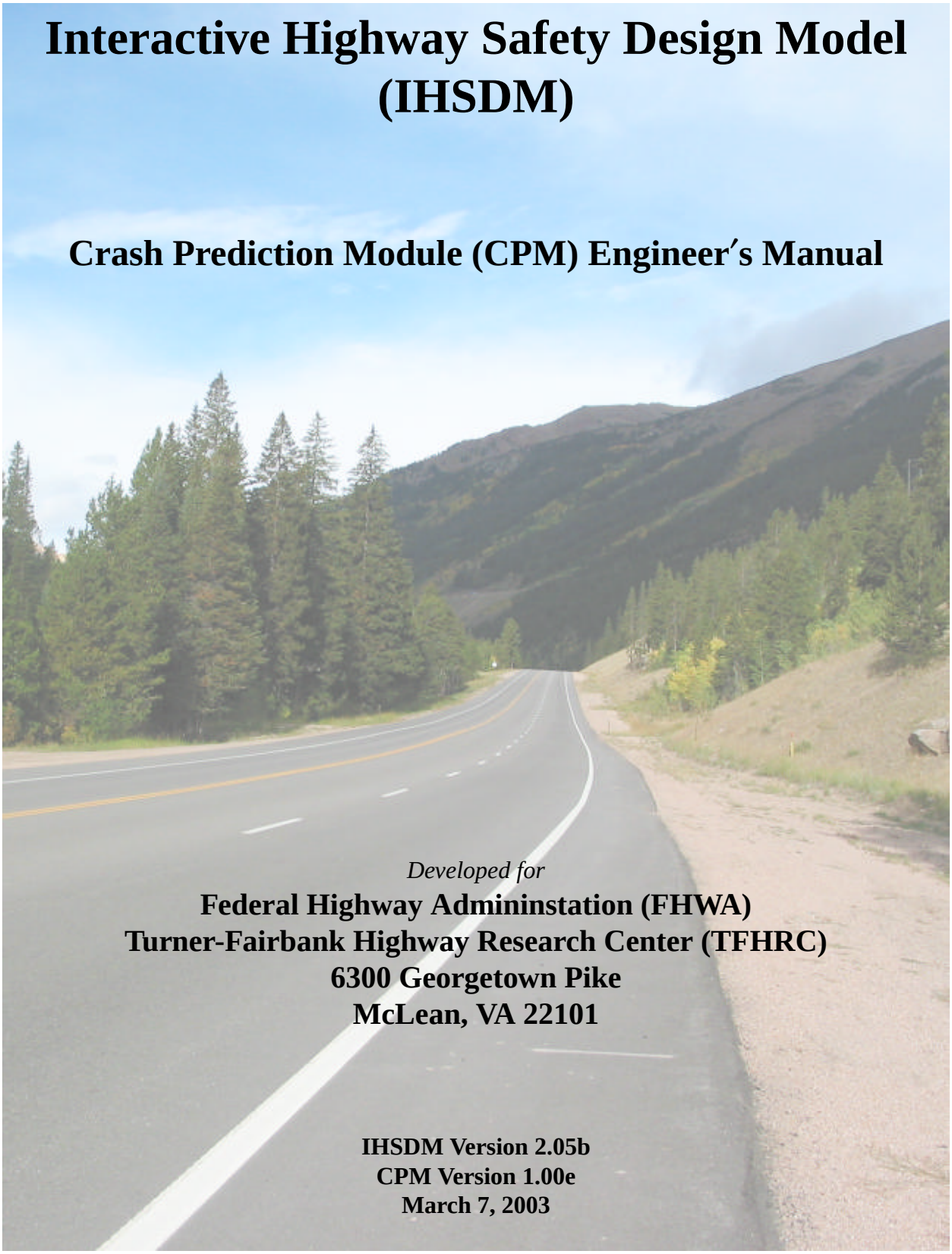
Interactive Highway Safety Design Model (IHSDM)

Crash Prediction Module (CPM) Engineer′s Manual



Developed for

Federal Highway Admininstation (FHWA)
Turner-Fairbank Highway Research Center (TFHRC)
6300 Georgetown Pike
McLean, VA 22101

IHSDM Version 2.05b
CPM Version 1.00e
March 7, 2003

Disclaimer

This document is disseminated under the sponsorship of the Department of Transportation in the interest of information exchange. The United States Government assumes no liability for its content or use thereof. This document does not constitute a standard, specification, or regulation.

The United States Government does not endorse products or manufacturers. Trade and manufacturers' names may appear in this document only because they are considered essential to the objective of the document.

Limited Warranty and Limitations of Remedies

This software product is provided "as-is," without warranty of any kind-either expressed or implied (but not limited to the implied warranties of merchantability and fitness for a particular purpose). The FHWA and distributor do not warrant that the functions contained in the software will meet the end-user's requirements or that the operation of the software will be uninterrupted and error-free.

Under no circumstances will the FHWA or the distributor be liable to the end-user for any damages or claimed lost profits, lost savings, or other incidental or consequential damages rising out of the use or inability to use the software (even if these organizations have been advised of the possibility of such damages), or for any claim by any other party.

Notice

The use and testing of the IHSDM software is being done strictly on a voluntary basis. In exchange for provision of IHSDM, the user agrees that the Federal Highway Administration (FHWA), U.S. Department of Transportation and any other agency of the Federal Government shall not be responsible for any errors, damage or other liability that may result from any and all use of the software, including installation and testing of the software. The user further agrees to hold the FHWA and the Federal Government harmless from any resulting liability. The user agrees that this hold harmless provision shall flow to any person to whom or any entity to which the user provides the IHSDM software. It is the user's full responsibility to inform any person to whom or any entity to which it provides the IHSDM software of this hold harmless provision.

Table of Contents

1. Introduction 1
1.1 Overview of IHSDM 1
1.2 Overview of Crash Prediction Module 2
1.3 Purpose and Organization of this Manual 3
2. Data Input Requirements 4
2.1 Analysis Data 4
2.2 Geometric and Traffic Control Data 4
2.2.1 Highway Segment Geometric and Traffic Control Data 4
2.2.2 Intersection Geometric and Traffic Control Data 5
2.3 Traffic Volume Data 6
2.3.1 Highway Segment Traffic Volume Data 6
2.3.2 Intersection Traffic Volume Data 6
2.4 Crash History Data 6
2.4.1 Highway Segment Crash History Data 6
2.4.2 Intersection Crash History Data 7
3. Crash Prediction Procedural Elements 7
3.1 Segmentation Procedure 7
3.2 Base Models (Segments and Intersections) 7
3.3 AMFs (Segments and Intersections) 7
3.4 Calibration Factors 8
3.5 Crash Prediction When Site-Specific Crash History Data are Available 8
3.5.1 Caution on Use of CPM Results When Site-Specific Crash History Data Are Available 9
3.5.2 Situations in Which the EB Procedure Should and Should Not Be Applied 9
3.5.3 Example to Illustrate When to Use/Not Use Historical Crash Data 10
4. CPM Output 11
4.1 Highway Data 11
4.1.1 Highway Segment Data (Current and Proposed) 11
4.1.2 Horizontal Curve Data (Current and Proposed) 12
4.1.3 Intersection Data (Current and Proposed) 13
4.1.4 Segment Traffic Volume (Current and Proposed) 13
4.1.5 Intersecting Highway Traffic Volume (Current and Proposed) 13
4.2 Analysis Report 14
4.2.1 Expected Crash Frequencies and Rates (for entire project within analysis limits) 14
4.2.2 Expected Crash Type Distributions 14
4.2.3 Reporting Results by Homogeneous Analysis Section 14
4.2.3.1 Caution on ’Microanalysis’ Use of CPM Results 14
4.2.3.2 Expected Crash Frequencies and Rates (by highway segment and intersection) 15
4.2.3.3 Expected Crash Frequencies and Rates by Horizontal Design Element 15
4.2.3.4 Plot of Expected Crash Rates by Highway Segment and Plot of Expected

Accident Rates Plotted Using a Sliding Scale 15
4.2.3.5 Plot of Expected Crash Rates by Highway Segment and Plot of Expected Accident Rates Plotted Using a Sliding Scale 16
4.2.3.6 Plot of Expected Crash Frequencies by Intersection 16
4.2.4 Historical Crash Data Plots (Future) 16
4.2.4.1 Historical Crashes Plotted by Specific Location 16
4.2.4.2 Plot of Historical Crashes When Crashes are Assigned to Homogeneous Highway Segments and Intersections 16
4.2.4.3 Plot of Historical Crashes When Crashes are Assigned to Horizontal Design Elements 16
5. Crash Prediction Algorithm 17
6. AMFs for Highway Segments 28
6.1 Lane Width (AMF_1) 28
6.2 Shoulder Width and Type (AMF_2) 29
6.3 Horizontal Curve Length, Radius, and Presence or Absence of Spiral Transition (AMF_3) 31
6.4 Superelevation (AMF_4) 31
6.5 Grades (AMF_5) 32
6.6 Driveway Density (AMF_6) 37
6.7 Passing Lanes and Short Four-Lane Sections (AMF_7) 37
6.8 Two-Way Left-Turn Lanes (AMF_8) 38
6.9 Roadside Hazard Rating (AMF_9) 38
6.10 Reference 38
7. AMFs for Intersections 38
7.1 Intersection Skew Angle (AMF_{10}) 38
7.2 Intersection Traffic Control (AMF_{11}) 39
7.3 Intersection Left-Turn Lanes (AMF_{12}) 39
7.4 Intersection Right-Turn Lanes (AMF_{13}) 39
7.5 Intersection Sight Distance (AMF_{14}) 40
8. Examples 40
8.1 Example 1: When Data are Stored by Station Range (Non-overlapping Crash Segments) 40
8.2 Example 2: When Data are Stored by Station Range (Overlapping Crash Segments) 43
8.3 Example 3: Applying the Sliding Scale Analysis 46
9. Default Tables 48
9.1 Default Percentage Distributions for Crash Severity Level and for Crash Type and Manner of Collision 48
9.2 Default Percentage Distributions for Crash Type and Manner of Collision on Rural Two-Lane Highways 49
10. Glossary 50
11. IHSDM Documentation 54
Index 58

List of Figures

Figure 1 Crash Prediction Algorithm Flowchart (Steps 1 - 4) 18
Figure 2 Crash Prediction Algorithm Flowchart (Steps 5 - 16) 20
Figure 3 Schematic of Example 1 highway with Historical Crash Data Stored by Station Range 41
Figure 4 Schematic of Example 2 Highway with Crash History Data Stored by Range 44
Figure 5 Plot of Predicted Crashes for Example Highway by Station 47
Figure 6 Example Highway with End Sections by Station Sliding Scale Plot (No Endpoints) 48
Figure 7 Example Highway with End Sections by Station Sliding Scale Plot (With Endpoints) 48
Figure 8 CPM Intersection Skew 53
Figure 9 User Documentation Map 57

List of Tables

Table 1. Overdispersion Parameters for Base Models 26
Table 2. Values of AMF for Lane Width of Highway Segments (AMF_{ra}) 29
Table 3. Values of AMF for Shoulder Width of Highway Segments (AMF_{wra}) 30
Table 4. Accident Modification Factors for Shoulder Width (SW) and Shoulder Type on Two-Lane Highways (AMF_{tra}) 30
Table 5. Design Superelevation (e_{design}) as a Function of Maximum Superelevation Rate, Curve Radius (e_{max}=0.04) and Design Speed (V mi/h) 33
Table 6. Design Superelevation (e_{design}) as a Function of Maximum Superelevation Rate, Curve Radius (e_{max}=0.06) and Design Speed (V mi/h) 34
Table 7. Design Superelevation (e_{design}) as a Function of Maximum Superelevation Rate, Curve Radius (e_{max}=0.08) and Design Speed (V mi/h) 35
Table 8. Design Superelevation (e_{design}) as a Function of Maximum Superelevation Rate, Curve Radius (e_{max}=0.10) and Design Speed (V mi/h) 36
Table 9. Design Superelevation (e_{design}) as a Function of Maximum Superelevation Rate, Curve Radius (e_{max}=0.12) and Design Speed (V mi/h) 37
Table 10. Accident Modification Factors for Installation of Left-turn Lanes on the Major Legs to Intersections 39
Table 11. Accident Modification Factors for Installation of Right-turn Lanes on the Major Legs to Intersections 40
Table 12. Accident Modification Factors for Intersection Sight Distance Limitations in Quadrants of Three-Leg and Four-Leg Intersections with Minor STOP Control 40
Table 13. Highway Segment Data and Calculations for Example 1 41
Table 14. Highway Segment Data and Calculations for Example 2, Crash Segment 2 45

Table 15. Default Percentage Distributions for Crash Severity Level and for Crash Type and Manner of Collision 49
Table 16. Default Percentage Distributions for Crash Type and Manner of Collision on Rural Two-Lane Highways 50

1. Introduction

This engineer's manual is one component of the documentation supporting the Interactive Highway Safety Design Model (IHSDM). This introductory section: (1) provides a brief overview of IHSDM, (2) summarizes the capabilities and intended uses of the Crash Prediction Module (CPM), and (3) states the purpose and organization of this engineer's manual.

1.1 Overview of IHSDM

IHSDM is a suite of software analysis tools for evaluating safety and operational effects of geometric design in the highway project development process. The scope of the current release of IHSDM is two-lane rural highways.

IHSDM is intended as a supplementary tool to augment the design process. This tool is designed and intended to predict the functionality of proposed or existing designs by applying chosen design guidelines and generalized data to predict performance of the design. This tool is NOT a substitute for engineering judgment and does not create a standard, guideline or prescriptive requirement that can be argued to create any standard of care upon a designer, highway agency or other governmental body or employee. The use of this tool for any purpose other than to aid a qualified design engineer in the review of a set of plans is beyond the designed scope of this tool and is not endorsed by the Federal Highway Administration (FHWA).

The suite of IHSDM tools includes the following evaluation modules. Each module of IHSDM evaluates an existing or proposed geometric design from a different perspective and estimates measures describing one aspect of the expected safety and operational performance of the design.

- Policy Review Module (PRM) - The Policy Review Module checks a design relative to the range of values for critical dimensions recommended in AASHTO design policy.
- Crash Prediction Module (CPM) - The Crash Prediction Module provides estimates of expected crash frequency and severity.
- Design Consistency Module (DCM) - The Design Consistency Module estimates expected operating speeds and measures of operating-speed consistency.
- Intersection Review Module (IRM) - The Intersection Review Module leads users through a systematic review of intersection design elements relative to their likely safety and operational performance.
- Traffic Analysis Module (TAM) - The Traffic Analysis Module estimates measures of traffic operations used in highway capacity and quality of service evaluations.

Intended users of IHSDM results are geometric design decision makers in the highway design process, including project managers, planners, designers, and reviewers. The Federal Highway Administration's Flexibility in Highway Design document (Publication No. FHWA-PD-97-062) explains the context within which these decision makers operate:

> An important concept in highway design is that every project is unique. The setting and character of the area, the values of the community, the needs of the highway users, and the challenges and opportunities are unique factors that designers must consider in each highway project. Whether the design to be developed is for a modest safety improvement or 10 miles of new-location rural freeway, there are no patented solutions. For each potential project, designers are faced with the task of balancing the need for the improvement with the need to safely integrate the design into the surrounding natural and human environment.

The measures of expected safety and operational performance estimated by IHSDM are intended as inputs to the decision making process. The value added by IHSDM is in providing quantitative estimates of effects that previously could be considered only in more general, qualitative terms. The advantage of these quantitative estimates is that, when used appropriately by knowledgeable decision makers, they permit more informed decision-making.

The following general cautions should be considered in using IHSDM:

- Measures of expected safety and operational performance from IHSDM are only a subset of the large number of inputs that must be considered in making design decisions.
- Estimates from IHSDM are expected values, in the statistical sense, i.e., they represent the estimated average performance over a long time period and among a large number of sites with similar characteristics. Actual performance may vary over time and among sites. The estimates from IHSDM should not substitute for, but rather should supplement and complement local knowledge.
- While derived from the best available data using the best available methods, both the available data and methods have limitations. The engineer's manuals for each module document limitations that should be understood to apply appropriately the resulting estimates.

1.2 Overview of Crash Prediction Module

The CPM estimates the frequency and severity of crashes that would be expected on a highway considering its geometric design and traffic characteristics. The crash prediction algorithm combines base models and accident modification factors (AMFs). Jurisdictions may enter calibration factors to adjust estimates to be comparable to the their reported crash experience. The algorithm also provides a procedure to combine model estimates with highway-specific crash data.

Separate base models have been calibrated, using state-of-the-art statistical techniques, for highway segments and three types of at-grade intersections: three-leg intersections with stop control on the minor-road approach, four-leg intersections with stop control on the minor-road approach, and four-leg signalized intersections. The base models estimate the total crash frequency during a specified time period on a highway segment or at an intersection under base geometric conditions and actual traffic conditions.

The AMFs adjust the base model estimates for individual geometric design dimensions and traffic control features. The factors are the product of expert panels, which reviewed related research findings, and reflect their consensus estimate of the quantitative safety effects of each design and traffic control feature.

Each base model was developed using data from one or two States. Reported crash frequencies for nominally similar highway segments or intersections are known to vary widely among States due to differences in climate, animal population, driver populations, and accident reporting thresholds and practices. Therefore, each State should develop and input their own calibration factor, in order to adjust model estimates to be more comparable to their crash experience.

The final component of the Crash Prediction Module is an Empirical Bayes procedure to combine model estimates with locally available crash data for an existing highway. Locally available crash data reflect highway-specific variables that are not accounted for by the base models or accident modification factors. In theory, a weighted combination of model estimates and locally available crash data should produce a more accurate estimate of expected crash

frequency than either one by itself.

The module provides a quantitative basis for comparing the expected safety performance of design alternatives. Intended uses include comparison of project-wide crash estimates against State-wide crash experience, comparisons of the relative safety performance of individuals segments or intersections within an existing or proposed design that may merit more detailed evaluation, and safety cost-effectiveness evaluations of the costs versus the safety benefits of design alternatives.

Since, in the past, expected crash frequencies have not been routinely available to project decision makers, there is a need to use caution and gain experience with this measure of safety performance. Some general cautions that should be exercised include:

- "Safe" does not mean an expected crash frequency estimate of zero. Conversely, a non-zero expected crash frequency estimate does not mean a highway is "unsafe." No highway that carries any amount of traffic will have an expected crash frequency of zero.
- There is no accepted "standard" for what is a good or acceptable expected crash frequency. A good starting point for comparison may be the jurisdiction-wide average crash frequency for similar highways at similar traffic volume levels, assuming the jurisdiction has developed and input its calibration factors.
- While the focus of the IHSDM Crash Prediction Module is highway and intersection geometry and traffic control, these elements account for a relatively small proportion of the variation in crash experience along a given highway and among different highways. In making comparisons, consideration must be given to local knowledge of the effect of other variables "including, but not limited to, climate, terrain, animal population, driver population" that may also contribute to above or below average crash experience. In making judgments, it must be recognized that many of the factors contributing to crash experience are beyond the control of the project decision maker.

1.3 Purpose and Organization of this Manual

This Crash Prediction Module (CPM) Engineer’s Manual documents the basic information that users should understand in order to make appropriate use of the Module. It details the data input requirements, explains the procedural elements of the module, enumerates the steps in the crash prediction algorithm, and describes the presentation of model outputs. The manual highlights limitations of the Module that users should consider in applying it and interpreting results.

The manual is organized as follows:

- Section 2., *Data Input Requirements*
- Section 3., *Crash Prediction Procedural Elements*
- Section 4., *CPM Output*
- Section 5., *Crash Prediction Algorithm*
- Section 6., *AMFs for Highway Segments*
- Section 7., *AMFs for Intersections*
- Section 8., *Examples*
- Section 9., *Default Tables*
- Section 10., *Glossary*

- Section 11., *IHSDM Documentation*

2. Data Input Requirements

In addition to the IHSDM system-level input (e.g., Project, Analysis), the input data to the CPM consists of the following categories of data:

- analysis data (analysis limits and analysis period)
- geometric and traffic control data
- traffic volume data
- crash history data

2.1 Analysis Data

The analysis data consist of the analysis limits and the analysis period. The analysis limits are the beginning and ending stations for the portion of the project to be evaluated. The analysis limits must be stations within the project selected and are set from the Project/Analysis Tab in IHSDM.

The analysis period is the time period for which the safety performance is to be predicted. The analysis period is usually (but not necessarily) a period in the future. The minimum time period for which safety performance can be predicted is one year; the CPM does not provide predictions for partial years. The analysis period is set from the Attributes Tab in the CPM.

2.2 Geometric and Traffic Control Data

The geometric and traffic control data required to run the CPM can be divided into two categories: the highway segment geometric and traffic control data , and the intersection geometric and traffic control data.

2.2.1 Highway Segment Geometric and Traffic Control Data

The following highway geometric design and traffic control data are required to run the CPM:

- lane width (right)
- lane width (left)
- shoulder width (right)
- shoulder width (left)
- shoulder type (right)
- shoulder type (left)
- driveway density (DD)
- roadside hazard rating (RHR)
- horizontal curve data:
    - start station
    - end station
    - radius
    - superelevation
    - design speed
    - presence or absence of spiral transition

  - start and end station of spirals
- grade
- passing lanes (right)
- passing lanes (left)
- center TWLTLs

Values of some geometric characteristics may differ by direction of travel (i.e., lane width, shoulder width, shoulder type, and passing lanes). These data are entered separately for each direction of travel: "right" refers to the right side of the centerline in the direction of increasing stations and "left" refers to the left side of the centerline in the direction of increasing stations. Values for driveway density (DD) and roadside hazard rating (RHR) are for both sides of the road combined. Climbing lanes are treated like passing lanes by the crash prediction algorithm.

When historical crash data are not used in the analysis, data for all the variables listed must be entered for the entire project within the analysis limits of the highway used in the after period or analysis period (proposed highway). The CPM uses the highway geometric design and traffic control data to create a Highway Segment Data Table (see Section 4.1.1, *Highway Segment Data (Current and Proposed)*) for the proposed highway.

When historical crash data are used in the analysis, data for all the variables listed must be entered for the entire project within the analysis limits of the highway used in the before period (current highway) and after period (proposed highway). The CPM uses these data to create a Highway Segment Data Table for both the current and proposed highway.

Notes:

- Values of some geometric characteristics may differ by direction of travel (e.g., lane and shoulder width). These data are entered separately for each side of the centerline; "right" refers to the right side of the centerline in the direction of increasing stations and "left" refers to the left side of the centerline in the direction of increasing stations.
- Values for driveway density (DD) and roadside hazard rating (RHR) are for both sides of the road combined.
- Climbing lanes are treated like passing lanes by the crash prediction algorithm.

2.2.2 Intersection Geometric and Traffic Control Data

The following intersection geometric design and traffic control data are required to run the CPM:

- number of intersection legs
- type of traffic control
- approach (leg) type (i.e., major leg/ minor leg)
- intersection skew angle (right)
- intersection skew angle (left)
- number of major-road approaches (legs) with exclusive left-turn lanes
- number of major-road approaches (legs) with exclusive right-turn lanes
- number of intersection quadrants with limited intersection sight distance

The CPM uses these data to create an intersection data table Section 4.1.3, *Intersection Data (Current and Proposed)* for the proposed intersection if historical data are not used in the

analysis.

If historical data are used in the analysis, the CPM uses these data to create an Intersection Data Table for both the current and proposed highways.

2.3 Traffic Volume Data

The traffic volume data required to run the CPM can be divided into two categories: the highway segment traffic volume data , and the intersection traffic volume data.

2.3.1 Highway Segment Traffic Volume Data

ADT data are needed for the entire project within the analysis limits for the analysis period specified when historical crash data are ***not used*** in the analysis.

ADT are needed for all years for both the period when crash history data are available (before period) and the analysis (after) period when historical crash data are used in the analysis.

If ADT data are missing for any years or segments that are included in the analysis, then the CPM will interpolate missing ADT data based on rules described in the Crash Prediction Algorithm, Step 4.

2.3.2 Intersection Traffic Volume Data

ADT data are needed for each intersection for the analysis period specified when historical crash data are ***not used*** in the analysis.

ADT data are needed for all years for both the period when crash history data are available (before period) and the analysis (after) period when historical crash data ***are used*** in the analysis.

The major leg ADTs for each intersection can be determined directly from the highway segment ADTs. The minor leg ADTs need to be entered and cannot be interpolated by the CPM.

2.4 Crash History Data

Crash history data can be used to improve the overall crash prediction from the CPM.

Section 3.5, *Crash Prediction When Site-Specific Crash History Data are Available* discusses when crash history data should and should not be used in the CPM. Values for the following variables are needed when historical crash data are used:

- year of crash occurrence
- severity level
- location
- relationship to intersection
- intersection station

Crashes for which the relationship to intersection is stored as "intersection-related" or "non-intersection-related" are analyzed as such by the CPM. Crashes for which the relationship to intersection is stored as "unknown" and which fall within 76 m (250 ft) of the center of an intersection are attributed to that intersection. If the centers of two intersections are closer than 500 feet, the historical crashes between the two intersections are assigned to the closest intersection. Crashes stored as "unknown" and located more than 76 m (250 ft) from the center of an intersection are attributed to the highway segment within which the crash falls.

Crash history data are applied to the CPM in Step 14 of the Crash Prediction Algorithm.

2.4.1 Highway Segment Crash History Data

Crashes assigned to highway segments are stored one of two ways: by specific location, or by station range.

Crashes stored by a specific location are assigned to a homogeneous analysis section (homogeneous highway segment or intersection).

For historical crashes stored by station ranges, crash segments matching the station ranges (one station range per crash segment) are created with the number of observed crashes assigned to the crash segment. If the available crash data consist only of totals at the project level, then there will be only one crash segment.

2.4.2 Intersection Crash History Data

Crashes that are stored as "intersection-related" and have a value defined for the "intersection location" are assigned to the intersection at the specified station. Crashes that are stored as "intersection-related" and do not have a value defined for the "intersection location" are assigned to the intersection closest to the crash location.

3. Crash Prediction Procedural Elements

This section describes the main procedural elements that the CPM uses to estimate the crash frequency for a given project. These elements, which comprise the crash prediction algorithm, are as follows: segmentation procedure, base models, AMFs, calibration factors, and the Empirical Bayes Procedure for crash prediction when site-specific crash history data are available.

3.1 Segmentation Procedure

The crash prediction algorithm first divides the project within the specified limits into homogeneous analysis sections (homogeneous highway segments and intersections). Estimates of crash frequency are made on the individual homogeneous analysis sections that make up the project. The total prediction for the project is the sum of the individual predictions for each of the homogeneous analysis sections. The segmentation procedure is described in detail in Steps 2-4 of the Crash Prediction Algorithm.

3.2 Base Models (Segments and Intersections)

The base model for highway segments is the best available regression model for predicting the total crash frequency of a homogeneous highway segment on a rural two-lane highway. The base model predicts the total expected crash frequency on the highway segment during a specified time period as a function of the highway segment’s traffic volume, geometry, and traffic control. The specific regression model used by the CPM is presented in Step 7 of the Crash Prediction Algorithm.

Separate base models have been developed for three-leg intersections with minor-road STOP control, four-leg intersections with minor-road STOP control, and four-leg signalized intersections. These models are available in the Report No. FHWA-RD-99-207 . The specific regression models used by the CPM for intersections are presented in Step 7 of the Crash Prediction Algorithm.

3.3 AMFs (Segments and Intersections)

AMFs are multiplicative factors used to adjust the expected base crash frequency for the effect of individual geometric design and traffic control features. Each AMF is formulated so that the nominal or base condition is represented by an AMF of 1.00. Conditions associated with higher

crash experience than the nominal or base condition will have AMFs greater than 1.00 and conditions associated with lower crash experience than the nominal or base condition will have AMFs less than 1.00. The AMFs are applied in Step 9 of the Crash Prediction Algorithm . For details, see AMFs for Highway Segments, and AMFs for Intersections.

3.4 Calibration Factors

A key element of the crash prediction algorithm is a set of calibration factors that allow individual highway agencies to tailor the safety prediction to their own local conditions. Geometric design factors and ADTs are accounted for in the crash prediction algorithm and do not require calibration. However, there are factors that lead to differences in reported crash frequencies between highway agencies in different geographical areas that are not accounted for by the crash prediction algorithm. These include:

- Differences in climate
- Differences in animal population
- Differences in driver populations and trip purposes
- Accident reporting thresholds established by State law
- Accident investigation practices

The calibration procedure is intended to account for these differences and provide crash predictions that are comparable to the estimates that a highway agency would obtain from its own crash records system.

The values of these calibration factors can be viewed and edited in the Administration Tool (see System Administrator’s Manual). The calibration factors include:

- the calibration factor for highway segments, C_r, used in Equation (4.1)
- the calibration factor for three-leg intersections with minor-road STOP-control, C_{i1}, used in Equation (4.3)
- the calibration factor for four-leg intersections with minor-road STOP-control, C_{i2}, used in Equation (4.3)
- the calibration factor for four-leg signalized intersections, C_{i3}, used in Equation (4.3)

The calibration factors C_r, C_{i1}, C_{i2}, and C_{i3} are initially set to the default value of 1.00.

Appendix C of Report No. FHWA-RD-99-207, Prediction of the Expected Safety Performance of Rural Two-Lane Highways, presents a process that can be used by highway agencies to develop calibration factors appropriate for their own local conditions. It is recommended that these tables be set up in a spreadsheet, to perform the calibration process externally. The results can then be implemented in the CPM.

3.5 Crash Prediction When Site-Specific Crash History Data are Available

Users have the option of specifying (on the CPM Evaluation Tab) whether or not to use crash history data in the crash prediction algorithm. When users select this option, the algorithm incorporates an Empirical Bayes (EB) procedure for combining expected crash frequencies (estimated using the base models and AMFs) with site-specific crash history data.

This section briefly explains the advantages of using the EB procedure and factors that should be considered in deciding whether or not the EB procedure should be applied in a particular evaluation. Report No. FHWA-RD-99-207 , Prediction of the Expected Safety Performance of

Rural Two-Lane Highways, provides additional details on the EB procedure.

3.5.1 Caution on Use of CPM Results When Site-Specific Crash History Data Are Available

The estimated crash frequencies derived from the base models and AMFs have the advantage of being based upon data from many locations and representing the long-term average crash frequency that would be expected among sites with similar geometry and traffic characteristics. These estimates do not, however, account for all of the factors that cause expected frequencies at individual locations to vary about the average of all similar locations. The advantage of using the EB procedure is to improve the accuracy of estimates for an individual location by factoring in the actual crash history of the location being evaluated.

One might ask, Why not rely solely upon site-specific crash history data? Although crash history data are an important indicator of the safety performance of a highway, they suffer from the weakness of being highly variable. Given this high variability, it is difficult to estimate the long-term expected crash frequency using the relatively few years of crash data generally available. This variability is an issue for two-lane rural highways where crashes are rare and many locations experience no crashes over a period of several years. If a location has experienced no crashes during the past several years, it is certainly not correct to think that it will never experience a crash (i.e., that its long-term average crash frequency will be zero, which is what relying solely upon a few years of crash history data would suggest).

In theory, when enough years of crash history data are available, it would be more accurate to appropriately combine estimates from the base models and AMFs with site-specific crash history data than to rely on either the model estimates or the site-specific data alone. The EB procedure determines the statistically appropriate weighting of estimates from the base models and AMFs and site-specific crash history data.

3.5.2 Situations in Which the EB Procedure Should and Should Not Be Applied

If the project being evaluated involves an existing highway for which sufficient crash history data are available, then the EB procedure is an option that the user should consider. For projects involving highways on new locations, there is no relevant crash history and, therefore, use of the EB procedure is not an option.

For projects involving existing highways, two primary factors should be considered in determining whether or not to select the analysis option to use crash history data: (1) the availability of a sufficiently large sample of crash history data, and (2) the relevance of the crash history on the existing highway to the project alternatives being evaluated.

The availability of a sufficiently large sample of data needs to be considered because of the high variability in crash data on two-lane rural highways. IHSDM specifies at least 2 years of crash history data to use the EB procedure.

The relevance of the crash history on the existing highway to the project alternatives being evaluated must also be considered. The EB procedure considers both the existing highway's and the proposed alternatives' geometric design and traffic control. When projects involve several alternatives, comparisons are generally made among each alternatives' estimated crash frequency. To ensure comparability among the estimates for individual alternatives, the EB procedure should be applied either in all alternatives or in none of the alternatives. Therefore, if the crash history on the existing highway is not relevant to one or more of the project alternatives, then it should not be applied in any of the alternatives being evaluated.

Judgment is required to assess the relevance of the crash history on the existing highway. The fundamental issue is whether or not the crash history on the existing highway would be indicative of the crash experience that should be expected in the future after a design alternative is implemented. The following guidance is provided on project types in which it would be appropriate and, then, in which it would not be appropriate to apply the EB procedure.

The existing highway's crash history is generally relevant and, therefore, use of the EB procedure is considered appropriate in the following situations:

- Sites at which the highway geometrics and traffic control are not being changed (e.g., the "do-nothing" alternative).
- Projects in which the highway cross-section is modified but the basic number of lanes remains the same. Examples include projects in which lanes and/or shoulders were widened or the roadside was improved, but the highway remained a rural two-lane highway.
- Projects in which minor changes in alignment are made, such as flattening individual horizontal curves while leaving most of the alignment intact.

The existing highway's crash history may not be relevant and, therefore, use of the EB procedure may not be appropriate involving significant changes to alignment geometry and/or intersection configuration. Examples include:

- Projects in which a new alignment is developed for a significant proportion of the project length.
- Individual intersections at which the basic number of intersection legs and/or type of traffic control is changed as part of a project. The EB procedure can be applied to the rest of any project containing such intersection changes, but the intersection(s) impacted by the changes should be omitted.

3.5.3 Example to Illustrate When to Use/Not Use Historical Crash Data

Conditions for appropriate use of historical crash data to predict future crashes are somewhat subjective, and require user judgment. The following example is provided to help determine when use of historical crash data is appropriate:

- "**Case 1:**" Use of historical crash data is appropriate for the extreme case in which the highway to be analyzed (i.e., the 'proposed' highway) is exactly the same as the highway with crash data (i.e., the 'before' or 'existing' highway).
- "**Case 2:**" Use of historical crash data is not appropriate for the extreme case in which the proposed highway alignment is totally different from 'before' highway both geographically (e.g., separated by a significant distance from the before highway) and in character. For example, given:
  - A 'before' highway (Highway A) characterized by a curvilinear alignment, steep vertical grades, no shoulders and narrow lanes, that winds around and over a mountain, and
  - A proposed alternative alignment (Highway B) characterized by few, flat horizontal curves, no steep vertical grades, wide lanes and shoulders, that bypasses the mountain by going through a valley.

  Then, historical crash data from Highway A should not be used to predict future crashes on Highway B.

For other (non-extreme) cases, user judgment is required to determine when to use historical crash data to predict future crashes. For situations more similar to Case 1 (e.g., minor improvements to an existing highway), historical crash data can be used. For situations more similar to Case 2, use of historical crash data is not appropriate and should not be used.

The safety effect of spot improvements (e.g., flattening a single horizontal curve) may be examined in the context of an extended length of highway. For a given spot improvement, the new alignment might differ substantially from the old alignment (e.g., if a curve is flattened or replaced by a tangent, the horizontal alignment centerline for the limits of the improvement will be shifted). However, if when an extended length of highway is considered, the percentage of alignment changed is small, the use of historical crash data might be appropriate.

4. CPM Output

The CPM generates tables describing the highway and intersection data used for the analysis (Highway Data) and various reports and plots showing the expected crash frequencies and rates for the project (CPM Analysis Report). This section describes these reports, which can be viewed from buttons on the Evaluation Tab.

4.1 Highway Data

The CPM generates tables describing the highway and intersection data used for the analysis. These tables organize data in such a way as to help with interpreting the results of the CPM. The following tables are created by the CPM for the current (before period) and proposed (after period) alignments:

- Highway Segment Data
- Horizontal Curve Data
- Intersection Data
- Segment Traffic Volume
- Intersecting Highway Traffic Volume

These tables can be viewed by selecting the "View Highway Segment Data" button or the "View Analysis Report" button on the Evaluation Tab.

4.1.1 Highway Segment Data (Current and Proposed)

The Highway Segment Data Table displays the homogeneous highway segments created by the CPM for the analysis limits specified as described in step 2 through step 4 of the Crash Prediction Algorithm. When one of the highway characteristics listed in the columns changes, a new homogeneous segment is created and therefore, a new row is created. Columns in the table are as follows:

- **Segment #:** Each homogeneous highway segment is assigned a number by the CPM for purposes of identification. Numbers are sequential beginning with 1.
- **Station (Start):** Beginning station for the homogeneous highway segment in the given row.
- **Station (End):** End station for the homogeneous highway segment in the given row.
- **Length:** The length of the homogeneous highway segment in the given row (equal to the difference between the Start Station and End Station).
- **Lane Width (Right):** The width of the lane to the right of the centerline, in the direction of increasing stations.

- **Lane Width (Left):** The width of the lane to the left of the centerline, in the direction of increasing stations.
- **Shoulder Width (Right):** The width of the shoulder to the right of the centerline, in the direction of increasing stations.
- **Shoulder Width (Left):** The width of the shoulder to the left of the centerline, in the direction of increasing stations.
- **Shoulder Type (Right):** The type of the shoulder to the right of the centerline, in the direction of increasing stations.
- **Shoulder Type (Left):** The type of the shoulder to the left of the centerline, in the direction of increasing stations.
- **Driveway Density:** The number of driveways per length of highway (English: driveways/mi; Metric: driveways/km).
- **Roadside Hazard Rating:** A seven-point categorical scale from 1 (best) to 7 (worst) to characterize the roadside environment on two-lane highways. The Roadside Hazard Rating is defined in Appendix D of Report No. FHWA-RD-99-207, Prediction of the Expected Safety Performance of Rural Two-Lane Highways.
- **Horizontal Curve Number:** Each horizontal curve is assigned a number by the CPM for purposes of identification. Numbers are sequential beginning with 1. If the section of highway is a tangent section, this field will display a dash.
- **Grade (%):** The percent grade on the given highway segment.
- **Passing Lane (Right):** The presence of a passing lane to the right of the centerline, in the direction of increasing stations. Entry will be "yes" if a passing lane is present and "no" if a passing lane is not present.
- **Passing Lane (Left):** The presence of a passing lane to the left of the centerline, in the direction of increasing stations. Entry will be "yes" if a passing lane is present and "no" if a passing lane is not present.
- **Center TWLTL:** The presence of a Two-Way Left-Turn Lane. Entry will be "yes" if a TWLTL lane is present and "no" if a TWLTL lane is not present.

4.1.2 Horizontal Curve Data (Current and Proposed)

The Horizontal Curve Data Table lists the characteristics of the horizontal curves that are within the limits of the analysis. There is a separate row for each curve. Interpolated values are shown in a blue font. Columns in the table can be described as follows:

- **Horizontal Curve Number:** Each curve is assigned a number by the CPM for purposes of identification. Numbers are sequential beginning with 1.
- **Station (Start):** Beginning station for the curve.
- **Station (End):** End station for the curve.
- **Length of Curve:** The length of the curve.
- **Radius:** The radius of the curve.
- **Superelevation (%):** The actual superelevation attained on the curve.
- **Design Speed:** The design speed for the curve.

- **Spiral Transition:** The presence of a spiral transition. The entry will read "both" if a spiral transition is on both the entry and exit portions of the curve and will read "one" in the rare case where there is a spiral transition on only one side of the curve.

4.1.3 Intersection Data (Current and Proposed)

The Intersection Data Table includes a separate row for each intersection within the analysis limits. Columns include:

- **Intersection #:** Each intersection is assigned a number by the CPM for purposes of identification. Numbers are sequential beginning with 1.
- **Title:** The title or name of the intersection for the given row.
- **Station:** The location of the intersection along the highway being analyzed.
- **No. of Legs:** The number of legs for the intersection for the given row.
- **Traffic Control:** The type of traffic control for the given intersection.
- **Skew Angle (Right):** Deviation (in degrees) from a 90 degree angle for intersection legs on the right side of the highway. A positive value indicates a clockwise deviation and a negative value indicates a counter-clockwise deviation.
- **Skew Angle (Left):** Deviation (in degrees) from a 90 degree angle for intersection legs on the left side of the highway. A positive value indicates a clockwise deviation and a negative value indicates a counter-clockwise deviation.
- **Major legs with exclusive LTLs:** The number of exclusive left-turn lanes at the intersection.
- **Major legs with exclusive RTLs:** The number of exclusive right-turn lanes at the intersection.
- **Quadrants with Limited ISD:** The number of intersection quadrants with limited sight distance. These data are entered when the CPM query dialog box about limited ISD appears after "Run Analysis" is selected.

4.1.4 Segment Traffic Volume (Current and Proposed)

The Segment Traffic Volume Table provides a row for each segment or groups of segments with the same ADT. Columns include:

- **Segment #:** Each homogeneous highway segment is assigned a number by the CPM for purposes of identification. Numbers are sequential beginning with 1. The segment numbers correspond to the numbers listed in the Highway Segment Data Table. A new row is begun only when ADT changes, therefore, more than one segment may be listed in a given row.
- **Station (Start):** Beginning station for the homogeneous highway segment(s) in the given row.
- **Station (End):** End station for the homogeneous highway segment(s) in the given row.
- **Analysis Period - ADT (v/day):** ADTs for each year in the analysis period.

4.1.5 Intersecting Highway Traffic Volume (Current and Proposed)

The Intersecting Highway Traffic Volume Table provides a separate row for each intersection. Columns include:

- **Intersection # -** Each intersection is assigned a number by the CPM for purposes of identification. Numbers are sequential beginning with 1. The intersection number

corresponds to the number in the Intersection Data Table.

- **Station -** The location of the intersection along the highway being analyzed.
- **Intersecting Highway (Name) -** The name of the intersection for the given row.
- **Intersecting Highway (Major/Minor)**
- **Year -** The year to which the ADT data in the given row applies.
- **Leg (MJ1) -** Major road, leg 1 (lower station).
- **Leg (MJ2) -** Major road, leg 2 (higher station).
- **Leg (MN1) -** Minor road, leg 1 (to right in direction of increasing station).
- **Leg (MN2) -** Minor road, leg 2 (to left in direction of increasing station).

4.2 Analysis Report

The CPM Analysis Report displays CPM results in tabular and graphical format and is accessed through the "View Analysis Report" button on the Evaluation Tab. CPM graphs can be viewed, edited, and printed separately by selecting the "Create Graph" button on the Evaluation Tab. The Analysis Report displays project, analysis, and highway information at the top of the report, followed by the highway data described in Section 4.1, *Highway Data*. The analysis results are then presented as described below.

4.2.1 Expected Crash Frequencies and Rates (for entire project within analysis limits)

This report summarizes the expected crash frequencies and rates for the entire project within the specified analysis limits. Crash severity breakdown is based on the default crash severity distribution. The values of this distribution can be viewed in the Administration Tool (see System Administrator’s Manual).

4.2.2 Expected Crash Type Distributions

This report summarizes the expected crash type distributions for the project within the specified analysis limits. Crash type breakdown is based on the default crash type distribution. The values of this distribution can be viewed in the Administration Tool (see System Administrator’s Manual).

4.2.3 Reporting Results by Homogeneous Analysis Section

The remaining output reports present the results by homogeneous analysis section, (homogeneous highway segment and intersection) as created in step 2 through step 4 of the Crash Prediction Algorithm.

4.2.3.1 Caution on ’Microanalysis’ Use of CPM Results

The user has the option of generating (and viewing in the CPM Analysis Report) several tables and graphs that summarize predicted crash rates and frequencies for the analysis section, including:

- A summary of predicted crash rates and frequencies for the entire analysis section
- Predicted crash rates and frequencies for individual CPM homogeneous highway segments
- Predicted crash rates and frequencies by horizontal design element (e.g., tangents and curves)

The user should apply caution in interpreting results for individual highway segments. The CPM algorithm is primarily intended for project-level crash predictions, rather than for microanalysis (i.e., predictions for small highway segments). Predictions for individual segments or short sections of highway will have a higher degree of uncertainty than predictions for longer highway sections. Therefore, the user should have much more confidence in the summary of predicted crashes for the entire analysis section, than in the predictions for individual homogeneous highway segments.

Crash predictions for individual highway segments are made available to the user to illustrate how the total was derived for the entire analysis section.

The user may compare relative levels of safety of individual highway segments (e.g., Curve A has a relatively higher number of predicted crashes than Curve B), but should understand that the absolute number of predicted crashes for a small segment will have a higher degree of uncertainty compared to the prediction for an extended length of highway.

4.2.3.2 Expected Crash Frequencies and Rates (by highway segment and intersection)

This report shows the expected safety performance of individual homogeneous highway segments and intersections in tabular form. Intersection entries include the intersection name or the name of the cross-road and are highlighted in yellow to make it easier to identify intersection locations.

4.2.3.3 Expected Crash Frequencies and Rates by Horizontal Design Element

This report shows the expected safety performance of individual homogeneous highway segments and intersections in tabular form. Intersection entries include the intersection name or the name of the cross-road and are highlighted in yellow to make it easier to identify intersection locations.

4.2.3.4 Plot of Expected Crash Rates by Highway Segment and Plot of Expected Accident Rates Plotted Using a Sliding Scale

This report includes two plots:

- A histogram of the expected crash rates (expressed in crashes/mi/year or crashes/km/year) by highway segment that were tabulated in the Expected Crash Frequencies and Rates (by homogeneous highway segment and intersection) Report
- A sliding scale plot of the expected crash rates (expressed in crashes/mi/year or crashes/km/year) for segments of a given fixed length at a given increment

The sliding scale plot has the effect of "smoothing" the data from the plot of expected crash rates by highway segment providing a better understanding of the effects of adjacent features rather than individual isolated features. Refer to Caution on Microanalysis? Use of CPM Results for interpreting results from the Expected Crash Rates by Highway Segment Plot.

Three parameters are needed for the sliding scale analysis: the smoothing window or fixed segment length (W), the smoothing factor (1/n), and the moving interval or increment (I). The moving interval is equal to the smoothing window times the smoothing factor (I=W/n), and n is an odd number for the purposes of our analysis.

The procedure for producing the sliding scale plot is as follows: (1) starting from the expected crash rate for each homogeneous highway segment, calculate the average crash rate within the window of length W, (2) plot this average at the mid-point of the smoothing window, (3) advance the window by the moving interval (I) and repeat the calculation.

4.2.3.5 Plot of Expected Crash Rates by Highway Segment and Plot of Expected Accident Rates Plotted Using a Sliding Scale

This plot is a histogram of the expected crash rates (expressed in crashes/mi/year or crashes/km/year) by horizontal design element that were tabulated in the Expected Crash Frequencies and Rates by Horizontal Design Element Report (see Section 4.2.3.3, *Expected Crash Frequencies and Rates by Horizontal Design Element*).

4.2.3.6 Plot of Expected Crash Frequencies by Intersection

This report is a plot of the expected crash frequencies (expressed in crashes/year) by intersection that were tabulated in the Expected Crash Frequencies and Rates (by homogeneous highway segment and intersection) Report.

4.2.4 Historical Crash Data Plots (Future)

Plots of the historical crash data are created when the analysis is run using crash history data, and crash data are stored by specific locations. Two different types of data are shown (if available) on these plots:

- Crashes on highway segments, stored by specific location
- Crashes stored or assigned as intersection-related

The actual crashes are displayed in three separate plots:

- By specific location
- Assigned to homogeneous highway segments and intersections
- Assigned to horizontal design elements (i.e., tangent and curves) and intersections

Crash data for the entire before period are displayed on one graph and there is nothing to distinguish crash data from year to year since it is irrelevant to the analysis. If some or all of the data are stored by location range, the option to display historical crash data plots is not available.

4.2.4.1 Historical Crashes Plotted by Specific Location

Crashes stored by a specific location are shown as points on the horizontal axis at the station at which they are stored. Crashes stored as intersection-related or assigned as intersection-related as defined in Section 2.4, *Crash History Data*, are shown as points on the horizontal axis at the intersection at which they are stored. If more than one crash occurs at a specific station (or within a tolerance of one meter or three feet), one single graphic feature represents the multiple crashes that occur at that location, with the number of crashes that occur at that location displayed above the graphic feature.

4.2.4.2 Plot of Historical Crashes When Crashes are Assigned to Homogeneous Highway Segments and Intersections

In this plot, the homogeneous highway segments with historical crashes occurring on them are plotted on the horizontal axis and the number of crashes falling within each segment are plotted on the vertical axis. Crashes stored as intersection-related or assigned as intersection-related as defined in Section 2.4, *Crash History Data*, are plotted at the station locating the intersection with the number of crashes plotted on the vertical axis.

4.2.4.3 Plot of Historical Crashes When Crashes are Assigned to Horizontal Design Elements

In this plot, the design elements (i.e., tangents and curves) with historical crashes occurring on them are plotted on the horizontal axis and the number of crashes falling within each segment are plotted on the vertical axis. For compound curves, each curve is considered individually (e.g., a compound curve with three radii is considered three curves). Spirals are considered to be tangent elements. Crashes stored as intersection-related or assigned as intersection-related as defined in Section 2.4, *Crash History Data*, are plotted at the station locating the intersection with the number of crashes plotted on the vertical axis.

5. Crash Prediction Algorithm

The CPM estimates expected crash frequency and severity on rural, two-lane highways by using a crash prediction algorithm. The algorithm is applied for two cases:

- safety prediction when no site-specific crash history data are available
- safety prediction when site-specific crash history data are available

The procedure for the crash prediction algorithm is described in this section and illustrated in the Crash Prediction Algorithm Flowchart.

CPM Quickstart Procedural Guides

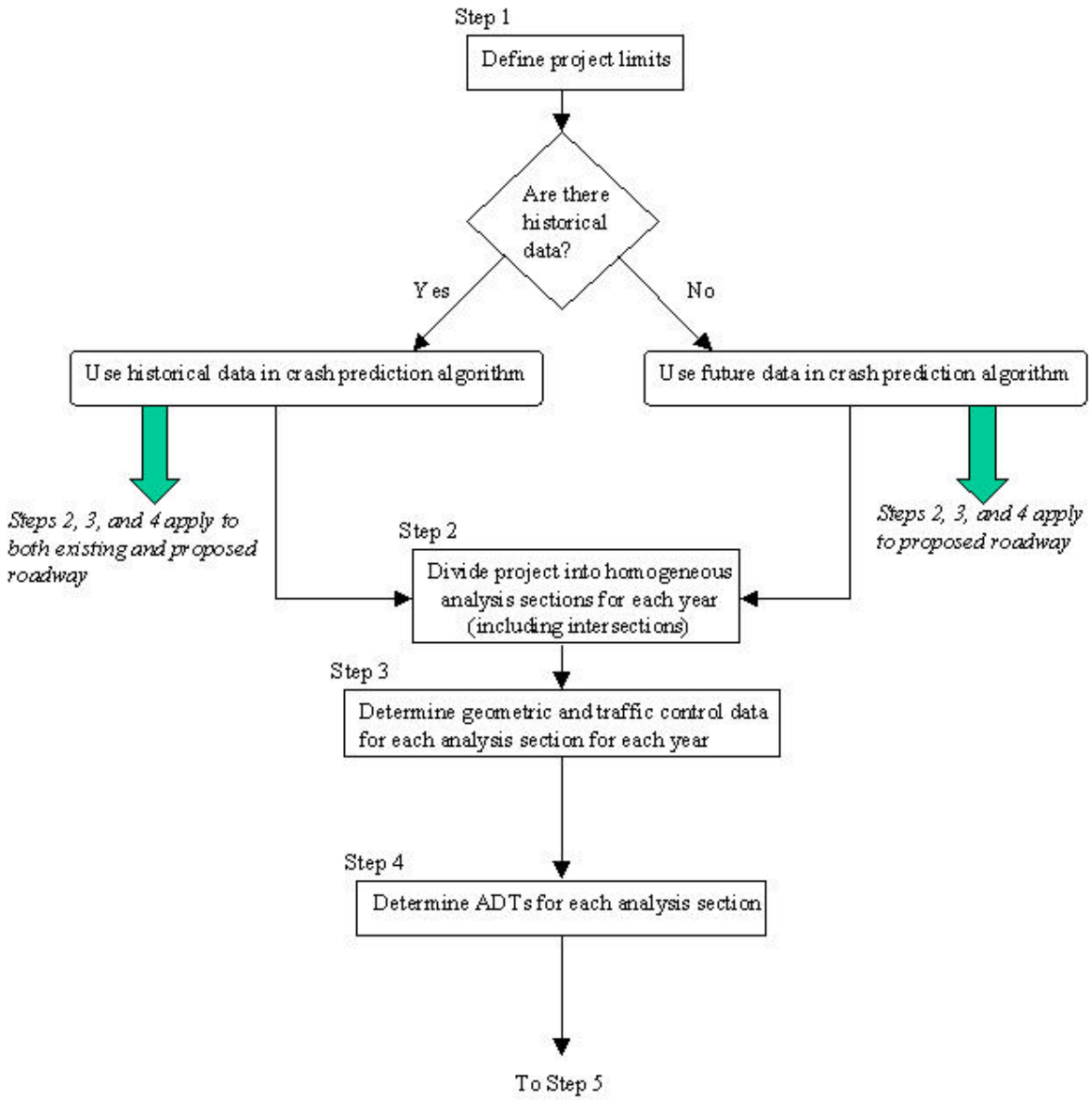


Figure 1 Crash Prediction Algorithm Flowchart (Steps 1 - 4)

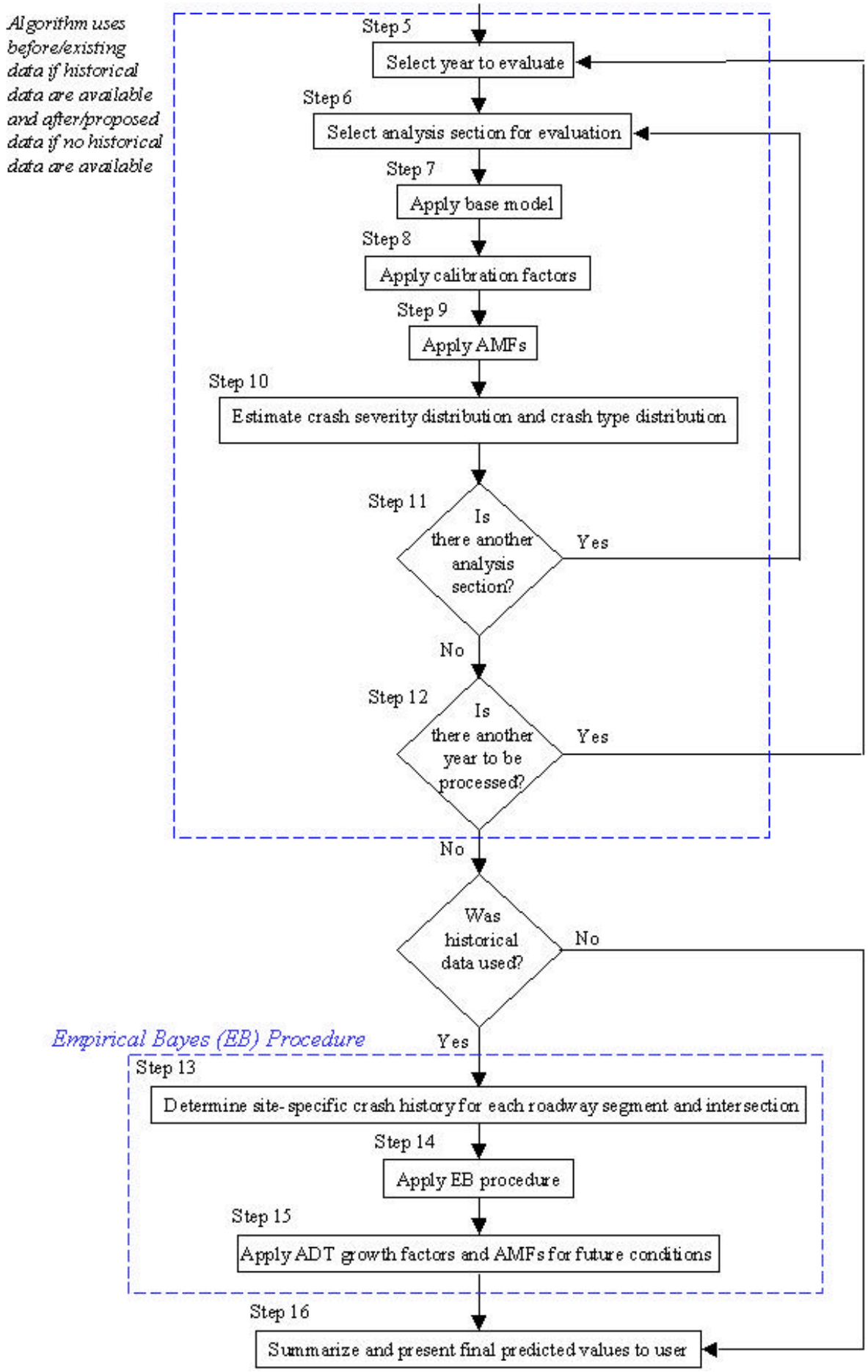

Algorithm uses before/existing data if historical data are available and after/proposed data if no historical data are available
Step 5
Select year to evaluate
Step 6
Select analysis section for evaluation
Step 7
Apply base model
Step 8
Apply calibration factors
Step 9
Apply AMFs
Step 10
Estimate crash severity distribution and crash type distribution
Step 11
Is there another analysis section?
Yes
No
Step 12
Is there another year to be processed?
Yes
No
Was historical data used?
No
Yes
Empirical Bayes (EB) Procedure
Step 13
Determine site-specific crash history for each roadway segment and intersection
Step 14
Apply EB procedure
Step 15
Apply ADT growth factors and AMFs for future conditions
Step 16
Summarize and present final predicted values to user

Figure 2 Crash Prediction Algorithm Flowchart (Steps 5 - 16)

Step 1

Define the limits of the project for which the expected safety performance is to be predicted (the analysis period and analysis limits).

Step 2

Divide the project into individual homogeneous analysis sections (highway segments and intersections) for each year.

The project within the specified analysis limits is divided into individual homogeneous highway segments and intersections. New highway segments begin at points 250 feet before and after the center of each intersection and at the center of each intersection. If the centers of two intersections are less than 500 feet apart, a new highway segment begins at the midpoint between the two centers of the intersections. New highway segments also begin at each point where the value of one of the following characteristics changes:

- average daily traffic volume (ADT, expressed in veh/day)
- lane width
- shoulder width
- shoulder type
- driveway density
- roadside hazard rating
- grade

To avoid fragmenting the project unnecessarily, use the same locations as ADT breakpoints for each year of data. (It is unlikely that the ADT break points for one year would not be appropriate for other years as well.) ADT break points should generally occur at intersections.

Also, a new highway segment starts at any of the following locations:

- beginning or end of a horizontal curve
- beginning or end of a passing lane, in either direction of travel, or short four-lane section provided for the purpose of increasing passing opportunities
- beginning or end of a center two-way left-turn lane (TWLTL)

When site-specific crash history data *are not* available:
The segmentation process is applied only to the proposed (after period) highway.

When site-specific crash history data *are* available:
The segmentation process is applied to both the existing (before period) highway and the proposed (analysis or after period) highway. Each station that represents a division point for either the existing or proposed highway is used as the beginning point of a new segment for purposes of the analysis.

Step 3

Determine the geometric design and traffic control features for each analysis section (individual highway segments and intersections).

When site-specific crash history data *are not* available:
Using data from the IHSDM highway file, a table of highway segment characteristics is

created by the CPM containing values for each of the variables described in Geometric and Traffic Control. Each of these variables will have a unique value within each highway segment (row) of this table.

When site-specific crash history data *are* available:
The procedures are analogous to the procedures when no crash history data are available except, in this case, two sets of tables are created, one set for the before period (existing highway geometrics, traffic control, and ADT data) and one for the after period (proposed highway geometrics, traffic control, and ADT data). The option of creating a single table displaying ADTs for the before and after periods is also available.

Step 4

Determine the ADTs for each highway segment and the major leg and minor leg ADTs for each intersection for each year of the analysis period.

When site-specific crash history data *are not* available:
Determine the ADTs for each highway segment and the major leg and minor leg ADTs for each intersection during each year of the analysis period for which the expected safety performance is to be predicted.

In this case, no ADT data for a historical period (the before period) are needed because accident history data are not available. It is possible that ADT data may not have been supplied for every year of the analysis period for all locations. In that case, the module uses the following rules to interpolate the missing ADT data:

- If ADT data are available for only a single year, that same value is assumed to apply to all years of the analysis period.
- If two or more years of ADT data are available, the ADTs for intervening years are computed by interpolation.
- ADTs for years before the first year for which data are available are assumed to be equal to the ADT for that first year.
- ADTs for years after the last year for which data are available are assumed to be equal to the last year (i.e., no extrapolation is performed by the algorithm).

The interpolated values are shown in blue and are in "normal" font (as opposed to bold for actual ADT data).

It is important to note that any extrapolation of ADT data must be performed outside of the CPM; extrapolation of ADT values is not performed by the module.

When site-specific crash history data *are* available:
Determine the ADTs for each highway segment and the major leg and minor leg ADTs for each intersection during each year of the before period for which observed crash history data are available and for each year of the analysis period for which the expected safety performance is to be predicted.

ADT data are placed in tables for highway segments and intersections in a manner analogous to the procedure when no crash history data are available except that two tables are created (one for the before period and one for the after period) that include all years of the before and after periods. The interpolation rules for filling in missing ADT data are the same as when no crash history data are available.

Step 5 through step 12 describe the procedure for applying the crash prediction algorithm to each of the individual homogeneous highway segments and intersections.

When site-specific crash history data *are not* available:
The algorithm is applied to each year of the proposed (after period) highway.

When site-specific crash history data *are* available:
The algorithm is applied to each year of the existing (before period) highway.

Step 5

Select a particular year of the specified evaluation period for the analysis section (highway segment or intersection) of interest.

The individual years of the evaluation period are evaluated one at a time for each highway segment or intersection. Separate estimates are made for each year because several of the AMFs considered in Step 9 are dependent on the ADT of the highway segment or intersection, which may change from year to year. Step 6 through step 11 are repeated for each year of the evaluation period as part of the evaluation of any particular highway segment or intersection.

Step 6

Select an individual highway segment or intersection for evaluation.

Highway segments and intersections are evaluated one at a time. Step 7 through step 10 are repeated for each highway segment and intersection.

Steps 7 - 9

Apply the crash prediction algorithm to each analysis section for each year of the analysis period. The steps are executed together and are defined as follows:

Step 7
Apply the base model

Step 8
Apply the calibration factors

Step 9
Apply AMFs

Four types of analysis segments and intersections are considered in the CPM. These are:

- highway segments
- three-leg minor-road STOP-control intersections
- four-leg minor-road STOP-control intersections
- four-leg signalized intersections

Three-leg intersections with minor leg YIELD control are treated as three-leg intersections with minor-road STOP-control for analysis purposes. Four-leg intersections with minor leg YIELD control are treated as four-leg intersections with minor-road STOP-control for analysis purposes. Three-leg signalized intersections and intersections with more than four-legs are outside the scope of the CPM.

Highway Segments
The predicted crash frequency for any given highway segment in any given year (N_{rs}) is determined from the following equations:

$N_{rs} = N_{br}\ C_r\ AMF_1\ AMF_2\ AMF_3\ AMF_4\ AMF_5\ AMF_6\ AMF_7\ AMF_8\ AMF_9$ **(4.1)**

$N_{br} = (ADT_n)\ (L)\ (365)\ (10^{-6})\ \exp(-0.4865)$ **(4.2)**

where:

N_{rs} = predicted number of total highway segment crashes per year

N_{br} = predicted number of total highway segment crashes per year for nominal or base conditions

C_r = calibration factor for highway segments

AMF_1, ..., AMF_9 = accident modification factors for highway segments

ADT_n = average daily traffic volume for specified year n (veh/day)

L = length of highway segment (mi)

The values of the parameters in Equation (4.1) and Equation (4.2) are determined as follows:

- The calibration factor (Cr) is determined as discussed in Section 3.4, *Calibration Factors*.
- The AMF (AMF_1 through AMF_9) values are determined as discussed in Section 3.3, *AMFs (Segments and Intersections)*.
- The highway segment ADT for a given year (ADT_n) is determined from the highway segment traffic volume data.
- The segment length (L) is the absolute value of the difference between the beginning and ending stations for the segment, corrected for any station equation that may be present and converted to miles.

For details on development of the highway segment crash prediction model, see Report No. FHWA-RD-99-207 , Prediction of the Expected Safety Performance of Rural Two-Lane Highways.

Intersections

The predicted crash frequency for any given intersection in any given year (N_i) is determined from the following equations:

$N_i = N_{bi}\ C_{in}\ AMF_{10}\ AMF_{11}\ AMF_{12}\ AMF_{13}\ AMF_{14}$ **(4.3)**

where:

N_i = predicted number of total intersection crashes per year

N_{bi} = predicted number of total crashes per year for nominal or base conditions

C_{in} = calibration factor for intersections

AMF_{10} - AMF_{14} = accident modification factors for intersections

For a three-leg intersections with minor-road STOP-control:

$N_{bi} = \exp(-10.9 + 0.79 \ln ADT_{1n} + 0.49 \ln ADT_{2n})$ **(4.4)**

where:

ADT_{1n} = average daily traffic volume for major leg for specified year *n* (veh/day)

ADT_{2n} = average daily traffic volume for minor leg for specified year *n* (veh/day)

For a four-leg intersections with minor-road STOP-control:

$N_{bi} = \exp(-9.34 + 0.60 \ln ADT_{1n} + 0.61 \ln ADT_{2n})$ **(4.5)**

For a four-leg signalized intersection:

$N_{bi} = \exp(-10.9 + 0.79 \ln ADT_{1n} + 0.49 \ln ADT_{2n})$ **(4.6)**

The values of the parameters in Equation (4.3) through equation (4.6) are determined as follows:

- The calibration factor (C_{in}) is determined as discussed in Section 3.4, *Calibration Factors*.
- The values of the AMFs (AMF_{10} to AMF_{14}) are determined as discussed in Section 3.3, *AMFs (Segments and Intersections)*.
- The intersection ADTs (ADT_{1n} and ADT_{2n}) are determined as:
  - ADT_{1n} is the average of the major leg (MJ1 and MJ2) ADTs from the intersection traffic volume data.
  - } ADT_{2n} is the average of the minor leg right and minor leg left ADTs (MN1 and MN2) from the intersection traffic volume data.

For details on development of the intersection crash prediction models, see Report No. FHWA-RD-99-207 , Prediction of the Expected Safety Performance of Rural Two-Lane Highways.

Step 10

Estimate the expected distribution of crash severities and crash types for the highway segments and intersections of interest from default distributions of crash severity and crash type.

Breakdowns of predicted crash frequencies by severity level and by crash type and manner of collision are estimated from default distributions for highway segments and intersections. Section 9.1, *Default Percentage Distributions for Crash Severity Level and for Crash Type and Manner of Collision* presents the default distributions in IHSDM.

These default distributions can also be viewed from the IHSDM Administration Tool. Agencies may use the Administration Tool to edit the distributions to jurisdiction-specific values.

Step 11

If there is another highway segment or intersection to be evaluated for the selected year, return to Step 6. Otherwise, proceed to Step 12.
This step creates a loop in Step 7 through step 10 that is repeated or each of the individual highway segments and intersections within the project, for the selected year.

Step 12

If there is another year to be evaluated, return to Step 5. Otherwise, proceed to step 16 (when site-specific crash history data are not available) or step 13 (when site-specific crash history data are available).
This step creates a loop in step 6 to step 11 that is repeated for each year of the evaluation period.

Steps 13 - 15

Steps 13 - 15 Describe the Empirical Bayes (EB) procedure for combining predicted and observed crash frequencies when site-specific crash history data are available.

Step 13

Begin application of the Empirical Bayes (EB) procedure: Determine the observed crash history during the before period for each of the highway segments (for crashes stored by a specific location) and for each intersection. For crashes stored by a station range, create crash segments and determine the observed crash history for each.

Note: A basic assumption is that the highway is unchanged during the before period and that the crash history data corresponds to the before period highway.

The observed crash history for the before period is obtained from one of two sources: (1) an available crash data file in IHSDM format or (2) key entry of crash data. (Crash data can be specified by either a crash location or a range, in stations, of where the crash occurred. When historical crash data are stored by a specific location, the CPM assigns each crash to a road segment or intersection. When historical crash data are stored by station ranges (i.e., crash segments), the predicted number of crashes for each highway segment is matched to a crash segment as described in step 14.)

Crashes that are stored as "intersection-related" or "non-intersection-related" are analyzed as such by the CPM. Crashes for which the "Relationship to Intersection" is stored as "unknown" and which fall within 76 m (250 ft) of the center of an intersection are attributed to that intersection. If the centers of two intersections are closer than 500 feet, the historical crashes between the two intersections are assigned to the closest intersection. Crashes stored as "unknown" and located more than 76 m (250 ft) from the center of an intersection are attributed to the highway segment within which the crash falls.

For historical crashes stored by station ranges, crash segments matching the station ranges (one station range per crash segment) are created with the number of observed crashes assigned to each crash segment. If the available crash data consist only of totals at the project level, then there will be only one crash segment.

Step 14

For each highway segment and intersection, apply the EB procedure by computing the expected crash frequency for the before period as a weighted average of the predicted and observed crash frequencies.

When Historical Crash Data are Stored by a Specific Location - When historical crash data are stored by a specific location, the EB procedure is applied by computing a weighted average of the predicted and observed crash frequencies for the before period. The expected crash frequency considering both the predicted and observed crash frequencies is computed as:

$$E_p = w\,(N_p) + (1-w)\,O \quad \textbf{(4.7)}$$

where:

E_p = expected crash frequency based on a weighted average of N_p and O

N_p = original predicted crash frequency for the individual homogeneous highway segment or intersection from step 5 through step 12, i.e., number of crashes predicted by the crash prediction algorithm during the before period (equal to N_{rs} for a highway segment or N_i for an intersection)

w = weight to be placed on the crash frequency predicted by the crash prediction algorithm

O = number of crashes observed during a specified period of time

The weight placed on the predicted crash frequency is determined by:

$$w = 1 / (1 + k (N_p/L)) \quad \textbf{(4.8)}$$

where:

k = overdispersion parameter of the relevant base model in crash prediction algorithm

L = length of highway segment in miles, or 1 for intersections

Table 1., *Overdispersion Parameters for Base Models* shows the values of the overdispersion parameters (k) for the four base models used in the crash prediction algorithm.

Table 1. Overdispersion Parameters for Base Models

Geometric element	Overdispersion Parameter for base model (k)
Highway segment	0.236 miles
Three-leg intersections with minor-road STOP-control	0.54
Four-leg intersections with minor-road STOP-control	0.24
Four-leg signalized intersection	0.11

The weight to be placed on predicted crash frequency using Equation (4.8) is computed and then the expected crash frequency is computed using Equation (4.7).

This step is applied both to total crash frequencies and to crash frequencies by crash severity level (i.e., fatal and injury crashes, and property damage only). Since these computations are independent, the expected crash frequencies by severity level may not sum to the expected total crash frequency. A correction is made as follows so that the expected crash frequencies for the individual severity levels do sum to the expected total crash frequency:

$$E_{fi/corr} = E_{tot} E_{fi} / (E_{fi} + E_{pdo}) \quad \textbf{(4.9)}$$

$$E_{pdo/corr} = E_{tot} E_{pdo} / (E_{fi} + E_{pdo}) \quad \textbf{(4.10)}$$

where:

$E_{fi/corr}$ = expected crash frequency for fatal and injury crashes (corrected)

$E_{pdo/corr}$ = expected crash frequency for property-damage-only crashes (corrected)

E_{tot} = expected crash frequency for total crashes as estimated with Equation (4.7) and Equation (4.8)

E_{fi} = expected crash frequency for fatal and injury crashes as estimated with Equation (4.7) and Equation (4.8)

E_{pdo} = expected crash frequency for property-damage-only crashes as estimated with Equation (4.7) and Equation (4.8)

When Historical Crash Data are Stored by Station Ranges - If historical crash data has crashes assigned by ranges -- station 1+230 to station 2+000, for instance -- then the expected crash frequencies are calculated as follows:

Step 14-1 - Calculate a unit length crash prediction for each highway Segment located within a given Crash Segment, based on total predicted crashes per length of Highway Segment.

Step 14-2 - Determine the predicted number of crashes occurring on each Crash Segment for each portion of a Highway Segment located within a given Crash Segment.

Step 14-3 - Calculate the total predicted number of crashes on each Crash Segment by summing the contributions of each Highway Segment overlapping a given Crash Segment.

Step 14-4 - Calculate a unit length crash prediction for each Crash Segment by taking the predictions from Step 14-3 and dividing by the length of a given crash segment.

Step 14-5 - Determine the expected crash frequency for each Crash Segment (calculating the weights to be placed on the predicted crash frequency and the number of crashes observed).

Step 14-6 - Calculate the proportion of crashes expected to occur on each Crash Segment for each Highway Segment overlapping a given Crash Segment.

Step 14-7 - Determine the expected crash frequency for each Highway Segment by summing the contributions of each Crash Segment overlapped by a given Highway Segment.

See Section 8.1, *Example 1: When Data are Stored by Station Range (Non-overlapping Crash Segments)* for example calculations when historical crash data are stored by station ranges.

Step 15

Apply ADT growth factors and/or AMFs for geometric changes to convert the expected crash frequency for the before period to an expected crash frequency for the proposed project during the specified future time period.

At the conclusion of Step 14 , E_p represents the expected crash frequency for a given highway segment or intersection during the before period. To obtain an estimate of expected accident frequency in a future period (the analysis or after period), the estimate must be corrected for (1) any difference in the duration of the before and after periods; (2) any growth or decline in ADT between the before and after periods; and (3) any changes in geometric design or traffic control features between the before and after periods that affect the values of the AMFs for the highway segment or intersection. The expected crash frequency for a highway segment or intersection in the after period can be estimated as:

$$E_f = E_p\,(N_{bf} / N_{bp})\,(AMF_{1f} / AMF_{1p})\,(AMF_{2f} / AMF_{2p}) \ldots (AMF_{nf} / AMF_{np}) \quad \textbf{(4.11)}$$

where:

E_f = expected crash frequency during the future time period for which crashes are being forecast for the analysis segment or intersection in question

E_p = expected crash frequency for the past time period for which crash history data were available cras

N_{bf} = number of crashes forecast by the base model using the future ADT data, the specified nominal values for geometric parameters, and -- in the case of an analysis segment -- the actual length of the analysis segment

N_{bp} = number of crashes forecast by the base model using the past ADT data, the specified nominal values for geometric parameters, and -- in the case of an analysis segment -- the actual length of the analysis segment

AMF_{nf} = value of the nth AMF for the geometric conditions for the future (i.e., proposed analysis or after period) design

AMF_{np} = value of the nth AMF for the geometric conditions for the past (i.e., existing or before period) design

Equation (4.11) applies to total crash frequency. The expected future crash frequencies by severity level are determined by multiplying the expected accident frequency from the before period for each severity level by the ratio

$E_{f(total)} / E_{p(total)}$ or

$E_{f(severity_level_n)} = E_{p(severity_level_n)} E_{f(total)} / E_{p(total)}$ **(4.12)**

In the case of minor changes in highway alignment (e.g., flattening a horizontal curve), the length of an analysis segment may change from the past to the future time period. In this case, the length of the analysis segment for the existing condition is used to determine N_{bp} and the modified length of the analysis segment for the planned condition is used to determine N_{bf}. This implicitly incorporates the assumption that if the length of the analysis segment is changed, its base accident rate (per million-vehicle miles) remains constant and, therefore, the crash frequency increases or decreases in proportion to length. Of course, the AMF ratios that also appear in Equation (4.11) will account for any change in geometrics (e.g., reduction in radius of curvature) that accompany a change in length.

Step 16

Summarize and present the predictions.

In the final step, the predicted crash frequencies are summarized and presented in the CPM Analysis Report. A detailed description of the output included in the CPM Analysis Report is provided in Section 4.2, *Analysis Report*.

6. AMFs for Highway Segments

The procedures for determining the values of the AMFs for highway segments (AMF_1 through AMF_9) are described in this section. The geometric and traffic control data needed to evaluate these AMFs are described in Highway Segment Geometric and Traffic Control Data. The AMFs for highway segments include all of the variables in the highway segment base model (with the exception of vertical curvature) plus additional variables. Report No. FHWA-RD-99-207 , Prediction of the Expected Safety Performance of Rural Two-Lane Highways, provides a more detailed description of the base models for highway segments.

6.1 Lane Width (AMF_1)

The value of the AMF for lane width (AMF_{ra}) is determined as shown in Table 2., *Values of AMF for Lane Width of Highway Segments (AMF_{ra})*. If the lane width is less than or equal to 9 ft, the AMF for 9 ft shown in the table is used. If the lane width is greater than or equal to 12 ft, the AMF for 12 ft is used. If the lane width is equal to 9, 10, 11, or 12 ft, the value of the AMF is shown or is computed with the formulas provided in the table. If the lane width falls between the integer values listed, the value of AMF_{ra} is determined by interpolation between the values for those integer values of lane width.

Table 2. Values of AMF for Lane Width of Highway Segments (AMF_{ra})

Lane width (ft)	ADT < = 400	ADT = 401 to 1999 [a]	ADT >= 2000
9	1.05	1.50 - 0.000281*(2000 - ADT)	1.50
10	1.02	1.30 - 0.000175*(2000 - ADT)	1.30
11	1.01	1.05 - 0.000025*(2000 - ADT)	1.05
12	1.00	1.00	1.00

[a] This column presents mathematical expressions used to evaluate AMF_{ra} as a function of ADT.

The value of AMF_{ra} determined from Table 2., *Values of AMF for Lane Width of Highway Segments (AMF_{ra})* is modified as follows to convert it from related accidents to total accidents:

$AMF_1 = (AMF_{ra} - 1.0)\ P_{ra} + 1.0$ **(A-1)**

where:

AMF_1 = accident modification factor for total accidents

AMF_{ra} = accident modification factor for related accidents (i.e., single-vehicle run-of-road and multiple-vehicle

opposite-direction accidents), such as the accident modification factor for lane width shown in Table 2., *Values of AMF for Lane Width of Highway Segments (AMF_{ra})*

P_{ra} = proportion of total accidents constituted by related accidents

The proportion of related accidents (P_{ra}) should be set equal to the sum of four values from Table 16., *Default Percentage Distributions for Crash Type and Manner of Collision on Rural Two-Lane Highways* expressed as a proportion rather than as a percentage. These four values are:

- Percentage of single vehicle run-off-road accidents
- Percentage of multiple-vehicle head-on collisions
- Percentage of multiple-vehicle opposite-direction sideswipe collisions
- Percentage of multiple-vehicle same-direction sideswipe collisions

Based on the values in Table 16., *Default Percentage Distributions for Crash Type and Manner of Collision on Rural Two-Lane Highways*, the value of P_{ra} is 0.35. If the default values are replaced with data from the accident records of a specific highway agency, those replacement values are used in determining P_{ra}.

If the lane width differs between the two directions of travel for any highway segment, AMF_1 is computed separately for each direction of travel and the results averaged.

6.2 Shoulder Width and Type (AMF_2)

The AMF for shoulder width (AMF_{wra}) is determined as shown in Table 3., *Values of AMF for Shoulder Width of Highway Segments (AMF_{wra})*. If the shoulder width is greater than or equal to 8 ft, the shoulder width shown in the table for 8 ft is used. If the shoulder width is equal to 0, 2, 4, 6, or 8 ft, the value of AMF_{wra} is shown or is computed with the formulas provided in the

table. If the shoulder width falls between these values, the value of AMF_{wra} is determined by interpolation.

The AMF for shoulder type (AMF_{tra}) is determined from Table 4., *Accident Modification Factors for Shoulder Width (SW) and Shoulder Type on Two-Lane Highways (AMF_{tra})* using interpolation, as needed, between the shoulder width values shown in the table.

The AMF for shoulder width and type combined is determined as follows:

$$AMF_2 = (AMF_{wra}\ AMF_{tra} - 1.0)\ P_{ra} + 1.0 \quad \textbf{(A-2)}$$

where:

AMF_{wra} = accident modification factor for related accidents based on shoulder width (from Table 3., *Values of AMF for Shoulder Width of Highway Segments (AMF_{wra})*)

AMF_{tra} = accident modification factor for related accidents based on shoulder type (from Table 4., *Accident Modification Factors for Shoulder Width (SW) and Shoulder Type on Two-Lane Highways (AMF_{tra})*)

Table 3. Values of AMF for Shoulder Width of Highway Segments (AMF_{wra})

Shoulder width (ft)	ADT <=400	ADT = 401 to 1999 [a]	ADT >=2000
0	1.10	1.50 - 0.000250*(2000 - ADT)	1.50
2	1.07	1.30 - 0.000144*(2000 - ADT)	1.30
4	1.02	1.15 - 0.0000813*(2000 - ADT)	1.15
6	1.00	1.00	1.00
8	0.98	0.87 + 0.0000688*(2000 - ADT)	0.87

[a] This column presents mathematical expressions used to evaluate AMF_{wra} as a function of ADT.

Table 4. Accident Modification Factors for Shoulder Width (SW) and Shoulder Type on Two-Lane Highways (AMF_{tra})

Shoulder type	SW=0 ft	SW=1 ft	SW=2 ft	SW=3 ft	SW=4 ft	SW=6 ft	SW=8 ft	SW=10 ft
Paved	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Gravel	1.00	1.00	1.01	1.01	1.01	1.02	1.02	1.03
Composite	1.00	1.01	1.02	1.02	1.03	1.04	1.06	1.07
Turf	1.00	1.01	1.03	1.04	1.05	1.08	1.11	1.14

The proportion of related accidents (P_{ra}) used in Equation (A-2) should be the same as that used in Equation (A-1).

If the shoulder width differs between the two directions of travel for any highway segment, AMF_2 is computed separately for each direction of travel and the results averaged.

A composite shoulder is a shoulder that is paved for a portion of its width and turf for the remainder. The AMF values in Table 4., *Accident Modification Factors for Shoulder Width (SW) and Shoulder Type on Two-Lane Highways (AMF_{tra})* assume that half of the composite shoulder

width is paved and half is turf.

6.3 Horizontal Curve Length, Radius, and Presence or Absence of Spiral Transition (AMF_3)

If a highway segment is located on a tangent highway or on a spiral transition curve (i.e., not on a circular horizontal curve), then the value of AMF_3 is 1.00.

If a highway segment is located on a horizontal curve, then the value of AMF_3 is determined as follows:

$$AMF_3 = (1.55\ L_c + 80.2 / R - 0.012\ S) / 1.55\ L_c \quad \textbf{(A-3)}$$

where:

L_c = length of horizontal curve (mi)

R = radius of curvature (ft)

S = 1 if spiral transition curve is present

0 if spiral transition curve is not present

0.5 if spiral transition curve is present on one end of circular curve

Some homogeneous highway segments may be shorter than the horizontal curve being analyzed; L_c represents the total length of the horizontal curve which may be greater than the length of the highway segment. Where spiral transitions are present, L_c represents the length of the circular curve plus the lengths of spiral transitions.

In applying Equation (A-3), if the radius of curvature is less than 100 ft, the radius is set equal to 100 ft. If the length of the horizontal curve is less than 100 ft, the length of the horizontal curve is set equal to 100 ft.

AMFs are computed for each curve in a compound curve set. The length of the horizontal curve (L_c) used in Equation (A-3) is the total length for the compound curve set and the radius of curvature (R) is the radius for the individual curve in the compound curve set that is being analyzed.

If the computed value of AMF_3 for a horizontal curve is less than 1.00, AMF_3 is set equal to 1.00. This AMF applies to total highway segment accidents.

6.4 Superelevation (AMF_4)

The AMF for the superelevation of a horizontal curve is based on the superelevation deficiency defined as the difference between the actual superelevation of the horizontal curve (e_{act}) and the design superelevation (e_{design}) specified in the 1994 AASHTO Green Book. Superelevation deficiency (SD) is computed as:

$$SD = 0.00 \text{ if } e_{act} > e_{design} \quad \textbf{(A-4)}$$

$$SD = e_{design} - e_{act} \text{ if } e_{act} < e_{design} \quad \textbf{(A-5)}$$

The value of e_{act} for each horizontal curve is that input by the user as part of the highway geometric data. In applying Equation (A-6), negative values of e_{act} are permitted; such negative values are associated with superelevation with the opposite cross slope to that intended, which may well be associated with a superelevation deficiency. The value of e_{design} is determined from

interpolation in Design Superelevation tables (see Table 5., *Design Superelevation (e_{design}) as a Function of Maximum Superelevation Rate, Curve Radius (e_{max}=0.04) and Design Speed (V mi/h)* to Table 9., *Design Superelevation (e_{design}) as a Function of Maximum Superelevation Rate, Curve Radius (e_{max}=0.12) and Design Speed (V mi/h)*). Use of the Design Superelevation tables requires the horizontal curve radius, the horizontal curve design speed, and the value of the maximum superelevation rate (e_{max}) used by a highway agency. Interpolation of values of e_{design} between the radii shown in the Design Superelevation tables for a given value of e_{max} is performed. Interpolation between design speeds will be necessary for design speeds in US customary units between those shown in the table and for design speeds converted from metric units. If e_{design} exceeds 0.120, e_{design} is set equal to 0.120.

The value of the AMF for superelevation (AMF_4) is determined as:

AMF_4 = 1.00 for SD < 0.01 **(A-6)**

AMF_4 = 1.00 + 6 (SD-0.01) for 0.01 <SD < 0.02 **(A-7)**

AMF_4 = 1.06 + 3 (SD-0.03) for 0.02 <SD **(A-8)**

6.5 Grades (AMF_5)

The AMF for percent grade (AMF_5) is determined as:

AMF_5 = 1.00 + 0.016 |PG| **(A-9)**

where:

PG = percent grade for the highway segment

If the percent grade exceeds 12 percent, PG is set equal to 12 percent. Grades are determined from Vertical Point of Intersection (VPI) to Vertical Point of Intersection. Vertical curves are not considered.

Table 5. Design Superelevation (e_{design}) as a Function of Maximum Superelevation Rate, Curve Radius (e_{max}=0.04) and Design Speed (V mi/h)

e_{max}	Radius(ft)	V=30	V=40	V=50	V=60	V=70
0.04	22918	0.000	0.000	0.000	0.000	0.000
0.04	11459	0.000	0.000	0.000	0.000	0.000
0.04	7639	0.000	0.000	0.000	0.000	0.000
0.04	5730	0.000	0.000	0.000	0.025	0.025
0.04	3820	0.000	0.000	0.024	0.029	0.029
0.04	2865	0.000	0.022	0.027	0.033	0.033
0.04	2292	0.000	0.025	0.030	0.036	0.036
0.04	1910	0.020	0.027	0.033	0.039	0.039
0.04	1637	0.022	0.028	0.035	0.040	0.040
0.04	1432	0.024	0.030	0.037	0.040	0.040
0.04	1146	0.026	0.033	0.039	0.040	0.040
0.04	955	0.028	0.036	0.040	0.040	0.040
0.04	819	0.030	0.037	0.040	0.040	0.040
0.04	716	0.031	0.039	0.040	0.040	0.040
0.04	637	0.033	0.040	0.040	0.040	0.040
0.04	573	0.034	0.040	0.040	0.040	0.040
0.04	521	0.035	0.040	0.040	0.040	0.040
0.04	477	0.036	0.040	0.040	0.040	0.040
0.04	441	0.037	0.040	0.040	0.040	0.040
0.04	409	0.038	0.040	0.040	0.040	0.040
0.04	358	0.039	0.040	0.040	0.040	0.040
0.04	318	0.040	0.040	0.040	0.040	0.040
0.04	286	0.040	0.040	0.040	0.040	0.040
0.04	260	0.040	0.040	0.040	0.040	0.040
0.04	239	0.040	0.040	0.040	0.040	0.040
0.04	220	0.040	0.040	0.040	0.040	0.040

Table 6. Design Superelevation (e_{design}) as a Function of Maximum Superelevation Rate, Curve Radius (e_{max}=0.06) and Design Speed (V mi/h)

e_{max}	Radius(ft)	V=30	V=40	V=50	V=60	V=70
0.06	22918	0.000	0.000	0.000	0.000	0.000
0.06	11459	0.000	0.000	0.000	0.000	0.000
0.06	7639	0.000	0.000	0.000	0.021	0.026
0.06	5730	0.000	0.000	0.020	0.027	0.033
0.06	3820	0.000	0.020	0.025	0.037	0.046
0.06	2865	0.000	0.025	0.030	0.045	0.055
0.06	2292	0.020	0.030	0.034	0.051	0.059
0.06	1910	0.023	0.034	0.038	0.055	0.060
0.06	1637	0.026	0.038	0.041	0.058	0.060
0.06	1432	0.029	0.041	0.046	0.060	0.060
0.06	1146	0.034	0.046	0.050	0.060	0.060
0.06	955	0.038	0.050	0.053	0.060	0.060
0.06	819	0.041	0.053	0.056	0.060	0.060
0.06	716	0.043	0.056	0.058	0.060	0.060
0.06	637	0.046	0.058	0.059	0.060	0.060
0.06	573	0.048	0.059	0.060	0.060	0.060
0.06	521	0.050	0.060	0.060	0.060	0.060
0.06	477	0.052	0.060	0.060	0.060	0.060
0.06	441	0.054	0.060	0.060	0.060	0.060
0.06	409	0.055	0.060	0.060	0.060	0.060
0.06	358	0.058	0.060	0.060	0.060	0.060
0.06	318	0.059	0.060	0.060	0.060	0.060
0.06	286	0.060	0.060	0.060	0.060	0.060
0.06	260	0.060	0.060	0.060	0.060	0.060
0.06	239	0.060	0.060	0.060	0.060	0.060
0.06	220	0.060	0.060	0.060	0.060	0.060

Table 7. Design Superelevation (e_{design}) as a Function of Maximum Superelevation Rate, Curve Radius (e_{max}=0.08) and Design Speed (V mi/h)

e_{max}	Radius(ft)	V=30	V=40	V=50	V=60	V=70
0.08	22918	0.000	0.000	0.000	0.000	0.000
0.08	11459	0.000	0.000	0.000	0.000	0.000
0.08	7639	0.000	0.000	0.000	0.022	0.028
0.08	5730	0.000	0.000	0.021	0.029	0.036
0.08	3820	0.000	0.021	0.030	0.041	0.051
0.08	2865	0.000	0.027	0.038	0.051	0.065
0.08	2292	0.021	0.033	0.046	0.061	0.075
0.08	1910	0.025	0.038	0.053	0.068	0.080
0.08	1637	0.028	0.043	0.058	0.074	0.080
0.08	1432	0.031	0.047	0.063	0.078	0.080
0.08	1146	0.038	0.055	0.071	0.080	0.080
0.08	955	0.043	0.062	0.077	0.080	0.080
0.08	819	0.048	0.067	0.080	0.080	0.080
0.08	716	0.053	0.071	0.080	0.080	0.080
0.08	637	0.056	0.075	0.080	0.080	0.080
0.08	573	0.060	0.078	0.080	0.080	0.080
0.08	521	0.063	0.079	0.080	0.080	0.080
0.08	477	0.065	0.080	0.080	0.080	0.080
0.08	441	0.068	0.080	0.080	0.080	0.080
0.08	409	0.070	0.080	0.080	0.080	0.080
0.08	358	0.074	0.080	0.080	0.080	0.080
0.08	318	0.077	0.080	0.080	0.080	0.080
0.08	286	0.079	0.080	0.080	0.080	0.080
0.08	260	0.080	0.080	0.080	0.080	0.080
0.08	239	0.080	0.080	0.080	0.080	0.080
0.08	220	0.080	0.080	0.080	0.080	0.080

Table 8. Design Superelevation (e_{design}) as a Function of Maximum Superelevation Rate, Curve Radius (e_{max}=0.10) and Design Speed (V mi/h)

e_{max}	Radius(ft)	V=30	V=40	V=50	V=60	V=70
0.10	22918	0.000	0.000	0.000	0.000	0.000
0.10	11459	0.000	0.000	0.000	0.000	0.000
0.10	7639	0.000	0.000	0.000	0.023	0.028
0.10	5730	0.000	0.000	0.021	0.030	0.037
0.10	3820	0.000	0.021	0.031	0.043	0.054
0.10	2865	0.000	0.028	0.040	0.055	0.070
0.10	2292	0.021	0.034	0.049	0.067	0.085
0.10	1910	0.025	0.040	0.057	0.077	0.096
0.10	1637	0.029	0.046	0.065	0.086	0.100
0.10	1432	0.033	0.051	0.072	0.093	0.100
0.10	1146	0.040	0.061	0.083	0.098	0.100
0.10	955	0.046	0.070	0.092	0.100	0.100
0.10	819	0.053	0.078	0.098	0.100	0.100
0.10	716	0.058	0.084	0.100	0.100	0.100
0.10	637	0.063	0.089	0.100	0.100	0.100
0.10	573	0.068	0.094	0.100	0.100	0.100
0.10	521	0.072	0.097	0.100	0.100	0.100
0.10	477	0.076	0.099	0.100	0.100	0.100
0.10	441	0.080	0.100	0.100	0.100	0.100
0.10	409	0.083	0.100	0.100	0.100	0.100
0.10	358	0.089	0.100	0.100	0.100	0.100
0.10	318	0.093	0.100	0.100	0.100	0.100
0.10	286	0.097	0.100	0.100	0.100	0.100
0.10	260	0.099	0.100	0.100	0.100	0.100
0.10	239	0.100	0.100	0.100	0.100	0.100

Table 9. Design Superelevation (e_{design}) as a Function of Maximum Superelevation Rate, Curve Radius (e_{max}=0.12) and Design Speed (V mi/h)

e_{max}	Radius(ft)	V=30	V=40	V=50	V=60	V=70
0.12	22918	0.000	0.000	0.000	0.000	0.000
0.12	11459	0.000	0.000	0.000	0.000	0.000
0.12	7639	0.000	0.000	0.000	0.023	0.029
0.12	5730	0.000	0.000	0.022	0.030	0.038
0.12	3820	0.000	0.022	0.032	0.044	0.056
0.12	2865	0.000	0.029	0.042	0.058	0.073
0.12	2292	0.022	0.035	0.051	0.070	0.090
0.12	1910	0.026	0.042	0.060	0.082	0.106
0.12	1637	0.030	0.048	0.069	0.094	0.118
0.12	1432	0.034	0.054	0.077	0.104	0.120
0.12	1146	0.041	0.065	0.092	0.117	0.120
0.12	955	0.049	0.075	0.104	0.120	0.120
0.12	819	0.055	0.085	0.113	0.120	0.120
0.12	716	0.068	0.094	0.119	0.120	0.120
0.12	637	0.068	0.101	0.120	0.120	0.120
0.12	573	0.074	0.107	0.120	0.120	0.120
0.12	521	0.079	0.112	0.120	0.120	0.120
0.12	477	0.084	0.116	0.120	0.120	0.120
0.12	441	0.089	0.119	0.120	0.120	0.120
0.12	409	0.093	0.120	0.120	0.120	0.120
0.12	358	0.101	0.120	0.120	0.120	0.120
0.12	318	0.108	0.120	0.120	0.120	0.120
0.12	286	0.113	0.120	0.120	0.120	0.120
0.12	260	0.116	0.120	0.120	0.120	0.120
0.12	239	0.119	0.120	0.120	0.120	0.120
0.12	220	0.120	0.120	0.120	0.120	0.120

6.6 Driveway Density (AMF_6)

The AMF for driveway density (AMF_6) is determined as:

$$AMF_6 = (0.2 + [0.05 - 0.005 \ln ADT_y] DD) / (0.2 + [0.05 - 0.005 \ln ADT_y] (5)) \quad \textbf{(A-10)}$$

where:

ADT_y = annual average daily traffic volume of the highway being evaluated (veh/day)

DD = driveway density for both sides of the road combined (driveways/mile)

6.7 Passing Lanes and Short Four-Lane Sections (AMF_7)

If no passing lane is present on a highway segment, then the value of AMF_7 is 1.00.

If a passing lane is present in one direction of travel (i.e., two lanes in one direction and one lane in the other direction of travel), then the value of AMF_7 is 0.75.

If a short four-lane section is provided on a two-lane highway, then the value of AMF_7 is 0.65. The value of 0.65 should be used for any cross section where two lanes are provided in both directions of travel; this value should be used for short four-lane sections that begin and end at the same station or for any area where passing lanes in opposing directions of travel overlap.

6.8 Two-Way Left-Turn Lanes (AMF_8)

If no center TWLTL is present on a highway section, the value of AMF_8 is 1.00.

If a center TWLTL is present, the value of the AMF is determined as:

$AMF_8 = 1.00 - 0.35\ P_{AP}$ **(A-11)**

where:

P_{AP} = access-point-related accidents as a proportion of total accidents

The value of P_{AP} is determined as:

$P_{AP} = (0.0047\ DD + 0.0024\ DD^2) / (1.199 + 0.0047\ DD + 0.0024\ DD^2)$ **(A-12)**

If the driveway density (DD) is less than five driveways per mile, the value of AMF_8 is 1.00.

6.9 Roadside Hazard Rating (AMF_9)

The AMF for roadside hazard rating (AMF_9) is determined as:

$AMF_9 = \exp (-0.6869 + 0.0668\ RHR) / \exp (-0.4865)$ **(A-13)**

where:

RHR = roadside hazard rating for the highway segment considering both sides of the road (1 to 7 scale)

The roadside hazard rating for a highway section ranges from 1 (best roadside) to 7 (poorest roadside). This roadside hazard rating scale is explained and illustrated Report FHWA-RD-99-207, Prediction of the Expected Safety Performance of Rural Two-Lane Highways.

6.10 Reference

Report No. FHWA-RD-99-207, *Prediction of the Expected Safety Performance of Rural Two-Lane Highways* is available online (`http://www.tfhrc.gov/safety/pubs/99207/index.htm`).

7. AMFs for Intersections

The procedures for determining the values of the AMFs for intersections (AMF_{10} through AMF_{14}) are described in this section. The values of the geometric and traffic control variables needed to determine the AMFs are found in Intersection Geometric and Traffic Control Data. In all cases, the values for the AMFs for minor leg YIELD-controlled intersections are the same as for minor STOP-controlled intersections.

7.1 Intersection Skew Angle (AMF_{10})

The AMF for intersection skew angle (AMF_{10}) is defined as follows:

For a three-leg intersection with minor-road STOP-control, the value of AMF_{10} is:

$AMF_{10} = \exp (0.0040\ SKEW)$ **(A-14)**

where:

SKEW = intersection skew angle (degrees) expressed as the absolute value of the difference between 90 degrees and the actual angle between the major legs and minor legs of the intersection. The skew angle is always within the range from 0 to 90 degrees (i.e., for nonzero skew angles, always measure the acute rather than the obtuse angle).

For a four-leg intersection with minor-road STOP-control, the value of AMF_{10} is:

$$AMF_{10} = \exp(0.0054\ SKEW) \quad \textbf{(A-15)}$$

If the skew angle for a four-leg intersection with minor-road STOP-control differs for the two minor legs , AMF_{10} is computed separately for each minor leg and the results averaged.

For a four-leg signalized intersection or an all-way STOP-controlled intersection, the value of AMF_{10} is 1.00.

7.2 Intersection Traffic Control (AMF_{11})

The value of AMF_{11} for an intersection with all-way STOP control is 0.53. For all other intersections, the value of AMF_{11} is 1.00.

7.3 Intersection Left-Turn Lanes (AMF_{12})

The values of the AMF for left-turn lanes (AMF_{12}) on one or more major legs to an intersection is specified in Table 10., *Accident Modification Factors for Installation of Left-turn Lanes on the Major Legs to Intersections*. If there are no left-turn lanes on any major legs to the intersection, the value of AMF_{12} is 1.00. These AMFs are based upon the research documented in Report No. FHWA-RD-02-089, Safety Effectiveness of Intersection Left- and Right-Turn Lanes. The values in this table are from the judgement of an expert panel combining results from several sources.

Table 10. Accident Modification Factors for Installation of Left-turn Lanes on the Major Legs to Intersections

Intersection type	Intersection traffic control	One approach (leg) on which left-turn lanes installed	Both approaches (legs) on which left-turn lanes installed
Three-leg intersection	STOP control	0.56	N/A
Three-leg intersection	Signal control	0.85	N/A
Four-leg intersection	STOP control	0.72	0.52
Four-leg intersection	Signal control	0.82	0.67

Note that three-leg, signal-control intersections are not modeled by CPM.

7.4 Intersection Right-Turn Lanes (AMF_{13})

The value of the AMF for right-turn lanes (AMF_{13}) on one or more major legs to an intersection is specified in Table 11., *Accident Modification Factors for Installation of Right-turn Lanes on the Major Legs to Intersections*. If there are no right-turn lanes on any major leg to the intersection, the value of AMF_{13} is 1.00. These AMFs are based upon the research documented in Report No. FHWA-RD-02-089, Safety Effectiveness of Intersection Left- and Right-Turn Lanes.

Table 11. Accident Modification Factors for Installation of Right-turn Lanes on the Major Legs to Intersections

Intersection type	Intersection traffic control	One approach (leg) on which right-turn lanes are installed	Both approaches (legs) on which right-turn lanes are installed
Three-leg intersection	STOP control	0.86	N/A
Three-leg intersection	Signal control	0.96	N/A
Four-leg intersection	STOP control	0.86	0.74
Four-leg intersection	Signal control	0.96	0.92

Note that three-leg, signal-control intersections are not modeled by CPM.

7.5 Intersection Sight Distance (AMF_{14})

The value of the AMF for l limited intersection sight distance (AMF_{14}) at three- and four-leg intersections with minor leg STOP control is specified in Table 12., *Accident Modification Factors for Intersection Sight Distance Limitations in Quadrants of Three-Leg and Four-Leg Intersections with Minor STOP Control*. For four-leg signalized intersections and all-way STOP-controlled intersections, the value of AMF_{14} is 1.00.

Table 12. Accident Modification Factors for Intersection Sight Distance Limitations in Quadrants of Three-Leg and Four-Leg Intersections with Minor STOP Control

Number of quadrants with limited intersection sight distance	Accident Modification Factor (AMF_{14})
0	1.00
1	1.05
2	1.10
3	1.15
4	1.20

8. Examples

8.1 Example 1: When Data are Stored by Station Range (Non-overlapping Crash Segments)

The following example illustrates how the CPM estimates the expected crash frequency when historical data are stored by station ranges and crash segments do not overlap.

Given:

- Six homogeneous highway segments (RS1-RS6)
- Three crash segments (CS1-CS3)
- Four crashes occurring on crash segment CS1 during the "before" period

Figure 3, *Schematic of Example 1 highway with Historical Crash Data Stored by Station Range* shows a schematic of the homogeneous highway segments and the crash segments for this example.

Table 13., *Highway Segment Data and Calculations for Example 1* contains data on highway segment lengths (second column) and lengths of highway segments within crash segments (third column).

```
|--RS1--|-----RS2-----|---RS3---|-RS4-|-RS5-|--RS6--| Roadway Segments

|-------CS1-------|---------CS2-----------|---CS3---| Crashes assigned to a
                                                      range of stations
                                                      (Step 13)
```

Figure 3 Schematic of Example 1 highway with Historical Crash Data Stored by Station Range

Step 14-1

$N_{rs(j)}$, the total predicted number of crashes per year for highway segment j obtained in Equation (4.1), must first be divided by the highway segment length ($L_{rs(j)}$) to obtain a unit length crash prediction for the highway segment, $U_{rs(j)}$. This can be expressed as follows:

$U_{rs(j)} = N_{rs(j)} / L_{rs(j)}$ **(Ex1.1)**

Table 13., *Highway Segment Data and Calculations for Example 1* shows the unit length crash prediction calculations for highway segments RS1 and RS2 using Equation (Ex1.1).

Table 13. Highway Segment Data and Calculations for Example 1

Highway Segment No.	Highway SegmentLength $L_{rs(j)}$ (mi.)	Length in CrashSegment 1 (CS1) l_{ij} (mi.)	Total Predicted No. of Crashes for Highway Segment $N_{rs(j)}$ (crashes/yr)	Step 14-1: Unit Length Crash Prediction $U_{rs(j)}$(crashes/mi.-yr)	Step 14-2: Predicted No. of Crashes Occurring on CrashSegment 1 (CS1) $N_{cs(ij)}$ (crashes/yr)
RS1	0.1 ($L_{rs(1)}$)	0.1 (l_{11})	1.5 ($N_{rs(1)}$)	15 ($U_{rs(1)}$)	1.50 ($N_{cs(11)}$)
RS2	0.3 ($L_{rs(2)}$)	0.2 (l_{12})	2.0 ($N_{rs(2)}$)	6.67 ($U_{rs(2)}$)	1.33 ($N_{cs(12)}$)
Total (Step 14-3)					2.83 ($N_{cs(1)}$)

Step 14-2

A predicted number of crashes that can be expected to occur on the given crash segment ($N_{cs(i)}$) must be estimated. Since it is unknown as to exactly where on the crash segment that the historical crashes occurred, the unit length predicted number of crashes for each highway segment ($U_{rs(j)}$) are used to predict the number of crashes on the crash segment, for the portion of a given highway segment that is located within the crash segment ($N_{cs(ij)}$). This can be expressed with the following equation:

$N_{cs(ij)} = U_{rs(j)} (L_{ij})$ **(Ex1.2)**

where:

$N_{cs(ij)}$ = predicted crashes on crash segment i, for portion of highway segment j located within crash segment i

$U_{rs(j)}$ = unit length crash prediction for highway segment j

l_{ij} = length of highway segment j within crash segment i

Since the entire length (0.1 mi.) of highway segment 1 (RS1) is within crash segment 1 (CS1), $N_{cs(11)}$ would be found as follows:

$N_{cs(11)} = U_{rs(1)} L_{11} = 15 (0.1) = 1.5$ crashes/yr

Since only 0.2 mi. of highway segment 2 (RS2) is within crash segment 1 (CS1), Ncs(12) would be found as follows:

$N_{cs(12)} = U_{rs(2)} L_{12} = 6.67 (0.2) = 1.33$ crashes/yr

These values are shown in the last column of Table 13., *Highway Segment Data and Calculations for Example 1*.

Step 14-3

Next find the total number of crashes occurring on crash segment CS1 ($N_{cs(1)}$). For this example,

$N_{cs(1)} = 2.83$ crashes/year; where ($N_{cs(1)} = N_{cs(11)} + N_{cs(12)}$).

In its basic form, the total predicted number of crashes occurring on crash segment i can be expressed as follows:

$N_{cs(i)} = N_{cs(i1)} + N_{cs(i2)} + N_{cs(i3)} + ...+ N_{cs(in)}$ **(Ex.1.3)**

where:

n = the number of highway segments overlapping crash segment i.

Step 14-4

The overall unit length crash prediction for the crash segment (P_i) can then be found by dividing the total predicted number of crashes occurring on crash segment i ($N_{cs(i)}$) by the length of the crash segment ($L_{cs(i)}$).

$P_i = N_{cs(i)} / L_{cs(i)}$ **(Ex.1.4)**

For this example, the unit length crash prediction on crash segment 1 (CS1) would be:

$P_1 = N_{cs(1)} / L_{cs(1)} = 2.83 / 0.3 = 9.44$ crashes/mi.

Step 14-5

The weight placed on the predicted crash frequency (*w*) can then be found by modifying Equation (4.8) as follows:

$w = 1 / (1 + k (P_1))$ **(Ex.1.5)**

which, for this example, would give the following result:

$w = 1 / (1 + 0.236 (9.44)) = 0.3097$

The expected crash frequency for crash segment i ($Ep_{cs(i)}$), based on a weighted average of the original number of predicted crashes ($N_{cs(i)}$) and the observed crashes (O), can then be found using the following equation:

$Ep_{cs(i)} = w(N_{cs(i)}) + (1-w)O$ **(Ex.1.6)**

which would yield the following result for this example:

$Ep_{cs(1)} = w(N_{cs(1)}) + (1-w)O = 0.3097(2.83) + (1-0.3097)(4) = 3.638$ crashes

Steps 14-6 and 14-7

Next, the proportion of the expected crashes that belongs to each highway segment ($Ep_{rs(j)}$) must

be determined from the expected crashes for crash segments ($Ep_{cs(i)}$, as determined in Equation Ex.1.6). This can be determined by distributing the crashes in the same proportion to the values in the last column in Table 13., *Highway Segment Data and Calculations for Example 1*.

Therefore, the expected crash frequency on highway segment *j* ($Ep_{rs(j)}$) would be found as follows:

$$Ep_{rs(j)} = Ep_{rs(1)} N_{cs(1j)} / N_{cs(1)} + Ep_{rs(2)} N_{cs(2j)} / N_{cs(2)} + ... + Ep_{rs(m)} N_{cs(mj)} / N_{cs(m)}$$
(Ex.1.7)

where:

m = the number of crash segments that highway segment *j* overlaps.

For this example, the expected crash frequency on highway segment 1 ($Ep_{rs(1)}$) would be:

$$Ep_{rs(1)} = Ep_{rs(1)} N_{cs(11)} / N_{cs(1)} = 3.638 \text{ x } 15.0 / 2.83 = 1.928 \text{ crashes}$$

The expected crash frequency on highway segment 2 (Eprs(2)) would be the sum of the contributions from crash segment 1 (CS1) and crash segment 2 (CS2):

$$Ep_{rs(2)} = Ep_{rs(1)} N_{cs(12)} / N_{cs(1)} + Ep_{rs(2)} N_{cs(22)} / N_{cs(2)}$$

where the contribution from crash segment 1 (CS1) is:

$$= Ep_{rs(1)} N_{cs(12)} / N_{cs(1)} = 3.638 \text{ x } 1.33 / 2.83 = 1.710 \text{ crashes}$$

and the contribution from crash segment 2 (CS2) would still need to be added.

Again, this step is applied both to total crash frequencies and to crash frequencies by crash severity level (i.e., fatal and injury crashes and property damage only). Equation 4.9 and Equation 4.10 are used as described earlier to ensure that the expected crash frequencies for the individual severity levels sum to the expected total crash frequency.

8.2 Example 2: When Data are Stored by Station Range (Overlapping Crash Segments)

The following example illustrates how the CPM estimates the expected crash frequency when historical data are stored by station ranges and ***crash segments*** overlap.

Given:

- Six homogeneous highway segments (RS1-RS6)
- Five crash segments (CS1-CS5)
- Four crashes occurring on crash segment CS1 during the "before" period, three crashes occurring on crash segment CS2, three crashes occurring on CS3, two crashes on CS4, and one crash on CS5.

Figure 4, *Schematic of Example 2 Highway with Crash History Data Stored by Range* shows a schematic of the homogeneous highway segments and the crash segments for this example.
Tables Table 13., *Highway Segment Data and Calculations for Example 1* and Table 14., *Highway Segment Data and Calculations for Example 2, Crash Segment 2* contain data on highway segment lengths (second column) and lengths of highway segments within crash segments (third column).

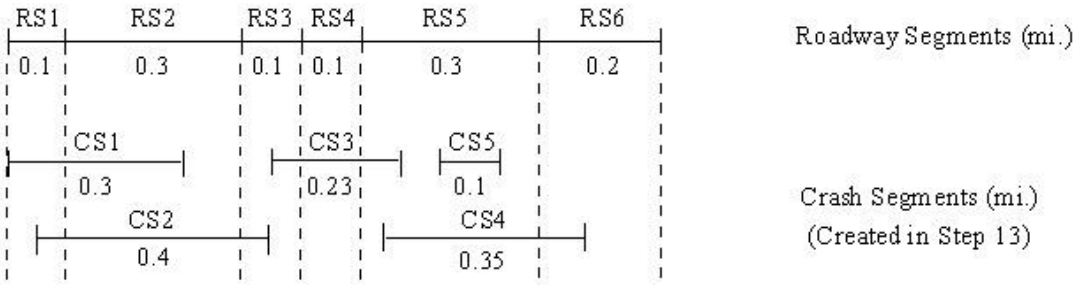


Figure 4 Schematic of Example 2 Highway with Crash History Data Stored by Range

Crash Segment 1 (CS1)

Steps 14-1 to 14-5:

Refer to Section 8.1, *Example 1: When Data are Stored by Station Range (Non-overlapping Crash Segments)* and the data and calculations in Table 13., *Highway Segment Data and Calculations for Example 1*.

Steps 14-6 and 14-7:

The proportion of the expected crashes that belongs to each highway segment ($Ep_{rs(j)}$) must be determined from the expected crashes for crash segments ($Ep_{cs(i)}$, as determined in Equation (Ex.1.6). This can be determined by distributing the crashes in the same proportion to the values in the last column in Table 13., *Highway Segment Data and Calculations for Example 1*. However, since two crash segments (CS1 and CS2) overlap highway segment 1 (RS1), $Ep_{rs(1)}$ will be the sum of the proportion of crashes from crash segment 1 (CS1) and crash segment 2 (CS2) as described by Equation (Ex.1.7). For this example the expected crash frequency on highway segment 1 ($Ep_{rs(1)}$) would be:

$$Ep_{rs(i)} = Ep_{cs(i)} N_{cs(11)} / N_{cs(1)} + Ep_{cs(i)} N_{cs(11)} / N_{cs(1)}$$

where:

$Ep_{cs(i)} N_{cs(11)} / N_{cs(1)}$ represents the influence of crash segment 1 (CS1) and $Ep_{cs(i)} N_{cs(11)} / N_{cs(1)}$

represents the influence of crash segment 2 (CS2), which has not yet been determined. Therefore, the influence of crash segment 2 (CS2) on highway segment 1 (RS1) must be found using the same eight-step procedure.

Crash Segment 2 (CS2)

Step 14-1:

$N_{rs(j)}$, the total predicted number of crashes per year for highway segment j obtained in Equation (4.1) must first be divided by the highway segment length ($L_{rs(j)}$) to obtain a unit length crash prediction for the highway segment, $U_{rs(j)}$. This can be done using Equation (Ex1.1). The results of the unit length crash prediction calculations for highway segments RS1, RS2, and RS3 using Equation (Ex1.1) are shown in Table 14., *Highway Segment Data and Calculations for Example 2, Crash Segment 2*.

Table 14. Highway Segment Data and Calculations for Example 2, Crash Segment 2

Highway Segment No.	Length Lrs(j) (mi.)	Length in Crash Segment 2(CS2) l_{ij} (mi.)	Total Predicted No. of Crashes for Highway Segment $N_{rs(j)}$ (crashes/yr)	*Step 14-1:* Unit Length CrashPrediction $U_{rs(j)}$ (crashes/mi.-yr)	*Step 14-2:* PredictedNo. of Crashes Occurring on Crash Segment 2 (CS2) Ncs(ij) (crashes/yr)
RS1	0.1 ($L_{rs(1)}$)	0.05 (l_{21})	1.5 ($N_{rs(1)}$)	15 ($U_{rs(1)}$)	0.75 ($N_{cs(21)}$)
RS2	0.3 ($L_{rs(2)}$)	0.3 (l_{22})	2.0 ($N_{rs(2)}$)	6.67 ($U_{rs(2)}$)	2.0 (Ncs(22))
RS3	0.1 ($L_{rs(3)}$)	0.05 (l_{23})	1.0 ($N_{rs(3)}$)	10 ($U_{rs(3)}$)	0.50 (Ncs(23))
Total (*Step 14-3*)					3.25 ($N_{cs(2)}$)

Step 14-2:

A predicted number of crashes that can be expected to occur on the given crash segment ($N_{cs(i)}$) must be estimated. Since it is unknown as to exactly where on the crash segment that the historical crashes occurred, the unit length predicted number of crashes for each highway segment ($U_{rs(j)}$) are used to predict the number of crashes on the crash segment, for the portion of a given highway segment that is located within the crash segment. This can be found using Equation (Ex1.2).

Since only 0.05 mi. of highway segment 1 (RS1) is within crash segment 2 (CS2), $N_{cs(21)}$ would be found as follows:

$$N_{cs(21)} = U_{rs(1)} (l_{21}) = 15 (0.5) = 0.75 \text{ crashes/yr}$$

Since the entire length (0.3 mi.) of highway segment 2 (RS2) is within crash segment 2 (CS2), $N_{cs(22)}$ would be found as follows:

$$N_{cs(22)} = U_{rs(2)} (l_{22}) = 6.67 (0.3) = 2.0 \text{ crashes/yr}$$

Likewise, $N_{cs(23)}$ would be found as follows:

$$N_{cs(23)} = U_{rs(3)} (l_{23}) = 10 (0.05) = 0.05 \text{ crashes/yr}$$

These values are shown in the last column of Table 14., *Highway Segment Data and Calculations for Example 2, Crash Segment 2*.

Step 14-3:

Next find the total number of crashes occurring on crash segment CS2 ($N_{cs(2)}$). This can be found using Equation (Ex1.3), which yields 3.25 crashes/year.

$$(N_{cs(2)} = N_{cs(21)} + N_{cs(22)} + N_{cs(23)}).$$

Step 14-4:

The overall unit length crash prediction for the crash segment (P_i) can then be found by using Equation (Ex1.4). For this example, the unit length crash prediction on crash segment 2 (CS2) would be:

$$P_2 = N_{cs(2)} / L_{cs(2)} = 3.25 / 0.4 = 8.125 \text{ crashes/mi.}$$

Step 14-5:

The weight placed on the predicted crash frequency (w) can then be found by using Equation (Ex1.5), which would give the following result:

$w = 1 / (1 + k (P_2)) = 1 / (1 + 0.236 (8.125)) = 0.3428$

The expected crash frequency for crash segment i ($Ep_{cs(i)}$), based on a weighted average of the original number of predicted crashes $N_{cs(i)}$) and the observed crashes (O), can then be found using Equation (Ex1.6), which would yield the following result:

$Ep_{cs(2)} = w(N_{cs(2)}) + (1-w)O = 0.3428(3.25) + (1-0.3428)(3) = 3.0857$ crashes

Steps 14-6 and 14-7:

Next, the proportion of the expected crashes that belongs to each highway segment ($Ep_{rs(j)}$) must be determined from the expected crashes for crash segments ($Ep_{cs(i)}$), as determined in Equation (Ex1.6). This can be determined by distributing the crashes in the same proportion to the values in the last column in Table 13., *Highway Segment Data and Calculations for Example 1* and Table 14., *Highway Segment Data and Calculations for Example 2, Crash Segment 2* and is expressed in Equation (Ex1.7). For this example, the expected crash frequency on highway segment 1 ($Ep_{rs(1)}$) would be the sum of the weighted contributions from crash segment 1 (CS1) and crash segment 2 (CS2) or:

$Ep_{rs(1)} = Ep_{rs(1)} N_{cs(11)} / N_{cs(1)} + Ep_{rs(2)} N_{cs(21)} / N_{cs(2)} = 3.638 \times 1.5 / 2.83 + 3.086 \times 0.75 / 3.25 = 1.928 + 0.7122 = 2.640$ crashes

Likewise, the expected crash frequency on highway segment 2 ($Ep_{rs(2)}$) would be:

$Ep_{rs(2)} = Ep_{rs(1)} N_{cs(12)} / N_{cs(1)} + Ep_{rs(2)} N_{cs(22)} / N_{cs(2)} = 3.638 \times 1.33 / 2.83 + 3.086 \times 2.00 / 3.25 = 1.710 + 1.899 = 3.6091$ crashes

Again, this step is applied both to total crash frequencies and to crash frequencies by crash severity level (i.e., fatal and injury crashes and property damage only). Equation 4.9 and Equation 4.10 are used as described earlier to ensure that the expected crash frequencies for the individual severity levels sum to the expected total crash frequency.

8.3 Example 3: Applying the Sliding Scale Analysis

The following example illustrates how the sliding scale analysis is applied to the expected crash rates by homogeneous highway segment and plotted.

Given:

$W = 500$ ft.,

$n = 5$,

and $I = W/n = 100$ft.

Figure 5, *Plot of Predicted Crashes for Example Highway by Station*, a plot of predicted crashes/yr for a highway 1200 ft. long

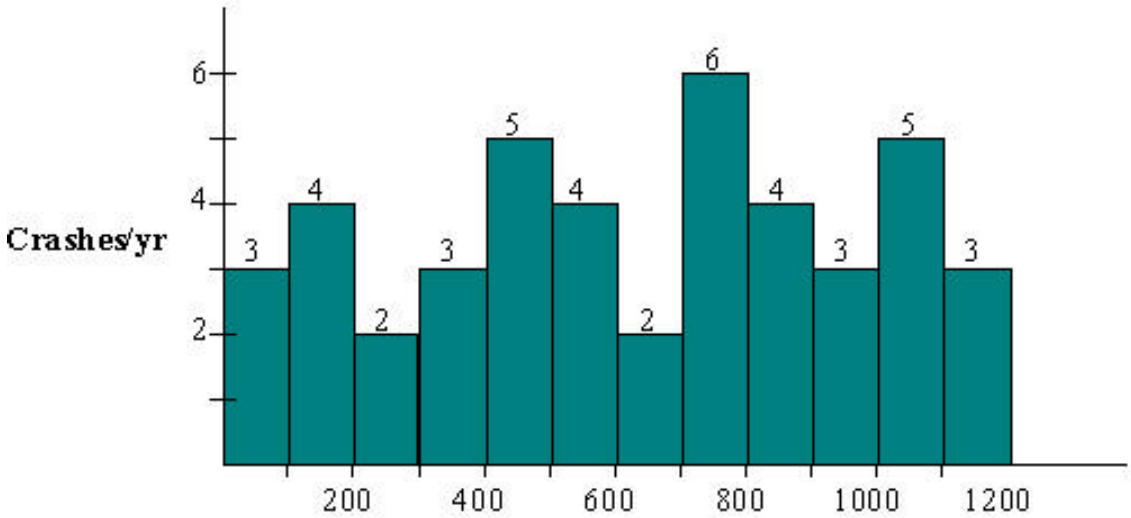


Figure 5 Plot of Predicted Crashes for Example Highway by Station

The first point is plotted at the midpoint of the first segment (W) and will be 250 ft. away from the beginning of the highway. The predicted crashes/yr are calculated as follows:

- @250 (midpoint of 0 to 500), plot average of 1st 5 intervals = (3+4+2+3+5)/5 = 3.4

Likewise, the next seven points are:

- @350 (midpoint of 100 to 600), plot average of intervals 2-6 = (4+2+3+5+4)/5 = 3.6
- @450 (midpoint of 200 to 700), plot average of intervals 3-7 = (2+3+5+4+2)/5 = 3.2
- @550 (midpoint of 300 to 800), plot average of intervals 4-8 = (3+5+4+2+6)/5 = 4.0
- @650 (midpoint of 400 to 900), plot average of intervals 5-9 = (5+4+2+6+4)/5 = 4.2
- @750 (midpoint of 500 to 1000), plot average of intervals 6-10 = (4+2+6+4+3)/5 = 3.8
- @850 (midpoint of 600 to 1100), plot average of intervals 7-11 = (2+6+4+3+5)/5 = 4.0
- @950 (midpoint of 700 to 1200), plot average of intervals 8-12 = (6+4+3+5+3)/5 = 4.2

These points are plotted in Figure Ex.3.2. The end points were not calculated since for the remaining points n < 5. Therefore, the average for the remaining points are found as follows:

- @50 (midpoint of -200 to 300), plot average of intervals 1-3 = (3+4+2)/3 = 3.0
- @150 (midpoint of -100 to 400), plot average of intervals 1-4 = (3+4+2+3)/4 = 3.0
- @1050 (midpoint of 800 to 1300), plot average of intervals 9-12 = (4+3+5+3)/4 = 3.75
- @1150 (midpoint of 900 to 1400), plot average of intervals 10-12 = (3+5+3)/3 = 3.67

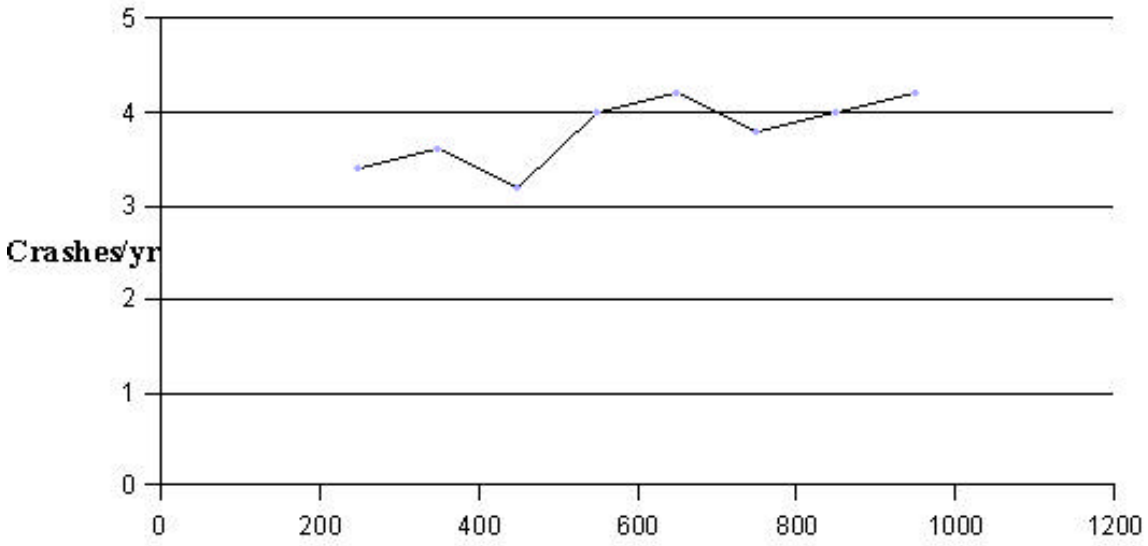


Figure 6 Example Highway with End Sections by Station Sliding Scale Plot (No Endpoints)

The complete sliding scale plot for this example is shown in Figure 7, *Example Highway with End Sections by Station Sliding Scale Plot (With Endpoints)*.

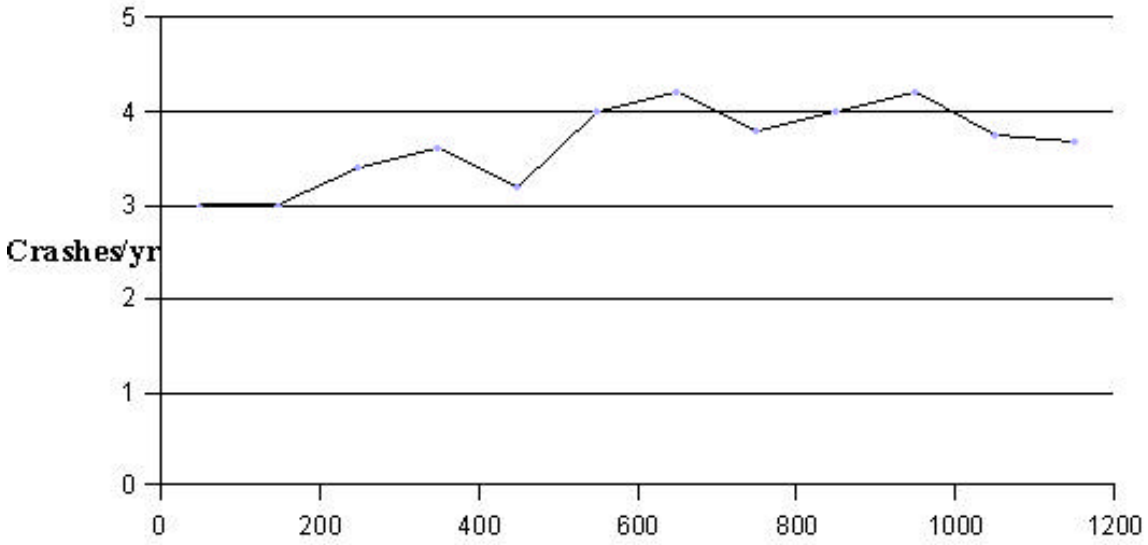


Figure 7 Example Highway with End Sections by Station Sliding Scale Plot (With Endpoints)

9. Default Tables

9.1 Default Percentage Distributions for Crash Severity Level and for Crash Type and Manner of Collision

Table 15. Default Percentage Distributions for Crash Severity Level and for Crash Type and Manner of Collision

Crash severity level	Highway segments[a]	Three-leg intersections[b] with minor-road STOP control	Four-leg intersections[b] with minor-road STOP control	Four-leg signalized intersections[b]
Fatal	1.3	1.1	1.9	0.4
Incapacitating Injury	5.4	5.0	6.3	4.1
Nonincapacitating injury	10.9	15.2	12.8	12.0
Possible injury	14.5	18.5	20.7	21.2
Total fatal plus injury	32.1	39.8	41.7	37.7
Property-damage-only	67.9	60.2	58.3	62.3
TOTAL	100.0	100.0	100.0	100.0

[a] Based on HSIS data for Illinois (1992), Michigan (1995), Minnesota (1996), and North Carolina (1995).

[b] Based on HSIS data for Michigan (1995) and Minnesota (1996).

9.2 Default Percentage Distributions for Crash Type and Manner of Collision on Rural Two-Lane Highways

Table 16. Default Percentage Distributions for Crash Type and Manner of Collision on Rural Two-Lane Highways

Crash type and manner of collision	Highway segments[a]	Three-leg intersections[b] with minor-road STOP control	Four-leg intersections[b] with minor-road STOP control	Four-leg signalized intersections[b]
SINGLE-VEHICLE CRASHES				
Collision with animal	30.9	2.1	0.6	0.3
Collision with bicycle	0.3	0.7	0.3	1.0
Collision with parked vehicle	0.7	0.1	0.1	0.1
Collision with pedestrian	0.5	0.4	0.2	1.3
Overturned	2.3	2.1	0.6	0.4
Ran off road	28.1	10.4	4.5	1.9
Other single-vehicle accident	3.6	3.9	1.4	1.6
Total single-vehicle accidents	66.3	19.7	7.7	6.6
MULTIPLE-VEHICLE CRASHES				
Angle collision	3.9	29.8	51.4	28.5
Head-on collision	1.9	2.0	1.4	1.8
Left-turn collision	4.2	6.4	5.9	9.0
Right-turn collision	0.6	0.4	0.2	0.4
Rear-end collision	13.9	26.2	17.2	36.2
Sideswipe opposite-direction collision	2.4	2.9	1.7	2.0
Sideswipe same-direction collision	2.6	4.5	4.4	5.5
Other multiple-vehicle collision	4.1	8.1	10.1	10.0
Total multiple-vehicle accidents	33.7	92.3	93.4	TOTAL CRASHES

[a] Based on HSIS data for Illinois (1992), Michigan (1995), Minnesota (1996), and North Carolina (1995).

[b] Based on HSIS data for Michigan (1995) and Minnesota (1996).

10. Glossary

After Period

Also called the analysis period, this is the period for which the crash frequency is being predicted. The minimum length of after-period is one year; there are no partial year predictions. The after period cannot overlap the before period.

AMF (Accident Modification Factor)

AMFs are multiplicative factors used to adjust the expected base crash frequency for the effect of individual geometric design and traffic control features. Each AMF is formulated so that the

nominal or base condition is represented by an AMF of 1.00. Conditions associated with higher crash experience than the nominal or base condition will have AMFs greater than 1.00 and conditions associated with lower crash experience than the nominal or base condition will have AMFs less than 1.00. The AMFs are applied in step 9 of the Crash Prediction Algorithm and are described in Section 6., *AMFs for Highway Segments* and Section 7., *AMFs for Intersections*.

Approach (Leg) Classification

Approach (leg) classification is an intersection geometric design variable with permitted values of major leg or minor leg. The values enumerate whether the leg being analyzed is the major or minor leg at a given intersection. The IHSDM automatically designates the legs of the same highway as major or minor based on the traffic control, volume, or simply being Base Highway or Intersecting Highway. The user has also the ability to overwrite these designations. These designations are used in calculation of the expected number of crashes at intersections.

Base Highway

An intersection definition is linked to a highway definition. The information about the same intersection could be accessed through the main HSDM interface or through the *Edit|View Highway Elements* menu item of any of the intersection highways involved in the information of that intersection. In the former case the Base Highway for the intersection would be the same as the Master Highway for the analysis. In the latter case the Base Highway for the intersection would be the intersecting highway through which the intersection is accessed.

Base Model

The base model for highway segments is the best available regression model for predicting the total crash frequency of a homogeneous highway segment on a rural two-lane highway. These models are available in the Report No. FHWA-RD-99-207 . The base model predicts the total expected crash frequency on the highway segment during a specified time period as a function of the highway segment's traffic volume, geometry, and traffic control. The specific regression model used by the CPM is presented in step 7 of the Crash Prediction Algorithm.

Separate base models have been developed for three-leg intersections with minor-road STOP control, four-leg intersections with minor-road STOP control, and four-leg signalized intersections. The specific regression models used by the CPM for intersections are presented in step 7 of the Crash Prediction Algorithm.

Before Period

The period for which historical crash data are applied to the model. As is the case with the after or analysis period, only full calendar years are permitted. The before period cannot overlap with the after period.

Calibration Factor

The calibration factors are applied to the base models in the crash prediction algorithm to allow highway agencies to tailor the safety prediction to their local conditions. By default all calibration factors are set to 1.00. The procedure for developing calibration factors is documented in Appendix C of Report No. FHWA-RD-99-207, Prediction of the Expected Safety Performance of Rural Two-Lane Highways.

Crash Segment

When historical crash data are stored by location ranges, crash segments matching the station ranges (one station range per crash segment) are created with the number of observed crashes

assigned to each segment. If the available data consist only of totals at the project level, then there will be only one crash segment.

Homogeneous Analysis Section

A homogeneous highway segment or intersection.

Homogeneous highway Segment

A homogeneous highway segment is a segment whose geometry, traffic volume, and traffic control are all constant throughout the entire length of the segment. The minimum length for a homogeneous highway segment is one meter. There is no maximum length for a homogeneous highway segment.

Intersection Skew Angle

The intersection skew angle is the deviation (in degrees) for the intersecting leg. The skew angle is reported for the left and right legs of the intersecting highway. The left/right orientation is relative to increasing station numbers on the master highway.

Figure 8, *CPM Intersection Skew* shows a four-legged intersection. The master highway is shown running vertically in the figure, the intersecting highway is shown running horizontally. The station numbers on the master highway increase when moving from the lower leg to the upper leg. The station numbers on the intersecting highway increase when moving from the left leg to the right leg.

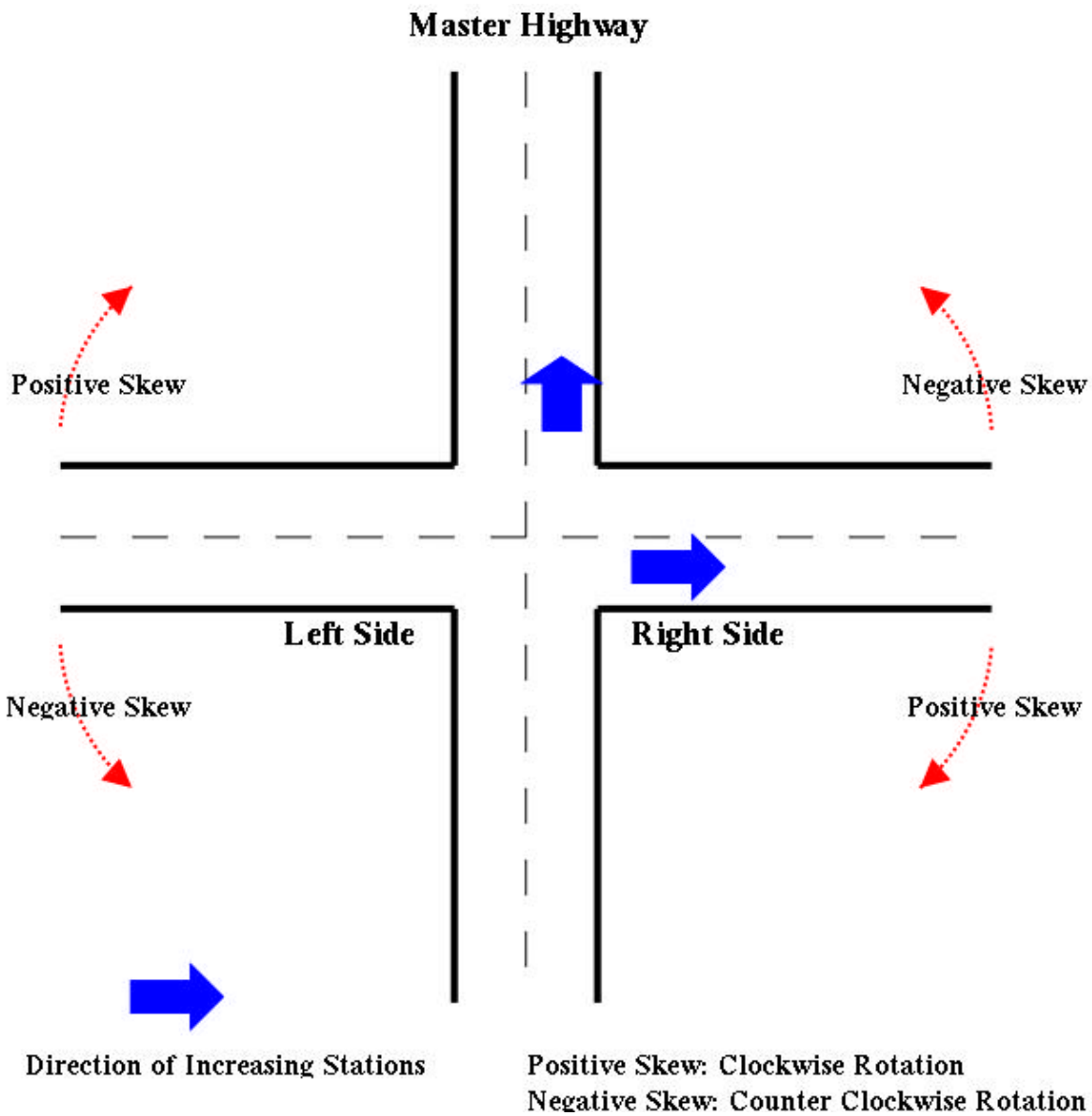


Figure 8 CPM Intersection Skew

The figure shows a intersection with zero degrees intersection skew on both the left and right intersecting highway legs. On the right side leg, as the orientation of the leg is moved counter clockwise (up), the skew angle decreases from zero (a negative value in degrees). Likewise, as the orientation of the right side leg is moved clockwise (down), the skew angle increases from zero (a positive angle in degrees). On the left side leg, as the orientation of the leg is moved clockwise (up), the skew angle increases from zero degrees (a postive value); as the orientation of the leg is moved counter clockwise (down), the skew angle decreases from zero degrees (a negative value).

Limited Intersection Sight Distance

Intersection sight distance in a quadrant is considered limited if the available sight distance is less than the sight distance specified by AASHTO policy for a design speed of 10 mph (16.093 km/h) less than the major leg design speed. For the purpose of the CPM this sight distance is calculated based on a required gap of 7.5 sec. This distance is measured along the major leg lane

from the intersection of the center of the minor leg lane and the center of the major leg lane beside the quadrant.

Major Leg

An intersection leg would be designated as a Major Leg if one of the following is true:

- The intersection traffic control designation is either **Stop** or **Yield** (i.e., minor leg STOP/YIELD-controlled) and the leg has no stop or yield sign.
- The traffic control is either **All-Way Stop**, or **None** or **Signal** and the leg has higher ADT than the other intersection highway.
- The traffic control is either **All-Way Stop**, or **None** or **Signal** and more than two legs have the same ADT (or the ADT data for some legs are missing) and the legs belon to the Master Highway at the intersection.

Master Highway

Master Highway for each analysis is the highway under analysis.

Minor Leg

An intersection leg would be designated as a Minor Leg if it is not designated as Major Leg.

Quadrant

Quadrant is a part of roadside that falls between adjacent legs of different intersecting highways. There are two quadrants at a three-leg intersection and four quadrants at a four-leg intersection. The limitation of the intersection sight distance for minor legs on minor-stop intersections created by each quadrant could be due to the vertical and horizontal curvature and are determined by the user.

Roadside Hazard Rating

A seven-point categorical scale from 1 (best) to 7 (worst) to characterize crash potential for roadside designs on two-lane highways. The Roadside Hazard Rating is defined in Appendix D of Report No. FHWA-RD-99-207, Prediction of the Expected Safety Performance of Rural Two-Lane Highways. For more information on roadside hazard rating, refer to Roadside Hazard Ratings used by IHSDM.

11. IHSDM Documentation

IHSDM documentation is organized in a series of manuals oriented to specific user types and information needs. User types include first-time users, regular users, and system administrators. Information needs include: installing and configuring IHSDM, the mechanics of using the various features of the software, engineering insights to ensure appropriate use of the software and interpretation of outputs, and administering and maintaining the software installation.

The structure of the series of manuals is illustrated in the User Documentation Map. The manuals are listed and described below by the users and information needs they support:

- Manuals for First-Time Users: These manuals are oriented to assist new users in installing and configuring IHSDM and running it for the first time. Manuals include:
  - Getting Started Guide - An overview of the installation and use of IHSDM. This Guide should be sufficient for stand-alone installations. For client-server installations, the more detailed IHSDM Installation Manual will be needed.

  - Installation Manual - A detailed reference to the installation and configuration of IHSDM.
  - Running IHSDM Software Manual - An overview of the basic operations in running the IHSDM software. The intent is to provide new users the information they need to run IHSDM for the first time.
- User's Manuals: These Manuals are intended as references that regular users can consult when issues arise about the mechanics of using the IHSDM graphical user interface. Manuals include:
  - IHSDM User's Manual - A reference for using the primary IHSDM graphical user interface. Other User's Manuals provide additional details on specific components of the IHSDM graphical user interface:
    - Policy Review Module (PRM) User's Manual - A reference for using the (stand-alone) Policy Review Module software graphical user interface.
    - Crash Prediction Module (CPM) User's Manual - A reference for using the (stand-alone) Crash Prediction Module software graphical user interface.
    - Design Consistency Module (DCM) User's Manual - A reference for using the (stand-alone) Design Consistency Module software graphical user interface.
    - Intersection Review Module (IRM) User's Manual - A reference for using the (stand-alone) Intersection Review Module software graphical user interface.
    - Traffic Analysis Module (TAM) User's Manual - A reference for using the (stand-alone) Traffic Analysis Module software graphical user interface.
    - Using the IHSDM Graphical User Interface - A reference for the operation of the individual components of the graphical user interface.
    - User Properties and Defaults Manual - A reference for editing IHSDM system properties, user properties, and user default values.
  - Frequently Asked Questions - A list of frequently asked questions related to the IHSDM software.
  - IHSDM Troubleshooting Guide - A reference for troubleshooting IHSDM software problems.
- Documentation of IHSDM Data: These documents provide detailed descriptions of all IHSDM data elements and references for importing and editing data.
  - IHSDM Highway Model - A reference for the IHSDM highway model, including descriptions of the data elements comprising the model.
  - LandXML Support - A reference for IHSDM support for the LandXML data standard.
  - Editing Highway Elements - A reference for using the Edit/View Highway Elements graphical user interface.
  - GEOPAK-TO-IHSDM Application Programmer's Interface (API) User's Manual - A reference for using the Application Program Interface (API) to export data from GEOPAK into a format that IHSDM can import.
- Engineer's Manual: The intent of these Manuals is to provide the engineering information necessary to make appropriate use of IHSDM evaluation capabilities and interpretation of results. Manuals include:

- Policy Review Module (PRM) Engineer's Manual - A reference for the engineering issues of using the Policy Review Module.
- Crash Prediction Module (CPM) Engineer's Manual - A reference for the engineering issues of using the Crash Prediction Module.
- Design Consistency Module (DCM) Engineer's Manual - A reference for the engineering issues of using the Design Consistency Module.
- Intersection Review Module (IRM) Engineer's Manual - A reference for the engineering issues of using the Intersection Review Module.
    - **Intersection Policy Review Sub-Manual** - Describes the procedures for checking an intersection design element against relevant policy, including references to the section of the AASHTO policy that contains the information used to develop the module and check the design. **(The Intersection Policy Review Sub-Manual is not available in the current release of IHSDM.)**
    - Intersection Diagnostic Review Engineer's Sub-manual - Describes in detail the concerns that the diagnostic review component considers and the models used to evaluate those concerns.
- Traffic Analysis Module (TAM) Engineer's Manual - A reference for the engineering issues of using the Traffic Analysis Module.

• Manuals for System Administrators: These Manuals provide system administrators the information they need to maintain IHSDM installations.

- System Administrator's Manual - A reference for using the IHSDM Administration Tool software graphical user interface. This manual also discusses customizing variable components of IHSDM, including analysis report templates, data dictionaries, and policy files.
- PRM/IRM Policy Table Maintenance - A reference for editing design policy tables used in the Policy Review Module and Intersection Review Module.

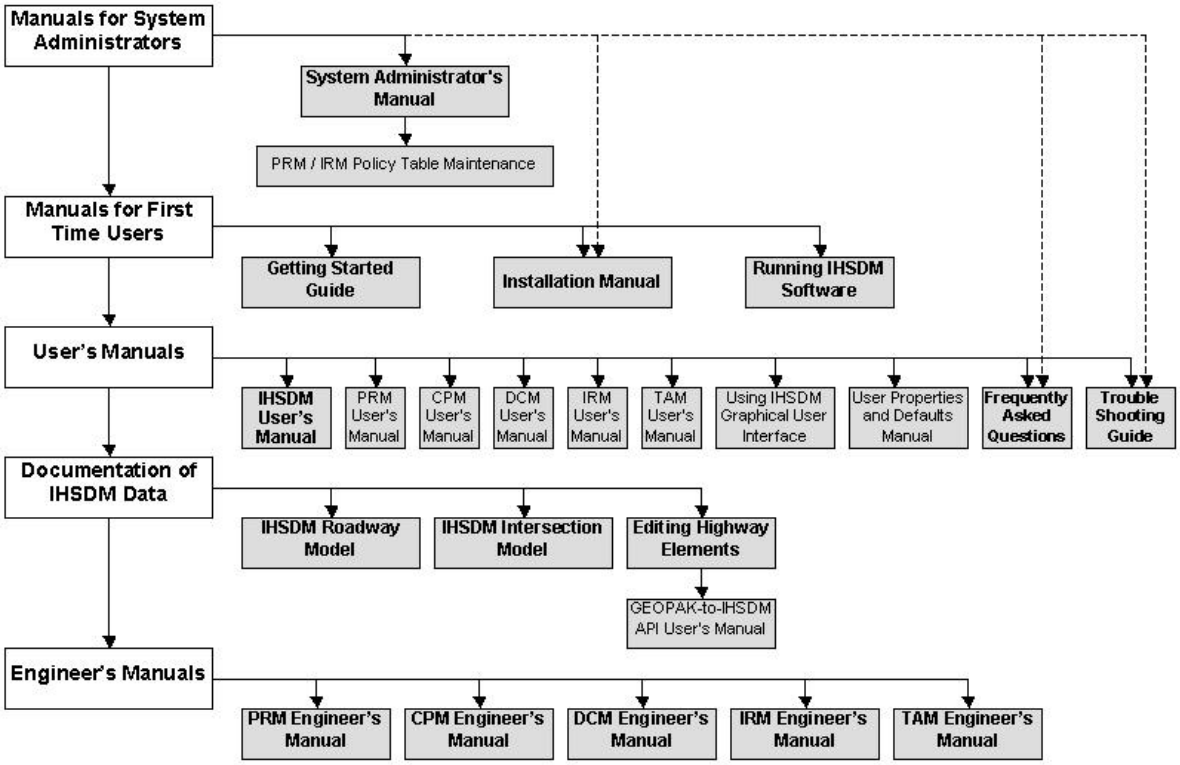


Figure 9 User Documentation Map

Index

A

accident modification factor
  See AMF

after
  period, 50

algorithm
  crash prediction, 17

AMF, 50
  highway segments, 7, 28
  intersections, 7, 38

analysis data, 4

analysis report, 14
  caution on microanalysis, 14
  expected crash frequencies and rates, 14, 15
  expected crash type distribution, 14
  historical crashes, 16
  homogeneous analysis section, 14
  plot of expected crash frequencies by intersection, 16
  plot of expected crash rates, 15, 15, 16
  plot of historical crashes, 16, 17

approach classification, 51

B

base highway, 51

base model, 51
  highway segments, 7
  intersections, 7

before
  period, 51

C

calibration factors, 8, 51

caution on microanalysis
  analysis report, 14

crash history data, 6, 10
  highway segment, 7
  intersection, 7
  stored by station, 40
  stored by station range, 43

crash prediction
  algorithm, 17
  procedural elements, 7
  site-specific crash history, 8

crash segment, 51

crash severity level on rural two-lane highways
  default tables, 49

crash type and manner of collision on rural two-lane highways
  default tables, 50

D

data input requirements, 4

default tables, 48
  crash severity level on rural two-lane highways, 49
  crash type and manner of collision on rural two-lane highways, 50

driveway density, 37

E

EB procedures, 9

expected crash frequencies and rates
  analysis report, 14, 15

expected crash type distribution
  analysis report, 14

G

geometric and traffic control data, 4

geometric data
  highway segment, 4
  intersection, 5

glossary, 50

grade, 32

H

highway data
  output, 11

highway segment
  crash history data, 7
  geometric data, 4
  output, 11
  traffic control data, 4
  traffic volume data, 6

highway segments
AMF, 7, 28
base model, 7

historical crashes
analysis report, 16

homogeneous analysis section, 52
analysis report, 14

homogeneous highway segment, 52

horizontal curve
length, 31
output data, 12
radius, 31
spiral transition, 31

I

intersecting highway traffic volume
output data, 13

intersection
crash history data, 7
geometric data, 5
left-turn lanes, 39
output data, 13
right-turn lanes, 39
sight distance, 40
skew angle, 38
traffic control, 39
traffic control data, 5
traffic volume data, 6

intersection skew angle, 52

intersections
AMF, 7, 38
base model, 7

L

lane width, 28

lanes
passing, 37
two-way left-turn, 38

left-turn lanes
intersection, 39

leg classification, 51

length
horizontal curve, 31

Limited Intersection Sight Distance, 53

M

major leg, 54

master highway, 54

minor leg, 54

O

output, 11
highway data, 11
highway segment, 11

output data
horizontal curve, 12
intersecting highway traffic volume, 13
intersection, 13
segment traffic volume, 13

overview, 1

P

passing
lanes, 37

period
after, 50
before, 51

plot of expected crash frequencies by intersection
analysis report, 16

plot of expected crash rates
analysis report, 15, 15, 16

plot of historical crashes
analysis report, 16, 17

procedural elements
crash prediction, 7

procedure
quickstart, 18
segmentation, 7

Q

quadrant, 54

quickstart
procedure, 18

R

radius
horizontal curve, 31

Report No. 99-207, 38

results
    site-specific crash history, 9

right-turn lanes
    intersection, 39

roadside hazard rating, 38, 54

S

segment traffic volume
    output data, 13

segmentation
    procedure, 7

shoulder type, 29

shoulder width, 29

sight distance
    intersection, 40

site-specific crash history
    crash prediction, 8
    results, 9

skew angle
    intersection, 38

sliding scale analysis, 46

spiral transition
    horizontal curve, 31

stored by station
    crash history data, 40

stored by station range
    crash history data, 43

superelevation, 31

T

traffic control
    intersection, 39

traffic control data
    highway segment, 4
    intersection, 5

traffic volume data, 6
    highway segment, 6
    intersection, 6

two-way left-turn
    lanes, 38