\text{total}}} + \frac{1}{N_{\text{expected},C,B,\text{total}}} + \frac{1}{N_{\text{expected},C,A,\text{total}}} \quad (9A.2\text{-}13)$$

Step 12—Using Equation 9A.2-14, calculate the weighted average log odds ratio, *R*, across all *n* treatment sites as:

$$R = \frac{\sum_n w_i R_i}{\sum_n w_i} \quad (9A.2\text{-}14)$$

Step 13—Exponentiating the result from Equation 9A.2-14, calculate the overall effectiveness of the treatment, expressed as an odds ratio, *OR*, averaged across all sites, as follows:

$$OR = e^R \quad (9A.2\text{-}15)$$

Step 14—Calculate the overall safety effectiveness, expressed as a percentage change in crash frequency averaged across all sites as:

$$\text{Safety Effectiveness} = 100 \times (1 - R) \quad (9A.2\text{-}16)$$

Step 15—To obtain a measure of the precision of the treatment effectiveness, calculate its standard error, *SE*(Safety Effectiveness), as follows:

$$SE(\text{Safety Effectiveness}) = 100 \frac{OR}{\sqrt{\sum_n w_i}} \quad (9A.2\text{-}17)$$

Step 16—Assess the statistical significance of the estimated safety effectiveness by making comparisons with the measure *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] and drawing conclusions based on the following criteria:

- If *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] < 1.7, conclude that the treatment effect is not significant at the (approximate) 90 percent confidence level.
- If *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] ≥ 1.7, conclude that the treatment effect is significant at the (approximate) 90 percent confidence level.
- If *Abs*[Safety Effectiveness/*SE*(Safety Effectiveness)] ≥ 2.0, conclude that the treatment effect is significant at the (approximate) 95 percent confidence level.

9A.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, follows.

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 "*CT*") are relative to total crashes (subscript "total").

- $N_{observed,B,total}$ denotes the observed number of total 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.
- $N_{observed,A,total}$ denotes the observed number of total 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 (total) at treatment site *i*, $P_{i(CT)B}$, across the entire before period as follows:

$$P_{i(CT)B} = \frac{N_{\text{obseved},B,CT}}{N_{\text{obseved},B,\text{total}}} \tag{9A.3-1}$$

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:

$$P_{i(CT)A} = \frac{N_{\text{obseved},A,CT}}{N_{\text{obseved},A,\text{total}}} \tag{9A.3-2}$$

Step 3—Determine the difference between the after and before proportions at each treatment site *i* as follows:

$$P_{i(CT)diff} = P_{i(CT)A} - P_{i(CT)B} \tag{9A.3-3}$$

Step 4—Calculate the average difference between after and before proportions over all *n* treatment sites as follows:

$$AvgP_{(CT)diff} = \frac{1}{n} \sum_{\substack{\text{Treat}\\ \text{sites}}} P_{i(CT)diff} \tag{9A.3-4}$$

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 9A.3-4 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 9A.3-4 is significantly different from zero at a predefined confidence level.

Step 5—Take the absolute value of the non-zero $P_{i(CT)diff}$ calculated in Equation 9A.3-3. For simplicity of notation, let Z_i denote the absolute value of $P_{i(CT)diff}$, thus:

$$Z_i = abs(P_{i(CT)\,diff}) \tag{9A.3-5}$$

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, for example, 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 positive differences (i.e., positive values of $P_{i(CT)diff}$), calculate the statistic T^+ as follows:

$$T^+ = \sum_{n^*} R_i^+ \qquad (9A.3\text{-}6)$$

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:

$$T^+ \geq t(a_2, n^*) \quad \text{or} \quad T^+ \leq \frac{n^*(n^*+1)}{2} - t(a_1, n^*) \qquad (9A.3\text{-}7)$$

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 Table 9A.3-1. 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 Table 9A.3-1 and α_1 and α_2 are each closest to $\alpha/2$. Often, $\alpha_1 = \alpha_2$ are the closest values to $\alpha/2$.

Table 9A.3-1 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, then 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). Table 9A.3-1 shows one-sided probability levels; since the test performed here is a two-sided test, the values in Table 9A.3-1 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 Table 9A.3-1

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 Table 9A.3-1, 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 percent confidence level [where $90.2 = 1 - 2 \times 0.049$] based on Equation 9A.3-7.

Table 9A.3-1. Upper Tail Probabilities for the Wilcoxon Signed Rank T^+ Statistic (n^* = 4 to 10)[a] (8)

	Number of sites (n^*)						
x	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 $t(\alpha, n^*) = x$.

Table 9A.3-1 (Continued). Upper Tail Probabilities for the Wilcoxon Signed Rank T^+ Statistic (n^* = 11 to 15)[a] (8)

	Number of sites (n^*)				
x	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 ($n^* > 15$)

Table 9A.3-1 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:**

$$T^* \leq \frac{T^+ - E_0(T^+)}{\sqrt{Var_0}(T^+)} \quad (9A.3\text{-}8)$$

Where:

$$E_0\left(T^+\right) = \frac{n^*\left(n^*+1\right)}{4} \quad (9A.3\text{-}9)$$

and

$$Var_0(T^+) = \frac{\left[n^*(n^*+1)(2n^*+1) - \frac{1}{2}\sum_{j=1}^{g} t_j(t_j-1)(t_j+1)\right]}{24} \quad (9A.3\text{-}10)$$

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:

$$T^* \geq z_{(\alpha/2)} \text{ or } T^* \leq -z_{(\alpha/2)} \quad (9A.3\text{-}11)$$

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.