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16. Abstract

Highway safety is an ongoing concern to the Texas Department of Transportation (TxDOT). As part of its proactive commitment to improving highway safety, TxDOT is moving toward including quantitative safety analyses earlier in the project development process. The objectives of this research project are: (1) the development of safety design guidelines and evaluation tools to be used by TxDOT designers, and (2) the production of a plan for the incorporation of these guidelines and tools in the planning and design stages of the project development process.

This document provides the best-available information describing the relationship between various highway geometric design components and crash frequency. It is intended to be used by engineers for the purpose of explicitly evaluating the potential safety trade-offs associated with various design alternatives. This document focuses on quantitative safety relationships for specific design components known to be correlated with crash frequency. It is intended for engineers responsible for the geometric design of streets and highways.

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ROADWAY SAFETY DESIGN WORKBOOK

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DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data published herein. The contents do not necessarily reflect the official view or policies of the Federal Highway Administration (FHWA) and/or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation. It is not intended for construction, bidding, or permit purposes. The engineer in charge of the project was James Bonneson, P.E. #67178.

NOTICE

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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- Ms. Aurora (Rory) Meza, Project Coordinator (TxDOT, Design Division);
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- Mr. David Bartz (FHWA);
- Mr. Stan Hall (TxDOT, Dallas District);
- Mr. Richard Harper (TxDOT, Paris District);
- Ms. Meg Moore (TxDOT, Traffic Operations Division);
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- Ms. Wendy Simmons (TxDOT, Tyler District);
- Ms. Joanne Wright (TxDOT, Office of the General Council); and
- Mr. Wade Odell, Research Engineer (TxDOT, Research and Technology Implementation Office).

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Introduction

PREFACE

The *Roadway Safety Design Workbook* provides information about the relationship between roadway geometric design and safety. It is based on a synthesis of current research that quantifies the correlation between various design elements (e.g., lane width) or design components (e.g., left-turn bay) and expected crash frequency. The information provided in the *Workbook* is intended to help designers make informed judgments about the benefits and costs of design alternatives.

The *Workbook* does not define design controls and does not represent a design requirement. It is not a substitute for engineering judgment. Further, it does not represent a legal requirement for roadway design. Any decision to not use the *Workbook*, or the information obtained from its use, is not evidence of negligence on the part of any person or organization.

Knowledge about the relationship between roadway design and safety is continually evolving. As additional information becomes available through experience, research, and/or in-service evaluation, this *Workbook* will be updated. However, the fact that it has been updated does not imply that existing facilities are unsafe. Nor should the publication of updated *Workbook* content be construed to imply the need for improvement to existing roadways. Rather, the implementation of the updated information should occur as projects are built, or rebuilt, in conjunction with the annual project programming process.

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OVERVIEW

Highway safety concerns are also evident in Texas. Crashes in Texas continue to increase and currently exceed 300,000 per year. Nearly 3800 motorists die annually on Texas highways. Public demand for safer streets and highways continues to grow. In response to this demand, state and national transportation agencies have developed safety programs that emphasize public education, accelerated highway renewal, community sensitive street systems, and innovative technology to facilitate safe highway design.

The objective of the *Roadway Safety Design Workbook* is to provide the best-available information describing the relationship between various geometric design components and highway safety. The *Workbook* is intended for use by engineers for the purpose of explicitly evaluating the relationship between various design alternatives and crash frequency. To this end, the *Workbook* focuses on the presentation of quantitative safety relationships for specific design components known to be directly correlated with crash frequency. The *Workbook* is intended for engineers responsible for the geometric design of streets and highways.

It is envisioned that the *Workbook* will be used throughout the design process. However, the insights provided through use of the safety relationships in this document will be most helpful in situations where the choice among design elements is not obvious or the trade-offs are not readily apparent (e.g., where atypical conditions exist, the design is complex, or construction costs are high). In this manner, the *Workbook* guidance can facilitate the thoughtful and balanced consideration of both safety and operational benefits as well as the costs associated with construction, maintenance, and environmental impacts.

The content of this document was derived from a synthesis of safety information in the literature and from original research. The findings from the synthesis are documented in the *Roadway Safety Design Synthesis* (*1*). The results of the original research are provided in a series of research reports (*2*, *3*, *4*). Users of the *Workbook* are encouraged to consult these documents if additional information is desired about the relationships in this *Workbook*.

Guidelines for using the information in this *Workbook* are provided in a companion document, *Procedure for Using Accident Modification Factors in the Highway Design Process* (*4*). The guidelines describe how the models in the *Workbook* can be used to evaluate the safety associated with a given highway or intersection.

The safety relationships in this document are derived from research conducted throughout the United States, including Texas. All of them were screened for applicability to Texas conditions. Most of the relationships were either compared to Texas crash data to confirm the stated trends or calibrated to Texas conditions.

At this time, quantitative safety relationships are not available for every element of roadway design. The reader is referred to the *Highway Safety Design and Operations Guide* (*5*) for a qualitative discussion of safety considerations associated with the various design-related factors for which quantitative information is not available herein.

Research is presently underway at the state and national levels. It will produce significant new information about the relationship between design components and safety. It is envisioned that this *Workbook* will be periodically updated to incorporate the findings and recommendations of this new research.

ROLE OF SAFETY IN THE DESIGN PROCESS

The project development process takes the design project from concept to letting. This process consists of six stages: planning and programming; preliminary design; environmental; right-of-way and utilities; plans, specification, and estimates (PS&E) development; and letting. The planning and programming, preliminary design, and PS&E development stages are stages where safety can be readily added to the design process. The sequence of these stages in the development process is shown in Figure 1-1.

As indicated by Figure 1-1, evaluation tools (like those provided in this *Workbook*) are used by the designer to verify the performance potential of alternative designs. The evaluation quantifies the design's performance in terms of safety, operations, construction cost, etc. The objective of this evaluation is to ensure that the design offers a reasonable balance between cost and effectiveness.

Table 1-1 identifies safety tasks that can be undertaken in the project development process. The steps shown in the table generally correspond to the tasks described in the *Project Development Process Manual* (*6*).

As indicated in Table 1-1, "key" design elements are identified in Step 4 of the preliminary design stage and then used to direct the safety evaluation tasks. Key design elements are those elements that: (1) are associated with the "controlling criteria" that dictate the need for a design exception or have a known effect on safety, and (2) are used in situations where atypical conditions exist, the design is complex, or construction costs are high. The controlling criteria vary by project type; those applicable to Rehabilitation Projects (3R) include:

- design speed,
- lane width,
- shoulder width,
- bridge width,
- structural capacity,
- horizontal alignment,
- vertical alignment,
- grade,
- superelevation, and
- deficient bridge rails.

The controlling criteria for New Location and Reconstruction Projects (4R) include:

- design speed,
- lane width,
- shoulder width,
- bridge width,
- structural capacity,
- horizontal alignment,
- vertical alignment,
- grade,
- stopping sight distance,
- cross slope,
- superelevation, and
- vertical clearance.

Additional design elements that may also be considered as "key" because of their known relationship with safety include: turn bays at intersections, median treatment, and horizontal clearance. For non-key design elements, the traditional design process (i.e., compliance with design criteria and warrants) will likely provide an acceptable level of safety.

Implementation of the safety-related tasks in Table 1-1 will add time to the design process. However, by limiting the evaluation of safety to "key" design elements, the additional time required should be kept to a minimum and incurred only where it is likely to provide some return in terms of improved safety, lower construction cost, or both. This added time represents an immediate and direct cost to the design process. However, it also represents a more cost-effective approach to design because additional benefit should be derived through fewer crashes and lower construction costs (by not over-designing some design elements).

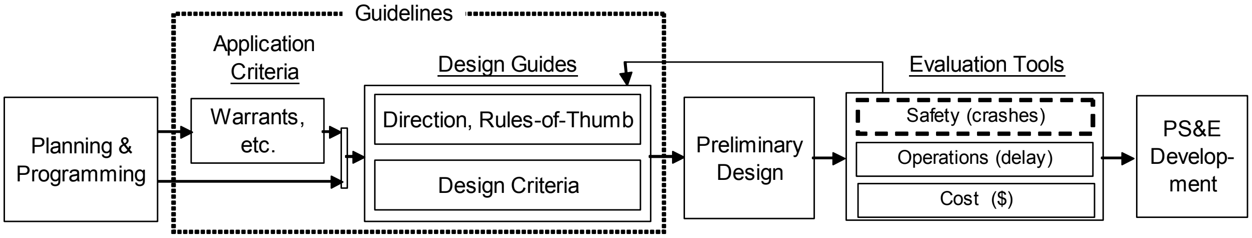


Figure 1-1. Components of the Project Development Process.

Table 1-1. Potential Safety Tasks in the Project Development Process.

Stage	Step	Potential Safety-Related Task
Planning and programming	Needs identification	• Screen facilities for locations with safety needs.
Preliminary design	Preliminary design conference	• Document safety needs. • Identify atypical conditions, complex elements, and high-cost components.
	Data collection/preliminary design preparation	• Diagnose safety data to identify crash patterns. • Refine project scope if necessary.
	Preliminary schematic	• Perform preliminary level of safety analysis for “key” design elements.[1]
	Geometric schematic	• Perform detailed level of safety analysis for “key” design elements.[1]
	Value engineering	• Compare cost of specific elements and overall roadway with safety and operational benefits.
	Geometric schematic approval	• Document safety of design choices (use results for design exception request, if necessary).
PS&E development	Final alignments/profiles	• Re-evaluate alignment, cross section, and roadside design to ensure acceptable level of safety.
	Traffic control plan	• Evaluate safety of long-term detour roadway design.

Note:
1 - Key design elements are those elements that: (1) are associated with the controlling criteria specified for the project or have a known effect on safety, and (2) are used in situations where atypical conditions exist, the design is complex, or construction costs are high.

ORGANIZATION

This *Workbook* provides quantitative information that can be used to evaluate the level of safety associated with various design alternatives. The *Workbook* chapters address the following facility types:

- freeways,
- rural highways,
- urban and suburban arterials,
- interchange ramps and frontage roads,
- rural intersections, and
- urban intersections.

Each chapter contains two main sections. The first section describes *base models* that can be used to predict the expected injury (plus fatal) crash frequency for a roadway segment, ramp, or intersection. These models have been calibrated using data representing Texas streets, highways, and intersections. The process used to calibrate these models is described in references *2* and *3*.

The procedure includes a technique for combining the reported crash count with the base model estimate to obtain a more reliable estimate of the expected crash frequency. This technique can be used if: (1) two or more years of crash data are available for the subject project, and (2) the project is not undergoing a fundamental change in character (e.g., change in area type, traffic control, number of lanes, or number of intersection legs).

In some instances, the nature of the alternatives analysis requires an estimate of the expected crash frequency for an alternative that would require a fundamental change in the project's character. In this situation, the expected crash frequency should be estimated using the base model for *all* alternatives, as well as the existing facility. This technique is necessary to ensure an equitable assessment of the incremental safety benefit of each alternative.

The second section of each chapter contains *accident modification factors* (AMFs) for various design-related factors that have been found to have some correlation with crash frequency. The AMFs in each chapter have been evaluated for their applicability to Texas streets and highways. They represent the current best knowledge regarding their relationship to crash frequency. The source of these AMFs is discussed in various documents (*1*, *2*, *3*, *4*).

An AMF indicates the relative change that occurs in crash frequency when a particular geometric design component is added or removed, or when a design element is changed in size. More precisely, an AMF represents the ratio of crashes during the "after" period to crashes during the "before" period. It typically ranges in value from 0.5 to 2.0, with a value of 1.0 representing no effect of the design change. AMFs less than 1.0 indicate that the design change is associated with fewer crashes.

All of the AMFs described in this section yield a value of 1.0 when the associated design component or element represents base conditions. A table is provided in each chapter to identify the applicable base conditions.

The AMFs and crash rates in this document are derived from research conducted throughout the United States, including Texas. All of the research findings were screened for applicability to Texas conditions. Several AMFs require the distribution of crashes (by crash type, number of lanes, or median type) as an input. The distributions tabulated herein for these AMFs were obtained from the crash database maintained by the Texas Department of Transportation.

CRASH DATA VARIABILITY

This part of the chapter examines variability in crash data. The discussion is presented in four sections. The first section focuses on how the random nature of crash data can cloud the interpretation of trends in crash frequency for a highway segment or intersection. The second section discusses the influence of design components or design element sizes on crash frequency and addresses how the variability in crash data can mask the detection of this influence. The third section discusses the precision of design-related AMFs. The last section summarizes the main points of the preceding three sections.

Crash Frequency at One Site

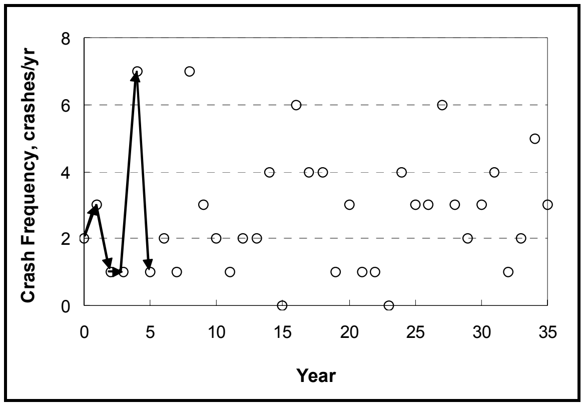
This section examines the variability in crash frequency on a street segment or at an intersection (hereafter, referred to as a "site"). It also explores how long-run averages can be used to reveal the underlying mean crash frequency at a site.

Variability in Crash Frequency

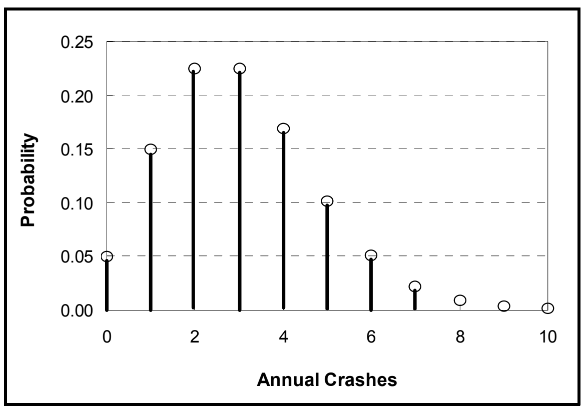
On a year-to-year basis, crash data typically exhibit a large variability in crash count. Figure 1-2a illustrates the pattern of crashes at a hypothetical site for a 35-year period, during which 98 crashes occurred. Traffic growth was negligible at this site, and its geometry did not change substantially over the 35 years. Figure 1-2b illustrates the distribution of crashes at this site. It indicates that the chance of three crashes occurring in a given year is 22 in 100; the chance of seven crashes is 2 in 100.

Figure 1-2a indicates that crash frequency ranged from 0 to 7 crashes in a given year at the site. Two crashes occurred in year 0 and three crashes occurred in year 1. Recall that this increase is due only to random events because traffic and geometry conditions did not change in a significant manner. In years 2 and 3, only one crash occurred. By the end of year 3, the agency responsible for this site would likely (incorrectly) assume at this point that the mean crash frequency at this site was less than 2.0 crashes/yr.

In year 4, seven crashes occurred—a 700 percent increase from the previous year. Most agencies would likely assume that safety at this site had deteriorated and that some type of improvement was justified. Of course, this action would be unjustified because the increase in crashes was due only to the random variation of crashes.



a. Yearly Crash Frequency Trend.



b. Crash Frequency Distribution.

Figure 1-2. Crash Frequency Time Trend at a Site.

In year 5, the crash count drops back down to a more typical level of one crash (an 84 percent decrease). If the agency had implemented an improvement in response to the unexpected seven crashes in year 4, they would likely (incorrectly) infer that the improvement at the start of year 5 was responsible for the 84 percent reduction in crashes. In fact, the return to one crash in year 5 would be solely due to the phenomena of "regression-to-the-mean." This phenomenon occurs because of the tendency of sites that have an exceptionally high crash count in one year to return to a lower crash count (i.e., one nearer the true mean) the following year.

The regression-to-the-mean phenomenon has implications on agency policy for "hazardous" site selection. Many agencies identify hazardous sites based on an examination of the reported crash count, as averaged over the last few years. However, this policy can lead to unnecessary design changes at some sites because of regression-to-the-mean. The relatively large number of crashes that may have occurred in the last few years at some sites may be solely due to random variation.

If not controlled, regression-to-the-mean will bias the findings of the before-after study by yielding an AMF that is overly optimistic about the effect of a design change on crash frequency. Techniques for identifying sites that would truly benefit from treatment and for evaluating treatment effectiveness are described in the safety literature (*7*, *8*).

Variability in Mean Crash Frequency

The underlying trend in the crash pattern at the hypothetical site in Figure 1-2 can be examined by taking a "running" average over time. In this examination, the running average for year 0 represents the reported crash count in that year. The running average for year 1 represents an average of the reported crash count for years 0 and 1. The running average for year 2 represents the average of reported counts for years 0, 1, and 2. This process repeats until the running average for year 35 represents the average of all years of data. The resulting running average is shown as a thick bold line in Figure 1-3.

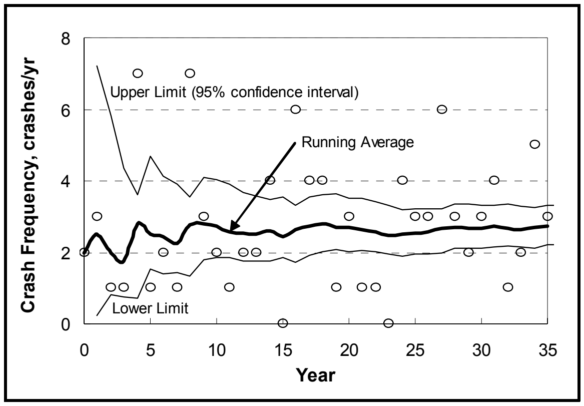


Figure 1-3. Running Average Crash Frequency.

The running average shown in Figure 1-3 varies widely for the first few years, gradually becoming more stable with an increase in the number of years over which the average is taken. The average of 35 years of data yields an average of 2.8 crashes/yr (= 98/35). Hereafter, the long-run average is more correctly referred to as the "expected crash frequency."

The 95 percent confidence interval of the expected crash frequency is also shown in Figure 1-3 using the thin trend lines. These confidence intervals were computed using a statistical technique developed by Nicholson (*9*). The confidence limits in Figure 1-3 indicate that, even with a foundation of 98 crashes, the 95 percent confidence interval for the true mean is about 20 percent of the expected crash frequency (i.e., the true mean is between 2.2 and 3.3 crashes/yr).

Averages and confidence intervals are the only tools available to engineers for evaluating crash trends. However, on a site-by-site basis, these statistics are not very telling because of the limited number of crashes that typically occur at a site and the relatively few years for which one can reasonably assume that traffic and geometry conditions do not change substantially.

Technique for Uncovering the True Mean

In the previous subsection, it was determined that an average crash frequency estimate based on 98 crashes had a 95 percent confidence interval of ±20 percent. To narrow this interval (i.e., reduce the percentage), the long-run average will need to be based on a larger number of reported crashes. Sample size analysis yields the relationship between crash frequency and confidence interval limit percentage shown in Table 1-2.

Table 1-2. Percentages for Average Crash Frequency Confidence Interval.

Total Crash Count	Limit Percent [1, 2]
10	62
20	44
50	28
100	20
200	14
500	8.8
1000	6.2
2000	4.4
10,000	2.0

Notes:
1 - Percentages correspond to a 95 percent confidence interval.
2 - N_{upper} = N × (1 + *Limit Percent*/100);
N_{lower} = N × (1 - *Limit Percent*/100); and
N = average crash frequency.

The percentages listed in Table 1-2 indicate that 500 crashes are needed to estimate the confidence interval for the true mean crash frequency as ±8.8 percent of the long-run average. To reduce this interval by one half (i.e., to ±4.4 percent), a total of 2000 crashes would need to be represented in the average. These crash totals exceed the crash frequency of any given site. However, they may be obtainable by aggregating the crash data for a group of "similar" sites and accepting that the group average is representative of any one site in the group. For this application, "similar" sites are defined to have traffic volume, traffic control, and geometric conditions that are very nearly the same at each site.

The insight to be taken from this exploration of confidence intervals is that the variability in crash data is so large that efforts to use the average crash frequency for a given site will not likely reveal telling information about the true mean crash frequency at that site. Obtaining a reasonably small confidence interval for a site's true mean crash frequency requires an average based on a very large number of crashes, more so than are likely to be reported at most sites during a reasonable time period (say, three to five years). The aggregation of crash data for similar sites provides a more practical method for obtaining a reasonably precise estimate of a site's true mean crash frequency.

Influence of Design Features

This section addresses the issue of whether the correlation between geometric design features and crash frequency can be detected in an examination of crash data. The first subsection examines the challenges faced when trying to quantify the change in crash frequency that occurs at a site following a change in its design (e.g., add a turn bay). The second subsection examines the challenges faced when trying to explain the variation in crash frequency that occurs between sites as a function of differences in design element size (e.g., lane width). The correlation between crash frequency variation and design element size is believed to reflect the influence of element size on crash risk. Hence, hereafter, this correlation is referred to as "influence."

Influence of a Change in Design

This subsection discusses the effect of crash frequency variability on the examination of trends in crash data as a result of a change in design. For this examination, engineers may compare the crash frequency before and after a specific change in site design. This analysis technique is commonly referred to as a "before-after" study. Its application to safety evaluation is described by Hauer (*8*).

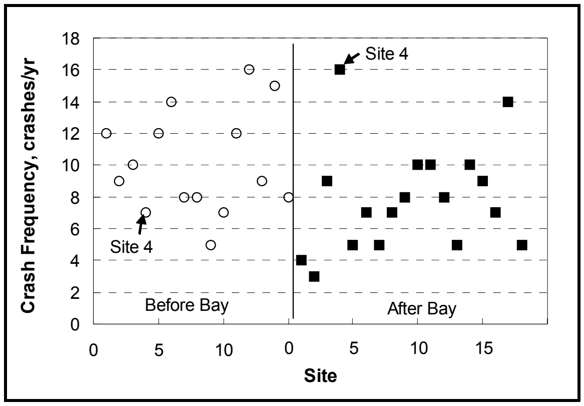
Challenges to Detection. It is generally recognized that some design components are used at a site because of their direct influence on safety. For example, the addition of a left-turn bay at a rural signalized intersection has been found to reduce the expected crash frequency by about 20 percent (*10*). Thus, an intersection with an expected crash frequency of 10 crashes/yr should have an expected crash frequency of 8 crashes/yr after the addition of a left-turn bay. However, the variability in crash frequency at this site may make it difficult to detect this reduction if only a few years of crash data are examined. This point is illustrated in Figure 1-4a.

Figure 1-4a shows the distribution of crashes one year before and one year after bay installation at each of 15 intersection sites, each with an expected crash frequency of 10 crashes/yr before treatment. The open circles indicate the reported crash count during the "before" period and the solid squares indicate the reported crash count during the "after" period.

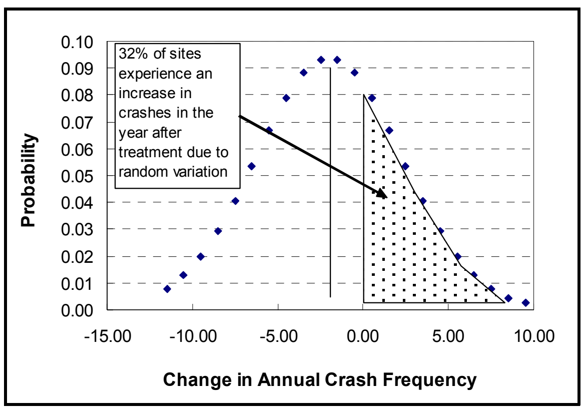
The data in Figure 1-4a indicate that there is a trend toward a decrease in crash frequency at the collective set of sites in the "after" period. However, random variation in the number of crashes makes the trend difficult to see at a given site. In fact, the average reduction of 2 crashes/yr (= 10 − 8) is small, relative to the variability in the crash data. The implication of this variability is that, in the year following the bay addition, the reported crash frequency can actually *increase* at some sites (even though the mean crash frequency has been reduced at all sites). In fact, Site 4 (and with closer inspection, three other sites shown in Figure 1-4a) realized an increase in crashes the year after the bay was added.

At first glance, an increase in the reported crash count the year following the implementation of a safety improvement would seem to be illogical and suggest that the bay did not yield its "advertised" safety benefit. Yet, the site's mean crash frequency *is* reduced as a result of bay addition. The number of reported crashes in the year after bay addition increased because of random variation in crash occurrence.

The potential for the aforementioned illogical trend to occur is shown in Figure 1-4b. This figure shows the distribution of "crash change" (i.e., reported crash frequency after change minus crash frequency before change). The distribution is centered on the average crash change of −2.0 crashes/yr. However, there is a portion of the distribution that lies to the right of the "0.0 crashes/yr" value. This portion (shown as a shaded triangular shape) equates to 32 percent of the distribution. It implies that there is a 32 percent chance that, in a given year, a site will show an increase in crashes following implementation of a design change that yields a 2.0 crash/yr reduction in mean crash frequency.



a. Before and after Crash Frequency.



b. Sites with Increase in Following Year.

Figure 1-4. Distribution of Crashes before and after a Change in Design.

In general, the variability in crash data can make it difficult to detect a change in crash frequency due to the implementation of a change in design at a single site. In fact, crash frequency in the year or two after a design change may increase when a decrease was expected. An examination of crash frequency in a group of sites is more likely to yield a definitive indication of the influence of the design change.

Detecting Influence. The before-after study is the most appropriate technique for quantifying the influence of design change on safety. The findings from the before-after study are used to estimate an AMF that describes the observed relationship between the design change and crash frequency.

Hauer (*8*) developed an equation that can be used to compute the minimum crash count needed to determine if a design change has a detectable influence on safety. The use of this equation requires a preliminary estimate of the AMF that the analyst expects to detect. The computed minimum crash count represents the total number of crashes reported in the period before the design change.

Table 1-3 lists the minimum crash count, as obtained from Hauer's equation. The first nine rows list the crash count needed to detect a reduction in mean crash frequency for a specific AMF. The last nine rows list the minimum crash count needed to detect an increase in mean crash frequency.

To illustrate the use of Table 1-3, consider a site selected for a change in design. This change is believed to be associated with about a 10 percent reduction in crashes. Thus, the preliminary estimate of the AMF is 0.9 (= 1.0 – 10/100). Table 1-3 indicates that the site would have to be associated with at least 514 crashes in the "before" years to detect a change in crash frequency corresponding to an AMF of 0.9. As noted in the discussion associated with Table 1-2, the only viable means of obtaining a sample of 514 crashes is to pool the crash data from several similar sites (all of which would undergo the same design change).

Table 1-3. Minimum Crash Count to Detect the Influence of a Change in Geometry.

AMF	Minimum Crash Count before Change [1]
0.1	4
0.2	5
0.3	7
0.4	11
0.5	16
0.6	27
0.7	51
0.8	122
0.9	514
1.1	568
1.2	149
1.3	69
1.4	41
1.5	27
1.6	20
1.7	15
1.8	12
1.9	10

Note:
1 - Crash counts correspond to a 95 percent level of confidence that a change occurred. The time duration for the "before" and "after" periods is the same. Multiply the crash count by 4.0 to obtain a 95 percent level of confidence in detecting a change equal in magnitude to the AMF listed.

The crash counts listed in Table 1-3 represent the minimum number of crashes needed to determine if a change in geometry has resulted in a change in the mean crash frequency (with 95 percent level of confidence). The crash frequencies listed in Table 1-3 would have to be increased by a factor of about 4.0 to obtain a 95 percent level of confidence in detecting a change equal in magnitude to the AMF listed (*11*). Thus, at least 2056 crashes (= 4.0 × 514) are needed to be reasonably sure that the true mean AMF is 0.90 or less.

The insight to take from this discussion is that crash variability is so large as to make it difficult to detect a change in crash frequency at one site due to a change in geometry. In fact, detection of a change at only one site is likely impossible when the geometric feature being considered has a relatively small influence on crash frequency. This challenge emerges because of the difficulty of finding a site that has enough crashes to detect the influence of a change in design. Changes in geometry that tend to have a subtle influence on crash frequency can only be evaluated using data for several years from many sites.

Influence of Different Design Element Sizes

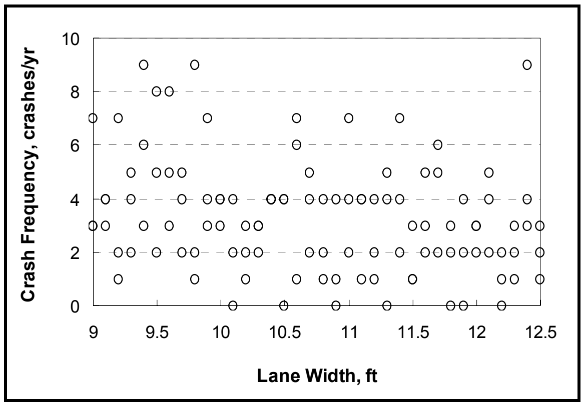
This subsection discusses the effect of crash count variability on the examination of trends in crash data from several sites that may differ in the size of one or more design elements (e.g., lane width). For this examination, engineers may compare the crash frequency of several sites that collectively have a range of sizes for specified geometric elements. A common analysis technique is the "cross section" study. It uses a regression model to quantify the influence of different design element sizes and to control for differences in traffic volume or segment length. A before-after study can also be used to quantify the effect of specific changes in design element size; however, it can be fairly expensive if used to develop AMFs for a range of sizes.

Challenges to Detection. In contrast to the before-after study, a cross section study does not have as strict a requirement for site similarity. Nominal differences in geometry or traffic volume are controlled by including variables in the regression model. Nevertheless, some similarity among the group of sites is important to minimize influences that are not of interest to the analyst. By algebraic manipulation of the calibrated regression model, an AMF can be derived that is inferred to characterize the influence of the geometric feature of interest.

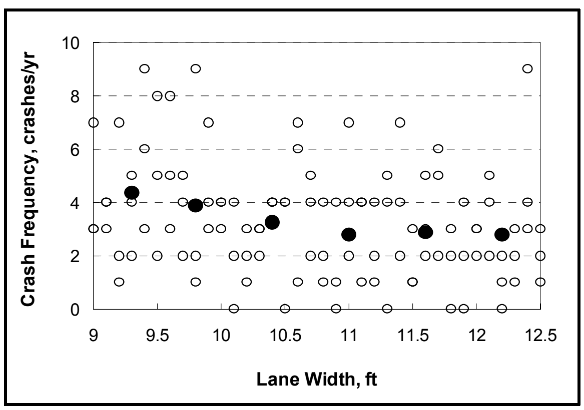
The following example illustrates the manner in which crash variability can obscure an assessment of the influence of a specific design element.

Consider an examination of the influence of lane width on crash frequency. Thirty-six sites are selected that have different lane widths. One of the 36 sites has a lane width of 9.0 ft, a second site has a lane width of 9.1 ft, a third site has a lane width of 9.2 ft, etc., with the last site having a lane width of 12.5 ft. This use of sites with unique lane widths is intended to facilitate the display of data in forthcoming plots–it is recognized that this approach does not reflect the actual distribution of lane widths among sites.

Three years of crash data are acquired for each site. They are plotted in Figure 1-5a. The trends in the data are highly variable and reflect the random nature of crashes at each site. If an engineer were asked to examine Figure 1-5a, he or she would not likely have any confidence that lane width is correlated with crash frequency.



a. Crash Frequency at 36 Sites.



b. Average Crash Frequency for Specific Lane Width Ranges.

Figure 1-5. Relationship between Lane Width and Crash Frequency.

Continuing with the example, consider that the data in Figure 1-5a are grouped into ranges of sites with nearly similar lane widths. The average crash frequency is then computed for each group. Specifically, assume that the crash data for the sites with lane widths of 9.0, 9.1, 9.2, 9.3, and 9.4 ft were averaged; the data for the sites with lane widths of 9.5, 9.6, 9.7, 9.8, and 9.9 ft were averaged; etc. The resulting averages are shown using large black dots in Figure 1-5b. Each dot represents the average of 15 site-years of data. The trend in these large dots now reveals that a relationship between lane width and crash frequency does exist. It suggests that the expected crash frequency is higher at locations with narrow lanes.

In spite of the trend demonstrated by the large dots in Figure 1-5b, the variability in the individual data points (i.e., the open circles) indicates that many sites with narrow lanes have *fewer* crashes than those with wide lanes. As such, the engineers that operate these sites may have difficultly accepting the trend shown by the large black dots because they may not be able to see it in the crash data for any one site. In general, this trend is most easily detected using data for a large number of sites. In other words, it is only "observable" on a regional or state level–an area that only engineers responsible for safety on a regional or state level would likely detect in their work.

Detecting Influence. Unlike the before-after study, the minimum crash count needed for regression analysis is not as well defined. This limitation is partly a consequence of the uncertain variability introduced by correlation among regression model variables. Nevertheless, some preliminary work in this area indicates that the minimum total crash count needed to detect the influence of various design elements is a function of the number of variables in the regression model and the average crash frequency at each site. Table 1-4 provides an estimate of the minimum crash count needed for regression analysis.

The minimum crash counts listed in Table 1-4 are partially dependent on the similarity of the sites used for the regression analysis. The counts listed are applicable to databases within which the sites are reasonably similar. If the sites in the database are less similar (i.e., they require more model variables to explain their differences), then the minimum total crash count needed to obtain the desired confidence interval will increase.

Table 1-4. Minimum Crash Count to Detect the Influence of Different Design Element Sizes.

Model Variables		Average Crash Frequency Per Site [1]									
		1	2	3	4	5	6	7	8	9	10
3	Minimum total crash count:	150	175	200	200	225	250	250	250	275	275
	Minimum number of sites:	150	88	67	50	45	42	36	31	31	28
8	Minimum total crash count:	525	675	825	950	1100	1200	1350	1500	1600	1700
	Minimum number of sites:	525	338	275	238	220	200	193	188	178	170

Notes:
1 - Average crash frequency per site equals the sum of crashes for one or more years at all sites divided by the number of sites.

To illustrate the use of Table 1-4, consider that an engineer desires to develop a regression model for rural frontage road segments. The average rural frontage road segment is determined to experience about two crashes each year. If the segments are reasonably similar such that they only differ in terms of their traffic volume and lane width, then the model would need only three model variables (i.e., intercept, traffic volume coefficient, and lane width coefficient). Table 1-4 indicates that this combination requires assembly of a database with 88 frontage road segments (i.e., sites) to quantify the effect of volume and lane width. If the segments are less similar such that eight model variables are needed to explain site differences, then the database would need 338 sites.

The large crash counts listed in Table 1-4 are a reminder of the points made in the previous subsections. Specifically, that engineers who implement geometric changes are only likely to detect the resulting change in crash frequency when the change is implemented on a district-wide or statewide basis. In this manner, the district-wide implementation is likely to yield a district crash history that satisfies the frequencies listed in Table 1-4 and, thereby, allows the effect of the design element to be visualized (as in Figure 1-5b).

For example, consider the district-wide addition of one foot of lane width to all two-lane highways. The reduction in crashes associated with this change may not be detectable in the year or two following the change on any specific highway segment, but it is likely to be observed in the average crash frequency for all such highways in the district.

AMF Precision

This section discusses the precision of design-related AMFs. The first subsection provides a definition of precision, as it relates to AMFs. The second subsection describes a technique for estimating the precision of AMFs obtained from before-after studies. The third subsection discusses the challenges associated with estimating AMF precision. The last subsection presents the recommendations made regarding the precision of the AMFs offered in the *Workbook*.

Definition of Precision

All AMFs offered in the *Workbook* are long-run averages and represent a best estimate of the true, but unknown, mean AMF value. This characterization is true regardless of whether the average AMF is read from a figure or computed from an equation. However, as with any statistic, there is inherently some unexplained variability in the data that ultimately makes it impossible to quantify the true mean AMF value with certainty. The degree of uncertainty associated with an AMF value is referred to as its precision. The precision of the AMF is described in terms of a range of values that bound the true mean AMF.

The standard deviation of the AMF S_{AMF} is the statistic used to describe AMF precision. The 68 percent confidence interval for the true mean AMF is centered on the average AMF and extends on one standard deviation above and below this average. The 95 percent confidence interval is more commonly used for engineering analyses and is defined as $AMF \pm 2.0\ S_{AMF}$. Hereafter, the "precision" of an AMF is defined to be its 95 percent confidence interval.

Technique for Estimating AMF Precision

Table 1-5 illustrates the relationship between the ratio S_{AMF}/AMF and crash frequency, where this ratio is multiplied by 100 to convert it into a "limit percentage." These percentages are approximate, but those listed are sufficiently accurate to estimate AMF confidence intervals. Their accuracy increases for larger crash frequencies.

The percentages in Table 1-5 are applicable to before-after studies. The crash frequency referred to represents the total number of crashes reported for the pool of sites in the period before the design change. The use of regression analysis to derive an AMF is likely to have additional variability introduced due to correlation among variables in the regression model. As a result, the percentages for AMFs from regression analysis are likely to be larger than those listed in Table 1-5.

Table 1-5. Percentages for Estimating AMF Precision.

Total Crash Count before Change	Approx. Limit Percent [1, 2]	AMF Range [3]	
		Low	High
10	not avail.	0.48	3.39
20	not avail.	0.58	2.13
50	not avail.	0.70	1.55
100	28	0.77	1.35
200	20	0.83	1.23
500	12	0.89	1.14
1000	8.8	0.92	1.09
2000	6.2	0.94	1.06
10,000	2.8	0.97	1.03

Notes:

1 - Percentages correspond to a 95 percent confidence interval.

2 - AMF_{upper} = $AMF \times (1 + \mathit{Limit\ Percent}/100)$;
AMF_{lower} = $AMF \times (1 - \mathit{Limit\ Percent}/100)$; and
AMF = average computed from before-after study.

3 - Because of statistical uncertainty in the average AMF estimate, AMF values between the low and high AMF values listed for a specific total crash count could actually have an effect on safety that is *opposite* to that expected.

To illustrate the concept of limit percentages, consider an AMF derived from a before-after study wherein the pool of sites experienced 1000 crashes in the "before" period. The AMF is derived to be 0.80, and from Table 1-5, the limit percentage is 8.8 percent. Thus, the 95 percent confidence interval for the AMF is 0.73 (= 0.80 × [1 − 8.8/100]) to 0.87 (= 0.80 × [1 + 8.8/100]), or ± 8.8 percent.

The percentages listed in Table 1-5 indicate that 500 crashes are needed in the pool of sites to estimate a confidence interval of ±12 percent. To reduce this interval by about one half (i.e., to ±6.2 percent), the number of crashes represented in the "before" database would have to total 2000.

The last two columns of Table 1-5 define a "cautionary" range of AMF values corresponding to the total crash frequencies listed. These values do not share the assumptions used to estimate the limit percentages. Thus, they can be considered as reasonably accurate for all crash frequencies.

AMFs within the cautionary range should be used with caution because there is a small chance that the expected change in crash frequency is opposite to that intended. This point is best illustrated by example. Consider a before-after study based on 1000 crashes in the "before" period. The AMF is derived to be 0.95. This AMF is less than 1.0 and, thus, implies that the corresponding design change is most likely going to reduce crashes by about 5 percent. However, from Table 1-5, AMFs in the range of 0.92 to 1.09 should be used with caution when based on 1000 crashes. There is enough uncertainty about AMFs in this range that it is possible that, following additional research, the true mean AMF for this design change could turn out to be larger than 1.0. If so, the design change actually *increased* crashes, which is opposite to the change that was expected.

Challenges to Estimation and Implications

Estimation Challenges. The statistics listed in Table 1-5 were computed using an equation derived by Hauer (*8*) for estimating the standard deviation of the AMF. However, this equation does not include all the factors that can influence the standard deviation and the corresponding limit percentages. Thus, the percentages in Table 1-5 represent a lower bound on the actual percentages.

There are several reasons why the actual percentages may be larger than those listed in Table 1-5. For example, the percentages can be increased by 50 percent or more if the AMF is computed from a "simple" before-after study that does not account for various external influences (e.g., regression-to-the-mean, changes in driver behavior over time, regional differences in driver behavior, regional differences in reporting threshold, etc.). The percentages can be increased by 100 percent or more if the AMF is derived from a regression model in which highly correlated variables are present.

Finally, it should be noted that some published reports that describe regression models do not state the standard deviation of the model coefficients. This omission makes it impossible to estimate the limit percentages for the AMFs derived from these models.

Implications. AMFs offered in the *Workbook* were derived from a synthesis of the literature or from an analysis of data for Texas streets and highways. Of those AMFs obtained from previous research, only those determined to be of good quality were synthesized. In this regard, studies of good quality were those that accounted for most external influences and correlated variables. These studies used databases that included hundreds of crashes.

The precision of the AMFs derived from an analysis of Texas data is provided in two research reports (*2*, *3*). Steps were taken to account for external influences and correlated variables. Databases were assembled to include as many intersections or segments as possible.

Nevertheless, for reasons cited in the previous subsection, the precision of each AMF offered in the *Workbook* is difficult to accurately quantify. The influence of external factors or the extent to which correlations are present can never be fully determined through an observational study. As a result, AMF precision is very difficult to accurately quantify.

If a conservative analysis is desired when using the *Workbook* AMFs, then a change in AMF value of less than 0.05 can be considered as not significantly different from 0.0 (i.e., there may be no detectable safety effect). For example, consider the AMF for an existing design is 1.07 and that for a proposed design is 1.04. The difference of 0.03 (= 1.07 − 1.04) is less than 0.05 and the analyst may conservatively judge that the new design may not have a detectable effect on safety. However, it should be remembered that the difference of 0.03 is still the best estimate of the likely change in safety for this design alternative based on current knowledge.

Summary

Engineers have been adequately and confidently guided by their first-hand experience with cause-and-effect for many years. Their observation of traffic events (e.g., queue discharge at a signal), coupled with similar experiences by others, gives them the confidence that they need to make decisions in their work. There is no question that an increase in green interval duration reduces delay to the movement receiving the additional time. A regression model of such a relationship would only confirm what the engineer has already witnessed. However, it may help with evaluations of unbuilt intersections or the improvement of signal timing at existing intersections.

Unfortunately, the influence of most geometric features on crash frequency is somewhat subtle, partly because of the design profession's long-standing adherence to conservative design criteria. This fact, combined with the large variability in crash data, indicates that the subtle influence of some geometric features (e.g., lane width, shoulder width, etc.) on crash frequency will not likely be observed by the engineer at a given site. The engineer that requires this experience to trust that such a trend exists may never be convinced. In fact, the engineer that has observed a reduction in crash frequency at a site and believes that it is due to a change that he or she made at the site is likely to have observed the regression-to-the-mean phenomenon. This phenomenon was discussed previously with regard to Figure 1-2a and occurs when safety improvements are made at a site that experienced an atypically large number of crashes in the year prior to treatment. The crash count observed at the site the year after the improvement is found to be smaller, and the reduction is incorrectly attributed to the improvement. In fact, research has shown that the reduction in crashes at a site (when it was selected because it was a "high crash" location) is partly due to the natural tendency for crash frequency to regress to a value nearer the true mean in the year that follows an above average year (*5*).

Unlike the effect of other traffic phenomena (e.g., the effect of signal timing on delay), the engineer will not likely be able to observe the influence of most geometric features and control devices on the mean crash frequency at a site. Rather, this influence can only be accurately quantified using large databases and statistical techniques. The subtle influence of a change in a geometry on crash frequency tends only to be observable through its implementation on a district-wide or statewide basis and a subsequent area-wide safety evaluation.

The precision of each AMF offered in the *Workbook* is difficult to accurately estimate for a variety of reasons. In all cases, the AMFs offered in the *Workbook* represent the current best estimate of the true mean AMF.

When comparing the AMF for an existing design with the AMF for a proposed design, the difference of the two AMFs represents the best estimate of the likely change in safety. However, if a conservative analysis is desired, then a difference in AMF values of less than 0.05 can be considered as not significantly different from 0.0 (i.e., there may be no detectable safety effect).

GLOSSARY

This part of the chapter defines several terms used in geometric design and safety-related documents. Separate sections are provided for design-related definitions and for safety-related definitions.

Design-Related Definitions

Several terms are used in this document to describe the entities that categorize and describe the design character of a roadway. These terms include: facility type, design category, design feature, design component, and design element. They also form a hierarchy in terms of their increasing focus and specificity. This hierarchy is illustrated by example in Table 1-6. Each term is defined in the following paragraphs.

Table 1-6. Hierarchy of Design Terms.

Descriptor	Examples
Facility type	Freeway, highway, intersection
Design categories	Geometry, traffic control devices, bridge
Design feature	Horizontal alignment, cross section, signing, markings
Design component	Horizontal curve, lane, warning sign, edge markings
Design element	Curve radius, lane width, "intersection ahead" sign

Design categories represent technical areas that are sufficiently complicated as to require designers with specific training and expertise. Each area typically includes its own stand-alone policies and/or guidelines. These categories include: geometric design, roadside design, traffic control device design, pavement design, lighting design, bridge design, rail-highway intersection design, work zone design, etc.

Design components are the fundamental entities (or building blocks) that are assembled for the roadway design. For example, a roadway design often includes the following components: horizontal curve, horizontal tangent, vertical curve, vertical tangent, lane, shoulder, median, warning signs, delineators, edgeline markings, driveway access points, etc.

Design elements are the physical characteristics of a specific design component (e.g., superelevation rate, lane width, etc.) or a unique descriptor of a part of the component (e.g., sign message, pavement marking color, etc.). The limiting value of a geometric design element can be designated as a "design control" in a design policy or guideline document.

Design features further separate the design categories into areas that historically have been designed together as a functional unit (or subsystem). As such, the selection of design components for a specific feature tends to be carefully coordinated such that the resulting design is safe, efficient, and consistent with driver expectation. Traditional design features include: horizontal alignment, vertical alignment, cross section, signing, delineation, marking, etc.

Facility type describes the main entities that comprise the transportation network. They include freeways, rural highways, urban streets, interchange ramps, and intersections. Design policies and guidelines often define controls that are specific to each of these facility types.

Rural area is any area outside the boundaries of an urban area.

Urban area, as defined by the U.S. Bureau of Census, are those places within boundaries set by state and local officials having a population of 5000 or more.

Safety-Related Definitions

Several safety-related terms are also used extensively in the literature and in this *Workbook*. They are defined in the following paragraphs.

Accident modification factor (AMF) is a constant or equation that represents the long-run change in safety following a change in the design or operation of a facility. An AMF can be computed as the ratio $N_w/N_{w/o}$, where N_w represents the expected number of crashes experienced by a highway facility *with* one or more specified design components and $N_{w/o}$ represents the expected number of crashes experienced by the same facility *without* the specified components. AMFs are often used as multiplicative factors to adjust the estimate obtained from a safety prediction model to a value that reflects the safety of a specific facility.

AMFs typically range in value from 0.5 to 2.0, with a value of 1.0 representing no effect on safety. AMFs less than 1.0 indicate that the specified component is associated with fewer crashes.

Barrier is located along the roadside or in the median. It can be categorized as rigid, semi-rigid, or cable. Rigid barrier includes concrete traffic barrier, retaining wall, or bridge railing. Semi-rigid barrier includes any type of metal beam guard fence or barrier terminal. Cable barrier includes any low- or high-tension system mounted on weak posts.

Crash reduction factor (CRF) is a constant that represents the proportion of crashes reduced as a result of a safety improvement at a specific location or along a specific road segment. CRFs typically range in value from 0.10 to 0.90. Larger CRFs in this range indicate a more significant reduction in crashes due to the improvement. To illustrate, consider a road segment that has a crash frequency of 3.0 crashes/yr. An improvement is made to the road's cross section and, after a period of time passes, a follow-up evaluation indicates that the change resulted in a crash frequency of 2.0 crashes/yr. The CRF for this improvement is 0.33 (= [3.0 − 2.0]/3.0). It represents a 33 percent reduction in crashes.

Injury crash is a crash wherein one or more of the persons involved is injured. The injury is reported as "incapacitating," "non-incapacitating," or "possible."

Safety is the expected crash frequency associated with a facility for a given set of design components, traffic control devices, and exposure conditions (e.g., traffic volume, segment length). Given that crashes are random events and that conditions can change over time, the safety of a specific type of facility is best conceptualized as the long-run average of the actual crash counts.

Safety evaluation tool is, at its simplest level, a set of equations that can be used to predict: (1) the safety of a given facility type, and (2) the safety effect associated with a change in its design features. At this "simple" level, a tool is equivalent to a model. However, complex tools can incorporate additional analysis techniques. For example, complex tools can include techniques for incorporating the reported crash history of a specific facility to improve the accuracy of the safety prediction. Complex tools can also include techniques for evaluating alternative designs using safety and other data (e.g., benefit-cost analysis). Tools are sometimes represented in software to facilitate their application.

Safety prediction model is an equation, or set of equations, that can be used to estimate the safety of a typical facility. The model includes factors related to crash risk and exposure. A figure or table is sometimes used to portray the relationship, instead of an equation. A model is typically derived to include one or more AMFs.

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Chapter 2

Freeways

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INTRODUCTION

Freeways are designed to serve long-distance, high-speed trips for automobile and truck traffic. They have full control of access such that points where traffic enter or exit the freeway are limited and intersecting roadways are accommodated by an interchange or grade separation, or they are terminated. Railroad crossings are served by grade separation and at-grade intersections are prohibited. The freeway design and operation are intended to preserve through capacity, maintain high speed, and promote safe travel.

The process of designing a freeway can include an evaluation of the operational and safety benefits associated with various design alternatives, with consideration to the overall cost-effectiveness of each alternative. The importance of this evaluation increases when right-of-way is more constrained, or when access to adjacent properties is adversely impacted.

The procedure described in this chapter can be used to quantify the safety associated with an existing freeway facility or with a proposed design. In this regard, safety is defined as the expected frequency of injury (plus fatal) crashes. The safety benefit of a proposed design can be obtained by comparing its expected crash frequency with that of the existing facility or of another alternative. Background information about the various equations and constants that comprise the procedure is provided in references *1, 2, 3,* and *4*. Procedures for estimating the operational or other impacts of a design alternative are beyond the scope of this *Workbook*.

PROCEDURE

This part of the chapter describes a procedure for evaluating the safety of freeway segments. A freeway segment is defined to be a length of roadway that is homogenous in terms of having reasonably constant cross section, adjacent land use, and traffic demand. A new segment begins at each horizontal curve or any significant change in grade, cross section, traffic volume, lane width, or other variable addressed by an applicable accident modification factor (AMF).

A procedure for evaluating interchange ramps is described in Chapter 5. Similarly, a procedure for evaluating the cross road is provided in Chapter 3 or 4, as appropriate. The ramp terminals can be evaluated using the procedure described in Chapter 6 or 7, as appropriate. Collectively, these procedures can be used together with the procedure in this chapter to evaluate the safety of a freeway and its interchanges.

The procedure described herein is based on the prediction of expected crash frequency. Specifically, the crash frequency for a typical segment is computed from a base model. This frequency is then adjusted using various AMFs to tailor the resulting estimate to a specific freeway segment. The base model includes variables for traffic volume, segment length, and access point frequency. AMFs are used to account for factors found to have some correlation with crash frequency, typically of a more subtle nature than the main factors. The AMFs are multiplied by the base crash frequency to obtain an expected crash frequency for the subject freeway segment.

The procedure described herein differs from that developed by Harwood et al. (*5*) because this procedure predicts injury (plus fatal) crash frequency, as opposed to total crash frequency. Otherwise, the procedure described herein is similar and shares the same strengths and weaknesses.

Base crash prediction models are described in the next section. The section that follows describes the AMFs to be used with these models. Example applications are provided throughout this *Workbook* to illustrate the use of the base models and the AMFs.

Base Models

Discussion

An examination of crash trends for freeways indicates that the crash rate varies with area type (urban or rural) and with the number of lanes in the cross section (*3*). In general, crash frequency is lower for freeways with many lanes than those with few lanes. Also, crash rates for the urban freeways tend to be higher than those for rural freeways. This latter influence is likely a reflection of the "busy" urban environment.

Safety Relationship

The relationship between crash frequency and traffic demand for base freeway conditions is shown in Figure 2-1. The trends shown in this figure apply to one-mile freeway segments that have one ramp entrance and one ramp exit along each side. Equations 2-1 through 2-22 should be used for other conditions.

Equations 2-1 through 2-22 are used to compute the expected crash frequency for freeway segments. Equations 2-1, 2-7, 2-13, and 2-18 are used for four-, six-, eight-, and ten-lane urban freeway segments, respectively. Equations 2-2 and 2-8 are used for four- and six-lane rural freeway segments, respectively. Each equation consists of four component equations that separately predict multiple-vehicle (non-ramp-related), single-vehicle, ramp entrance, and ramp exit crashes.

Table 2-1 lists the over-dispersion parameter *k* for each component equation. The use of this parameter is described in reference *6*.

Guidance

The crash frequency obtained from a base model is applicable to segments having base conditions. These conditions generally represent uncomplicated geometry, straight alignment, and typical cross section elements. The complete set of base conditions is identified in Table 2-2.

If a particular segment has some characteristics that differ from the base conditions, then the AMFs described in the next section can be used

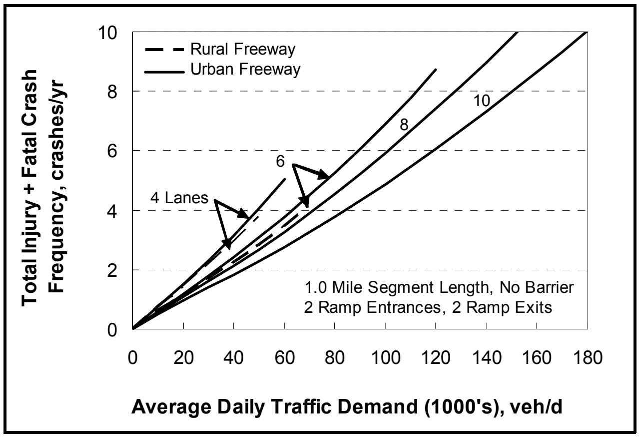

Figure 2-1. Illustrative Freeway Crash Trends.

Urban four-lane segments:

$$C_{b,4} = (C_{mv,4} + C_{sv,4} + C_{enr,4} + C_{exr,4})\, f_{4u} \qquad \textbf{(2-1)}$$

Rural four-lane segments:

$$C_{b,4} = (0.860 C_{mv,4} + 0.991 C_{sv,4} + 0.638 C_{enr,4} + 3.51 C_{exr,4})\, f_{4r} \qquad \textbf{(2-2)}$$

with,

$$C_{mv,4} = 0.00532\ (0.001\ ADT)^{1.55}\ L \qquad \textbf{(2-3)}$$

$$C_{sv,4} = 0.134\ (0.001\ ADT)^{0.646}\ L \qquad \textbf{(2-4)}$$

$$C_{enr,4} = 0.00704\ (ADT/15000)^{1.33}\ n_{enr} \qquad \textbf{(2-5)}$$

$$C_{exr,4} = 0.00174\ (ADT/15000)^{1.68}\ n_{exr} \qquad \textbf{(2-6)}$$

where:

C_b = base injury (plus fatal) crash frequency, crashes/yr;
C_{mv} = multiple-vehicle non-ramp crash frequency (urban), crashes/yr;
C_{sv} = single-vehicle crash frequency (urban), crashes/yr;
C_{enr} = ramp entrance crash frequency (urban), crashes/yr;
C_{exr} = ramp exit crash frequency (urban), crashes/yr;
ADT = average daily traffic volume, veh/d;
n_{enr} = number of ramp entrances;
n_{exr} = number of ramp exits;
L = segment length, mi; and
$f_{i,j}$ = local calibration factor for lanes *i* and area type *j*.

to obtain a more accurate estimate of segment crash frequency.

A local calibration factor is shown for each of the equations. The factor can be used to adjust the computed value so that it is more consistent with typical freeways in the agency's jurisdiction. A calibration procedure is identified in reference *4*. A calibration factor of 1.0 should be used unless a local calibration indicates another value is more appropriate.

The number of lanes noted with each equation represents the count of basic through lanes that are continuous for the segment. High-occupancy vehicle (HOV) lanes are not included in this count, unless they are separated from the adjacent through lane by just the paint stripe on their common lane line. Auxiliary lanes associated with a weaving section are not included in this count, unless the weaving section length exceeds 0.75 mi.

Ramp "entrance" refers to a point of freeway entry; ramp "exit" refers to a point of freeway departure. When determining the number of ramp entrances n_{enr} (or exits n_{exr}), an entrance or exit is counted if its gore point is on the segment (regardless of whether it is associated with a weaving section).

Example Application

The Question: What is the expected crash frequency for a typical six-lane urban freeway?

The Facts:

- Through lanes: 6
- Area type: urban
- Ramp entrances: 2
- Ramp exits: 2
- Segment length: 1.0 mi
- ADT: 60,000 veh/d

The Solution: From Equations 2-7 and 2-9 to 2-12, find that the typical urban six-lane freeway segment experiences about 3.76 crashes/yr (2.01 multiple-vehicle, 1.67 single-vehicle, 0.07 entrance ramp, and 0.01 exit ramp crashes). These crashes are designated as either injury or fatal.

Urban six-lane segments:

$$C_{b,6} = (C_{mv,6} + C_{sv,6} + C_{enr,6} + C_{exr,6})\, f_6 \qquad \textbf{(2-7)}$$

Rural six-lane segments:

$$C_{b,6} = (0.860 C_{mv,6} + 0.991 C_{sv,6} + 0.638 C_{enr,6} + 3.51 C_{exr,6})\, f_6 \qquad \textbf{(2-8)}$$

with,

$$C_{mv,6} = 0.00352\ (0.001\ ADT)^{1.55}\ L \qquad \textbf{(2-9)}$$

$$C_{sv,6} = 0.119\ (0.001\ ADT)^{0.646}\ L \qquad \textbf{(2-10)}$$

$$C_{enr,6} = 0.00532\ (ADT/15000)^{1.33}\ n_{enr} \qquad \textbf{(2-11)}$$

$$C_{exr,6} = 0.000640\ (ADT/15000)^{1.68}\ n_{exr} \qquad \textbf{(2-12)}$$

Urban eight-lane segments:

$$C_{b,8} = (C_{mv,8} + C_{sv,8} + C_{enr,8} + C_{exr,8})\, f_8 \qquad \textbf{(2-13)}$$

with,

$$C_{mv,8} = 0.00289\ (0.001\ ADT)^{1.55}\ L \qquad \textbf{(2-14)}$$

$$C_{sv,8} = 0.113\ (0.001\ ADT)^{0.646}\ L \qquad \textbf{(2-15)}$$

$$C_{enr,8} = 0.00199\ (ADT/15000)^{1.33}\ n_{enr} \qquad \textbf{(2-16)}$$

$$C_{exr,8} = 0.000482\ (ADT/15000)^{1.68}\ n_{exr} \qquad \textbf{(2-17)}$$

Urban ten-lane segments:

$$C_{b,10} = (C_{mv,10} + C_{sv,10} + C_{enr,10} + C_{exr,10})\, f_{10} \qquad \textbf{(2-18)}$$

with,

$$C_{mv,10} = 0.00220\ (0.001\ ADT)^{1.55}\ L \qquad \textbf{(2-19)}$$

$$C_{sv,10} = 0.104\ (0.001\ ADT)^{0.646}\ L \qquad \textbf{(2-20)}$$

$$C_{enr,10} = 0.00212\ (ADT/15000)^{1.33}\ n_{enr} \qquad \textbf{(2-21)}$$

$$C_{exr,10} = 0.000491\ (ADT/15000)^{1.68}\ n_{exr} \qquad \textbf{(2-22)}$$

Table 2-1. Over-Dispersion Parameters.

Crash Type	Over-Dispersion Parameter (*k*)
Multiple-vehicle	4.40 mi^{-1}
Single-vehicle	9.05 mi^{-1}
Ramp-entrance	3.62
Ramp-exit	0.695

Table 2-2. Base Conditions.

Characteristic	Base Condition
Horizontal curve radius	tangent (no radius)
Grade	flat (0% grade)
Lane width	12 ft
Outside shoulder width	10 ft
Inside shoulder width	4 ft (= 4 lanes) 10 ft (> 4 lanes)
Median width	36 ft (= 4 lanes + median barrier) 26 ft (> 4 lanes + median barrier) 56 ft (no median barrier)
Rigid or semi-rigid barrier	not present
Shoulder rumble strips	not present
Horizontal clearance	30 ft
Ramp entrance	not present
Weaving section	not present
Truck presence	20% trucks

Accident Modification Factors

Discussion

This section describes AMFs that can be used to evaluate the relationship between a design change and the corresponding change in injury (plus fatal) crash frequency. The topics addressed are listed in Table 2-3. There are many additional factors, other than those listed in Table 2-3, that are likely to have some influence on crash frequency. However, their relationship has yet to be quantified through research. The list of available AMFs for freeways is likely to increase as new research in this area is undertaken.

All of the AMFs described in this section yield a value of 1.0 when the associated design component or element represents base conditions. A condition that is more generous (i.e., desirable) than the base condition results in an AMF of less than 1.0. A condition that is less generous will result in an AMF of more than 1.0.

Table 2-3. AMFs for Freeway Segments.

Application	Accident Modification Factor
Geometric design	Horizontal curve radius Grade Lane width Outside shoulder width Inside shoulder width Median width (no barrier) [1] Median width (some barrier) [1] Median width (full barrier) [1] Shoulder rumble strips
Roadside design	Outside clearance (no barrier) [1] Outside clearance (some barrier) [1] Outside clearance (full barrier) [1]
Access	Aggregated ramp entrance Aggregated weaving section
Freeway environment	Truck presence

Note:
1 - Barrier can be either rigid or semi-rigid.
Rigid barrier: concrete traffic barrier or retaining wall.
Semi-rigid barrier: metal beam guard fence.

Safety Relationship

The expected injury (plus fatal) crash frequency for a specific freeway segment is computed using Equation 2-23. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are different from the base conditions.

$$C = C_b \times AMF_{lw} \times AMF_{cr} \ldots \qquad \textbf{(2-23)}$$

where:
C = expected injury (plus fatal) crash frequency, crashes/yr;
C_b = base injury (plus fatal) crash frequency, crashes/yr;
AMF_{lw} = lane width accident modification factor; and
AMF_{cr} = horizontal curve radius accident modification factor.

Guidance

In application, all applicable AMFs should be quantified for the subject segment and then multiplied together. The base crash frequency C_b for freeways is obtained from Equation 2-1, 2-2, 2-7, 2-8, 2-13, or 2-18. The product of the AMFs and C_b represents the expected injury (plus fatal) crash frequency for the subject freeway segment.

If the crash history is available for the segment, then the over-dispersion parameters in Table 2-1 can be used with the empirical Bayes adjustment procedure described in reference *6* to increase the accuracy of the expected crash frequency (over that obtained from Equation 2-23).

Example Application

The Question: What is the expected crash frequency for a specific six-lane urban freeway segment?

The Facts:

- Through lanes: 6
- Area type: urban
- Base crash frequency C_b: 3.76 crashes/yr
- Average lane width: 10 ft

The Solution: The segment of interest has typical characteristics with the exception that its average lane width is 10 ft. As described later, the AMF for a lane width of 10 ft is 1.06. This AMF can be used with Equation 2-23 to estimate the expected injury (plus fatal) crash frequency for the subject segment as 4.0 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 3.76 \times 1.06 \\ &= 4.0\ crashes/yr \end{aligned} \qquad (2\text{-}24)$$

Horizontal Curve Radius - AMF_{cr}

Discussion

Larger radius horizontal curves improve safety in several ways. The larger radius increases the margin of safety against vehicle crash by rollover or slide out. The larger radius is often accompanied by an improved preview distance of the road ahead and, thereby, more driver sight distance. When a curve of near-minimum radius is used, the designer should ensure that adequate sight distance is available around retaining walls and embankments.

Safety Relationship

The relationship between curve radius and injury (plus fatal) crash frequency can be estimated using Equation 2-25. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a tangent highway section (i.e., infinite radius). Thus, the AMF yields a value of 1.0 when the radius is infinite.

Guidance

This AMF is applicable to any curve with a radius that corresponds to an AMF value of 2.0 or less when the ratio L_c/L is set to 1.0. The speed limit variable in the AMF is used only as a surrogate for the actual operating speed. Research indicates that a change in speed limit is rarely accompanied by an equivalent change in operating speed. As such, this AMF should not be used as a basis for decisions regarding a proposed change in speed limit.

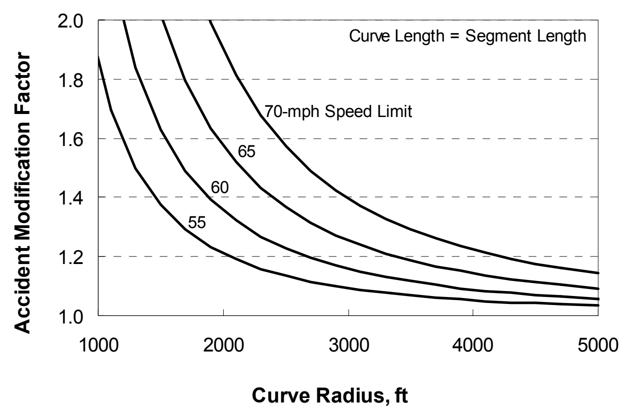

Figure 2-2. Horizontal Curve Radius AMF.

$$AMF_{cr} = 1.0 + 0.97\,(0.147\,V)^4 \,\frac{(1.47\,V)^2}{32.2\,R^2}\left(\frac{L_c}{L}\right) \quad \textbf{(2-25)}$$

where:
AMF_{cr} = horizontal curve radius accident modification factor;
V = speed limit, mph;
L_c = horizontal curve length, mi;
L = segment length, mi; and
R = curve radius, ft.

Base Condition: tangent alignment

Example Application

The Question: What is the AMF for a proposed horizontal curve on a freeway?

The Facts: Curve radius: 1700 ft. Speed limit: 60 mph. Curve length: 0.2 mi. Segment lenth: 0.2 mi.

The Solution: From Figure 2-2, find the AMF of 1.49. This value suggests that 49 percent more crashes may occur on this curve, relative to a tangent section.

Grade - AMF_g

Discussion

Grade can indirectly influence safety by influencing the speed of the traffic stream. Differences in speed between cars and trucks are most notable on ascending grades. Significant differences in speed among vehicles on ascending grades may increase the frequency of lane changes and related crashes. Descending grades accelerate the vehicle and place additional demand on vehicle braking and maneuverability.

Figure 2-3. Grade AMF.

$$AMF_g = e^{0.019\,g} \qquad (2\text{-}26)$$

where:
AMF_g = grade accident modification factor; and
g = percent grade (absolute value), %.

Base Condition: flat (0% grade)

Safety Relationship

The relationship between grade and injury (plus fatal) crash frequency can be estimated using Figure 2-3 or Equation 2-26. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is flat (i.e., 0 percent grade). In other words, the AMF yields a value of 1.0 when the grade is zero.

Guidance

This AMF is applicable to grades of 8 percent or less. It was developed for segments of constant grade; however, it can be applied to vertical curves. A procedure for using this AMF to evaluate a vertical curve is described in reference *6*.

Example Application

The Question: What is the AMF for a freeway segment with an uphill grade of 4 percent?

The Facts:

- Curve grade: +4 percent

The Solution: From Figure 2-3, find the AMF of 1.08. This value suggests that 8 percent more crashes may occur on this curve, relative to a flat segment.

Lane Width - AMF_{lw}

Discussion

It is generally recognized that lane width has some influence on driving comfort and efficiency. A narrow lane reduces the lateral clearance to vehicles in adjacent lanes and is most notable when large trucks are present in the traffic stream. Research indicates that narrow lanes have a lower capacity than wider lanes.

Safety Relationship

The relationship between lane width and injury (plus fatal) crash frequency can be estimated using Figure 2-4 or Equation 2-27. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 12-ft lane width.

Guidance

This AMF is applicable to lane widths ranging from 10 to 12 ft. If the lane width is more than 12 ft, then the AMF value for 12 ft should be used. If Equation 2-27 is used, then the proportion needed is obtained from Table 2-4.

Example Application

The Question: What is the AMF for a lane width of 10 ft?

The Facts:
- Area type: urban
- Through lanes: 6
- Lane width: 10 ft

The Solution: From Figure 2-4 for "Urban," find the AMF of 1.06. This value implies the 10-ft lane width is associated with 6 percent more crashes than the 12-ft lane width.

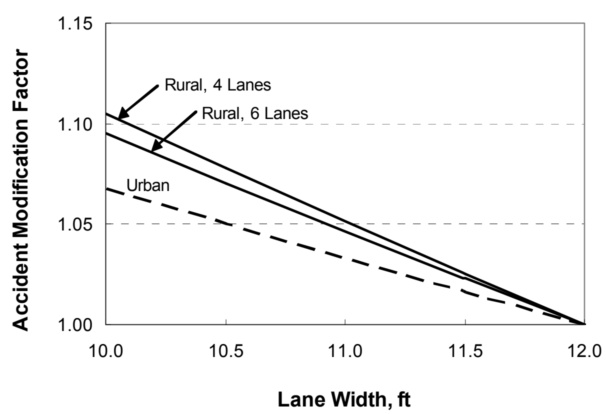

Figure 2-4. Lane Width AMF.

$$AMF_{lw} = (e^{-0.050\,(W_l - 12)} - 1.0)\frac{P_i}{0.62} + 1.0 \qquad \textbf{(2-27)}$$

where:
AMF_{lw} = lane width accident modification factor;
P_i = proportion of influential crashes of type i (see Table 2-4); and
W_l = lane width, ft.

Base Condition: 12-ft lane width

Table 2-4. Crash Distribution for Lane Width AMF.

Area Type	Through Lanes	Proportion of Crashes [1]
Rural	4	0.62
	6	0.56
Urban	4	0.44
	6	0.37
	8	0.38
	10	0.41

Note:
1 - Single-vehicle run-off-road, same-direction sideswipe, and multiple-vehicle opposite direction crashes.

Outside Shoulder Width - AMF_{osw}

Discussion

Shoulders offer numerous safety benefits for freeways. Depending on their width, shoulders may provide space for disabled vehicles and evasive maneuvers. Because of these safety benefits, wide outside (i.e., right-hand) shoulders are typically provided on freeways in rural and urban areas.

Safety Relationship

The relationship between outside shoulder width and injury (plus fatal) crash frequency can be estimated using Figure 2-5 or Equation 2-28. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 10-ft shoulder width.

Guidance

This AMF is applicable to outside shoulder widths ranging from 6 to 12 ft. If the shoulder width is greater than 12 ft, use the AMF value for 12 ft. If the shoulder width is less than 6 ft, use the AMF value for 6 ft. If Equation 2-28 is used, then the proportion needed is obtained from Table 2-5. The value used for W_s should be an average for both travel directions.

Example Application

The Question: What is the AMF for an outside shoulder width of 8 ft?

The Facts:
- Area type: urban
- Through lanes: 4
- Outside shoulder width: 8 ft

The Solution: From Figure 2-5, find the AMF of 1.03. This value implies that an 8-ft shoulder is likely to be associated with 3 percent more crashes than a 10-ft shoulder.

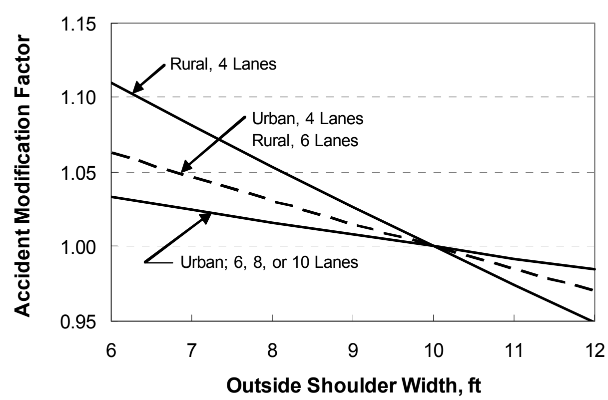

Figure 2-5. Outside Shoulder Width AMF.

$$AMF_{osw} = (e^{-0.026\,(W_s - 10)} - 1.0)\,\frac{P_i}{0.26} + 1.0 \qquad \textbf{(2-28)}$$

where:
AMF_{osw} = outside shoulder width accident modification factor;
P_i = proportion of influential crashes of type i (see Table 2-5); and
W_s = outside shoulder width, ft.

Base Condition: 10-ft outside shoulder width

Table 2-5. Crash Distribution for Outside Shoulder Width AMF.

Area Type	Through Lanes	Proportion of Crashes [1]
Rural	4	0.26
	6	0.14
Urban	4	0.15
	6	0.089
	8	0.066
	10	0.071

Note:
1 - Single-vehicle run-off-road crashes, right side only.

$$AMF_{osw} = (e^{-0.026 \times (8 - 10)} - 1.0)\,\frac{0.15}{0.26} + 1.0 = 1.03 \qquad \textbf{(2-29)}$$

Inside Shoulder Width - AMF_{isw}

Discussion

Inside (i.e., left-hand) shoulders offer similar safety benefits for freeways as do outside shoulders. Specifically, they provide storage space for disabled vehicles and additional room for evasive maneuvers.

Safety Relationship

The relationship between inside shoulder width and injury (plus fatal) crash frequency can be estimated using Figure 2-6 or Equation 2-30. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition a 4-ft inside shoulder width when there are four lanes and a 10-ft inside shoulder width when there are six or more lanes.

Guidance

This AMF is applicable to inside shoulder widths ranging from 0 to 10 ft. If the shoulder width is greater than 10 ft, then the AMF value for 10 ft should be used. If Equation 2-30 is used, then the proportion needed is obtained from Table 2-6. The value used for W_{is} should be an average for both travel directions.

Example Application

The Question: What is the AMF for an inside shoulder width of 6 ft?

The Facts:

- Area type: urban
- Through lanes: 4
- Inside shoulder width: 6 ft

The Solution: From Figure 2-6 for "Urban, 4 lanes," find the AMF of 0.97. This value implies a 3 percent reduction in crashes if a 6-ft shoulder width is used instead of a 4-ft width.

Figure 2-6. Inside Shoulder Width AMF.

$$AMF_{isw} = (e^{-0.026\,(W_{is} - W_{isb})} - 1.0)\,\frac{P_i}{0.30} + 1.0 \qquad \textbf{(2-30)}$$

where:
AMF_{isw} = inside shoulder width accident modification factor;
P_i = proportion of influential crashes of type *i* (see Table 2-6);
W_{is} = inside shoulder width, ft; and
W_{isb} = base inside shoulder width, ft.

Base Condition: 4-ft inside shoulder width for 4 lanes, 10-ft inside shoulder width for 6 or more lanes

Table 2-6. Crash Distribution for Inside Shoulder Width AMF.

Area Type	Through Lanes	Proportion of Crashes [1]
Rural	4	0.30
	6	0.32
Urban	4	0.20
	6	0.16
	8	0.14
	10	0.15

Note:
1 - Single-vehicle run-off-road (left side only) and multiple-vehicle opposite direction crashes.

$$AMF_{isw} = (e^{-0.026 \times (6-4)} - 1.0)\,\frac{0.20}{0.30} + 1.0 \qquad \textbf{(2-31)}$$
$$= 0.97$$

Median Width (no barrier) - AMF_{mwnb}

Discussion

A median provides several functions including separation of opposing traffic, a recovery area for errant vehicles, and a reduction in the glare of oncoming vehicle headlights. The benefits derived from these functions tend to increase with wider medians. Medians on a freeway are typically depressed, but they may also be flush paved if a barrier is used.

Safety Relationship

The relationship between median width and injury (plus fatal) crash frequency is shown in Figure 2-7; however, it should be estimated using Equation 2-32. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 56-ft median width and either: (1) a 4-ft inside shoulder width for four lanes or (2) a 10-ft inside shoulder width for six or more lanes.

Guidance

This AMF is applicable to freeway segments with a depressed median ranging from 30 to 80 ft in width and no barrier in the median. Median width is measured between the near edges of the left- and right-side traveled way (i.e., it includes the width of the inside shoulders).

If there are short lengths of rigid (or semi-rigid) barrier or bridge rail in the median, then the Median Width (some barrier) AMF should be used.

If a rigid (or semi-rigid) barrier is present in the median for the length of the segment, then the Median Width (full barrier) AMF should be used.

Figure 2-7. Median Width (no barrier) AMF.

$$AMF_{mwnb} = e^{-0.0296\,([W_m - 2W_{is}]^{0.5} - [56 - 2W_{isb}]^{0.5})} \qquad \textbf{(2-32)}$$

where:
AMF_{mwnb} = median width (no barrier) accident modification factor;
W_m = median width, ft;
W_{is} = inside shoulder width, ft; and
W_{isb} = base inside shoulder width, ft.

Base Condition: 56-ft median width, 4-ft inside shoulder width for 4 lanes, 10-ft inside shoulder width for 6 or more lanes

Example Application

The Question: What is the percent increase in crash frequency increase if a 64-ft median is reduced to 48 ft?

The Facts:

- Lanes: 6
- Inside shoulder width: 10 ft

The Solution: From Figure 2-7, find the AMF of 0.98 for the 64-ft median and the AMF of 1.02 for the 48-ft median. The ratio of these two AMFs indicates a 4.1 percent increase in crashes (= [1.02/0.98 - 1]×100).

Median Width (some barrier) - AMF_{mwsb}

Discussion

Barrier may be used in the median to protect motorists from collision with a fixed object such as a sign support or bridge abutment. The barrier itself is a fixed object, but one that is designed to reduce crash severity. An increase in the number of injury and property-damage-only crashes may be observed when a barrier is used.

Safety Relationship

The relationship between barrier presence in the median and injury (plus fatal) crash frequency is shown in Figure 2-8. The AMF value should be estimated using Equation 2-33. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition is stated in the box to the right.

Guidance

This AMF is applicable when there are short lengths of barrier or bridge rail in the median. The barrier can be rigid or semi-rigid. It can be located adjacent to one roadbed or at a specified distance W_{off} from the edge of traveled way. This AMF applies to median widths of 14 ft or more. It should not be used to justify the addition or removal of barrier.

The distance from the edge of shoulder to the barrier face W_{icb} is an average for the segment considering all individual short lengths of barrier in both travel directions. If each barrier is located at the same distance W_{off}, then $W_{icb} = W_{off} - W_{is}$. Otherwise, Equation 2-36 should be used to estimate W_{icb}. Similarly, Equation 2-37 should be used to estimate the proportion of the segment length with barrier in the median. The summation term "Σ" in the denominator of Equation 2-36 indicates that the ratio of barrier length $L_{ib,off}$ to offset distance "$W_{off} - W_{is}$" is computed for each length of barrier.

If a rigid (or semi-rigid) barrier is present in the median for the length of the segment, then the Median Width (full barrier) AMF should be used.

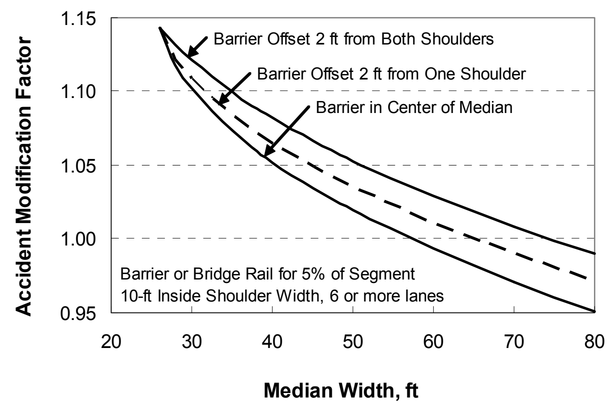

Figure 2-8. Median Width (some barrier) AMF.

$$AMF_{mwsb} = (1.0 - P_{ib})\, AMF_{mwnb} + P_{ib}\, AMF_{ib,ir} \quad \textbf{(2-33)}$$

with,

for 4 lanes:

$$AMF_{ib,ir} = e^{0.890/W_{icb} - 0.0296\,([2\,W_{icb}]^{0.5} - 6.93)} \quad \textbf{(2-34)}$$

for 6 or more lanes:

$$AMF_{ib,ir} = e^{0.890/W_{icb} - 0.0296\,([2\,W_{icb}]^{0.5} - 6.00)} \quad \textbf{(2-35)}$$

where:

AMF_{mwsb} = median width (some barrier) acc. modification factor;
AMF_{mwnb} = median width (no barrier) acc. modification factor;
$AMF_{ib,ir}$ = barrier or rail in median accident modification factor;
W_{icb} = width from edge of shoulder to barrier face, ft; and
P_{ib} = proportion of segment length with barrier in median.

Base Condition: median barrier not present, 56-ft median width, 4-ft inside shoulder width for 4 lanes, 10-ft inside shoulder width for 6 or more lanes

$$W_{icb} = \frac{\sum L_{ib,off}}{\sum \dfrac{L_{ib,off}}{W_{off} - W_{is}}} \quad \textbf{(2-36)}$$

$$P_{ib} = \frac{\sum L_{ib,off}}{2\,L} \quad \textbf{(2-37)}$$

where:

$L_{ib,off}$ = length of inside lane paralleled by a barrier located at a distance W_{off} from the traveled way, mi;
L = segment length, mi;
W_{is} = inside shoulder width, ft; and
W_{off} = width from edge of the traveled way to the face of a specific short length of barrier, ft.

Median Width (full barrier) - AMF$_{mwfb}$

Discussion

Barrier may be used with narrower medians to minimize cross-median crashes. The barrier can be located near the center of the median or nearer to one of the roadbeds. In some instances, a barrier is adjacent to both roadbeds. The barrier itself is a fixed object, but one that is designed to reduce crash severity. An increase in the number of injury and property-damage-only crashes may be observed when a barrier is used.

Safety Relationship

The relationship between median barrier presence and injury crash frequency is shown in Figure 2-9. The AMF value should be estimated using Equation 2-38 or 2-39. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is stated in the box to the right.

Guidance

This AMF is applicable when a median barrier extends the length of the segment. The barrier can be rigid or semi-rigid. It can be located in the center of the median or adjacent to a roadbed. The AMF applies to segments with a median width of 14 ft or more. It should not be used to justify the addition or removal of barrier.

The distance from the edge of shoulder to the barrier face W_{icb} is an average for the length of the segment considering both travel directions. Equation 2-40 or 2-41 should be used to estimate this distance. Both equations also account for short lengths of barrier that may exist in addition to the continuous barrier (e.g., for a sign support or bridge abutment).

If the barrier in the median does not extend for the length of the segment, then the Median Width (some barrier) AMF should be used.

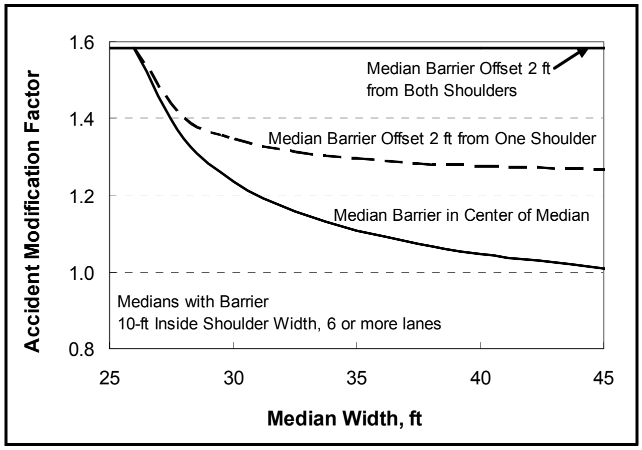

Figure 2-9. Median Width (full barrier) AMF.

For 4 lanes:

$$AMF_{mwfb} = e^{0.890/W_{icb} - 0.0296\,([2W_{icb}]^{0.5} - 5.29)} \qquad \textbf{(2-38)}$$

For 6 or more lanes:

$$AMF_{mwfb} = e^{0.890/W_{icb} - 0.0296\,([2W_{icb}]^{0.5} - 2.45)} \qquad \textbf{(2-39)}$$

where:
AMF_{mwfb} = median width (full barrier) acc. modification factor;
W_{icb} = width from edge of shoulder to barrier face, ft.

Base Condition: 36-ft median width for 4 lanes, or 26-ft median width for 6 or more lanes

For barrier in center of median:

$$W_{icb} = \frac{2\,L}{\sum \dfrac{L_{ib,off}}{W_{off} - W_{is}} + \dfrac{2\,L - \sum L_{ib,off}}{0.5\,(W_m - 2\,W_{is} - W_{ib})}} \qquad \textbf{(2-40)}$$

For barrier adjacent to one roadbed:

$$W_{icb} = \frac{2\,L}{\dfrac{L}{2.0} + \sum \dfrac{L_{ib,off}}{W_{off} - W_{is}} + \dfrac{L - \sum L_{ib,off}}{W_m - 2\,W_{is} - W_{ib} - 2.0}} \qquad \textbf{(2-41)}$$

where:
$L_{ib,\,off}$ = length of inside lane paralleled by a barrier located at a distance W_{off} from the traveled way, mi;
L = segment length, mi;
W_{off} = width from edge of the traveled way to the face of a specific short length of barrier, ft;
W_m = median width, ft;
W_{is} = inside shoulder width, ft; and
W_{ib} = inside barrier width (measured between barrier faces), ft.

Shoulder Rumble Strips - AMF_{srs}

Discussion

Shoulder rumble strips offer the benefit of both an audible and a tactile warning to drivers that have drifted laterally from the traveled way. These warnings tend to alert unaware drivers and, thereby, reduce run-off-road crashes.

Safety Relationship

The relationship between rumble strip presence and injury (plus fatal) crash frequency can be estimated using Table 2-7 or Equation 2-42. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is no shoulder rumble strips.

Guidance

This AMF is based on the installation of continuous rumble strips along all shoulders. If there are no shoulder rumble strips, then AMF_{srs} equals 1.0.

Example Application

The Question: What percent reduction in crashes may result if shoulder rumble strips are installed on both the inside and outside shoulders of a rural four-lane freeway?

The Facts:
- Area type: rural
- Through lanes: 4

The Solution: From Table 2-7, find the AMF of 0.94. This value implies that crashes may be reduced by 6 percent by the installation of shoulder rumble strips.

Table 2-7. Shoulder Rumble Strips AMF.

Rumble Strips	Area Type	Through Lanes	AMF_{srs}
Present	Rural	4	0.94
		6	0.95
	Urban	4	0.96
		6	0.97
		8	0.97
		10	0.97
Not present	Any	Any	1.00

$$AMF_{srs} = (0.88 - 1.0)\,P_i + 1.0 \qquad \textbf{(2-42)}$$

where:
AMF_{srs} = shoulder rumble strips accident modification factor; and
P_i = proportion of influential crashes of type *i* (see Table 2-8).

Base Condition: shoulder rumble strips not present

Table 2-8. Crash Distribution for Shoulder Rumble Strips AMF.

Area Type	Through Lanes	Proportion of Crashes [1]
Rural	4	0.51
	6	0.43
Urban	4	0.33
	6	0.24
	8	0.21
	10	0.21

Note:
1 - Single-vehicle run-off-road crashes, either side.

Outside Clearance (no barrier) - AMF_{ocnb}

Discussion

A clear roadside provides a space for errant vehicles to recover and an openness to the roadway that creates driving ease. Right-of-way constraints and other factors can sometimes make it difficult to provide desirable clearance distances for the full length of the roadway.

Safety Relationship

The relationship between horizontal clearance and injury (plus fatal) crash frequency is shown in Figure 2-10. The AMF value should be estimated using Equation 2-43. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 30-ft horizontal clearance and a 10-ft outside shoulder width.

Guidance

This AMF is applicable to clearance distances up to 30 ft. The clearance distance is measured from the edge of traveled way to any continuous or repetitive vertical obstruction (e.g., utility poles, fence line, etc.). The values used for W_{hc} and W_s should be an average for both travel directions. If Equation 2-43 is used, the proportion needed is obtained from Table 2-9.

If there are short lengths of rigid (or semi-rigid) barrier or bridge rail on the roadside, then the Outside Clearance (some barrier) AMF should be used.

If a rigid (or semi-rigid) barrier is present on the roadside for the length of the segment, then the Outside Clearance (full barrier) AMF should be used.

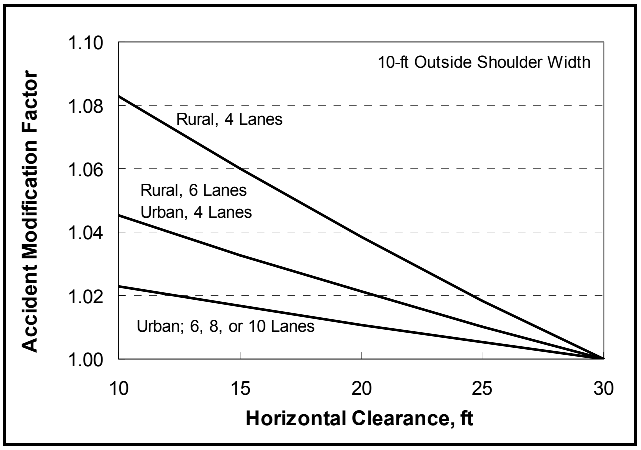

Figure 2-10. Outside Clearance (no barrier) AMF.

$$AMF_{ocnb} = (e^{-0.014\,(W_{hc} - W_s - 20)} - 1.0)\,P_i + 1.0 \qquad \textbf{(2-43)}$$

where:
AMF_{ocnb} = outside clearance (no barrier) accident modification factor;
P_i = proportion of influential crashes of type i (see Table 2-9);
W_s = outside shoulder width, ft; and
W_{hc} = horizontal clearance (average for both sides of segment length), ft.

Base Condition: 30-ft horizontal clearance, 10-ft outside shoulder width

Table 2-9. Crash Distribution for Outside Clearance AMF.

Area Type	Through Lanes	Proportion of Crashes [1]
Rural	4	0.26
	6	0.14
Urban	4	0.15
	6	0.089
	8	0.066
	10	0.071

Note:
1 - Single-vehicle run-off-road crashes, right side only.

Outside Clearance (some barrier) - AMF_{ocsb}

Discussion

Barrier may be used on the roadside to protect motorists from collision with a fixed object such as a sign support or bridge abutment. The barrier itself is a fixed object, but one that is designed to reduce crash severity. An increase in the number of injury and property-damage-only crashes may be observed when a barrier is used.

Safety Relationship

The relationship between barrier presence on the roadside and injury (plus fatal) crash frequency is shown in Figure 2-11. The AMF value should be estimated using Equation 2-44. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition is stated in the box to the right.

Guidance

This AMF is applicable when there are short lengths of barrier or bridge rail on the roadside. The barrier can be rigid or semi-rigid. It can be located adjacent to one roadbed, both roadbeds, or at a specified distance W_{off} from the edge of traveled way. This AMF should not be used to justify the addition or removal of barrier.

The distance from the edge of shoulder to the barrier face W_{ocb} is an average for the segment considering all individual short lengths of barrier in both travel directions. If each barrier is located at the same distance W_{off}, then $W_{ocb} = W_{off} - W_s$. Otherwise, Equation 2-47 should be used to estimate W_{ocb}. Similarly, Equation 2-48 should be used to estimate the proportion of the segment length with barrier on the roadside. The summation term "Σ" in the denominator of Equation 2-47 indicates that the ratio of barrier length $L_{ob,off}$ to offset distance "$W_{off} - W_s$" is computed for each length of barrier.

If a rigid (or semi-rigid) barrier is present on the roadside for the length of the segment, then the Outside Clearance (full barrier) AMF should be used.

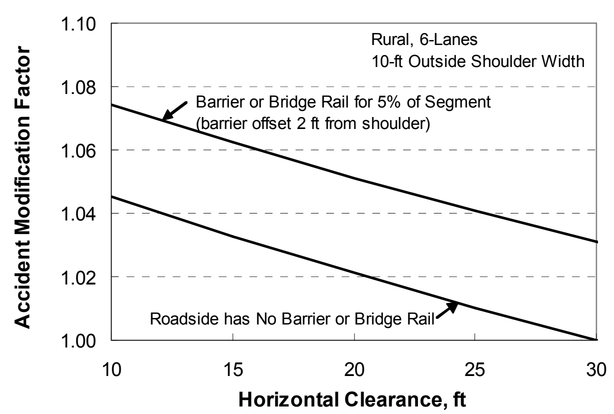

Figure 2-11. Outside Clearance (some barrier) AMF.

$$AMF_{ocsb} = (1.0 - P_{ob})\, AMF_{ocnb} + P_{ob}\, AMF_{oc|ob}\, F_{b|ob} \quad (2\text{-}44)$$

with,

$$AMF_{oc|ob} = (e^{-0.014\,(W_{ocb} - 20)} - 1.0)\, P_i + 1.0 \quad (2\text{-}45)$$

$$F_{b|ob} = e^{0.890/W_{ocb}} \quad (2\text{-}46)$$

where:
AMF_{ocsb} = outside clearance (some barrier) accident modification factor;
AMF_{ocnb} = outside clearance (no barrier) accident modification factor;
$AMF_{oc|ob}$ = outside clearance accident modification factor when outside barrier is present;
$F_{b|ob}$ = barrier adjustment factor;
W_{ocb} = width from edge of shoulder to barrier face, ft;
P_{ob} = proportion of segment length with barrier on roadside; and
P_i = proportion of influential crashes of type i (see Table 2-9).

Base Condition: roadside barrier not present, 30-ft horizontal clearance, 10-ft outside shoulder width

$$W_{ocb} = \frac{\sum L_{ob,off}}{\sum \dfrac{L_{ob,off}}{W_{off} - W_s}} \quad (2\text{-}47)$$

$$P_{ob} = \frac{\sum L_{ob,off}}{2\,L} \quad (2\text{-}48)$$

where:
$L_{ob,off}$ = length of outside lane paralleled by a barrier located at a distance W_{off} from the traveled way, mi;
L = segment length, mi;
W_s = outside shoulder width, ft; and
W_{off} = width from edge of the traveled way to the face of a specific short length of barrier, ft.

Outside Clearance (full barrier) - AMF_{ocfb}

Discussion

Barrier may be used in constrained rights-of-way and may be adjacent to both roadbeds on bridges, in mountainous terrain, and in other situations where hazards exist along the length of a freeway segment. The barrier itself is a fixed object, but one that is designed to reduce crash severity. An increase in the number of injury and property-damage-only crashes may be observed when a barrier is used.

Safety Relationship

The relationship between roadside barrier presence and injury crash frequency is shown in Figure 2-12. The AMF value should be estimated using Equation 2-49. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition is stated in the box to the right.

Guidance

This AMF is applicable when a roadside barrier extends the length of the segment on both sides. The barrier can be rigid or semi-rigid. This AMF should not be used to justify the addition or removal of barrier.

The distance from the edge of shoulder to the barrier face W_{ocb} is an average for the segment considering both travel directions. If the barrier is located at the same distance W_{off} in each direction of travel, then $W_{ocb} = W_{off} - W_s$. Otherwise, Equation 2-52 should be used to estimate this distance.

If the barrier on the roadside does not extend on both sides for the length of the segment, then the Outside Clearance (some barrier) AMF should be used.

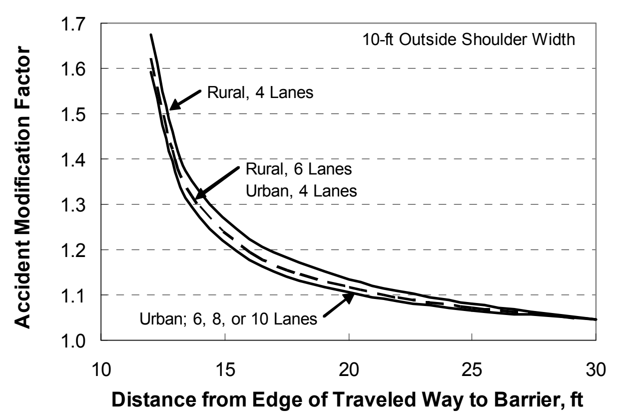

Figure 2-12. Outside Clearance (full barrier) AMF.

$$AMF_{ocfb} = AMF_{oc|ob} \, F_{b|ob} \quad \textbf{(2-49)}$$

with,

$$AMF_{oc|ob} = (e^{-0.014\,(W_{ocb} - 20)} - 1.0)\,P_i + 1.0 \quad \textbf{(2-50)}$$

$$F_{b|ob} = e^{0.890/W_{ocb}} \quad \textbf{(2-51)}$$

where:
- AMF_{ocfb} = outside clearance (full barrier) accident modification factor;
- $AMF_{oc|ob}$ = outside clearance accident modification factor when outside barrier is present;
- W_{ocb} = width from edge of shoulder to barrier face, ft;
- $F_{b|ob}$ = barrier adjustment factor; and
- P_i = proportion of influential crashes of type i (see Table 2-9).

Base Condition: varies with P_i, 30-ft horizontal clearance, 10-ft outside shoulder width

$$W_{ocb} = \frac{2}{\dfrac{1}{W_{off,1} - W_s} + \dfrac{1}{W_{off,2} - W_s}} \quad \textbf{(2-52)}$$

where:
- $W_{off,\,i}$ = width from the edge of the traveled way to the face of the barrier in travel direction i, ft; and
- W_s = outside shoulder width, ft.

Example Application

The Question: What is the AMF when a continuous roadside barrier is located 12 ft from the traveled way in one travel direction and 16 ft in the other travel direction?

The Facts: Area type: rural; Through lanes: 6; Outside shoulder width: 10 ft

The Solution: From Equation 2-52, find W_{ocb} = 3.0 ft (= 2/[1/{12 -10} + 1/{16 -10}]). From Equation 2-51, find $F_{b|ob}$ = 1.35. From Equation 2-50, find $AMF_{oc|ob}$ = 1.04. From Equation 2-49, find AMF_{ocfb} = 1.40.

Aggregated Ramp Entrance - $AMF_{enr|agg}$

Discussion

The portion of the freeway adjacent to a ramp entrance is often associated with increased turbulence in the freeway traffic stream when ramp vehicles attempt to merge into the through lanes. To some degree, this turbulence can be lessened through the provision of a longer ramp entrance length because it allows more time for vehicles to adjust their speed and for gaps to open up in the freeway traffic stream.

Safety Relationship

The relationship between ramp entrance length and injury (plus fatal) crash frequency is shown in Figure 2-13. The AMF value should be estimated using Equation 2-53. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF is applicable to right-side ramp entrances with either a parallel or taper design. The ramp entrance plan views in Figure 2-13 show the reference points used to measure ramp entrance length. These reference points are based on the marked pavement edge line location.

This AMF is applicable to ramp entrances that are 0.30 mi in length or less. It should be used when any portion of the ramp entrance length L_{enr} is within the subject segment.

Equation 2-54 can be used to estimate the average ramp entrance length when the subject segment has two or more entrances. Similarly, Equation 2-55 can be used to estimate the proportion of the segment length with a ramp entrance. The summation term "Σ" in the denominator of Equation 2-54 indicates that the ratio of $L_{enr,seg}$ to L_{enr} is computed for each individual ramp entrance on the segment.

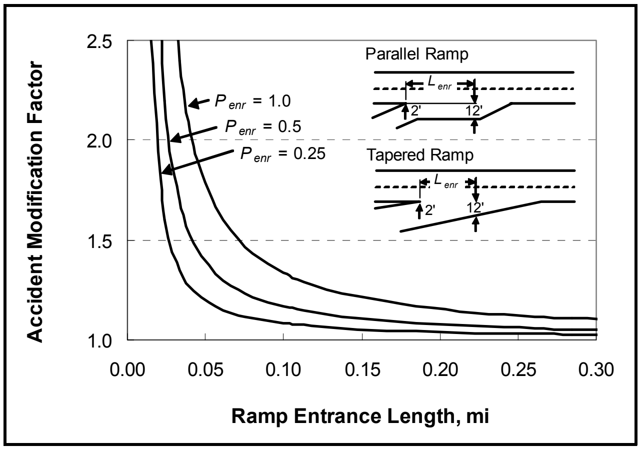

Figure 2-13. Aggregated Ramp Entrance AMF.

$$AMF_{enr|agg} = (1.0 - P_{enr})\,1.0 + P_{enr}\,e^{152.9/l_{enr}} \qquad \textbf{(2-53)}$$

where:
$AMF_{enr|agg}$ = aggregated ramp entrance acc. modification factor;
l_{enr} = average ramp entrance length, ft; and
P_{enr} = proportion of segment length adjacent to a ramp entrance.

Base Condition: no ramp entrance

$$l_{enr} = 5280\,\frac{\sum L_{enr,seg}}{\sum \dfrac{L_{enr,seg}}{L_{enr}}} \qquad \textbf{(2-54)}$$

$$P_{enr} = \frac{\sum L_{enr,seg}}{2\,L} \qquad \textbf{(2-55)}$$

where:
$L_{enr,\,seg}$ = length of ramp entrance that exists within the subject segment, mi;
L = segment length, mi; and
L_{enr} = length of ramp entrance (may extend beyond segment boundaries), mi.

Aggregated Weaving Section - $AMF_{wev|agg}$

Discussion

The portion of the freeway adjacent to a weaving section is often associated with increased turbulence in the freeway traffic stream when ramp vehicles attempt to merge into the through lanes and freeway vehicles attempt to exit at the downstream ramp. To some degree, this turbulence can be lessened through the provision of a longer weaving section length because it allows more time for vehicles to adjust their speed and for gaps to open up in the traffic stream.

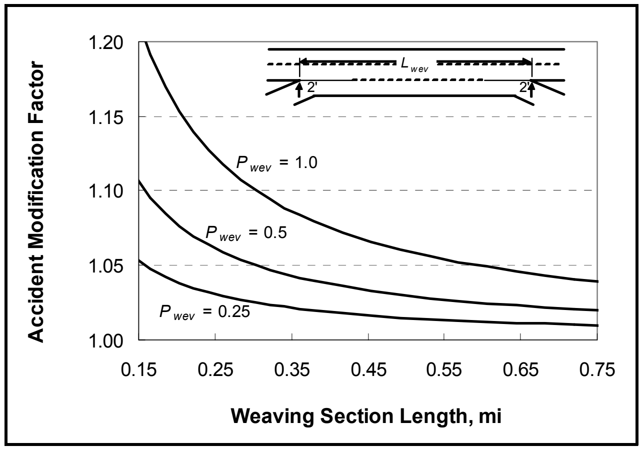

Figure 2-14. Aggregated Weaving Section AMF.

Safety Relationship

The relationship between weaving section length and injury (plus fatal) crash frequency is shown in Figure 2-14. The AMF value should be estimated using Equation 2-56. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

$$AMF_{wev|agg} = (1.0 - P_{wev})\,1.0 + P_{wev}\, e^{152.9/l_{wev}} \qquad \textbf{(2-56)}$$

where:
$AMF_{wev|agg}$ = aggregated weaving section acc. mod. factor;
l_{wev} = average weaving section length, ft; and
P_{wev} = proportion of segment length adjacent to a weaving section.

Base Condition: no weaving section

Guidance

This AMF is applicable to weaving sections with the ramp entrance and exit on the same side of the freeway. The weaving section plan view in Figure 2-14 shows the reference points used to measure weaving section length. These reference points are based on the marked pavement edge line location.

This AMF is applicable to weaving sections that are 0.15 to 0.75 mi in length. It should be used when any portion of the weaving section length L_{wev} is within the subject segment.

$$l_{wev} = 5280 \frac{\sum L_{wev,seg}}{\sum \frac{L_{wev,seg}}{L_{wev}}} \qquad \textbf{(2-57)}$$

$$P_{wev} = \frac{\sum L_{wev,seg}}{2\,L} \qquad \textbf{(2-58)}$$

where:
$L_{wev,seg}$ = length of weaving section that exists within the subject segment, mi;
L = segment length, mi; and
L_{wev} = length of weaving section (may extend beyond segment boundaries), mi.

Equation 2-57 can be used to estimate the average weaving section length when the subject segment has two or more weaving sections. Similarly, Equation 2-58 can be used to estimate the proportion of the segment length with a weaving section. The summation term "Σ" in the denominator of Equation 2-57 indicates that the ratio of $L_{wev,seg}$ to L_{wev} is computed for each individual weaving section on the segment.

Truck Presence - AMF_{tk}

Discussion

An analysis of crash data indicates that freeways with higher truck percentages are associated with fewer crashes. This trend suggests that drivers may be more cautious when there are many trucks in the traffic stream.

Safety Relationship

The relationship between truck presence and injury (plus fatal) crash frequency can be estimated using Figure 2-15 or Equation 2-59. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition is 20 percent trucks.

Guidance

This AMF is applicable to truck percentages ranging from 0.0 to 30 percent. It should not be used as a basis for design decisions regarding truck percentage. Rather, it should be used as a means of adjusting the base crash frequency to accurately reflect the presence of trucks on the subject freeway segment.

Example Application

The Question: What is the crash frequency for a specific freeway segment?

The Facts:

- Base crash frequency C_b: 1.5 crashes/yr
- Truck percentage: 10%

The Solution: The segment of interest has typical characteristics with the exception that it has 10 percent trucks. As shown in Figure 2-15, the AMF for 10 percent trucks is 1.11. This value can be used with Equation 2-23 to estimate the expected crash frequency for the subject segment as 1.67 crashes/yr.

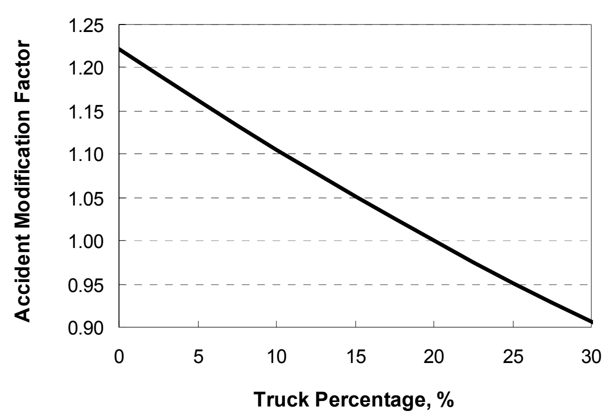

Figure 2-15. Truck Presence AMF.

$$AMF_{tk} = e^{-0.010\,(P_t - 20)} \qquad \textbf{(2-59)}$$

where:
AMF_{tk} = truck presence accident modification factor; and
P_t = percent trucks represented in ADT, %.

Base Condition: 20% trucks

$$\begin{aligned} C &= C_b \times AMF_{tk} \\ &= 1.50 \times 1.11 \\ &= 1.67 \textit{ crashes/yr} \end{aligned} \qquad \textbf{(2-60)}$$

Safety Appurtenances

AMFs for comparing specific roadside safety appurtenances are not described in this document. A comprehensive procedure for evaluating appurtenances is outlined in a report by Mak and Sicking (*7*) and automated in the *Roadside Safety Analysis Program* (RSAP) (*8*). RSAP can be used to evaluate alternative roadside safety appurtenances on individual freeway segments. The program accepts as input information about the freeway segment geometry and traffic characteristics. It also allows the analyst to describe the roadside cross section, location of fixed objects, and safety appurtenance design. The output from RSAP includes an estimate of annual crash frequency as well as the road-user costs associated with these crashes. The crash reduction potential realized by adding a roadside safety appurtenance (or changing the roadside cross section) can be estimated by specifying the changed condition as an "alternative."

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7. Mak, K., and D.L. Sicking. *NCHRP Report 492: Roadside Safety Analysis Program (RSAP) – Engineer's Manual.* National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C., 2003.

8. Mak, K., and D.L. Sicking. *Roadside Safety Analysis Program (RSAP) – User's Manual*. NCHRP Project 22-9. National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C., June 2002.

Chapter 3

Rural Highways

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INTRODUCTION

Rural highway cross sections vary from undivided, two-lane facilities with unlimited access to divided, multilane highways with partial access control. They are intended for both long-distance and moderate-distance trips. They also provide important at-grade access to county roads and adjacent property. Rural highways are sometimes difficult to design because of unusual or constrained conditions in the rural environment.

The process of designing a rural highway can include an evaluation of the operational and safety benefits associated with various design alternatives, with consideration to the overall cost-effectiveness of each alternative. The importance of this evaluation increases when right-of-way is more constrained, or when access to adjacent properties is adversely impacted.

The procedure described in this chapter can be used to quantify the safety associated with an existing rural highway facility or with a proposed design. In this regard, safety is defined as the expected frequency of injury (plus fatal) crashes. The safety benefit of a proposed design can be obtained by comparing its expected crash frequency with that of the existing facility or of another alternative. Background information about the various equations and constants that comprise the procedure is provided in references *1, 2, 3,* and *4*. Procedures for estimating the operational or other impacts of a design alternative are beyond the scope of this *Workbook*.

PROCEDURE

This part of the chapter describes a procedure for evaluating the safety of rural highway segments. A rural highway segment is defined to be a length of roadway that is homogeneous in terms of having a reasonably constant cross section, adjacent land use, and traffic demand. A new segment begins at each intersection, horizontal curve, or any significant change in cross section, median type, traffic volume, lane width, shoulder width, driveway density, or other variable addressed by an applicable accident modification factor (AMF).

A procedure for evaluating highway intersections is described in Chapter 6. This procedure can be used together with the procedure in this chapter to evaluate a rural highway and its intersections.

The procedure described herein is based on the prediction of expected crash frequency. Specifically, the crash frequency for a typical segment is computed from a base model. This frequency is then adjusted using various AMFs to tailor the resulting estimate to a specific highway segment. The base model includes variables for traffic volume, segment length, and access point frequency. AMFs are used to account for factors found to have some correlation with crash frequency, typically of a more subtle nature than the main factors. The AMFs are multiplied by the base crash frequency to obtain an expected crash frequency for the subject highway segment.

The procedure described herein differs from that developed by Harwood et al. (*5*) because this procedure predicts injury (plus fatal) crash frequency, as opposed to total crash frequency. Otherwise, the procedure described herein is similar and shares the same strengths and weaknesses.

Base crash prediction models are described in the next section. The section that follows describes the AMFs to be used with these models. Example applications are provided throughout this *Workbook* to illustrate the use of the base models and the AMFs.

Base Models

Discussion

An examination of crash trends for rural highways indicates that the crash rate varies with the median type used in the cross section and with the number of lanes (*3*). In general, crash rates are lower for highways with four lanes than those with two lanes. Also, four-lane highways with a restrictive median (i.e., depressed median) tend to have a lower crash rate than highways with a nonrestrictive median (i.e., two-way left-turn lane [TWLTL] or flush paved median) or undivided cross section.

Safety Relationship

The relationship between crash frequency and traffic demand for base rural highway conditions is shown in Figure 3-1. The trends shown in this figure apply to highway segments that are one mile long and located in a residential or undeveloped area. Equations 3-1 through 3-14 should be used for other conditions.

Equations 3-1 through 3-14 are used to compute the expected crash frequency for rural highway segments. Equations 3-1 and 3-2 are used for two- and four-lane undivided highways, respectively. Equation 3-7 is used for four-lane highways with a nonrestrictive median. Equation 3-11 is used for four-lane highways with a restrictive median. Each equation consists of three component equations that separately predict multiple-vehicle (non-driveway), single-vehicle, and driveway-related crashes.

Table 3-1 lists the over-dispersion parameter k for each equation. The use of this parameter is described in reference *6*.

Guidance

The crash frequency obtained from a base model is applicable to segments having base conditions. These conditions generally represent uncomplicated geometry, straight alignment, and

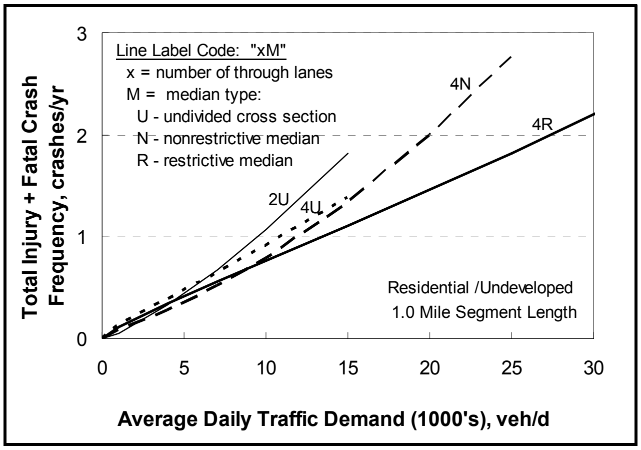

Figure 3-1. Illustrative Highway Crash Trends.

Two-lane, undivided segments:

$$C_{b,2u} = 0.0537\ (0.001\ ADT)^{1.30}\ L\ f_{2u} \qquad \textbf{(3-1)}$$

where:
C_b = base injury (plus fatal) crash frequency, crashes/yr;
ADT = average daily traffic volume, veh/d;
L = segment length, mi; and
$f_{i,j}$ = local calibration factor for lanes i and median type j.

Four-lane, undivided segments:

$$C_{b,4u} = (C_{mv,4u} + C_{sv,4u} + C_{dw,4u})\ f_{4u} \qquad \textbf{(3-2)}$$

with,

$$C_{mv,4u} = 0.00749\ (0.001\ ADT)^{1.63}\ L \qquad \textbf{(3-3)}$$

$$C_{sv,4u} = 0.109\ (0.001\ ADT)^{0.631}\ L \qquad \textbf{(3-4)}$$

$$C_{dw,4u} = 0.0169\ (ADT/15000)^{0.738}\ n_e \qquad \textbf{(3-5)}$$

$$n_e = n_{res} + 2.68\,n_{ind} + 2.33\,n_{bus} + 9.76\,n_{off} \qquad \textbf{(3-6)}$$

where:
C_{mv} = multiple-vehicle non-driveway crash frequency, crashes/yr;
C_{sv} = single-vehicle crash frequency, crashes/yr;
C_{dw} = driveway-related crash frequency, crashes/yr;
n_e = number of equivalent residential driveways;
n_{res} = number of driveways serving residential land uses;
n_{ind} = number of driveways serving industrial land uses;
n_{bus} = number of driveways serving business land uses; and
n_{off} = number of driveways serving office land uses.

Four-lane, nonrestrictive median segments:

$$C_{b,4n} = (C_{mv,4n} + C_{sv,4n} + C_{dw,4n})\ f_{4n} \qquad \textbf{(3-7)}$$

with,

$$C_{mv,4n} = 0.00527\ (0.001\ ADT)^{1.80}\ L \qquad \textbf{(3-8)}$$

$$C_{sv,4n} = 0.0776\ (0.001\ ADT)^{0.667}\ L \qquad \textbf{(3-9)}$$

$$C_{dw,4n} = 0.0170\ (ADT/15000)^{1.44}\ n_e \qquad \textbf{(3-10)}$$

typical cross section elements. The complete set of base conditions is identified in Table 3-2.

If a particular segment has characteristics that differ from the base conditions, then the AMFs described in the next section can be used to obtain a more accurate estimate of segment crash frequency.

Equations 3-5, 3-10, and 3-14 should not be used to evaluate the effect of adding or removing one driveway. Rather, the procedure described in Chapter 6 should be used for this purpose.

A local calibration factor is shown for each of the equations. The factor can be used to adjust the computed value so that it is more consistent with typical highways in the agency's jurisdiction. A calibration procedure is identified in reference *4*. A calibration factor of 1.0 should be used unless a local calibration indicates another value is more appropriate.

Land Use and Driveway Count

The land use served by a driveway is categorized as residential, industrial, business, or office. Analysis indicates that driveway volume and land use are highly correlated. In recognition of this correlation, Equation 3-6 uses land use as a convenient surrogate for driveway traffic volume because these data are not generally available. Table 3-3 can be used to determine the land use associated with each driveway along the subject highway segment.

Two types of driveway are recognized in the count of driveways. A "full driveway" allows left and right turns in and out of the property. A "partial driveway" allows only right turns in and out of the property. When counting driveways along a segment, a full driveway is counted as "1" driveway, and a partial driveway is counted as "0.5" driveways. Partial driveways are most commonly found on segments with a restrictive median.

Driveways that are unused should not be counted. Similarly, driveways leading into

Four-lane, restrictive median segments:

$$C_{b,4r} = (C_{mv,4r} + C_{sv,4r} + C_{dw,4r})\, f_{4r} \qquad \textbf{(3-11)}$$

with,

$$C_{mv,4r} = 0.00549\ (0.001\, ADT)^{1.49}\, L \qquad \textbf{(3-12)}$$

$$C_{sv,4r} = 0.106\ (0.001\, ADT)^{0.707}\, L \qquad \textbf{(3-13)}$$

$$C_{dw,4r} = 0.0152\ (ADT/15000)^{1.04}\, n_e \qquad \textbf{(3-14)}$$

Table 3-1. Over-Dispersion Parameters.

Lanes	Crash Type	Over-Disp. Parameter (*k*)
2	All	15.3 mi^{-1}
4	Multiple-vehicle	3.08 mi^{-1}
	Single-vehicle	4.30 mi^{-1}
	Driveway-related	1.11

Table 3-2. Base Conditions.

Characteristic	Base Condition
Rural Highways – Two or Four Lanes	
Horizontal curve radius	tangent (no curve)
Grade	flat (0% grade)
Lane width	12 ft
Outside shoulder width	8 ft
Rigid or semi-rigid barrier	not present
Horizontal clearance	30 ft
Side slope	1V:4H
Rural Highways – Two Lanes	
Spiral transition curve	not present
Shoulder rumble strips	not present
Centerline rumble strip	not present
Median type	Undivided (no TWLTL)
Superelevation	no deviation
Passing lane	not present
Driveway density	5 driveways/mi
Rural Highways – Four Lanes	
Inside shoulder width [1]	4 ft
Median width [2]	16 ft for nonrestrictive median 76 ft for restrictive median
Truck presence	16% trucks

Notes:
1 - Applies to highways with a restrictive median.
2 - Nonrestrictive median: TWLTL or flush-paved median. Restrictive median: depressed median.

fields, small utility installations (e.g., cellular phone tower), and abandoned buildings should not be counted. A circular driveway at a residence is counted as one driveway even though both ends of the driveway intersect the subject segment. Similarly, a small business (e.g., gas station) that has two curb cuts separated by only 10 or 20 ft is considered to have effectively one driveway.

Example Application

The Question: What is the expected crash frequency for a four-lane, undivided rural highway segment?

The Facts:

- Median type: undivided
- Land use: residential
- Driveways: 10
- Segment length: 1.0 mi
- ADT: 20,000 veh/d

The Solution: From Equations 3-2 to 3-6, find that the typical four-lane, undivided rural highway segment with these characteristics experiences 1.92 crashes/yr (0.99 multiple-vehicle, 0.72 single-vehicle, and 0.21 driveway-related crashes). These crashes are designated as either injury or fatal.

Table 3-3. Adjacent Land Use Characteristics.

Land Use	Characteristics	Examples
Residential or Undeveloped	• Buildings are small • A small percentage of the land is paved • If driveways exist, they have very low volume • Ratio of land-use acreage to parking stalls is large	• Single-family home • Undeveloped property, farmland • Graveyard • Park or green-space recreation area
Industrial	• Buildings are large and production oriented • Driveways and parking may be designed to accommodate large trucks • Driveway volume is moderate at shift change times and is low throughout the day • Ratio of land-use acreage to parking stalls is moderate	• Factory • Warehouse • Storage tanks • Farmyard with barns and machinery
Commercial Business	• Buildings are larger and separated by <u>convenient</u> parking between building and roadway • Driveway volume is moderate from mid-morning to early evening • Ratio of land-use acreage to parking stalls is small	• Strip commercial, shopping mall • Apartment complex, trailer park • Airport • Gas station • Restaurant
Office	• Buildings typically have two or more stories • Most parking is distant from the building or behind it • Driveway volume is high at morning and evening peak traffic hours; otherwise, it is low • Ratio of land-use acreage to parking stalls is small	• Office tower • Public building, school • Church • Clubhouse (buildings at a park) • Parking lot for "8 to 5" workers

Accident Modification Factors – Two or Four Lanes

Discussion

This section describes AMFs that can be used to evaluate the relationship between a design change and the corresponding change in injury (plus fatal) crash frequency. The AMFs described in this section apply to rural highways with either two or four lanes. They are listed in Table 3-4. AMFs that are only applicable to two-lane highways are described in the next section. AMFs that are only applicable to four-lane highways are provided in a subsequent section.

There are many additional factors, other than those listed in Table 3-4, that are likely to have some influence on crash frequency. However, their relationship has yet to be quantified through research. The list of available AMFs for rural highways is likely to increase as new research in this area is undertaken.

All of the AMFs described in this section yield a value of 1.0 when the associated design component or element represents base conditions. A condition that is more generous (i.e., desirable) than the base condition results in an AMF of less than 1.0. A condition that is less generous will result in an AMF of more than 1.0.

Safety Relationship

The expected injury (plus fatal) crash frequency for a specific rural highway segment is computed using Equation 3-15. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are different from base conditions.

Guidance

In application, all applicable AMFs should be quantified for the subject segment and then multiplied together. The base crash frequency C_b for rural highways is obtained from

Table 3-4. AMFs for Rural Highways – Two or Four Lanes.

Application	Accident Modification Factor
Geometric design	Horizontal curve radius Grade
Roadside design	Outside clearance (no barrier) [1] Outside clearance (some barrier) [1] Outside clearance (full barrier) [1] Side slope

Note:
1 - Barrier can be either rigid or semi-rigid.
Rigid barrier: concrete traffic barrier or retaining wall.
Semi-rigid barrier: metal beam guard fence.

$$C = C_b \times AMF_{cr} \times AMF_g \ldots \qquad \textbf{(3-15)}$$

where:
C = expected injury (plus fatal) crash frequency, crashes/yr;
C_b = base injury (plus fatal) crash frequency, crashes/yr;
AMF_{cr} = horizontal curve accident modification factor; and
AMF_g = grade accident modification factor.

Equation 3-1, 3-2, 3-7, or 3-11. The product of the AMFs and C_b represents the expected injury (plus fatal) crash frequency for the subject highway segment.

If the crash history is available for the segment, then the over-dispersion parameters in Table 3-1 can be used with the empirical Bayes adjustment procedure described in reference *6* to increase the accuracy of the expected crash frequency (over that obtained from Equation 3-15).

Example Application

The Question: What is the expected crash frequency for a specific four-lane, undivided rural highway segment?

The Facts:

- Through lanes: 4
- Median type: undivided
- Base crash frequency C_b: 1.92 crashes/yr
- Grade: 4 percent

The Solution: The segment of interest has typical characteristics with the exception that its grade is 4 percent. As described later, the AMF for this grade is 1.08. This AMF can be used with Equation 3-15 to estimate the expected crash frequency for the subject segment as 2.07 crashes/yr.

$$
\begin{aligned}
C &= C_b \times AMF_g \\
&= 1.92 \times 1.08 \\
&= 2.07 \ crashes/yr
\end{aligned}
\qquad (3\text{-}16)
$$

Horizontal Curve Radius - AMF$_{cr}$

Discussion

Larger radius horizontal curves improve safety in several ways. The larger radius increases the margin of safety against vehicle crash by rollover or slide out. The larger radius is often accompanied by an improved preview distance of the road ahead and, thereby, more driver sight distance. When a curve of near-minimum radius is used, the designer should ensure that adequate sight distance is available around retaining walls and embankments.

Safety Relationship

The relationship between curve radius and injury (plus fatal) crash frequency can be estimated using Equation 3-17. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a tangent highway section (i.e., infinite radius). Thus, the AMF yields a value of 1.0 when the radius is infinite.

Guidance

This AMF is applicable to any curve with a radius that corresponds to an AMF value of 2.0 or less when the ratio L_c/L is set to 1.0. The speed limit variable in the AMF is used only as a surrogate for the actual operating speed. Research indicates that a change in speed limit is rarely accompanied by an equivalent change in operating speed. As such, this AMF should not be used as a basis for decisions regarding a proposed change in speed limit. The variable L_c equals the length of the circular portion of the curve *plus* the length of both spiral transition curves (if present).

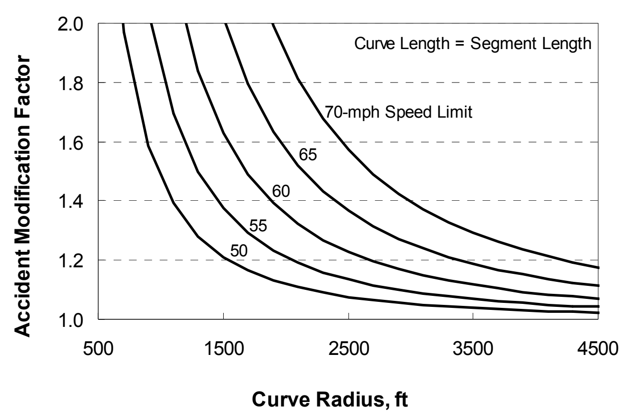

Figure 3-2. Horizontal Curve Radius AMF.

$$AMF_{cr} = 1.0 + 0.97\,(0.147\,V)^4\,\frac{(1.47\,V)^2}{32.2\,R^2}\left(\frac{L_c}{L}\right) \quad \textbf{(3-17)}$$

where:
AMF_{cr} = horizontal curve radius accident modification factor;
V = speed limit, mph;
L_c = horizontal curve length (plus spirals), mi;
L = segment length, mi; and
R = curve radius, ft.

Base Condition: tangent alignment

Example Application

The Question: What is the AMF for a proposed horizontal curve on a two-lane highway?

The Facts: Curve radius: 1600 ft. Speed limit: 55 mph. Curve length: 0.2 mi. Segment lenth: 0.2 mi.

The Solution: From Figure 3-2 for "55 mph," find the AMF of 1.33. This value suggests that 33 percent more crashes may occur on this curve, relative to a tangent section.

Grade - AMF_g

Discussion

Grade can indirectly influence safety by influencing the speed of the traffic stream. Differences in speed between cars and trucks are most notable on ascending grades. Significant differences in speed among vehicles on ascending grades may increase the frequency of lane changes and related crashes. Descending grades accelerate the vehicle and place additional demand on vehicle braking and maneuverability.

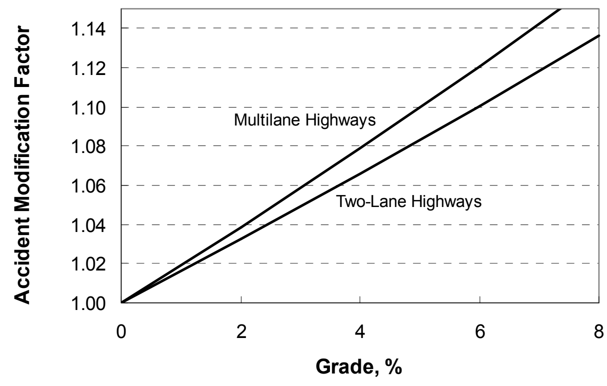

Figure 3-3. Grade AMF.

Safety Relationship

The relationship between grade and injury (plus fatal) crash frequency can be estimated using Figure 3-3, Equation 3-18, or Equation 3-19. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is flat (i.e., 0 percent grade). In other words, the AMF yields a value of 1.0 when the grade is zero.

For two-lane highways:

$$AMF_g = e^{0.016\,g} \qquad \textbf{(3-18)}$$

For multilane highways:

$$AMF_g = e^{0.019\,g} \qquad \textbf{(3-19)}$$

where:
AMF_g = grade accident modification factor; and
g = percent grade (absolute value), %.

Base Condition: flat (0% grade)

Guidance

This AMF is applicable to grades of 8 percent or less. It was developed for segments of constant grade; however, it can be applied to vertical curves. A procedure for using this AMF to evaluate a vertical curve is described in reference *6*.

Example Application

The Question: A grade reduction is being considered for an existing two-lane highway. What is the likely impact on crash frequency?

The Facts:

- Existing grade: 4.0 percent
- Proposed grade: 2.8 percent

The Solution: From Figure 3-3, find AMFs of 1.07 and 1.05 for grades of 4.0 and 2.8 percent, respectively. A reduction in crashes of 1.9 percent (= 100 ×[1 − 1.05/1.07]) is likely.

Outside Clearance (no barrier) - AMF_{ocnb}

Discussion

A clear roadside provides space for errant vehicles to recover and an openness to the roadway that creates driving ease. Right-of-way constraints and other factors can sometimes make it difficult to provide desirable clearance distances for the full length of the roadway.

Safety Relationship

The relationship between horizontal clearance and injury (plus fatal) crash frequency is shown in Figure 3-4. The AMF value should be estimated using Equation 3-20. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 30-ft horizontal clearance and an 8-ft outside shoulder width.

Guidance

This AMF is applicable to clearance distances up to 30 ft. The clearance distance is measured from the edge of traveled way to any continuous or repetitive vertical obstruction (e.g., utility poles, fence line, etc.). The values used for W_{hc} and W_s should be an average for both travel directions. If Equation 3-20 is used, the proportion needed is obtained from Table 3-5.

If there are short lengths of rigid (or semi-rigid) barrier or bridge rail on the roadside, then the Outside Clearance (some barrier) AMF should be used.

If a rigid (or semi-rigid) barrier is present on the roadside for the length of the segment, then the Outside Clearance (full barrier) AMF should be used.

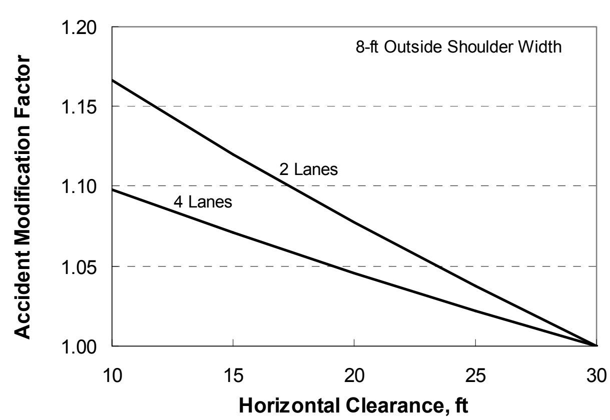

Figure 3-4. Outside Clearance (no barrier) AMF.

$$AMF_{ocnb} = (e^{-0.014\,(W_{hc} - W_s - 22)} - 1.0)\,P_i + 1.0 \qquad \textbf{(3-20)}$$

where:

AMF_{ocnb} = outside clearance (no barrier) accident modification factor;

P_i = proportion of influential crashes of type i (see Table 3-5);

W_s = outside shoulder width, ft; and

W_{hc} = horizontal clearance (average for both sides of segment length), ft.

Base Condition: 30-ft horizontal clearance, 8-ft outside shoulder width

Table 3-5. Crash Distribution for Outside Clearance AMF.

Median Type	Through Lanes	Proportion of Crashes
Restrictive [1]	4	0.30
Undivided or nonrestrictive [2]	2	0.52
	4	0.32

Notes:
1 - Single-vehicle run-off-road crashes, right side only.
2 - Single-vehicle run-off-road crashes, either side.

Outside Clearance (some barrier) - AMF_{ocsb}

Discussion

Barrier may be used on the roadside to protect motorists from collision with a fixed object such as a sign support or bridge abutment. The barrier itself is a fixed object, but one that is designed to reduce crash severity. An increase in the number of injury and property-damage-only crashes may be observed when a barrier is used.

Safety Relationship

The relationship between barrier presence on the roadside and injury (plus fatal) crash frequency is shown in Figure 3-5. The AMF value should be estimated using Equation 3-21. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition is stated in the box to the right.

Guidance

This AMF is applicable when there are short lengths of barrier or bridge rail on the roadside. The barrier can be rigid or semi-rigid. It can be located adjacent to one roadbed, both roadbeds, or at a specified distance W_{off} from the edge of traveled way. This AMF should not be used to justify the addition or removal of barrier.

The distance from the edge of shoulder to the barrier face W_{ocb} is an average for the segment considering all individual short lengths of barrier in both travel directions. If each barrier is located at the same distance W_{off}, then $W_{ocb} = W_{off} - W_s$. Otherwise, Equation 3-24 should be used to estimate W_{ocb}. Similarly, Equation 3-25 should be used to estimate the proportion of the segment length with barrier on the roadside. The summation term "Σ" in the denominator of Equation 3-24 indicates that the ratio of barrier length $L_{ob,off}$ to offset distance "$W_{off} - W_s$" is computed for each length of barrier.

If a rigid (or semi-rigid) barrier is present on the roadside for the length of the segment, then the Outside Clearance (full barrier) AMF should be used.

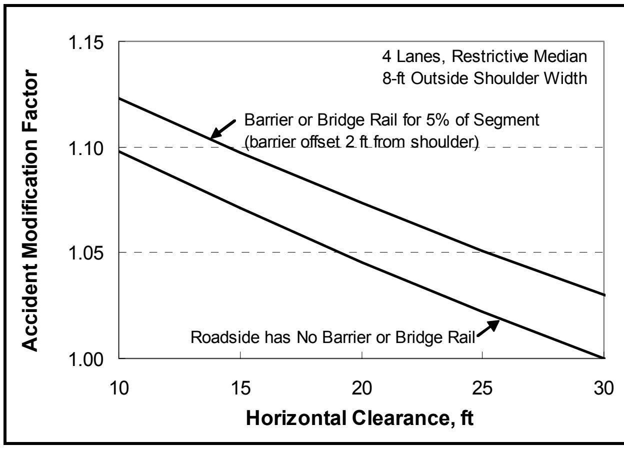

Figure 3-5. Outside Clearance (some barrier) AMF.

$$AMF_{ocsb} = (1.0 - P_{ob})\,AMF_{ocnb} + P_{ob}\,AMF_{oc|ob}\,F_{b|ob} \qquad (3\text{-}21)$$

with,

$$AMF_{oc|ob} = (e^{-0.014\,(W_{ocb} - 22)} - 1.0)\,P_i + 1.0 \qquad (3\text{-}22)$$

$$F_{b|ob} = e^{0.757/W_{ocb}} \qquad (3\text{-}23)$$

where:
AMF_{ocsb} = outside clearance (some barrier) accident modification factor;
AMF_{ocnb} = outside clearance (no barrier) accident modification factor;
$AMF_{oc|ob}$ = outside clearance accident modification factor when outside barrier is present;
$F_{b|ob}$ = barrier adjustment factor;
W_{ocb} = width from edge of shoulder to barrier face, ft;
P_{ob} = proportion of segment length with barrier on roadside; and
P_i = proportion of influential crashes of type i (see Table 3-5).

Base Condition: roadside barrier not present, 30-ft horizontal clearance, 8-ft outside shoulder width

$$W_{ocb} = \frac{\sum L_{ob,off}}{\sum \dfrac{L_{ob,off}}{W_{off} - W_s}} \qquad (3\text{-}24)$$

$$P_{ob} = \frac{\sum L_{ob,off}}{2\,L} \qquad (3\text{-}25)$$

where:
$L_{ob,off}$ = length of outside lane paralleled by a barrier located at a distance W_{off} from the traveled way, mi;
L = segment length, mi;
W_s = outside shoulder width, ft; and
W_{off} = width from edge of the traveled way to the face of a specific short length of barrier, ft.

Outside Clearance (full barrier) - AMF_{ocfb}

Discussion

Barrier may be used in constrained rights-of-way and may be adjacent to both roadbeds on bridges, in mountainous terrain, and in other situations where hazards exist along the length of a highway segment. The barrier itself is a fixed object, but one that is designed to reduce crash severity. An increase in the number of injury and property-damage-only crashes may be observed when a barrier is used.

Safety Relationship

The relationship between roadside barrier presence and injury crash frequency is shown in Figure 3-6. The AMF value should be estimated using Equation 3-26. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition is stated in the box to the right.

Guidance

This AMF is applicable when a roadside barrier extends the length of the segment on both sides. The barrier can be rigid or semi-rigid. This AMF should not be used to justify the addition or removal of barrier.

The distance from the edge of shoulder to the barrier face W_{ocb} is an average for the segment considering both travel directions. If the barrier is located at the same distance W_{off} in each direction of travel, then $W_{ocb} = W_{off} - W_s$. Otherwise, Equation 3-29 should be used to estimate this distance.

If the barrier on the roadside does not extend on both sides for the length of the segment, then the Outside Clearance (some barrier) AMF should be used.

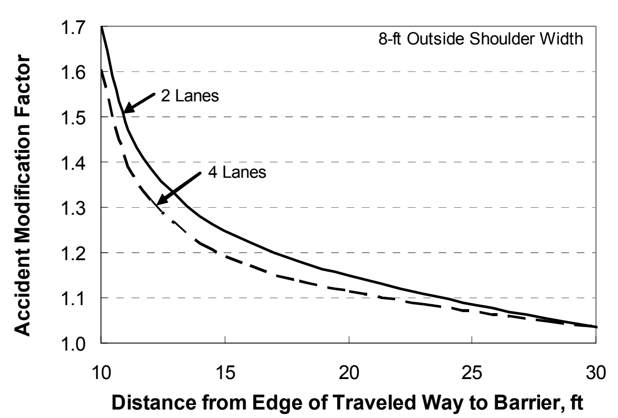

Figure 3-6. Outside Clearance (full barrier) AMF.

$$AMF_{ocfb} = AMF_{oc|ob}\, F_{b|ob} \tag{3-26}$$

with,

$$AMF_{oc|ob} = (e^{-0.014\,(W_{ocb} - 22)} - 1.0)\, P_i + 1.0 \tag{3-27}$$

$$F_{b|ob} = e^{0.757/W_{ocb}} \tag{3-28}$$

where:

AMF_{ocfb} = outside clearance (full barrier) accident modification factor;
$AMF_{oc|ob}$ = outside clearance accident modification factor when outside barrier is present;
W_{ocb} = width from edge of shoulder to barrier face, ft;
$F_{b|ob}$ = rigid barrier adjustment factor; and
P_i = proportion of influential crashes of type *i* (see Table 3-5).

Base Condition: varies with P_i, 30-ft horizontal clearance or more, 8-ft outside shoulder width

$$W_{ocb} = \frac{2}{\dfrac{1}{W_{off,1} - W_s} + \dfrac{1}{W_{off,2} - W_s}} \tag{3-29}$$

where:

$W_{off,\,i}$ = width from the edge of the traveled way to the face of the barrier in travel direction *i*, ft; and
W_s = outside shoulder width, ft.

Example Application

The Question: What is the AMF when a continuous roadside barrier is located 10 ft from the traveled way in one travel direction and 14 ft in the other travel direction?
The Facts: Median type: restrictive. Through lanes: 4. Outside shoulder width: 8 ft.
The Solution: From Equation 3-29, find W_{ocb} = 3.0 ft (= 2/[1/{10 - 8} + 1/{14 - 8}]). From Equation 3-28, find $F_{b|ob}$ = 1.29. From Equation 3-27, find $AMF_{oc|ob}$ = 1.09. From Equation 3-26, find AMF_{ocfb} = 1.41.

Side Slope - AMF_{ss}

Discussion

Flatter slopes improve the potential for the driver of an errant vehicle to safely regain control of the vehicle. Flatter slopes also minimize erosion and facilitate maintenance activities. For these reasons, side slopes are typically graded as flat as practical while still providing for necessary drainage.

Safety Relationship

The relationship between side slope and injury (plus fatal) crash frequency can be estimated using Figure 3-7 or Equation 3-30. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 1V:4H side slope (i.e., S_s = 4 ft).

Guidance

This AMF was developed for side slopes ranging from 1V:3H to 1V:6H. However, the trends are sufficiently stable that the AMF can be extended to slopes ranging from 1V:2H to 1V:7H with reasonable confidence. If Equation 3-30 is used, the proportion needed is obtained from Table 3-6.

Example Application

The Question: What percent increase in crashes is likely if side slopes of 1V:3H are used instead of 1V:4H?

The Facts:

- Median type: TWLTL
- Through lanes: 4

The Solution: From Figure 3-7, find the AMF of 1.02. This value suggests that crashes may increase 2 percent if a 1:3 side slope is used for the highway instead of a 1:4 slope.

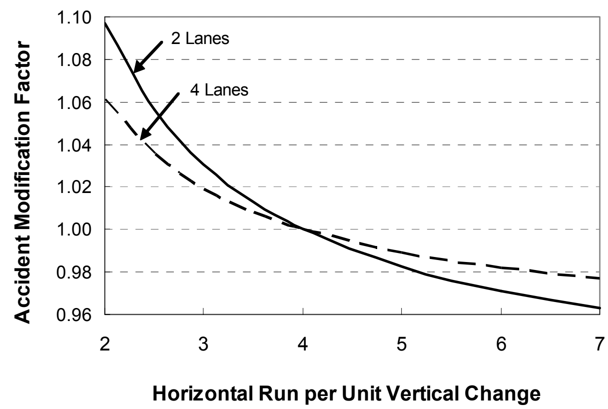

Figure 3-7. Side Slope AMF.

$$AMF_{ss} = (e^{0.69\,(1/S_s\,-\,1/4)} - 1.0)\,P_i + 1.0 \qquad \textbf{(3-30)}$$

where:
AMF_{ss} = side slope accident modification factor;
P_i = proportion of influential crashes of type *i* (see Table 3-6); and
S_s = horizontal run for a 1-ft change in elevation (average for segment length), ft.

Base Condition: 1V:4H

Table 3-6. Crash Distribution for Side Slope AMF.

Median Type	Through Lanes	Proportion of Crashes
Restrictive [1]	4	0.30
Undivided or nonrestrictive [2]	2	0.52
	4	0.32

Notes:
1 - Single-vehicle run-off-road crashes, right side only.
2 - Single-vehicle run-off-road crashes, either side.

$$\begin{aligned} AMF_{ss} &= (e^{0.69\,(1/3\,-\,1/4)} - 1.0)\,0.32 + 1.0 \\ &= 1.02 \end{aligned} \qquad \textbf{(3-31)}$$

Accident Modification Factors – Two Lanes

Discussion

This section describes AMFs that can be used to evaluate the relationship between a design change and the corresponding change in injury (plus fatal) crash frequency. The AMFs described in this section only apply to rural two-lane highways. They are listed in Table 3-7. AMFs applicable to either two- or four-lane rural highways are described in the previous section. AMFs that are only applicable to four-lane highways are provided in the next section.

There are many additional factors, other than those listed in Table 3-7, that are likely to have some influence on crash frequency. However, their relationship has yet to be quantified through research. The list of available AMFs for rural highways is likely to increase as new research in this area is undertaken.

All of the AMFs described in this section yield a value of 1.0 when the associated design component or element represents base conditions. A condition that is more generous (i.e., desirable) than the base condition results in an AMF of less than 1.0. A condition that is less generous will result in an AMF of more than 1.0.

Table 3-7. AMFs for Rural Highways – Two Lanes.

Application	Accident Modification Factor
Geometric design	Spiral transition curve Lane and shoulder width Shoulder rumble strips Centerline rumble strip TWLTL median type Superelevation Passing lane
Access	Driveway density

Safety Relationship

The expected injury (plus fatal) crash frequency for a specific rural highway segment is computed using Equation 3-15, repeated here as Equation 3-32. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are different from base conditions.

$$C = C_b \times AMF_{sp} \times AMF_{dd} \ldots \quad \text{(3-32)}$$

where:
C = expected injury (plus fatal) crash frequency, crashes/yr;
C_b = base injury (plus fatal) crash frequency, crashes/yr;
AMF_{sp} = spiral transition curve accident modification factor; and
AMF_{dd} = driveway density accident modification factor.

Guidance

In application, all AMFs should be quantified for the subject segment and then multiplied together. The base crash frequency C_b for rural highways is obtained from Equation 3-1, 3-2, 3-7, or 3-11. The product of the AMFs and C_b represents the expected injury (plus fatal) crash frequency for the subject highway segment.

If the crash history is available for the segment, then the over-dispersion parameters in Table 3-1 can be used with the empirical Bayes adjustment procedure described in reference *6* to increase the accuracy of the expected crash frequency (over that obtained from Equation 3-15).

Example Application

The Question: What is the expected crash frequency for a specific rural two-lane highway segment?

The Facts:

- Through lanes: 2
- Base crash frequency C_b: 1.92 crashes/yr
- Driveway density: 16 driveways/mi

The Solution: The segment of interest has typical characteristics with the exception that its driveway density is 16 driveways/mi. As described later, the AMF for this driveway density is 1.08. This AMF can be used with Equation 3-15 to estimate the expected crash frequency for the subject segment as 2.07 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{dd} \\ &= 1.92 \times 1.08 \\ &= 2.07\ crashes/yr \end{aligned} \tag{3-33}$$

Spiral Transition Curve - AMF_{sp}

Discussion

Spiral transition curves provide a gradually changing radius that is consistent with the natural path drivers follow as they steer into, or out of, a horizontal curve. Their benefit to safety and operation are most notable when the horizontal curve is long and relatively sharp. Drivers can steer a suitable transition path within the limits of a normal lane width for shorter curves or those that are relatively flat.

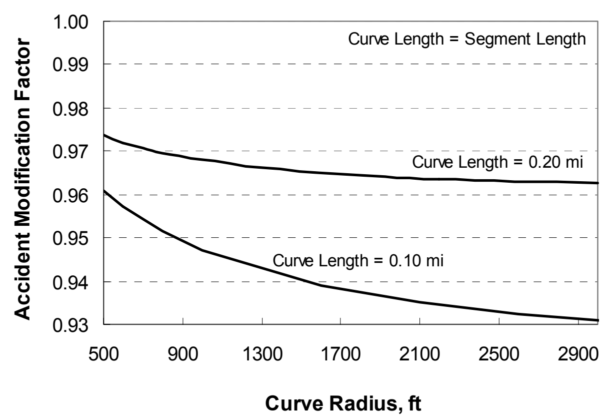

Figure 3-8. Spiral Transition Curve AMF.

Safety Relationship

The relationship between spiral presence and injury (plus fatal) crash frequency is shown in Figure 3-8. The AMF value is computed using Equation 3-34. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is that spiral transition curves are not present. Thus, the AMF has a value of 1.0 when spiral curves are not present.

$$AMF_{sp} = 1 - \frac{0.012}{1.55\,L_c + \frac{80.2}{R}}\left(\frac{L_c}{L}\right) \qquad \textbf{(3-34)}$$

where:
AMF_{sp} = spiral transition curve accident modification factor;
L_c = horizontal curve length (plus spirals), mi;
L = segment length, mi; and
R = curve radius, ft.

Base Condition: spiral transition curves not present

Guidance

This AMF is applicable to two-lane highway curves with a radius of 500 ft or more. If a horizontal curve has spiral transitions, both the horizontal curve AMF and the spiral transition curve AMF should be used. The variable L_c equals the length of the circular portion of the curve *plus* the length of both spiral transition curves.

Example Application

The Question: What is the safety benefit of adding spiral transitions to a horizontal curve?

The Facts:
- Curve radius: 1350 ft
- Curve plus spiral length: 0.10 mi
- Segment length: 0.10 mi

The Solution: From Figure 3-8, find the AMF of 0.94. It suggests that crash frequency on the curve may be reduced by 6 percent with the addition of spiral transition curves.

Lane and Shoulder Width - $AMF_{lw,sw}$

Discussion

It is generally recognized that lane width has some influence on driving comfort and efficiency. A narrow lane reduces the lateral clearance to vehicles in adjacent lanes and is most notable when large trucks are present in the traffic stream. Research indicates that narrow lanes have a lower capacity than wider lanes. On low-volume highways, drivers can use the full width of the roadbed to their advantage because of infrequent meetings between vehicles from opposing directions. This behavior is believed to explain the tendency for low-volume highways to have a lower crash rate than busier highways.

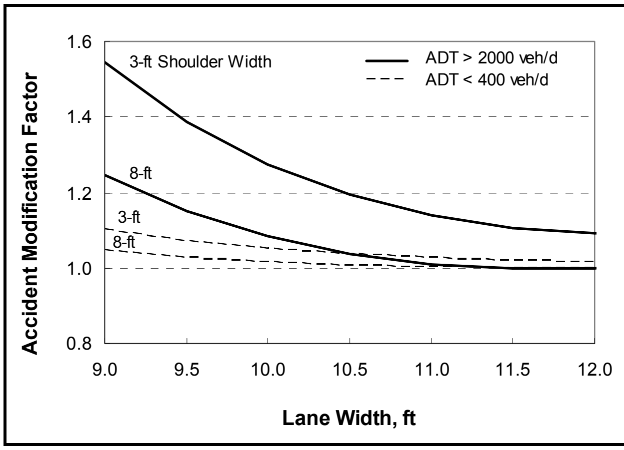

Figure 3-9. Lane and Shoulder Width AMF.

Safety Relationship

The relationship between lane width, shoulder width, traffic volume, and injury (plus fatal) crash frequency is shown in Figure 3-9; however, it should be estimated using Equation 3-35. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition lane for this AMF is a 12-ft lane width and an 8-ft shoulder width.

$$AMF_{lw,sw} = AMF_{hv}\, f_{ADT} + AMF_{lv}\,(1.0 - f_{ADT}) \quad \textbf{(3-35)}$$

$$f_{ADT} = \frac{1.0}{1.0 + e^{-0.00393\,(ADT - 1200)}} \quad \textbf{(3-36)}$$

$$AMF_{lv} = (AMF_{hv} - 1.0)\,0.15 + 1.0 \quad \textbf{(3-37)}$$

$$AMF_{hv} = \left(F_{lw}\, F_{sw}\, F_{lw,sw} - 1\right)0.54 + 1.0 \quad \textbf{(3-38)}$$

$$F_{lw} = e^{0.0533\,([W_l - 12.5]^2 - 0.250)} \quad \textbf{(3-39)}$$

$$F_{sw} = e^{-0.163\,(W_s - 8)} \quad \textbf{(3-40)}$$

$$F_{lw,sw} = e^{0.011\,(W_l\, W_s - 96)} \quad \textbf{(3-41)}$$

where:
$AMF_{lw,sw}$ = lane and shoulder width accident modification factor;
AMF_{hv} = accident modification factor for high volume;
AMF_{lv} = accident modification factor for low volume;
f_{ADT} = traffic volume adjustment factor;
ADT = average daily traffic volume, veh/d;
W_l = lane width, ft; and
W_s = outside shoulder width, ft.

Base Condition: 12-ft lane width, 8-ft shoulder width

Guidance

This AMF is applicable to two-lane highways having paved or gravel shoulders with a lane width ranging from 9 to 12 ft and a shoulder width ranging from 0 to 10 ft . If the lane width is less than 9 ft, then the AMF value for 9 ft should be used. If the lane width is greater than 12 ft, then the AMF value for 12 ft should be used. If the shoulder width is greater than 10 ft, then the AMF value for 10 ft should be used.

Shoulder Rumble Strips - AMF_{srs}

Discussion

Shoulder rumble strips offer the benefit of both an audible and a tactile warning to drivers that have drifted laterally from the traveled way. These warnings tend to alert unaware drivers and, thereby, reduce run-off-road crashes.

Safety Relationship

The relationship between rumble strip presence and injury (plus fatal) crash frequency can be estimated using Table 3-8 or Equation 3-42. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is no shoulder rumble strips.

Guidance

This AMF is applicable to two-lane highways. It is based on the installation of continuous rumble strips along both shoulders. If there are no shoulder rumble strips, then AMF_{srs} equals 1.0.

Example Application

The Question: What percent increase in crashes may result if shoulder rumble strips are removed from a rural highway?

The Facts:

- Median type: undivided
- Through lanes: 2

The Solution: From Table 3-8, find the AMF of 0.91 for having rumble strips and 1.00 for no rumble strips. The ratio of these two values implies that the removal of rumble strips may correspond to a 10 percent (= 100×[1.00/0.91 − 1.0]) increase in crashes.

Table 3-8. Shoulder Rumble Strips AMF.

Median Type	Rumble Strips	AMF_{srs}
Nonrestrictive	Present	unknown
	Not present	1.00
Undivided	Present	0.91
	Not present	1.00

For two-lane highways:

$$AMF_{srs} = (0.82 - 1.0)\, P_{sv} + 1.0 \qquad \textbf{(3-42)}$$

where:

AMF_{srs} = shoulder rumble strips accident modification factor; and

P_{sv} = proportion of single-vehicle run-off-road crashes (= 0.52).

Base Condition: shoulder rumble strips not present

Centerline Rumble Strip - AMF_{crs}

Discussion

A centerline rumble strip can offer the benefit of both an audible and a tactile warning to drivers that have drifted laterally into an oncoming traffic lane. These warnings tend to alert unaware drivers and, thereby, reduce head-on crashes.

Safety Relationship

The relationship between centerline rumble strip presence and injury (plus fatal) crash frequency is listed in Table 3-9. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is no centerline rumble strips.

Guidance

This AMF is applicable to two-lane highways. It is based on the installation of a continuous rumble strip along the centerline. If a centerline rumble strip is not installed, then AMF_{crs} equals 1.0.

Example Application

The Question: What percent decrease in crashes may result if a centerline rumble strip is added to a two-lane, undivided highway?

The Facts:
- Through lanes: 2
- Median type: undivided

The Solution: From Table 3-9, find the AMF of 0.85 for having rumble strips and 1.00 for no rumble strips. The ratio of these two values implies that the addition of the rumble strip may reduce crashes by 15 percent (= 100 ×[1.0 −0.85/1.00]).

Table 3-9. Centerline Rumble Strip AMF.

Median Type	Rumble Strip	AMF_{crs}
Nonrestrictive	Present	unknown
	Not present	1.00
Undivided	Present	0.85
	Not present	1.00

Base Condition: centerline rumble strip not present

TWLTL Median Type - AMF_T

Discussion

A TWLTL has the advantages of: (1) removing left-turning traffic from the through lanes, (2) providing access to adjacent properties, (3) providing a refuge area for vehicles turning left from a driveway, and (4) separating the opposing through traffic streams.

Safety Relationship

The relationship between TWLTL presence and injury (plus fatal) crash frequency can be estimated using Figure 3-10 or Equation 3-43. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is no TWLTL (i.e, undivided cross section).

Guidance

If the driveway density is less than 5.0 driveways/mi, then AMF_T equals 1.0. This AMF is applicable to two-lane highways with densities ranging from 0 to 20 driveways/mi. Driveway density is the count of driveways on both sides of the segment divided by segment length. The discussion in the section titled Land Use and Driveway Count describes the procedure for counting driveways. No distinction is made in this count for the type of land use served by the driveway.

Example Application

The Question: What percent reduction in crashes is likely to occur if a TWLTL is added to a two-lane, undivided highway?

The Facts:

- Through lanes: 2
- Driveway density: 10 driveways/mi

The Solution: From Figure 3-10, find the AMF of 0.93. This value implies that crashes may be reduced by 7 percent by the addition of a TWLTL.

Figure 3-10. TWLTL Median Type AMF.

$$AMF_T = 1.0 - 0.35\,P_D \qquad \textbf{(3-43)}$$

with,

$$P_D = \frac{0.0047\,D_d + 0.0024\,D_d^2}{1.199 + 0.0047\,D_d + 0.0024\,D_d^2} \qquad \textbf{(3-44)}$$

where:

AMF_T = TWLTL median type accident modification factor;
D_d = driveway density (two-way total), driveways/mi; and
P_D = driveway-related crashes susceptible to correction by TWLTL as a proportion of total crashes.

Base Condition: no TWLTL

Superelevation - AMF_e

Discussion

Superelevation is provided on horizontal curves to offset some of the centrifugal force associated with curve driving. It reduces side friction demand and increases the margin of safety relative to vehicle slide out or roll over. If the superelevation provided on a curve is significantly less than the amount specified by the applicable design guide, then the potential for a crash may increase.

Safety Relationship

The relationship between superelevation deviation and injury (plus fatal) crash frequency can be estimated using Figure 3-11. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The amount by which the superelevation provided is lower than that specified in the applicable design guideline is defined as "superelevation deviation." The base condition for this AMF is no deviation.

Guidance

This AMF is applicable to two-lane highway curves with a deviation of 5 percent or less. If the deviation exceeds 5 percent, then the AMF value for 5 percent should be used. If the superelevation rate *exceeds* that specified in the applicable design guide, then the AMF is 1.0.

Example Application

The Question: A curve has 2.7 percent superelevation. Current design guidelines call for 5.7 percent. How many crashes might be prevented by increasing the superelevation rate?

The Facts:

- Crash frequency: 0.32 crashes/yr

The Solution: The deviation is 3 percent (= 5.7 −2.7). From Figure 3-11, find the AMF of 1.09. If 5.7 percent is used, the expected crash frequency is 0.29 crashes/yr (= 0.32/1.09). Thus, the improvement may yield a reduction of 0.03 crashes/yr.

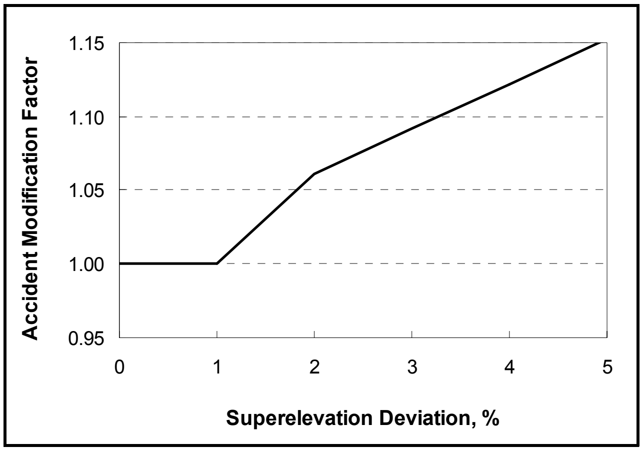

Figure 3-11. Superelevation AMF.

Base Condition: no superelevation deviation

Passing Lane - AMF_{pass}

Discussion

Passing and climbing lanes provide a way for drivers on two-lane highways to pass slower moving vehicles without entering the opposing traffic lane. As a result, these lanes can provide substantial safety benefit because drivers can pass without conflict with the opposing traffic stream.

Safety Relationship

The relationship between passing lane presence and injury (plus fatal) crash frequency can be estimated using Table 3-10. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition is no climbing lane or passing lane provided.

Guidance

This AMF is applicable to two-lane highways with passing lanes, and then only if the passing lane has a length sufficient to provide safe and efficient passing opportunities. Highways with passing lanes longer than required to provide a nominal passing opportunity should be treated as multilane highways.

Example Application

The Question: What percent reduction in crashes may result if a passing lane is installed in one travel direction on a specific rural two-lane highway segment?

The Facts:
- Cross section: two lanes, undivided
- Lane addition: passing in one direction

The Solution: From Table 3-10, find the AMF of 0.75. This AMF implies that crashes may be reduced 25 percent by the addition of the passing lane.

Table 3-10. Passing Lane AMF.

Climbing Lane or Passing Lane Type	AMF_{pass}
None provided	1.00
One direction (three-lane cross section)	0.75
Two directions (four-lane cross section)	0.65

Base Condition: climbing lane or passing lane not present

Driveway Density - AMF_{dd}

Discussion

Uncontrolled access on a rural highway creates numerous safety and operational problems. Proper design and spacing of access points can minimize these problems. Access management is an effective technique for guiding the process of locating driveways such that conflicts associated with turning vehicles are minimized.

Safety Relationship

The relationship between driveway density and injury (plus fatal) crash frequency can be estimated using Figure 3-12 or Equation 3-45. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is 5 driveways/mi.

Guidance

This AMF is applicable to two-lane highways with a driveway density ranging from 0 to 20 driveways/mi. Driveway density is the count of driveways on both sides of the segment divided by segment length. The discussion in the section titled Land Use and Driveway Count describes the procedure for counting driveways. No distinction is made in this count for the type of land use served by the driveway.

This AMF should not be used to evaluate the effect of adding or removing one driveway. Rather, the procedure described in Chapter 6 should be used for this purpose.

Example Application

The Question: What percent increase in crashes may occur if driveway density is allowed to increase from 2 to 10 driveways/mi?

The Solution: From Figure 3-12, find the AMFs of 0.98 and 1.04 for driveway densities of 2 and 10 driveways/mi, respectively. The percent increase in crashes is 6 percent (= 100 × [1.04/0.98 − 1]).

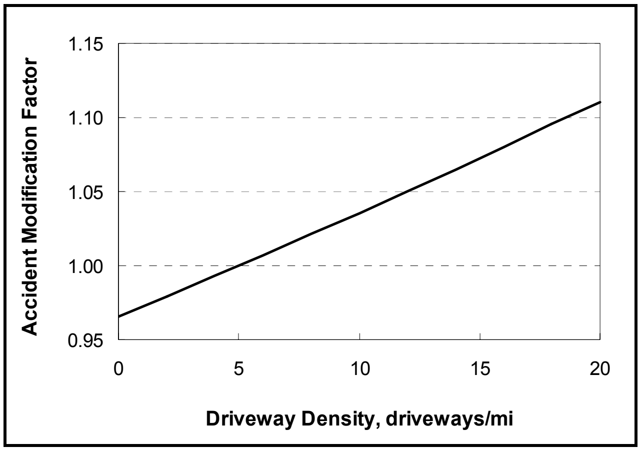

Figure 3-12. Driveway Density AMF.

$$AMF_{dd} = e^{0.007\,(D_d - 5)} \quad \textbf{(3-45)}$$

where:
AMF_{dd} = driveway density accident modification factor; and
D_d = driveway density (two-way total); driveways/mi.

Base Condition: 5 driveways per mile

$$\begin{aligned} AMF_{dd} &= e^{0.007\,(10 - 5)} \\ &= 1.04 \end{aligned} \quad \textbf{(3-46)}$$

Accident Modification Factors – Four Lanes

Discussion

This section describes AMFs that can be used to evaluate the relationship between a design change and the corresponding change in injury (plus fatal) crash frequency. The AMFs described in this section only apply to rural four-lane highways. They are listed in Table 3-11. AMFs applicable to either two- or four-lane rural highways are described in a previous section. AMFs that are only applicable to two-lane highways are also provided in a previous section.

There are many additional factors, other than those listed in Table 3-11, that are likely to have some influence on crash frequency. However, their relationship has yet to be quantified through research. The list of available AMFs for rural highways is likely to increase as new research in this area is undertaken.

All of the AMFs described in this section yield a value of 1.0 when the associated design component or element represents base conditions. A condition that is more generous (i.e., desirable) than the base condition results in an AMF of less than 1.0. A condition that is less generous will result in an AMF of more than 1.0.

Safety Relationship

The expected injury (plus fatal) crash frequency for a specific rural highway segment is computed using Equation 3-15, repeated here as Equation 3-47. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are different from base conditions.

Guidance

In application, all AMFs should be quantified for the subject segment and then multiplied together. The base crash frequency C_b for rural

Table 3-11. AMFs for Rural Highways – Four Lanes.

Application	Accident Modification Factor
Geometric design	Lane width Outside shoulder width Inside shoulder width Median width (no barrier) [1] Median width (some barrier) [1] Median width (full barrier) [1]
Highway environment	Truck presence

Note:
1 - Barrier can be either rigid or semi-rigid.
Rigid barrier: concrete traffic barrier or retaining wall.
Semi-rigid barrier: metal beam guard fence.

$$C = C_b \times AMF_{lw} \times AMF_{osw} \ldots \qquad \textbf{(3-47)}$$

where:
C = expected injury (plus fatal) crash frequency, crashes/yr;
C_b = base injury(plus fatal) crash frequency, crashes/yr;
AMF_{lw} = lane width accident modification factor; and
AMF_{osw} = outside shoulder width accident modification factor.

highways is obtained from Equation 3-1, 3-2, 3-7, or 3-11. The product of the AMFs and C_b represents the expected injury (plus fatal) crash frequency for the subject highway segment.

If the crash history is available for the segment, then the over-dispersion parameters in Table 3-1 can be used with the empirical Bayes adjustment procedure described in reference *6* to increase the accuracy of the expected crash frequency (over that obtained from Equation 3-15).

Example Application

The Question: What is the expected crash frequency for a specific four-lane, undivided rural highway segment?

The Facts:

- Through lanes: 4
- Median type: undivided
- Base crash frequency C_b: 1.92 crashes/yr
- Average lane width: 10 ft

The Solution: The segment of interest has typical characteristics with the exception that its average lane width is 10 ft. As described later, the AMF for a lane width of 10 ft is 1.08. This AMF can be used with Equation 3-15 to estimate the expected crash frequency for the subject segment as 2.07 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 1.92 \times 1.08 \\ &= 2.07 \ crashes/yr \end{aligned} \qquad (3\text{-}48)$$

Lane Width - AMF_{lw}

Discussion

It is generally recognized that lane width has some influence on driving comfort and efficiency. A narrow lane reduces the lateral clearance to vehicles in adjacent lanes and is most notable when large trucks are present in the traffic stream. Research indicates that narrow lanes have a lower capacity than wider lanes.

Safety Relationship

The relationship between lane width and injury (plus fatal) crash frequency can be estimated using Figure 3-13 or Equation 3-49. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 12-ft lane width.

Guidance

This AMF is applicable to four-lane highways with lane widths ranging from 9 to 12 ft. If the lane width is more than 12 ft, then the AMF value for 12 ft should be used. If the lane width is less than 9 ft, then the AMF value for 9 ft should be used. If Equation 3-49 is used, then the proportion needed is obtained from Table 3-12.

Example Application

The Question: What is the AMF for a lane width of 10 ft?

The Facts:

- Median type: undivided
- Through lanes: 4
- Lane width: 10 ft

The Solution: From Figure 3-13, for "Undivided, 4 lanes" find the AMF of 1.08. This value suggests that 10-ft lanes are associated with an 8 percent increase in crashes.

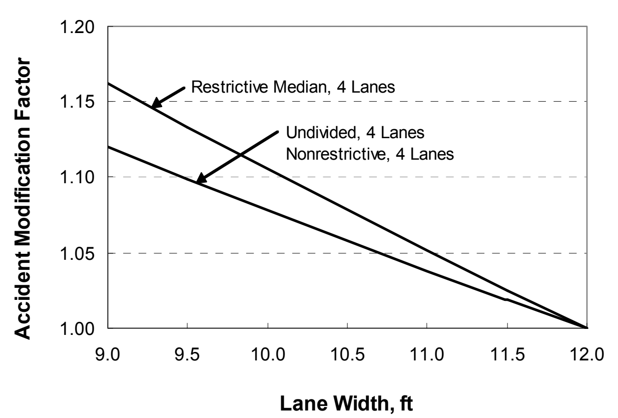

Figure 3-13. Lane Width AMF.

$$AMF_{lw} = (e^{-0.050\,(W_l - 12)} - 1.0)\frac{P_i}{0.59} + 1.0 \qquad \textbf{(3-49)}$$

where:
AMF_{lw} = lane width accident modification factor;
P_i = proportion of influential crashes of type i (see Table 3-12); and
W_l = lane width, ft.

Base Condition: 12-ft lane width

Table 3-12. Crash Distribution for Lane Width AMF.

Median Type	Through Lanes	Proportion of Crashes [1]
Restrictive	4	0.59
Undivided or nonrestrictive	2	not applicable
	4	0.44

Note:
1 - Single-vehicle run-off-road, same-direction sideswipe, and multiple-vehicle opposite direction crashes.

Outside Shoulder Width - AMF_{osw}

Discussion

Shoulders offer numerous safety benefits for rural highways. Depending on their width, shoulders may provide space for disabled vehicles, evasive maneuvers, and space within which right-turning vehicles can decelerate. Because of these safety benefits, wide outside (i.e., right-hand) shoulders are often provided on rural highways.

Safety Relationship

The relationship between outside shoulder width and injury (plus fatal) crash frequency can be estimated using Figure 3-14 or Equation 3-50. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is an 8-ft outside shoulder width.

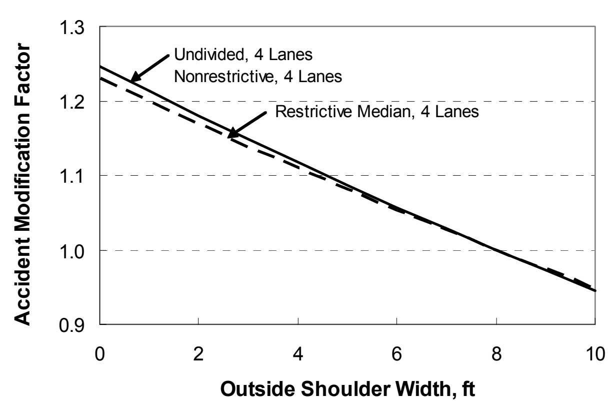

Figure 3-14. Outside Shoulder Width AMF.

$$AMF_{osw} = (e^{-0.026\,(W_s - 8)} - 1.0)\,\frac{P_i}{0.30} + 1.0 \qquad \textbf{(3-50)}$$

where:
AMF_{osw} = outside shoulder width accident modification factor;
P_i = proportion of influential crashes of type i (see Table 3-13); and
W_s = outside shoulder width, ft.

Base Condition: 8-ft outside shoulder width

Guidance

This AMF is applicable to four-lane highways having a paved outside shoulder with a width ranging from 0 to 10 ft. If the shoulder width is greater than 10 ft, then the AMF value for 10 ft should be used. If Equation 3-50 is used, then the proportion needed is obtained from Table 3-13.

Table 3-13. Crash Distribution for Outside Shoulder Width AMF.

Median Type	Through Lanes	Proportion of Crashes
Restrictive [1]	4	0.30
Undivided or nonrestrictive [2]	2	not applicable
	4	0.32

Notes:
1 - Single-vehicle run-off-road crashes, right side only.
2 - Single-vehicle run-off-road crashes, either side.

Example Application

The Question: What is the AMF for an outside shoulder width of 6 ft?

The Facts:

- Median type: undivided
- Through lanes: 4
- Outside shoulder width: 6 ft

The Solution: From Figure 3-14, find the AMF of 1.06. This value implies that a 6-ft shoulder width may be associated with 6 percent more crashes than an 8-ft shoulder.

$$AMF_{osw} = (e^{-0.026 \times (6 - 8)} - 1.0)\,\frac{0.32}{0.30} + 1.0 = 1.06 \qquad \textbf{(3-51)}$$

Inside Shoulder Width - AMF$_{isw}$

Discussion

Inside (i.e., left-hand) shoulders offer similar safety benefits for rural multilane highways as do outside shoulders. Specifically, they provide storage space for disabled vehicles and additional room for evasive maneuvers.

Safety Relationship

The relationship between inside shoulder width and injury (plus fatal) crash frequency can be estimated using Figure 3-15 or Equation 3-52. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 4-ft inside shoulder width.

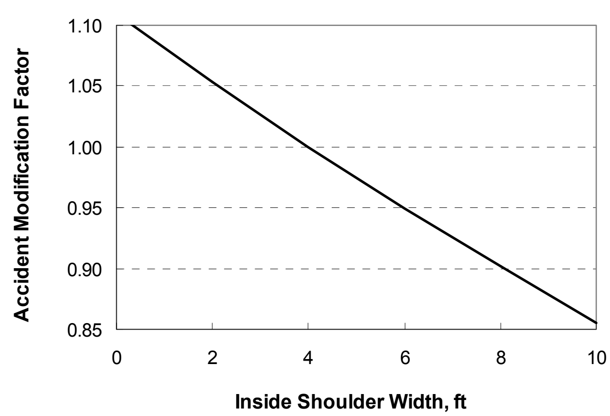

Figure 3-15. Inside Shoulder Width AMF.

$$AMF_{isw} = (e^{-0.026\,(W_{is} - 4)} - 1.0)\,\frac{P_i}{0.24} + 1.0 \qquad \textbf{(3-52)}$$

where:
AMF_{isw} = inside shoulder width accident modification factor;
P_i = proportion of influential crashes of type *i* (see Table 3-14); and
W_{is} = inside shoulder width, ft.

Base Condition: 4-ft inside shoulder width

Guidance

This AMF is applicable to four-lane highways with a restrictive median and an inside shoulder width ranging from 0 to 10 ft. If the shoulder width is greater than 10 ft, then the AMF value for 10 ft should be used. If Equation 3-52 is used, then the proportion is obtained from Table 3-14.

Table 3-14. Crash Distribution for Inside Shoulder Width AMF.

Median Type	Through Lanes	Proportion of Crashes [1]
Restrictive	4	0.24

Note:
1 - Single-vehicle run-off-road (left side only) and multiple-vehicle opposite direction crashes.

Example Application

The Question: What is the AMF for an inside shoulder width of 6 ft?

The Facts:
- Through lanes: 4
- Inside shoulder width: 6 ft

The Solution: From Figure 3-15, find the AMF of 0.95. This value implies a 5 percent reduction in crashes if a 6-ft shoulder width is used instead of a 4-ft width.

$$AMF_{isw} = (e^{-0.026 \times (6 - 4)} - 1.0)\,\frac{0.24}{0.24} + 1.0 = 0.95 \qquad \textbf{(3-53)}$$

Median Width (no barrier) - AMF$_{mwnb}$

Discussion

A median provides several functions including positive separation between opposing traffic streams, a recovery area for errant vehicles, space for left-turn bays, and control of access. The benefits derived from these functions tend to increase with wider medians. Medians on a rural highway are typically restrictive; however, a nonrestrictive median is sometimes used.

Safety Relationship

The relationship between median width and injury (plus fatal) crash frequency is shown in Figure 3-16; however, it should be estimated using Equation 3-54 or 3-55. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is stated in the box to the right.

Guidance

This AMF is applicable to four-lane highway segments with a nonrestrictive median ranging from 10 to 16 ft in width, and to those with a restrictive median ranging from 30 to 80 ft in width. It does not apply to segments with barrier in the median. Median width is measured between the near edges of the left- and right-side traveled way (i.e., it includes the width of the inside shoulders).

If there are short lengths of rigid (or semi-rigid) barrier or bridge rail in the median, then the Median Width (some barrier) AMF should be used.

If a rigid (or semi-rigid) barrier is present in the median for the length of the segment, then the Median Width (full barrier) AMF should be used.

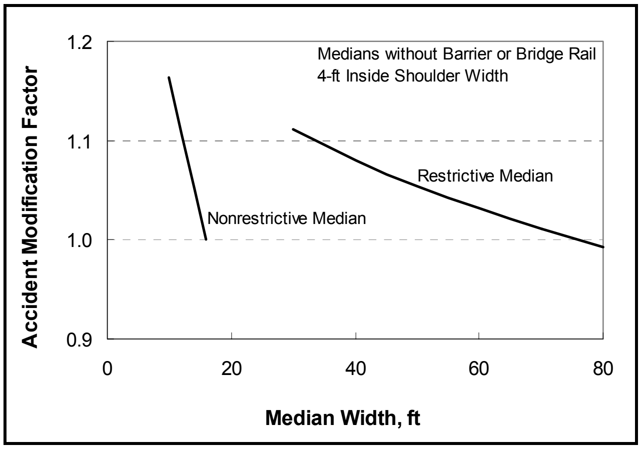

Figure 3-16. Median Width (no barrier) AMF.

For restrictive medians:

$$AMF_{mwnb} = e^{-0.0296\,([W_m - 2W_{is}]^{0.5} - 8.25)} \qquad \textbf{(3-54)}$$

For nonrestrictive medians:

$$AMF_{mwnb} = e^{-0.0253\,(W_m - 16)} \qquad \textbf{(3-55)}$$

where:

AMF_{mwnb} = median width (no barrier) accident modification factor;
W_m = median width, ft; and
W_{is} = inside shoulder width, ft.

Base Condition: 16-ft median width for nonrestrictive medians, 76-ft median width and 4-ft inside shoulder width for restrictive medians

Example Application

The Question: What is the percent decrease in crash frequency if a 12-ft median is increased to 16 ft?

The Facts:

- Median type: nonrestrictive

The Solution: From Figure 3-16, find AMFs of 1.11 and 1.00 for the 12- and 16-ft median widths, respectively. The ratio of the two AMFs indicates a 10 percent decrease in crashes (= [1.00/1.11 − 1]×100).

Median Width (some barrier) - AMF$_{mwsb}$

Discussion

Barrier may be used in the median to protect motorists from collision with a fixed object such as a sign support or bridge abutment. The barrier itself is a fixed object, but one that is designed to reduce crash severity. An increase in the number of injury and property-damage-only crashes may be observed when a barrier is used.

Safety Relationship

The relationship between barrier presence in the median and injury (plus fatal) crash frequency is shown in Figure 3-17. The AMF value should be estimated using Equation 3-56. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition is stated in the box to the right.

Guidance

This AMF is applicable four-lane highways with short lengths of barrier or bridge rail in a restrictive median. The barrier can be rigid or semi-rigid. It can be located adjacent to one roadbed or at a specified distance W_{off} from the edge of traveled way. This AMF applies to median widths of 14 ft or more. It should not be used to justify the addition or removal of barrier.

The distance from the edge of shoulder to the barrier face W_{icb} is an average for the segment considering all individual short lengths of barrier in both travel directions. If each barrier is located at the same distance W_{off}, then $W_{icb} = W_{off} - W_{is}$. Otherwise, Equation 3-58 should be used to estimate W_{icb}. Similarly, Equation 3-59 should be used to estimate the proportion of the segment length with barrier in the median. The summation term "Σ" in the denominator of Equation 3-58 indicates that the ratio of barrier length $L_{ib,off}$ to offset distance "$W_{off} - W_{is}$" is computed for each length of barrier.

If a rigid (or semi-rigid) barrier is present in the median for the length of the segment, then the Median Width (full barrier) AMF should be used.

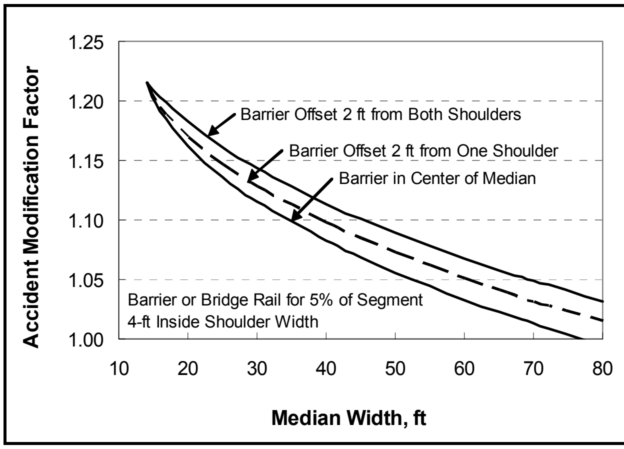

Figure 3-17. Median Width (some barrier) AMF.

$$AMF_{mwsb} = (1.0 - P_{ib})\,AMF_{mwnb} + P_{ib}\,AMF_{ib,ir} \qquad \textbf{(3-56)}$$

with,

$$AMF_{ib,ir} = e^{0.757/W_{icb} - 0.0296\,([2\,W_{icb}]^{0.5} - 8.25)} \qquad \textbf{(3-57)}$$

where:

$AMF_{ic|agg}$ = median width (some barrier) acc. modification factor;
AMF_{mwnb} = median width (no barrier) acc. modification factor;
$AMF_{ib,ir}$ = barrier or rail in median accident modification factor;
W_{icb} = width from edge of shoulder to barrier face, ft; and
P_{ib} = proportion of segment length with barrier in median.

Base Condition: median barrier not present, 76-ft median width, 4-ft inside shoulder width

$$W_{icb} = \frac{\sum L_{ib,off}}{\sum \dfrac{L_{ib,off}}{W_{off} - W_{is}}} \qquad \textbf{(3-58)}$$

$$P_{ib} = \frac{\sum L_{ib,off}}{2\,L} \qquad \textbf{(3-59)}$$

where:

$L_{ib,\,off}$ = length of inside lane paralleled by a barrier located at a distance W_{off} from the traveled way, mi;
L = segment length, mi;
W_{is} = inside shoulder width, ft; and
W_{off} = width from the edge of the traveled way to the face of a specific short length of barrier, ft.

Median Width (full barrier) - AMF_{mwfb}

Discussion

Barrier may be used with narrower medians to minimize cross-median crashes. The barrier can be located near the center of the median or nearer to one of the roadbeds. In some instances, a barrier is adjacent to both roadbeds. The barrier itself is a fixed object, but one that is designed to reduce crash severity. An increase in the number of injury and property-damage-only crashes may be observed when a barrier is used.

Safety Relationship

The relationship between median barrier presence and injury crash frequency is shown in Figure 3-18. The AMF value should be estimated using Equation 3-60. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is an 80-ft median width.

Guidance

This AMF is applicable to four-lane highways with a restrictive median and a barrier that extends the length of the segment. The barrier can be rigid or semi-rigid. It can be located in the center of the median or adjacent to a roadbed. The AMF applies to segments with a median width of 14 ft or more. It should not be used to justify the addition or removal of barrier.

The distance from the edge of shoulder to the barrier face W_{icb} is an average for the length of the segment considering both travel directions. Equation 3-61 or 3-62 should be used to estimate this distance. Both equations also account for short lengths of barrier that may exist in addition to the continuous barrier (e.g., for a sign support or bridge abutment).

If the barrier in the median does not extend for the length of the segment, then the Median Width (some barrier) AMF should be used.

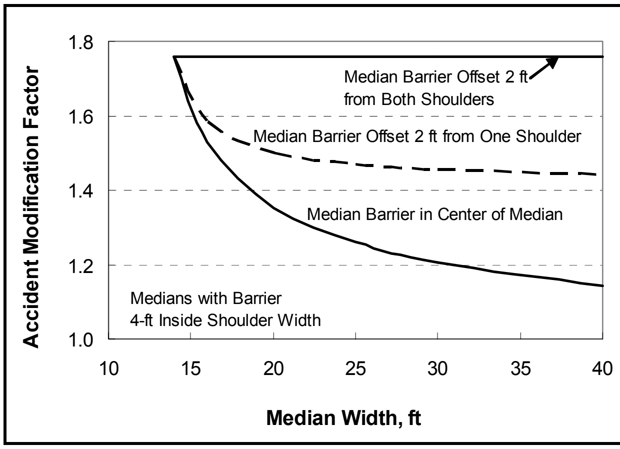

Figure 3-18. Median Width (full barrier) AMF.

$$AMF_{wmfb} = e^{0.757/W_{icb} - 0.0296([2W_{icb}]^{0.5} - 8.25)} \quad \textbf{(3-60)}$$

where:
AMF_{mwfb} = median width (full barrier) acc. modification factor;
W_{icb} = width from edge of shoulder to barrier face, ft.

Base Condition: 80-ft median width

For barrier in center of median:

$$W_{icb} = \frac{2\,L}{\sum \dfrac{L_{ib,off}}{W_{off} - W_{is}} + \dfrac{2\,L - \sum L_{ib,off}}{0.5\,(W_m - 2\,W_{is} - W_{ib})}} \quad \textbf{(3-61)}$$

For barrier adjacent to one roadbed:

$$W_{icb} = \frac{2\,L}{\dfrac{L}{2.0} + \sum \dfrac{L_{ib,off}}{W_{off} - W_{is}} + \dfrac{L - \sum L_{ib,off}}{W_m - 2\,W_{is} - W_{ib} - 2.0}} \quad \textbf{(3-62)}$$

where:
$L_{ib,\,off}$ = length of inside lane paralleled by a barrier located at a distance W_{off} from the traveled way, mi;
L = segment length, mi;
W_{off} = width from edge of the traveled way to the face of a specific short length of barrier, ft;
W_m = median width, ft;
W_{is} = inside shoulder width, ft; and
W_{ib} = inside barrier width (measured between barrier faces), ft.

Truck Presence - AMF_{tk}

Discussion

An analysis of crash data indicates that highways with higher truck percentages are associated with fewer crashes. This trend suggests that drivers may be more cautious when there are many trucks in the traffic stream.

Safety Relationship

The relationship between truck presence and injury (plus fatal) crash frequency can be estimated using Figure 3-19 or Equation 3-63. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition is 16 percent trucks.

Guidance

This AMF is applicable to four-lane highways with truck percentages ranging from 0.0 to 25 percent. It should not be used as a basis for design decisions regarding truck percentage. Rather, it should be used as a means of adjusting the base crash frequency to accurately reflect the presence of trucks on the subject highway segment.

Example Application

The Question: What is the crash frequency for a specific highway segment?

The Facts:
- Base crash frequency C_b: 1.5 crashes/yr
- Truck percentage: 10%

The Solution: The segment of interest has typical characteristics with the exception that it has 10 percent trucks. As shown in Figure 3-19, the AMF for 10 percent trucks is 1.03. This value can be used with Equation 3-15 to estimate the expected crash frequency for the subject segment as 1.55 crashes/yr.

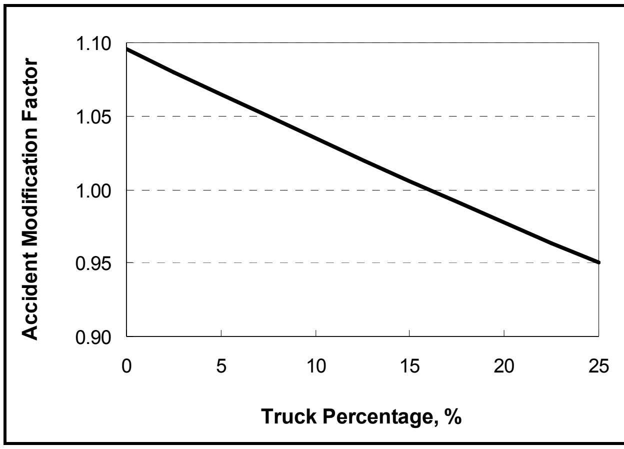

Figure 3-19. Truck Presence AMF.

$$AMF_{tk} = e^{-0.0057\,(P_t - 16)} \quad \textbf{(3-63)}$$

where:
AMF_{tk} = truck presence accident modification factor; and
P_t = percent trucks represented in ADT, %.

Base Condition: 16% trucks

$$\begin{aligned} C &= C_b \times AMF_{tk} \\ &= 1.50 \times 1.03 \\ &= 1.55 \; crashes/yr \end{aligned} \quad \textbf{(3-64)}$$

Safety Appurtenances

AMFs for comparing specific roadside safety appurtenances are not described in this document. A comprehensive procedure for evaluating appurtenances is outlined in a report by Mak and Sicking (*7*) and automated in the *Roadside Safety Analysis Program* (RSAP) (*8*). RSAP can be used to evaluate alternative roadside safety appurtenances on individual rural highway segments. The program accepts as input information about the highway segment geometry and traffic characteristics. It also allows the analyst to describe the roadside cross section, location of fixed objects, and safety appurtenance design. The output from RSAP includes an estimate of annual crash frequency as well as the road-user costs associated with these crashes. The crash reduction potential realized by adding a roadside safety appurtenance (or changing the roadside cross section) can be estimated by specifying the changed condition as an "alternative."

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Chapter 4

Urban and Suburban Arterials

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INTRODUCTION

Relative to rural highways, urban and suburban arterials are characterized by higher traffic volumes, lower speeds, densely developed adjacent land uses, limited right-of-way, shorter intersection spacing, and frequent driveways. Urban and suburban arterials are also more frequently used by non-automobile travel modes, such as truck, transit, pedestrian, and bicycle. These characteristics and varied travel modes complicate arterial street design.

The process of designing an arterial street can include an evaluation of the operational and safety benefits associated with various design alternatives, with consideration to the overall cost-effectiveness of each alternative. The importance of this evaluation increases when right-of-way is constrained, or when access to adjacent properties is adversely impacted.

The procedure described in this chapter can be used to quantify the safety associated with an existing arterial facility or with a proposed design. In this regard, safety is defined as the expected frequency of injury (plus fatal) crashes. The safety benefit of a proposed design can be obtained by comparing its expected crash frequency with that of the existing facility or of another alternative. Background information about the various equations and constants that comprise the procedure is provided in references *1, 2, 3,* and *4*. Procedures for estimating the operational or other impacts of a design alternative are beyond the scope of this *Workbook*.

PROCEDURE

This part of the chapter describes a procedure for evaluating the safety of urban and suburban arterial segments. An arterial segment is defined to be a length of roadway that is homogenous in terms of having a reasonably constant cross section, adjacent land use, and traffic demand. A new segment begins at each intersection, horizontal curve, or any significant change in cross section, median type, traffic volume, lane width, shoulder width, driveway density, or other variable addressed by an applicable accident modification factor (AMF).

A procedure for evaluating urban intersections is described in Chapter 7. This procedure can be used together with the procedure in this chapter to evaluate the safety of an urban or suburban arterial and its intersections.

The procedure described herein is based on the prediction of expected crash frequency. Specifically, the crash frequency for a typical segment is computed from a base model. This frequency is then adjusted using various AMFs to tailor the resulting estimate to a specific arterial segment. The base model includes variables for traffic volume, segment length, and access point frequency. AMFs are used to account for factors found to have some correlation with crash frequency, typically of a more subtle nature than the main factors. The AMFs are multiplied by the base crash frequency to obtain an expected crash frequency for the subject arterial segment.

The procedure described herein differs from that developed by Harwood et al. (*5*) because this procedure predicts injury (plus fatal) crash frequency, as opposed to total crash frequency. Otherwise, the procedure described herein is similar and shares the same strengths and weaknesses.

Base crash prediction models are described in the next section. The section that follows describes the AMFs to be used with these models. Example applications are provided throughout this *Workbook* to illustrate the use of the base models and the AMFs.

Base Models

Discussion

An examination of crash trends for arterials indicates that the crash rate varies with the number of lanes, median type, and the adjacent land use (*3*). In general, crash rates are lower for arterials with many lanes than those with few lanes. Also, arterials with a restrictive median (i.e., raised-curb or depressed median) tend to have a lower crash rate than arterials with a nonrestrictive median (i.e., two-way left-turn lane [TWLTL] or flush-paved median) or undivided cross section. Crash rates also tend to be higher in areas with commercial, business, or office land uses, relative to rates in residential or industrial areas. This influence is likely a reflection of more frequent driveway activity in commercial, business, and office areas.

Safety Relationship

The relationship between crash frequency and traffic demand for base arterial conditions is shown in Figure 4-1. The trends shown in this figure apply to arterial segments that are one mile long and located in a residential area. Equations 4-1 through 4-30 should be used for other conditions.

Equations 4-1 through 4-30 are used to compute the expected crash frequency for arterial segments. Equations 4-1 and 4-11 are used for two- and four-lane undivided arterials, respectively. Equations 4-7, 4-15, and 4-23 are used for two-, four-, and six-lane arterials with a nonrestrictive median. Equations 4-19 and 4-27 are used with four- and six-lane arterials with a restrictive median. Each equation consists of three component equations that separately predict multiple-vehicle (non-driveway), single-vehicle, and driveway-related crashes.

Table 4-1 lists the over-dispersion parameter *k* for each equation. The use of this parameter is described in reference *6*.

Guidance

The crash frequency obtained from a base model is applicable to segments having base conditions. These conditions generally represent

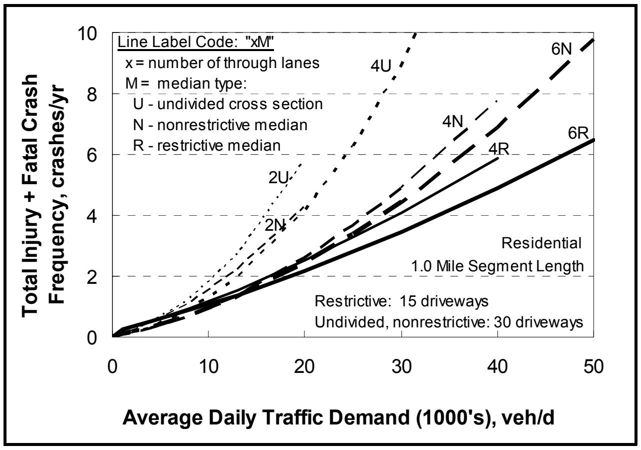


Figure 4-1. Illustrative Arterial Crash Trends.

Two-lane, undivided segments:

$$C_{b,2u} = (C_{mv,2u} + C_{sv,2u} + C_{dw,2u})\ f_{2u} \quad \textbf{(4-1)}$$

with,

$$C_{mv,2u} = 0.00362\ (0.001\ ADT)^{2.31}\ L\ F_{lu} \quad \textbf{(4-2)}$$

$$C_{sv,2u} = 0.0399\ (0.001\ ADT)^{1.06}\ L\ F_{lu} \quad \textbf{(4-3)}$$

$$C_{dw,2u} = 0.120\ (ADT/15000)^{1.04}\ n_e\ S_d^{0.518} \quad \textbf{(4-4)}$$

$$n_e = (n_{res} + 1.32\,n_{ind} + 4.11\,n_{bus} + 2.91\,n_{off}) \quad \textbf{(4-5)}$$

$$F_{lu} = e^{(0.210\,L_{ind} + 0.448\,L_{bus} + 0.113\,L_{off})/L} \quad \textbf{(4-6)}$$

where:

C_b = base injury (plus fatal) crash frequency, crashes/yr;
C_{mv} = multiple-vehicle non-driveway crash freq., crashes/yr;
C_{sv} = single-vehicle crash frequency, crashes/yr;
C_{dw} = driveway-related crash frequency, crashes/yr;
f = local calibration factor;
ADT = average daily traffic volume, veh/d;
L = segment length, mi;
n_e = number of equivalent residential driveways;
n_{res} = number of driveways serving residential land uses;
n_{ind} = number of driveways serving industrial land uses;
n_{bus} = number of driveways serving business land uses;
n_{off} = number of driveways serving office land uses;
S_d = driveway spacing (= 2 L / [n_{res} + n_{ind} + n_{bus} + n_{off} + 1.0]), mi/driveway;
F_{lu} = land use adjustment factor;
L_{ind} = curb miles with industrial land use (two-way total), mi;
L_{off} = curb miles with office land use (two-way total), mi; and
L_{bus} = curb miles with business land use (two-way total), mi.

Two-lane, nonrestrictive median segments:

$$C_{b,2n} = (C_{mv,2n} + C_{sv,2n} + C_{dw,2n})\ f_{2n} \quad \textbf{(4-7)}$$

with,

$$C_{mv,2n} = 0.0116\ (0.001\ ADT)^{1.82}\ L\ F_{lu} \quad \textbf{(4-8)}$$

$$C_{sv,2n} = 0.0700\ (0.001\ ADT)^{0.630}\ L\ F_{lu} \quad \textbf{(4-9)}$$

$$C_{dw,2n} = 0.103\ (ADT/15000)^{1.29}\ n_e\ S_d^{0.518} \quad \textbf{(4-10)}$$

uncomplicated geometry, straight alignment, and typical cross section elements. The complete set of base conditions is identified in Table 4-2.

If a particular segment has characteristics that differ from the base conditions, then the AMFs described in the next section can be used to obtain a more accurate estimate of segment crash frequency.

Equations 4-4, 4-10, 4-14, 4-18, 4-22, 4-26, and 4-30 should not be used to evaluate the effect of adding or removing one driveway. Rather, the procedure described in Chapter 7 should be used for this purpose.

A local calibration factor is shown for each of the equations. The factor can be used to adjust the computed value so that it is more consistent with typical highways in the agency's jurisdiction. A calibration procedure is identified in reference *4*. A calibration factor of 1.0 should be used unless a local calibration indicates another value is more appropriate.

Land Use and Driveway Count

The land use served by a driveway is categorized as residential, industrial, business, or office. Analysis indicates that driveway volume and land use are highly correlated. In recognition of this correlation, Equation 4-6 uses land use as a convenient surrogate for driveway traffic volume because these data are not generally available. Table 4-3 can be used to determine the land use associated with each driveway along the subject arterial segment.

Two types of driveway are recognized in the count of driveways. A "full driveway" allows left and right turns in and out of the property. A "partial driveway" allows only right turns in and out of the property. When counting driveways along a segment, a full driveway is counted as "1" driveway, and a partial driveway is counted as "0.5" driveways. Partial driveways are most commonly found on segments with a restrictive median.

Driveways that are unused should not be counted. Similarly, driveways leading into fields, small utility installations (e.g., cellular phone tower), and abandoned buildings should

Four-lane, undivided segments:

$$C_{b,4u} = (C_{mv,4u} + C_{sv,4u} + C_{dw,4u})\ f_{4u} \qquad \textbf{(4-11)}$$

with,

$$C_{mv,4u} = 0.00255\ (0.001\ ADT)^{2.31}\ L\ F_{lu} \qquad \textbf{(4-12)}$$

$$C_{sv,4u} = 0.0236\ (0.001\ ADT)^{1.06}\ L\ F_{lu} \qquad \textbf{(4-13)}$$

$$C_{dw,4u} = 0.102\ (ADT/15000)^{1.04}\ n_e\ S_d^{0.518} \qquad \textbf{(4-14)}$$

Four-lane, nonrestrictive median segments:

$$C_{b,4n} = (C_{mv,4n} + C_{sv,4n} + C_{dw,4n})\ f_{4n} \qquad \textbf{(4-15)}$$

with,

$$C_{mv,4n} = 0.00645\ (0.001\ ADT)^{1.82}\ L\ F_{lu} \qquad \textbf{(4-16)}$$

$$C_{sv,4n} = 0.0461\ (0.001\ ADT)^{0.630}\ L\ F_{lu} \qquad \textbf{(4-17)}$$

$$C_{dw,4n} = 0.0740\ (ADT/15000)^{1.29}\ n_e\ S_d^{0.518} \qquad \textbf{(4-18)}$$

Four-lane, restrictive median segments:

$$C_{b,4r} = (C_{mv,4r} + C_{sv,4r} + C_{dw,4r})\ f_{4r} \qquad \textbf{(4-19)}$$

with,

$$C_{mv,4r} = 0.0236\ (0.001\ ADT)^{1.38}\ L\ F_{lu} \qquad \textbf{(4-20)}$$

$$C_{sv,4r} = 0.193\ (0.001\ ADT)^{0.201}\ L\ F_{lu} \qquad \textbf{(4-21)}$$

$$C_{dw,4r} = 0.0897\ (ADT/15000)^{1.25}\ n_e\ S_d^{0.518} \qquad \textbf{(4-22)}$$

Six-lane, nonrestrictive median segments:

$$C_{b,6n} = (C_{mv,6n} + C_{sv,6n} + C_{dw,6n})\ f_{6n} \qquad \textbf{(4-23)}$$

with,

$$C_{mv,6n} = 0.00527\ (0.001\ ADT)^{1.82}\ L\ F_{lu} \qquad \textbf{(4-24)}$$

$$C_{sv,6n} = 0.0609\ (0.001\ ADT)^{0.630}\ L\ F_{lu} \qquad \textbf{(4-25)}$$

$$C_{dw,6n} = 0.0734\ (ADT/15000)^{1.29}\ n_e\ S_d^{0.518} \qquad \textbf{(4-26)}$$

Six-lane, restrictive median segments:

$$C_{b,6r} = (C_{mv,6r} + C_{sv,6r} + C_{dw,6r})\ f_{6r} \qquad \textbf{(4-27)}$$

with,

$$C_{mv,6r} = 0.0197\ (0.001\ ADT)^{1.38}\ L\ F_{lu} \qquad \textbf{(4-28)}$$

$$C_{sv,6r} = 0.244\ (0.001\ ADT)^{0.201}\ L\ F_{lu} \qquad \textbf{(4-29)}$$

$$C_{dw,6r} = 0.0657\ (ADT/15000)^{1.25}\ n_e\ S_d^{0.518} \qquad \textbf{(4-30)}$$

Table 4-1. Over-Dispersion Parameters.

Crash Type	Over-Dispersion Parameter (*k*)
Multiple-vehicle	4.98 mi^{-1}
Single-vehicle	6.12 mi^{-1}
Driveway-related	1.89

not be counted. A circular driveway at a residence is counted as one driveway even though both ends of the driveway intersect the subject segment. Similarly, a small business (e.g., gas station) that has two curb cuts separated by only 10 or 20 ft is considered to have effectively one driveway.

Example Application

The Question: What is the expected crash frequency for a typical four-lane, undivided arterial street segment?

The Facts:

- Median type: undivided
- Land use: business
- Driveways: 30
- Segment length: 1.0 mi
- ADT: 20,000 veh/d

The Solution: From Equations 4-11 to 4-14, find that the typical arterial segment with these characteristics experiences 11.7 crashes/yr (6.3 multiple-vehicle, 1.3 single-vehicle, and 4.1 driveway-related crashes). These crashes are designated as either injury or fatal.

Table 4-2. Base Conditions.

Characteristic	Base Condition
Horizontal curve radius	tangent (no curve)
Lane width	12 ft
Shoulder width [1]	1.5 ft (curb-and-gutter)
Median width [2]	12 ft for nonrestrictive median 16 ft for restrictive median
Curb parking	none
Utility pole density and offset	50 poles/mi 2.0 ft average offset
Truck presence	6% trucks

Note:
1 - Curb-and-gutter is assumed as typical. Width shown is an "effective" shoulder width for curb-and-gutter.
2 - Nonrestrictive median: TWLTL or flush-paved median. Restrictive median: raised-curb or depressed median.

Table 4-3. Adjacent Land Use Characteristics.

Land Use	Characteristics	Examples
Residential or Undeveloped	• Buildings are small • A small percentage of the land is paved • If driveways exist, they have very low volume • Ratio of land-use acreage to parking stalls is large	• Single-family home • Undeveloped property, farmland • Graveyard • Park or green-space recreation area
Industrial	• Buildings are large and production oriented • Driveways and parking may be designed to accommodate large trucks • Driveway volume is moderate at shift change times and is low throughout the day • Ratio of land-use acreage to parking stalls is moderate	• Factory • Warehouse • Storage tanks • Farmyard with barns and machinery
Commercial Business	• Buildings are larger and separated by <u>convenient</u> parking between building and roadway • Driveway volume is moderate from mid-morning to early evening • Ratio of land-use acreage to parking stalls is small	• Strip commercial, shopping mall • Apartment complex, trailer park • Airport • Gas station • Restaurant
Office	• Buildings typically have two or more stories • Most parking is distant from the building or behind it • Driveway volume is high at morning and evening peak traffic hours; otherwise, it is low • Ratio of land-use acreage to parking stalls is small	• Office tower • Public building, school • Church • Clubhouse (buildings at a park) • Parking lot for "8 to 5" workers

Accident Modification Factors

Discussion

This section describes AMFs that can be used to evaluate the relationship between a design change and the corresponding change in injury (plus fatal) crash frequency. Topics addressed are listed in Table 4-4. There are many additional factors, other than those listed in Table 4-4, that are likely to have some influence on crash frequency. However, their relationship has yet to be quantified through research. The list of available AMFs for urban and suburban arterials is likely to increase as new research in this area is undertaken.

All of the AMFs described in this section yield a value of 1.0 when the associated design component or element represents base conditions. A condition that is more generous (i.e., desirable) than the base condition results in an AMF of less than 1.0. A condition that is less generous will result in an AMF of more than 1.0.

Safety Relationship

The expected injury (plus fatal) crash frequency for a specific arterial segment is computed using Equation 4-31. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are different from base conditions.

Guidance

In application, all applicable AMFs should be quantified for the subject segment and then multiplied together. The base crash frequency C_b for arterials is obtained from Equation 4-1, 4-7, 4-11, 4-15, 4-19, 4-23, or 4-27. The product of the AMFs and C_b represents the expected injury (plus fatal) crash frequency for the subject arterial segment.

If the crash history is available for the segment, then the over-dispersion parameters in Table 4-1

Table 4-4. AMFs for Arterial Segments.

Application	Accident Modification Factor
Geometric design	Horizontal curve radius Lane width Shoulder width Median width Curb parking
Roadside design	Utility pole offset
Roadway environment	Truck presence

$$C = C_b \times AMF_{lw} \times AMF_{cr} \ldots \qquad \textbf{(4-31)}$$

where:
- C = expected injury (plus fatal) crash frequency, crashes/yr;
- C_b = base injury (plus fatal) crash frequency, crashes/yr;
- AMF_{lw} = lane width accident modification factor; and
- AMF_{cr} = horizontal curve radius accident modification factor.

can be used with the empirical Bayes adjustment procedure described in reference *6* to increase the accuracy of the expected crash frequency (over that obtained from Equation 4-31).

Example Application

The Question: What is the expected crash frequency for a specific four-lane, undivided urban arterial?

The Facts:

- Through lanes: 4
- Median type: undivided
- Adjacent land use: business
- Segment length: 1.0 mi
- ADT: 20,000 veh/d
- Base crash frequency C_b: 11.7 crashes/yr
- Average lane width: 10 ft

The Solution: The segment of interest has typical characteristics with the exception that its average lane width is 10 ft. As described later, the AMF for a lane width of 10 ft is 1.06. This AMF can be used with Equation 4-31 to estimate the expected crash frequency for the subject segment as 12.4 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 11.7 \times 1.06 \\ &= 12.4 \; crashes/yr \end{aligned} \tag{4-32}$$

Horizontal Curve Radius - AMF_{cr}

Discussion

Larger radius horizontal curves improve safety in several ways. The larger radius increases the margin of safety against vehicle crash by rollover or slide out. The larger radius is often accompanied by an improved preview distance of the road ahead and, thereby, more driver sight distance. When a curve of near-minimum radius is used, the designer should ensure that adequate sight distance is available around retaining walls and embankments.

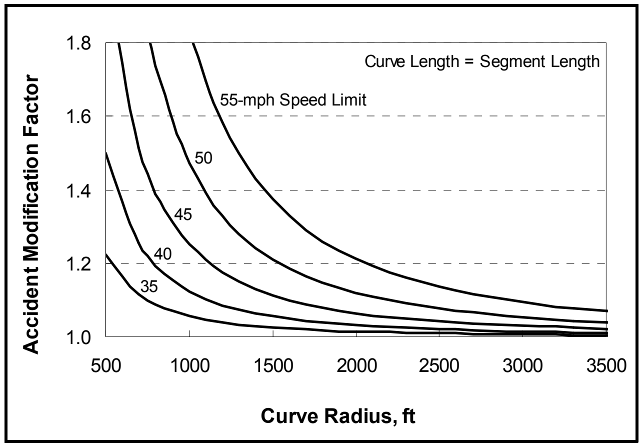


Figure 4-2. Horizontal Curve Radius AMF.

Safety Relationship

The relationship between curve radius and injury (plus fatal) crash frequency can be estimated using Equation 4-33. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a tangent roadway section (i.e., infinite radius). Thus, the AMF yields a value of 1.0 when the radius is infinite.

$$AMF_{cr} = 1.0 + 0.97\,(0.147\,V)^4 \,\frac{(1.47\,V)^2}{32.2\,R^2}\left(\frac{L_c}{L}\right) \qquad \textbf{(4-33)}$$

where:
AMF_{cr} = horizontal curve radius accident modification factor;
V = speed limit, mph;
L_c = horizontal curve length, mi;
L = segment length, mi; and
R = curve radius, ft.

Base Condition: tangent section

Guidance

This AMF is applicable to any curve with a radius that exceeds 500 ft and that corresponds to an AMF value of 1.8 or less when the ratio L_c/L is set to 1.0. The speed limit variable in the AMF is used only as a surrogate for the actual operating speed. Research indicates that a change in speed limit is rarely accompanied by an equivalent change in operating speed. As such, this AMF should not be used as a basis for decisions regarding a proposed change in speed limit.

Example Application

The Question: What is the AMF for a proposed horizontal curve on an arterial street?

The Facts: Radius: 900 ft. Speed limit: 40 mph. Curve length: 0.15 mi. Segment lenth: 0.15 mi.

The Solution: From Figure 4-2, find the AMF of 1.15. This value suggests that 15 percent more crashes may occur on this curve, relative to a tangent section.

Lane Width - AMF_{lw}

Discussion

It is generally recognized that lane width has some influence on driving comfort and efficiency. A narrow lane reduces the lateral clearance to vehicles in adjacent lanes and is most notable when large trucks are present in the traffic stream. Research indicates that narrow lanes have a lower capacity than wider lanes.

Safety Relationship

The relationship between lane width and injury (plus fatal) crash frequency can be estimated using Figure 4-3 or Equation 4-34. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 12-ft lane width.

Guidance

This AMF is applicable to lane widths ranging from 9 to 12 ft. If lane width is more than 12 ft, then the AMF value for 12 ft should be used. If Equation 4-34 is used, then the proportion needed is obtained from Table 4-5.

Example Application

The Question: What is the AMF for a lane width of 10 ft?

The Facts:
- Median type: undivided
- Through lanes: 4
- Lane width: 10 ft

The Solution: From Figure 4-3 for "Undivided, 4 lanes," find the AMF of 1.06. This value implies a 6 percent increase in crashes.

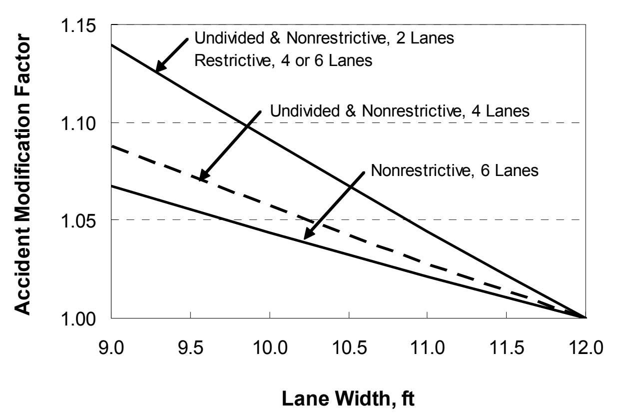


Figure 4-3. Lane Width AMF.

$$AMF_{lw} = (e^{-0.042\,(W_l - 12)} - 1.0)\,\frac{P_i}{0.26} + 1.0 \qquad \textbf{(4-34)}$$

where:
AMF_{lw} = lane width accident modification factor;
P_i = proportion of influential crashes of type i (see Table 4-5); and
W_l = lane width, ft.

Base Condition: 12-ft lane width

Table 4-5. Crash Distribution for Lane Width AMF.

Median Type	Through Lanes	Proportion of Crashes [1]
Undivided or Nonrestrictive	2	0.27
	4	0.17
Nonrestrictive	6	0.13
Restrictive	4	0.26
	6	0.26

Note:
1 - Single-vehicle run-off-road, same-direction sideswipe, and multiple-vehicle opposite direction crashes.

Shoulder Width - AMF_{sw}

Discussion

Shoulders offer numerous safety benefits for arterials. Depending on their width, shoulders may provide space for disabled vehicles, bicycle traffic, evasive maneuvers, and space within which right-turning vehicles can decelerate. In urban areas, the need to control access and drainage is often facilitated by the use of curb along the outside edge of the shoulder.

Safety Relationship

The relationship between shoulder width and injury (plus fatal) crash frequency can be estimated using Figure 4-4 or Equation 4-35. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 1.5-ft shoulder width.

Guidance

This AMF is applicable to shoulder widths ranging from 0 to 5 ft. If shoulder width is more than 5 ft, then the AMF value for 5 ft should be used. A curb-and-gutter section can be assumed to have a 1.5-ft "effective" shoulder width. If Equation 4-35 is used, then the proportion needed is obtained from Table 4-6.

Example Application

The Question: What is the AMF for a shoulder width of 4 ft?

The Facts:
- Median type: undivided
- Through lanes: 4

The Solution: From Figure 4-4 for "Undivided, 4 Lanes," find the AMF of 0.93. This value implies that crashes may be reduced by about 7 percent if a 4-ft shoulder is included in the cross section.

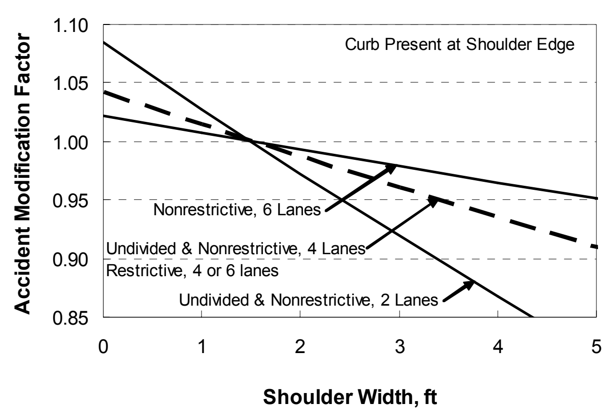


Figure 4-4. Shoulder Width AMF.

$$AMF_{sw} = \left(e^{-0.032\,(W_s - 1.5)} - 1.0\right)\frac{P_i}{0.11} + 1.0 \qquad \textbf{(4-35)}$$

where:
AMF_{sw} = shoulder width accident modification factor;
P_i = proportion of influential crashes of type *i* (see Table 4-6); and
W_s = shoulder width, ft.

Base Condition: 1.5-ft shoulder width

Table 4-6. Crash Distribution for Shoulder Width AMF.

Median Type	Through Lanes	Proportion of Crashes
Undivided or Nonrestrictive [1]	2	0.19
	4	0.094
Nonrestrictive [1]	6	0.050
Restrictive [2]	4	0.11
	6	0.080

Notes:
1 - Single-vehicle run-off-road crashes, either side.
2 - Single-vehicle run-off-road crashes, right side.

$$AMF_{sw} = \left(e^{-0.032 \times (4 - 1.5)} - 1.0\right)\frac{0.094}{0.11} + 1.0 = 0.93 \qquad \textbf{(4-36)}$$

Median Width - AMF_{mw}

Discussion

A median provides several functions including positive separation between opposing traffic streams, space for left-turn bays, refuge for pedestrians, and control of access. The benefits derived from these functions tend to increase with wider medians. Restrictive medians (i.e., raised-curb or depressed median) can also improve the aesthetics of the arterial environment. However, aesthetic treatments installed in the median should be designed to allow good visibility and be "forgiving" if impacted by errant vehicles.

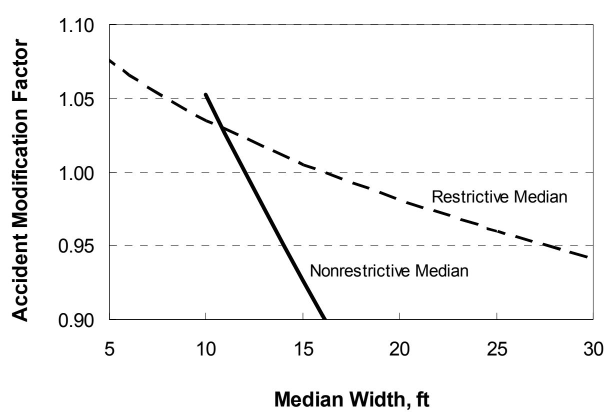


Figure 4-5. Median Width AMF.

Safety Relationship

The relationship between median width and injury (plus fatal) crash frequency can be estimated using Figure 4-5, Equation 4-37, or Equation 4-38. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is stated in the box to the right.

For restrictive medians:

$$AMF_{mw} = e^{-0.041\,(W_m^{0.5} - 16^{0.5})} \qquad \textbf{(4-37)}$$

For nonrestrictive medians:

$$AMF_{mw} = e^{-0.0255\,(W_m - 12)} \qquad \textbf{(4-38)}$$

where:
AMF_{mw} = median width accident modification factor; and
W_m = median width, ft.

Base Condition: 16-ft median width for restrictive medians, 12-ft median width for nonrestrictive medians

Guidance

This AMF is applicable to arterials with restrictive median widths ranging from 4 to 30 ft and those with nonrestrictive median widths ranging from 10 to 17 ft. It does not apply to arterials with barrier in the median. Median width is measured between the near edges of the left- and right-side traveled way (i.e., it includes the width of the inside shoulders). TWLTLs and other surfaced medians are considered to be nonrestrictive.

Example Application

The Question: What may be the percent increase in crash frequency if a 24-ft median is reduced to 12 ft?
The Facts: Median type: raised curb
The Solution: From Figure 4-5, find the AMF of 0.96 for the 24-ft median. Also, find the AMF of 1.02 for the 12-ft median. The ratio of these two AMFs indicates a 6.2 percent (= 100×[1.02/0.69 − 1]) increase in crash frequency.

Curb Parking - AMF_{pk}

Discussion

The provision of parallel or angle curb parking is an important consideration in arterial street design. Curb parking offers convenient, and sometimes essential, access to adjacent property. However, parking maneuvers may increase crash potential, especially if angle parking is provided. Crash frequency increases with the frequency of parking maneuvers. This frequency is often difficult to quantify but has been found to be highly correlated with adjacent land use. Business and office land uses are often found to be associated with more frequent parking than residential or industrial areas.

Safety Relationship

The relationship between curb parking and injury (plus fatal) crash frequency can be estimated using Figure 4-6 or Equation 4-39. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

The proportion of segment length with curb parking should be based on an assessment of both curb faces. For example, a 0.12-mi segment with parking along 0.10 mi of one curb face and along 0.05 mi of the other face has 0.15 mi (= 0.10 + 0.05) allocated to curb parking. The proportion of segment with curb parking P_{pk} is 0.625 (= 0.5 × 0.15/0.12).

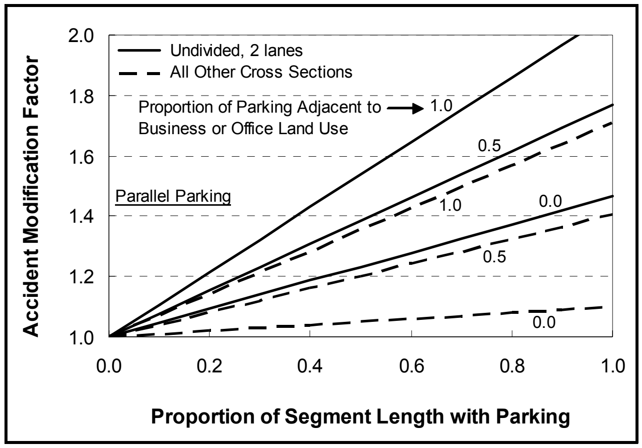
Example Application

The Question: What percent increase in crashes may result if a two-lane undivided street is widened to add parallel parking?

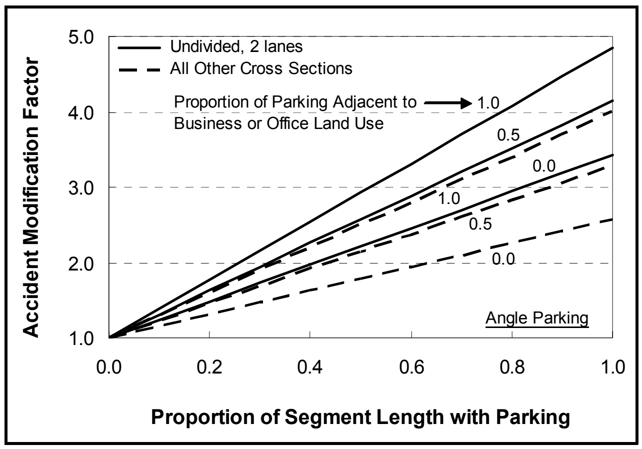
The Facts:

- Land use for that part of segment with parking: 0.50 business, 0.50 residential
- Proportion of segment with parking: 0.50

The Solution: From Figure 4-6a, find the AMF of 1.38, which equates to a 38-percent increase.



a. Parallel Parking.



b. Angle Parking.

Figure 4-6. Curb Parking AMF.

Base Condition: no parking

$$AMF_{pk} = 1 + P_{pk}\,(B_{pk} - 1) \qquad \textbf{(4-39)}$$

with,

$$B_{pk} = (1.10 + 0.365\,I_{nl} + 0.609\,P_{b/o})\,[1.34\,P_{ap} + 1.0] \qquad \textbf{(4-40)}$$

where:

AMF_{pk} = curb parking accident modification factor;
P_{pk} = proportion of segment length with parallel or angle parking (= 0.5 L_{pk} / L);
L_{pk} = curb miles allocated to parking (two-way total), mi;
I_{nl} = indicator variable for cross section (1 for two-lane undivided arterial; 0 otherwise);
$P_{b/o}$ = for that part of the segment with parking, the proportion that has business or office as an adjacent land use; and
P_{ap} = for that part of the segment with parking, the proportion with angle parking.

Utility Pole Offset - AMF_{pd}

Discussion

Utility poles are often identified as the first object struck by errant vehicles. Removal of these poles, or their relocation to a more distant offset from the arterial, is desirable when conditions allow. Research has shown that such relocation significantly reduces the frequency of pole-related crashes.

Safety Relationship

The relationship between utility pole presence and injury (plus fatal) crash frequency can be estimated using Figure 4-7 or Equation 4-41. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a pole offset of 2.0 ft and a pole density of 50 poles/mi.

Guidance

This AMF is applicable to utility or luminaire poles with densities ranging from 20 to 70 poles/mi and pole offsets ranging from 1 to 30 ft. Traffic volume and pole density have a small effect on the AMF value. If Equation 4-41 is used, then the proportion needed is obtained from Table 4-7.

Example Application

The Question: What percent reduction in crashes is likely to be realized if pole offset is increased from 2.0 to 15 ft?

The Facts:

- Median type: undivided
- Through lanes: 4
- Pole density: 50 poles/mi
- ADT: 10,000 veh/d

The Solution: From Figure 4-7, find the AMF of 0.98 (rounded from 0.979).

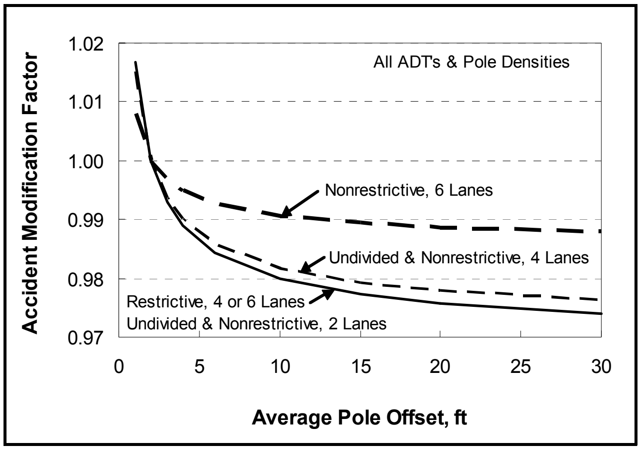


Figure 4-7. Utility Pole Offset AMF.

$$AMF_{pd} = (f_p - 1.0)\, P_i + 1.0 \tag{4-41}$$

with,

$$f_p = \frac{(0.0000984\, ADT + 0.0354\, D_p)\, W_o^{-0.6} - 0.04}{0.0000649\, ADT + 1.128} \tag{4-42}$$

where:
AMF_{pd} = utility pole offset accident modification factor;
P_i = proportion of influential crashes of type i (see Table 4-7);
D_p = utility pole density (two-way total), poles/mi; and
W_o = average pole offset from nearest edge of traveled way, ft.

Base Conditions: 2.0-ft pole offset and 50 poles/mi

Table 4-7. Crash Distribution for Utility Pole Offset AMF.

Median Type	Through Lanes	Proportion of Crashes [1]
Undivided or Nonrestrictive	2	0.032
	4	0.029
Nonrestrictive	6	0.015
Restrictive	4	0.035
	6	0.033

Note:
1 - Single-vehicle-with-pole crashes.

Truck Presence - AMF_{tk}

Discussion

An analysis of truck crash data indicates that arterials with higher truck percentages are associated with fewer crashes. This trend suggests that drivers may be more cautious when there are many trucks in the traffic stream.

Safety Relationship

The relationship between truck presence and injury (plus fatal) crash frequency can be estimated using Figure 4-8 or Equation 4-43. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition is 6 percent trucks.

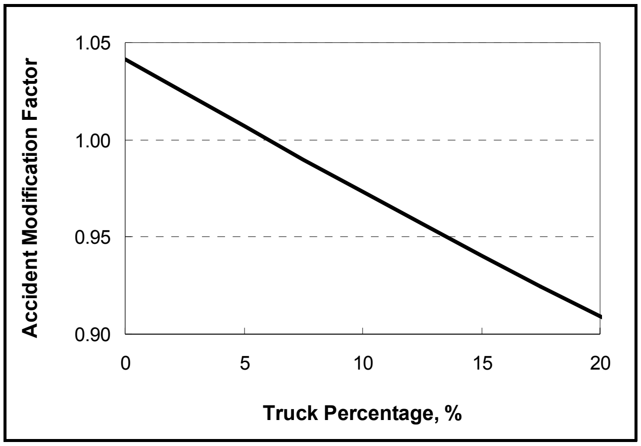


Figure 4-8. Truck Presence AMF.

$$AMF_{tk} = e^{-0.0068\,(P_t - 6)} \qquad \textbf{(4-43)}$$

where:
AMF_{tk} = truck presence accident modification factor; and
P_t = percent trucks represented in ADT, %.

Base Condition: 6% trucks

Guidance

This AMF is applicable to truck percentages ranging from 0.0 to 20 percent. It should not be used as a basis for design decisions regarding truck percentage. Rather, it should be used as a means of adjusting the base crash frequency to accurately reflect the presence of trucks on the subject arterial segment.

Example Application

The Question: What is the crash frequency for a specific four-lane urban arterial?

The Facts:
- Base crash frequency C_b: 1.5 crashes/yr
- Truck percentage: 20%

The Solution: The segment of interest has typical characteristics with the exception that it has 20 percent trucks. As shown in Figure 4-8, the AMF for 20 percent trucks is 0.91. This value can be used with Equation 4-31 to estimate the expected crash frequency for the subject segment as 1.4 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{tk} \\ &= 1.5 \times 0.91 \\ &= 1.4 \; crashes/yr \end{aligned} \qquad \textbf{(4-44)}$$

Safety Appurtenances

AMFs for comparing specific roadside safety appurtenances are not described in this document. A comprehensive procedure for evaluating appurtenances is outlined in a report by Mak and Sicking (*7*) and automated in the *Roadside Safety Analysis Program* (RSAP) (*8*). RSAP can be used to evaluate alternative roadside safety appurtenances on individual arterial segments. The program accepts as input information about the arterial segment geometry and traffic characteristics. It also allows the analyst to describe the roadside cross section, location of fixed objects, and safety appurtenance design. The output from RSAP includes an estimate of annual crash frequency as well as the road-user costs associated with these crashes. The crash reduction potential realized by adding a roadside safety appurtenance (or changing the roadside cross section) can be estimated by specifying the changed condition as an "alternative."

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6. Bonneson, J., and K. Zimmerman. *Procedure for Using Accident Modification Factors in the Highway Design Process.* Report No. FHWA/TX-07/0-4703-P5. Texas Department of Transportation, Austin, Texas, February 2007.

7. Mak, K., and D.L. Sicking. *NCHRP Report 492: Roadside Safety Analysis Program (RSAP) – Engineer's Manual.* National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C., 2003.

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Chapter 5

Interchange Ramps and Frontage Roads

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INTRODUCTION

Access to and from grade-separated facilities is obtained by way of interchange ramps. These ramps are essentially free-flow facilities with one or more lanes that allow ramp traffic to merge with freeway traffic while maintaining a relatively high speed. Ramps can connect two freeway facilities, a freeway to an arterial, or a freeway to a frontage road. Ramps are configured in a variety of shapes to accommodate heavy turn movements and topography. They are associated with a speed change that occurs along their length, often coupled with horizontal curves and grade changes. These attributes complicate the ramp driving task.

Frontage roads serve many functions, such as: provide freeway access control, provide access to properties adjacent to the freeway, and preserve the safety and capacity of the freeway. They also tend to constrain ramp design and increase traffic demand at the intersection of the crossroad and frontage road. One-way frontage roads tend to be favored over two-way frontage roads based on safety and operational considerations.

The procedure described in this chapter can be used to quantify the safety associated with existing or proposed designs. In this regard, safety is defined as the expected frequency of injury (plus fatal) crashes. The safety benefit of a proposed design can be obtained by comparing its expected crash frequency with that of the existing facility or of another alternative. Background information about the various equations and constants that comprise the procedure is provided in references *1*, *2*, *3*, and *4*. Procedures for estimating the operational or other impacts of a design alternative are beyond the scope of this *Workbook*.

PROCEDURE

This part of the chapter describes a procedure for evaluating the safety of ramp segments and a procedure for evaluating frontage road segments. The segments addressed herein do not include the intersections or ramp terminals.

The procedures in Chapter 2 can be used to evaluate the speed-change lane at the junction of a ramp and freeway mainlane segment.

The procedures in Chapters 6 and 7 can be used to evaluate ramp-crossroad or ramp-frontage-road intersections. They were not explicitly developed for evaluating these intersection types; however, they can still be used if their results are carefully reviewed for reasonableness.

The procedure described herein is based on the prediction of expected crash frequency. Separate base models are provided for the calculation of crash frequency for a ramp segment or a frontage road segment. The base model for ramps includes a sensitivity to traffic volume, ramp type, and ramp configuration. Currently, no accident modification factors (AMFs) are available for use with the ramp segment base model. The base model for frontage roads includes variables for traffic volume and segment length. AMFs are available for this model to account for factors found to have some correlation with crash frequency. The AMFs are multiplied by the base crash frequency to estimate the expected crash frequency for a given frontage road segment.

The procedure described herein differs from those developed by Harwood et al. (*5*) because this procedure predicts injury (plus fatal) crash frequency, as opposed to total crash frequency. Otherwise, the procedures described herein are similar and share the same strengths and weaknesses.

Base crash prediction models for interchange ramps are described in the next section. A subsequent section describes the models for frontage roads. Example applications are provided throughout this *Workbook* to illustrate the use of the base models and the AMFs.

Base Models – Interchange Ramps

Discussion

A wide range of ramp configurations are in use at interchanges. This variation is due to the unique traffic and topographic constraints placed on ramp design at each interchange location. However, most ramps can be placed into one of nine basic configurations. Exit ramp variations of each of the basic configurations are illustrated in Figure 5-1. Entrance ramp versions have a similar alignment. Of the ramps shown, the button hook, scissors, and slip ramps are used at interchanges in frontage-road settings.

Research indicates that ramp crash rate varies with interchange type, ramp type (i.e., entrance or exit), and ramp configuration (*1*). In general, crash rates tend to be higher for the ramp configurations used at interchanges in frontage-road settings, compared to those ramps used at interchanges not in a frontage-road system. Also, exit ramps tend to have higher crash rates than entrance ramps. This trend may be due to the significant deceleration, combined with the horizontal curvature, often associated with exit ramps.

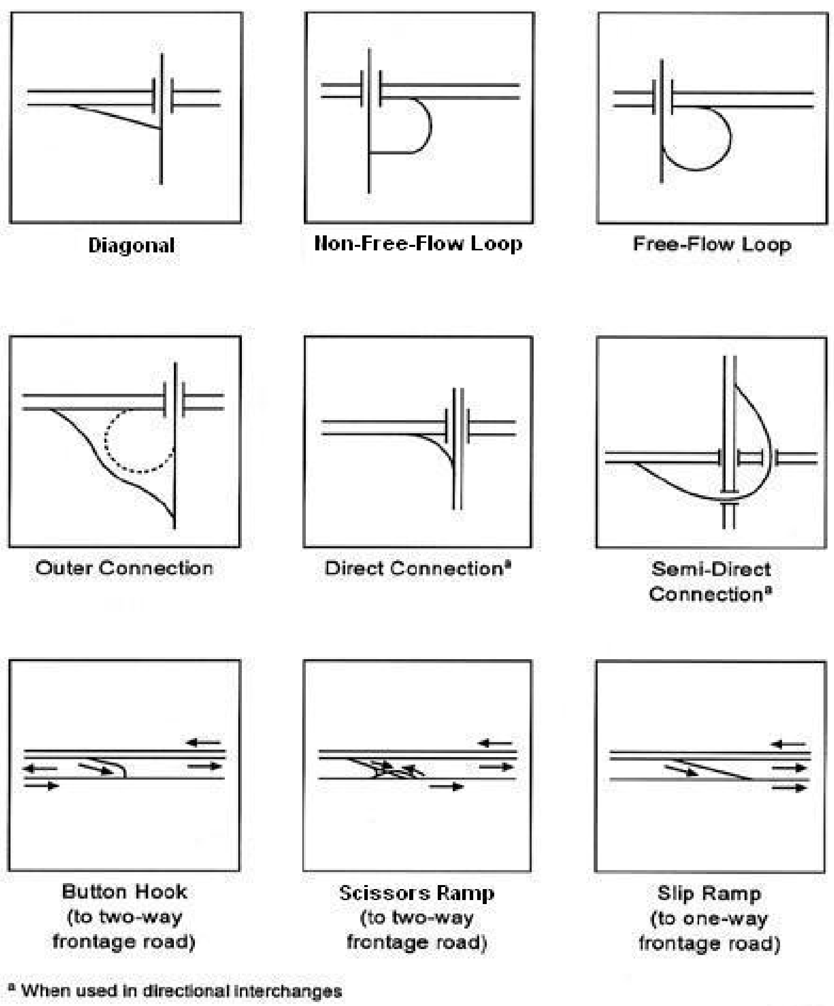

Figure 5-1. Basic Interchange Ramp Configurations.

Safety Relationship

Crash rates for various interchange ramps are provided in Table 5-1. The rates are categorized in terms of interchange setting, ramp type, and ramp configuration. There is some evidence that crash rate among similar ramps in urban areas is higher than in rural areas. However, this evidence is not conclusive, and the rates listed in the table are considered to be applicable to both urban and rural interchange ramps.

The rates provided in Table 5-1 are in units of injury (plus fatal) crashes per million vehicles. They can be used with Equation 5-1 to compute the expected severe crash frequency for a given ramp. It should be noted that the rates do not have a sensitivity to ramp length. Research indicates that ramp length may have some effect on crash frequency; however, this effect has not been accurately quantified.

Guidance

Equation 5-1 can be used to estimate the expected crash rate for any ramp listed in Table 5-1. The crash estimate relates to crashes that occur on the ramp proper. It does not include crashes that occur at the ramp terminals (i.e., at the speed-change lane or crossroad intersection). Procedures are provided in Chapters 2, 6, and 7 for estimating ramp-terminal-related crash frequency.

Equation 5-1 does not include a sensitivity to ramp length. The crash rates in Table 5-1 are applicable to ramps that have sufficient length to allow for reasonable transition in design speed between the freeway and crossroad (or frontage road) and, if needed, queue storage at the controlled ramp terminal. Equation 5-1 may underestimate the crash frequency of ramps that are too short to allow for reasonable speed change and queue storage.

A local calibration factor is identified in Equation 5-1. The factor can be used to adjust the computed value so that it is more consistent with typical ramps in the agency's jurisdiction.

Table 5-1. Base Crash Rates for Interchange Ramps.

Interchange Setting	Ramp Type	Ramp Configuration	Base Crash Rate, cr/mv [1]
Non-frontage road	Exit	Diagonal	0.28
		Non-free-flow loop	0.51
		Free-flow loop	0.20
		Outer connection	0.33
		Semi-direct conn.	0.25
		Direct connection	0.21
	Entrance	Diagonal	0.17
		Non-free-flow loop	0.31
		Free-flow loop	0.12
		Outer connection	0.20
		Semi-direct conn.	0.15
		Direct connection	0.13
Frontage road	Exit	Button hook	0.57
		Scissor	0.48
		Slip	0.36
	Entrance	Button hook	0.28
		Scissor	0.21
		Slip	0.23

Note:
1 - cr/mv: injury (plus fatal) crashes per million vehicles.

$$C_b = 0.000365\ Base\ ADT_{ramp}\ f \qquad \textbf{(5-1)}$$

where:
C_b = base injury (plus fatal) crash frequency, crashes/yr;
$Base$ = injury (plus fatal) crash rate (see Table 5-1), crashes/mv;
ADT_{ramp} = average daily traffic volume on the ramp, veh/d; and
f = local calibration factor.

A calibration procedure is described in reference *4*. A calibration factor of 1.0 should be used unless a local calibration indicates another value is more appropriate.

Example Application

The Question: What is the expected crash frequency for an exit diagonal ramp at a diamond interchange?

The Facts:
- Interchange setting: non-frontage road
- Ramp type: exit
- Ramp configuration: diagonal
- Ramp ADT: 10,000 veh/d

The Solution: From Table 5-1, find that the base crash rate for an exit diagonal ramp is 0.28 crashes/mv. For the ADT provided, this ramp would have an expected crash frequency of 1.0 crashes/yr.

$$\begin{aligned} C_b &= 0.000365\ Base\ ADT_{ramp}\ f \\ &= 0.000365 \times 0.28 \times 10{,}000 \times 1.0 \\ &= 1.0\ crashes/yr \end{aligned} \qquad \textbf{(5-2)}$$

Example Application

The Question: What is the expected crash frequency for an exit slip ramp used at a diamond interchange?

The Facts:
- Interchange setting: frontage road
- Ramp type: exit
- Ramp configuration: slip
- Ramp ADT: 10,000 veh/d

The Solution: From Table 5-1, find that the base crash rate for an exit slip ramp is 0.36 crashes/mv. For the ADT provided, this ramp would have an expected crash frequency of 1.3 crashes/yr.

$$\begin{aligned} C_b &= 0.000365\ Base\ ADT_{ramp}\ f \\ &= 0.000365 \times 0.36 \times 10{,}000 \times 1.0 \\ &= 1.3\ crashes/yr \end{aligned} \qquad \textbf{(5-3)}$$

Accident Modification Factors – Interchange Ramps

Discussion

This section is intended to describe AMFs that can be used to evaluate the relationship between a change in ramp design and a corresponding change in injury (plus fatal) crash frequency. As Table 5-2 indicates, there are no documented AMFs for ramp segments available at the time of publication. There are many factors that are likely to have some influence on crash frequency. However, their relationship has yet to be quantified through research. AMFs for ramps are likely to become available as new research in this area is undertaken.

Table 5-2. AMFs for Interchange Ramps.

Application	Accident Modification Factor
Geometric design	none
Roadside design	none

Base Models – Frontage Roads

Discussion

Frontage roads are typically located on each side of the freeway (or highway) and have a horizontal alignment that follows that of the freeway. Frontage roads separate local traffic from the high-speed freeway traffic and provide access to properties adjacent to the freeway or highway.

Frontage roads on either side of a freeway are often operated as a one-way pair; however, in some remote settings, each frontage road is allowed to operate with two-way traffic flow. One-way frontage road operation offers several operational and safety advantages. Relative to two-way operation, one-way operation reduces the number of vehicular and pedestrian conflicts at ramp terminals and at crossroad intersections.

Ramp connections between the frontage road and freeway are an important design element. As shown in Figure 5-1, a slip ramp design is often used with one-way frontage roads. The button hook or scissors ramp are used with two-way frontage roads.

Safety Relationship

The relationship between crash frequency and traffic demand for base frontage road conditions is shown in Figure 5-2. The trends shown in this figure apply to frontage road segments that are one mile long. Equation 5-4 should be used for other segment lengths.

Figure 5-2 and Equation 5-4 are applicable to one-way and two-way rural frontage roads with two lanes. A similar equation is not available for urban frontage road segments, segments with one lane, or segments with three or more lanes.

Table 5-3 lists the over-dispersion parameter k for each equation. The use of this parameter is described in reference 6.

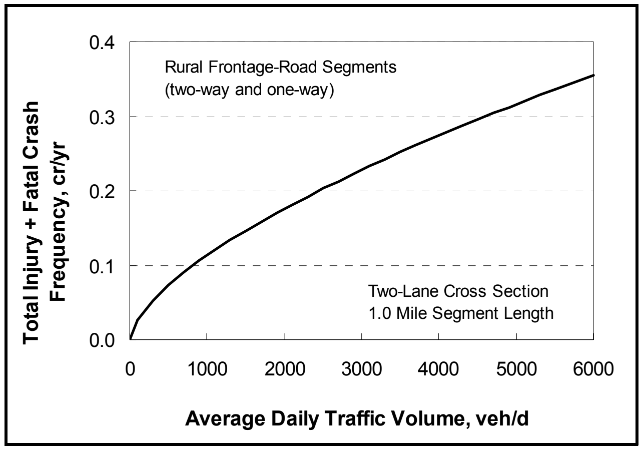

Figure 5-2. Illustrative Frontage Road Crash Trends.

$$C_b = 0.00134\ ADT^{0.641}\ L\ f \qquad \textbf{(5-4)}$$

where:
C_b = base injury (plus fatal) crash frequency, crashes/yr;
ADT = average daily traffic volume, veh/d;
L = segment length, mi; and
f = local calibration factor.

Table 5-3. Over-Dispersion Parameter.

Lanes	Crash Type	Over-Disp. Parameter (k)
2	All	1.37 mi^{-1}

Guidance

The crash frequency obtained from a base model is applicable to segments having base conditions. These conditions generally represent a straight alignment and typical cross section elements. The complete set of base conditions is identified in Table 5-4. If a particular segment has characteristics that differ from the base conditions, then the AMFs described in the next section can be used to obtain a more accurate estimate of segment crash frequency.

Equation 5-4 is limited to rural frontage roads with an ADT of 6000 veh/d or less.

A local calibration factor is shown in Equation 5-4. This factor can be used to adjust the computed value so that it is more consistent with typical frontage roads in the agency's jurisdiction. A calibration procedure is identified in reference *4*. A calibration factor of 1.0 should be used unless a local calibration indicates another value is more appropriate.

Table 5-4. Base Conditions for Frontage Roads.

Characteristic	Base Condition
Horizontal curve radius	tangent (no curve)
Lane width	12 ft
Inside shoulder width	1.5 ft
Outside shoulder width	1.5 ft

Example Application

The Question: What is the expected crash frequency for a typical rural two-lane frontage road segment?

The Facts:
- Area type: rural
- Frontage road ADT: 3900 veh/d
- Segment length: 0.5 mi

The Solution: From Equation 5-4, find that the typical two-lane frontage road segment with these characteristics experiences 0.13 crashes/yr. The crashes are designated as either injury or fatal.

$$\begin{aligned} C_b &= 0.00134\ ADT^{0.641}\ L\ f \\ &= 0.00134 \times (3900)^{0.641} \times 0.5 \times 1.0 \\ &= 0.13\ crashes/yr \end{aligned} \quad \textbf{(5-5)}$$

Accident Modification Factors – Frontage Roads

Discussion

This section describes AMFs that can be used to evaluate the relationship between a design change and the corresponding change in injury (plus fatal) crash frequency. The topics addressed are listed in Table 5-5.

There are many additional factors, other than those listed in Table 5-5, that are likely to have some influence on crash frequency. However, their relationship has yet to be quantified through research. The list of available AMFs for frontage roads is likely to increase as new research in this area is undertaken.

All of the AMFs described in this section yield a value of 1.0 when the associated design component or element represents base conditions. A condition that is more generous (i.e., desirable) than the base condition results in an AMF of less than 1.0. A condition that is less generous will result in an AMF of more than 1.0.

Table 5-5. AMFs for Frontage Roads.

Application	Accident Modification Factor
Geometric design	Lane width Shoulder width

Safety Relationship

The expected injury (plus fatal) crash frequency for a specific rural frontage road segment is computed using Equation 5-6. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are different from base conditions.

$$C = C_b \times AMF_{lw} \times AMF_{sw} \quad \textbf{(5-6)}$$

where:
C = expected injury (plus fatal) crash frequency, crashes/yr;
C_b = base injury (plus fatal) crash frequency, crashes/yr;
AMF_{lw} = lane width accident modification factor; and
AMF_{sw} = shoulder width accident modification factor.

Guidance

In application, all AMFs should be quantified for the subject segment and then multiplied together. The base crash frequency C_b for rural frontage roads is obtained from Equation 5-4. The product of the AMFs and C_b represents the expected injury (plus fatal) crash frequency for the subject segment.

If the crash history is available for the segment, then the over-dispersion parameters in Table 5-3 can be used with the empirical Bayes adjustment procedure described in reference *6* to increase the accuracy of the expected crash frequency (over that obtained from Equation 5-6).

Example Application

The Question: What is the expected crash frequency for a specific rural two-lane frontage road segment?

The Facts:

- Base crash frequency C_b: 0.13 crashes/yr
- Average lane width: 10 ft

The Solution: The segment of interest has typical characteristics with the exception that its average lane width is 10 ft. As described later, the AMF for a lane width of 10 ft is 1.46. This AMF can be used with Equation 5-6 to estimate the expected crash frequency for the subject segment as 0.19 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 0.13 \times 1.46 \\ &= 0.19 \; crashes/yr \end{aligned} \tag{5-7}$$

Lane Width - AMF_{lw}

Discussion

It is generally recognized that lane width has some influence on driving comfort and efficiency. A narrow lane reduces the lateral clearance to vehicles in adjacent lanes and is most notable when large trucks are present in the traffic stream. Research indicates that narrow lanes have a lower capacity than wider lanes.

Safety Relationship

The relationship between lane width and injury (plus fatal) crash frequency can be estimated using Figure 5-3 or Equation 5-8. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 12-ft lane width.

Guidance

This AMF is applicable to two-lane frontage roads with lane widths ranging from 9 to 12 ft. If the lane width is more than 12 ft, then the AMF value for 12 ft should be used. If the lane width is less than 9 ft, then the AMF value for 9 ft should be used.

Example Application

The Question: What is the AMF for a lane width of 10 ft?

The Facts:
- Lane width: 10 ft

The Solution: From Figure 5-3, find the AMF of 1.46. This value suggests that 10-ft lanes are typically associated with a 46 percent increase in crashes.

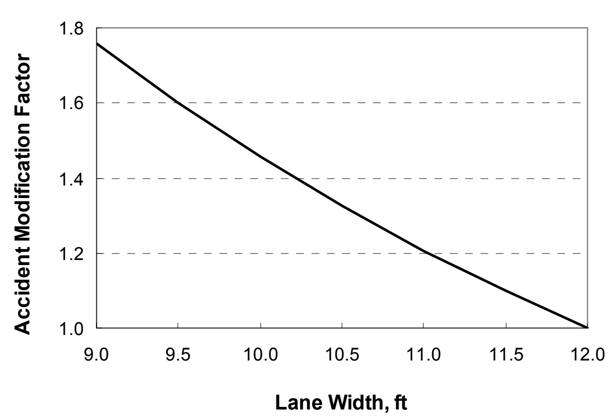

Figure 5-3. Lane Width AMF.

$$AMF_{lw} = e^{-0.188\,(W_l - 12)} \quad (5\text{-}8)$$

where:
AMF_{lw} = lane width accident modification factor; and
W_l = lane width, ft.

Base Condition: 12-ft lane width

Shoulder Width - AMF_{sw}

Discussion

Shoulders offer numerous safety benefits for rural highways. Depending on their width, shoulders may provide space for disabled vehicles, evasive maneuvers, and space within which right-turning vehicles can decelerate.

Safety Relationship

The relationship between shoulder width and injury (plus fatal) crash frequency can be estimated using Figure 5-4 or Equation 5-9. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 1.5-ft right-side shoulder width and a 1.5-ft left-side shoulder width.

Guidance

This AMF is applicable to two-lane frontage roads having paved shoulders with a width ranging from 0 to 5 ft. If the shoulder width is greater than 5 ft, then the AMF value for 5 ft should be used.

Example Application

The Question: What is the AMF for the following proposed paved shoulder width combination?

The Facts:
- Right-side shoulder width: 5 ft
- Left-side shoulder width: 3 ft

The Solution: The average shoulder width is computed as 4.0 ft (= [5+3]/2). From Figure 5-4, find the AMF of 0.84. This value implies that the proposed shoulder width combination may be associated with 16 percent fewer crashes than a shoulder width combination that averages 1.5 ft.

Figure 5-4. Shoulder Width AMF.

$$AMF_{sw} = e^{-0.070\,(W_s - 1.5)} \quad \textbf{(5-9)}$$

with,

$$W_s = \frac{W_{s,r} + W_{s,l}}{2} \quad \textbf{(5-10)}$$

where:
AMF_{sw} = shoulder width accident modification factor;
W_s = average paved shoulder width, ft;
$W_{s,r}$ = right-side paved shoulder width, ft; and
$W_{s,l}$ = left-side paved shoulder width, ft.

Base Condition: 1.5-ft right-side shoulder width, 1.5-ft left-side shoulder width

$$\begin{aligned} AMF_{sw} &= e^{-0.070 \times (4.0 - 1.5)} \\ &= 0.84 \end{aligned} \quad \textbf{(5-11)}$$

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2. Bonneson, J., D. Lord, K. Zimmerman, K. Fitzpatrick, and M. Pratt. *Development of Tools for Evaluating the Safety Implications of Highway Design Decisions.* Report No. FHWA/TX-07/0-4703-4. Texas Department of Transportation, Austin, Texas, September 2006.

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5. Harwood, D.W., F.M. Council, E. Hauer, W.E. Hughes, and A. Vogt. *Prediction of the Expected Safety Performance of Rural Two-Lane Highways.* Report No. FHWA-RD-99-207. Federal Highway Administration, Washington, D.C., 2000.

6. Bonneson, J., and K. Zimmerman. *Procedure for Using Accident Modification Factors in the Highway Design Process.* Report No. FHWA/TX-07/0-4703-P5. Texas Department of Transportation, Austin, Texas, February 2007.

Chapter 6

Rural Intersections

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INTRODUCTION

In Texas, about one-third of all crashes on rural highways occur at intersections. The combination of high speed and multiple, complex guidance and navigational choices at rural intersections complicate the driving task and increase the potential for a severe crash. The design of the intersection can have a significant impact on its safety and operation. Design elements that are consistent with driver expectation and that provide positive separation for turning movements tend to provide the greatest safety benefit.

The process of designing a rural intersection can include an evaluation of the operational and safety benefits associated with various design alternatives, with consideration to the overall cost-effectiveness of each alternative. The importance of this evaluation increases when right-of-way is more constrained, or when access to adjacent properties is adversely impacted.

The procedure described in this chapter can be used to quantify the safety associated with an existing rural intersection or with a proposed design. In this regard, safety is defined as the expected frequency of injury (plus fatal) crashes. The safety benefit of a proposed design can be obtained by comparing its expected crash frequency with that of the existing facility or of another alternative. Background information about the various equations and constants that comprise the procedure is provided in references *1, 2, 3,* and *4*. Procedures for estimating the operational or other impacts of a design alternative are beyond the scope of this *Workbook*.

PROCEDURE

This part of the chapter describes a procedure for evaluating the safety of rural intersections. An intersection is defined to be the pavement area common to two or more crossing public highways, plus a length of each road 250 ft back from the point of crossing. Intersection crashes include all crashes classified as "at intersection" or "intersection-related;" all other crashes are segment crashes.

A procedure for evaluating rural highway segments is described in Chapter 3. This procedure can be used together with the procedure in this chapter to evaluate a rural highway and its intersections.

The procedure described herein is based on the prediction of expected crash frequency. Specifically, the crash frequency for a typical intersection is computed from a base model. This frequency is then adjusted using various accident modification factors (AMFs) to tailor the resulting estimate to a specific intersection. The base model includes a sensitivity to traffic volume, traffic control mode, the number of intersection legs, and the main factors known to be uniquely correlated with crash frequency for the subject intersection. AMFs are used to account for factors found to have some correlation with crash frequency, typically of a more subtle nature than the main factors. The AMFs are multiplied by the base crash frequency to obtain an expected crash frequency for the subject intersection.

The procedure described herein differs from that developed by Harwood et al. (*5*) because this procedure predicts injury (plus fatal) crash frequency, as opposed to total crash frequency. Otherwise, the procedure described herein is similar and shares the same strengths and weaknesses.

Base crash prediction models are described in the next section. The two sections that follow describe the AMFs to be used with these models. Example applications are provided throughout this *Workbook* to illustrate the use of the base models and the AMFs.

Base Models

Discussion

An examination of crash trends indicates that crash rates for rural intersections are dependent on traffic volume, traffic control mode, and the number of intersection approach legs (*1*). In general, crash rates tend to be lower for lower volume intersections. Also, crash rates are typically lower at signalized intersections than two-way stop-controlled intersections, for the same volume levels. Crash rates at intersections with three legs are often lower than those at intersections with four legs. This latter influence is likely a reflection of the fewer number of conflict points at a three-leg intersection, compared to a four-leg intersection.

Safety Relationship

The relationship between crash frequency and traffic demand for base intersection conditions is shown in Figure 6-1. The trends shown in this figure apply to intersections at which the minor road volume equals one-half of the major road volume. Equations 6-1 through 6-4 should be used for other volume conditions.

Equations 6-1 and 6-2 apply to unsignalized intersections that have an uncontrolled major road and a stop-controlled minor road. Equations 6-3 and 6-4 apply to signalized intersections. Equations are not available for four-way stop-controlled intersections.

Table 6-1 lists the over-dispersion parameter k for each equation. The use of this parameter is described in reference *6*.

Guidance

The crash frequency obtained from a base model is applicable to intersections having base conditions. These conditions generally represent uncomplicated geometry, straight alignment, and typical cross section elements. The complete set of base conditions is identified in Table 6-2.

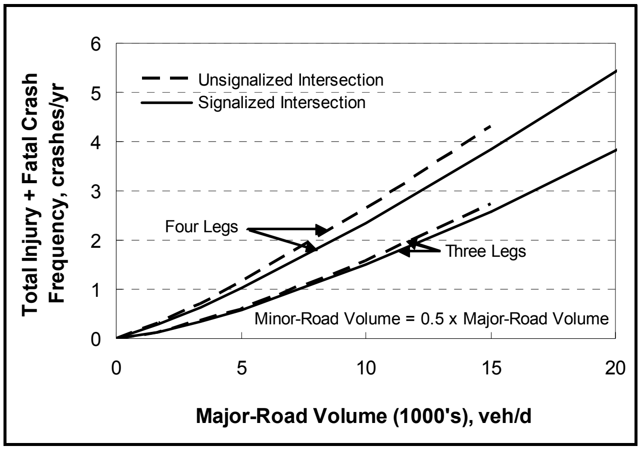


Figure 6-1. Illustrative Intersection Crash Trends.

For three-leg, unsignalized intersections:

$$C_{b,3u} = 0.0973 \left(\frac{ADT_{major}}{1000}\right)^{0.863} \left(\frac{ADT_{minor}}{1000}\right)^{0.497} f_{3u} \qquad \textbf{(6-1)}$$

where:
C_b = base injury (plus fatal) crash frequency, crashes/yr;
ADT_{major} = average daily traffic volume on the major road, veh/d;
ADT_{minor} = average daily traffic volume on the minor road, veh/d; and
f = local calibration factor.

For four-leg, unsignalized intersections:

$$C_{b,4u} = 0.235 \left(\frac{ADT_{major}}{1000}\right)^{0.692} \left(\frac{ADT_{minor}}{1000}\right)^{0.514} f_{4u} \qquad \textbf{(6-2)}$$

For three-leg, signalized intersections:

$$C_{b,3s} = 0.0973 \left(\frac{ADT_{major}}{1000}\right)^{0.782} \left(\frac{ADT_{minor}}{1000}\right)^{0.577} f_{3s} \qquad \textbf{(6-3)}$$

For four-leg, signalized intersections:

$$C_{b,4s} = 0.221 \left(\frac{ADT_{major}}{1000}\right)^{0.611} \left(\frac{ADT_{minor}}{1000}\right)^{0.595} f_{4s} \qquad \textbf{(6-4)}$$

If a particular intersection has characteristics that differ from the base conditions, then the AMFs described in the next two sections can be used to obtain a more accurate estimate of intersection crash frequency.

A local calibration factor is identified in Equations 6-1 through 6-4. The factor can be used to adjust the computed value so that it is more consistent with typical intersections in the agency's jurisdiction. A calibration procedure is identified in reference *4*. A calibration factor of 1.0 should be used unless a local calibration indicates another value is more appropriate.

Example Application

The Question: What is the expected crash frequency for a rural signalized intersection?

The Facts:
- Control mode: signalized
- Intersection legs: 4
- Major-road volume: 5000 veh/d
- Minor-road volume: 5000 veh/d

The Solution: Equation 6-4 is used to compute the expected crash frequency of 1.54 crashes/yr. The use of this equation is illustrated in the box at the right.

Example Application

The Question: What is the expected crash frequency for a rural unsignalized intersection?

The Facts:
- Control mode: unsignalized
- Intersection legs: 4
- Major-road volume: 8000 veh/d
- Minor-road volume: 800 veh/d

The Solution: Equation 6-2 is used to compute the expected crash frequency of 0.88 crashes/yr.

Table 6-1. Over-Dispersion Parameters.

Control Mode	Number of Intersection Legs	Over-Disp. Parameter (*k*)
Unsignalized[1]	3	2.59
	4	1.61
Signalized	3	unknown
	4	3.15

Note:
1 - Unsignalized intersections have an uncontrolled major road and a stop-controlled minor road.

Table 6-2. Base Conditions.

Characteristic	Approach Type	Base Condition by Control Type	
		Signalized	Unsignalized
Left-turn lanes	Major	both legs	none
	Minor	none	
Right-turn lanes	Major	none	none
	Minor	none	
Number of lanes	Major	2	2
	Minor	2	2
Shoulder width	Major		4 ft
	Minor		4 ft
Median presence	Major		not present
	Minor		
Alignment skew angle			no skew
Driveway frequency	Major	2	1
	Minor	2	0
Truck presence		11% trucks	15% trucks

$$C_{b,4s} = 0.221 \left(\frac{5000}{1000}\right)^{0.611} \left(\frac{5000}{1000}\right)^{0.595} \times 1.0 \qquad \textbf{(6-5)}$$
$$= 1.54 \ crashes/yr$$

$$C_{b,4u} = 0.235 \left(\frac{8000}{1000}\right)^{0.692} \left(\frac{800}{1000}\right)^{0.514} \times 1.0 \qquad \textbf{(6-6)}$$
$$= 0.88 \ crashes/yr$$

Accident Modification Factors – Signalized Intersections

Discussion

This section describes AMFs that can be used to evaluate the relationship between a design change and the corresponding change in injury (plus fatal) crash frequency. The AMFs that apply to signalized intersections are listed in Table 6-3. AMFs applicable to unsignalized intersections are presented in the next section.

There are many additional factors, other than those listed in Table 6-3, that are likely to have some influence on crash frequency. However, their relationship has yet to be quantified through research. The list of available AMFs for signalized intersections is likely to increase as new research in this area is undertaken.

All of the AMFs described in this section yield a value of 1.0 when the associated design component or element represents base conditions. A condition that is more generous (i.e., desirable) than the base condition results in an AMF of less than 1.0. A condition that is less generous will result in an AMF of more than 1.0.

Table 6-3. AMFs for Signalized Intersections.

Application	Accident Modification Factor
Geometric design	Left-turn lane Right-turn lane Number of lanes
Access control	Driveway frequency
Other	Truck presence

Safety Relationship

The expected injury (plus fatal) crash frequency for a specific signalized intersection is computed using Equation 6-7. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are different from base conditions.

$$C = C_b \times AMF_{RT} \times AMF_{nd} \ldots \qquad \textbf{(6-7)}$$

where:
C = expected injury (plus fatal) crash frequency, crashes/yr;
C_b = base injury (plus fatal) crash frequency, crashes/yr;
AMF_{RT} = right-turn lane accident modification factor; and
AMF_{nd} = driveway frequency accident modification factor.

Guidance

In application, all applicable AMFs should be quantified for the subject intersection and then multiplied together. The base crash frequency C_b for signalized intersections is obtained from Equation 6-3 or 6-4. The product of the AMFs and C_b represents the expected injury (plus fatal) crash frequency for the subject intersection.

If the crash history is available for the intersection, then the over-dispersion parameters in Table 6-1 can be used with the empirical Bayes adjustment procedure described in reference *6* to increase the accuracy of the expected crash frequency (over that obtained from Equation 6-7).

Example Application

The Question: What is the expected crash frequency for a specific rural signalized intersection?

The Facts:
- Control mode: signalized
- Intersection legs: 4
- Major-road volume: 5000 veh/d
- Minor-road volume: 5000 veh/d
- Base crash frequency C_b: 1.54 crashes/yr
- Major-road driveways: 3
- Minor-road driveways: 2

The Solution: The intersection of interest has typical characteristics with the exception that its driveway frequency on the major-road approaches is above average. As described later, the AMF for this frequency is 1.05. This AMF can be used with Equation 6-7 to estimate the expected crash frequency for the subject intersection as 1.62 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{nd} \\ &= 1.54 \times 1.05 \\ &= 1.62 \ \textit{crashes/yr} \end{aligned} \qquad \textbf{(6-8)}$$

Example Application

The Question: What is the expected change in crash frequency for the previous example intersection if all driveways on the major road are removed?

The Facts:
- Control mode: signalized
- Intersection legs: 4
- Proposed major-road driveways: 0

The Solution: The AMF for zero driveways is 0.91. This AMF can be used with Equation 6-7 to estimate the expected crash frequency for the subject intersection as 1.40 crashes/yr (= 1.54 ×0.91). This analysis suggests that the change may result in a decrease of 0.22 crashes/yr (= 1.62 − 1.40).

Left-Turn Lane - AMF_{LT}

Discussion

An exclusive left-turn lane (or bay) at an intersection provides a length of roadway within which left-turning vehicles can decelerate and store without disrupting the smooth flow of traffic in the adjacent through lane. The lack of a lane, or a bay of inadequate length, can lead to conflict between left-turning and through vehicles as well as poor traffic operations.

Safety Relationship

The relationship between left-turn lane presence and injury (plus fatal) crash frequency is shown in Table 6-4. The AMF value should be estimated using Equation 6-9. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF is applicable to the major- and minor-road approaches at a signalized intersection. At three-leg intersections, the minor road is the discontinuous route. This AMF is based on the concurrent provision of a left-turn phase with the left-turn lane.

The values from Equation 6-10 are appropriate when an exclusive turn lane is not provided or when it is provided but is not of adequate length. Equation 6-11 is appropriate when an exclusive turn lane of adequate length is provided. A turn lane is of adequate length if turning vehicles decelerate and store in it without impeding the flow of through traffic.

Example Application

The Question: What is AMF_{LT} when only one bay is provided on the major road?

The Facts:

- Intersection legs: 4
- Major-road leg volume: 5000 veh/d
- Minor-road leg volume: 5000 veh/d

The Solution: Table 6-4 applies for this volume combination. It indicates that AMF_{LT} equals 1.32 (i.e., a 32-percent increase in crashes).

Table 6-4. AMF for Left-Turn Lanes at a Signalized Intersection.

Number of Intersection Legs	Approach Type	AMF Based on Number of Legs Not Meeting Base Conditions [1]	
		One	Both
3	Major	1.42	not applicable [2]
4	Major	1.32	1.74
	Minor	0.86	0.74

Notes:
1 - Values based on minor-road volume equal to the major-road volume.
2 - Only one left-turn lane per roadway is likely at a three-leg intersection.

For four-leg intersections:

$$AMF_{LT} = AMF_1 \times AMF_2 \times AMF_3 \times AMF_4 \qquad \textbf{(6-9)}$$

Evaluate Rule 1 for each major-road leg i (i = 1, 2).
Evaluate Rule 2 for each minor-road leg i (i = 3, 4).

Rule 1: If leg i does *not* have a turn lane, then:

$$AMF_i = 2.27\ P_{leg,i} + 1.0\ (1 - P_{leg,i}) \qquad \textbf{(6-10)}$$

otherwise, if leg i has a turn lane, then AMF_i = 1.0.

Rule 2: If leg i has a turn lane, then:

$$AMF_i = 0.44\ P_{leg,i} + 1.0\ (1 - P_{leg,i}) \qquad \textbf{(6-11)}$$

otherwise, if leg i does *not* have a turn lane, then AMF_i = 1.0.

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3 + ADT_4} \qquad \textbf{(6-12)}$$

where:
AMF_{LT} = left-turn lane accident modification factor;
AMF_i = leg i accident modification factor;
ADT_i = average daily traffic volume on leg i, veh/d; and
$P_{leg,i}$ = proportion of average daily traffic volume on leg i.

For three-leg intersections:

$$AMF_{LT} = AMF_1 \qquad \textbf{(6-13)}$$

Evaluate Rule 1 for the major-road leg i (i = 1).

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3} \qquad \textbf{(6-14)}$$

Base Condition:
major-road legs: left-turn lane (or bay) on both legs
minor-road legs: left-turn lane (or bay) not present

Right-Turn Lane - AMF_{RT}

Discussion

An exclusive right-turn lane (or bay) at an intersection provides a length of roadway within which right-turning vehicles can decelerate and store without disrupting the smooth flow of traffic in the adjacent through lane. The lack of a lane, or a bay of inadequate length, can lead to conflict between right-turning and through vehicles as well as poor traffic operations.

Safety Relationship

The relationship between right-turn lane presence and injury (plus fatal) crash frequency is shown in Table 6-5. The AMF value should be estimated using Equation 6-15. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF is applicable to the major- and minor-road approaches at a signalized intersection. At three-leg intersections, the minor road is the discontinuous route.

The value from Equation 6-15 is appropriate when an exclusive turn lane of adequate length is provided. A turn lane is of adequate length if turning vehicles decelerate and store in it without impeding the flow of through traffic.

Example Application

The Question: How many crashes will likely be prevented by the addition of right-turn lanes on both major-road approaches?

The Facts:

- Intersection legs: 4
- Major-road leg volume: 5000 veh/d
- Minor-road leg volume: 5000 veh/d
- Base crash frequency C_b: 1.54 crashes/yr

The Solution: Table 6-5 applies for this volume combination. It indicates that AMF_{RT} equals 0.81. The expected crash frequency with the lanes installed is 1.24 crashes/yr (= 1.54 ×0.81). Thus, about 0.30 crashes/yr are prevented.

Table 6-5. AMF for Right-Turn Lanes at a Signalized Intersection.

Number of Intersection Legs	Approach Type	AMF Based on Number of Legs with a Right-Turn Lane [1]	
		One	Both
3	Major	0.86	not applicable [2]
4	Major	0.90	0.81
	Minor	0.90	0.81

Notes:
1 - Values based on minor-road volume equal to the major-road volume.
2 - Only one right-turn lane per roadway is likely at a three-leg intersection.

For four-leg intersections:

$$AMF_{RT} = AMF_1 \times AMF_2 \times AMF_3 \times AMF_4 \qquad \textbf{(6-15)}$$

Evaluate Rule 1 for each intersection leg i (i = 1, 2, 3, 4).

Rule 1: If leg i has a turn lane, then:

$$AMF_i = 0.59\, P_{leg,i} + 1.0\,(1 - P_{leg,i}) \qquad \textbf{(6-16)}$$

otherwise, if leg i does *not* have a turn lane, then AMF_i = 1.0.

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3 + ADT_4} \qquad \textbf{(6-17)}$$

where:
AMF_{RT} = right-turn lane accident modification factor;
AMF_i = leg i accident modification factor;
ADT_i = average daily traffic volume on leg i, veh/d; and
$P_{leg,i}$ = proportion of average daily traffic volume on leg i.

For three-leg intersections:

$$AMF_{RT} = AMF_1 \qquad \textbf{(6-18)}$$

Evaluate Rule 1 for the major-road leg i (i = 1).

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3} \qquad \textbf{(6-19)}$$

Base Condition:
major-road legs: right-turn lane (or bay) not present
minor-road legs: right-turn lane (or bay) not present

Number of Lanes - AMF_{lane}

Discussion

Research indicates that the number of lanes at a signalized intersection is correlated with the frequency of crashes. The trend is one of more crashes with an increase in the number of lanes. The number of lanes in the cross section tends to increase the size of the intersection conflict area, which could increase the exposure of vehicles to conflict with crossing movements.

Safety Relationship

The relationship between the number of through lanes and injury (plus fatal) crash frequency can be estimated from Table 6-6. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is two lanes on the major road and two lanes on the minor road.

Guidance

The number of through lanes is counted at the intersection, regardless of whether the lanes are added or dropped away from the intersection. The number of lanes provided at the intersection is often dictated by capacity considerations. The AMF from Table 6-6 should be used to obtain an accurate estimate of the expected crash frequency for a given cross section. This AMF is not intended to be used to justify a change in cross section.

Example Application

The Question: What is the AMF for the intersection of two, four-lane roads?

The Facts:
- Major-road through lanes: 4
- Minor-road through lanes: 4

The Solution: From Table 6-6, find the combined AMF of 1.03. This intersection typically has about 3 percent more crashes than one with two or three lanes on each road.

Table 6-6. AMF for Number of Through Lanes at a Signalized Intersection.

Number of Major-Road Through Lanes	AMF Based on Number of Minor-Road Through Lanes		
	2	3	4
2	1.00		
3	1.00	1.00	
4	1.01	1.01	1.03
5	1.01	1.01	1.03
6	1.03	1.03	1.04

Base Condition:
major road: 2 lanes
minor road: 2 lanes

Driveway Frequency - AMF_{nd}

Discussion

For most rural highways, provision of driveway access is consistent with the highway's function and essential to adjacent property owners. However, traffic movements associated with these driveways add turbulence to the traffic stream as vehicles enter or exit the highway. This turbulence can be more pronounced when the intersection is close to a busy intersection.

Safety Relationship

The relationship between driveway frequency and injury (plus fatal) crash frequency is shown in Figure 6-2. The AMF value should be estimated using Equation 6-20. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

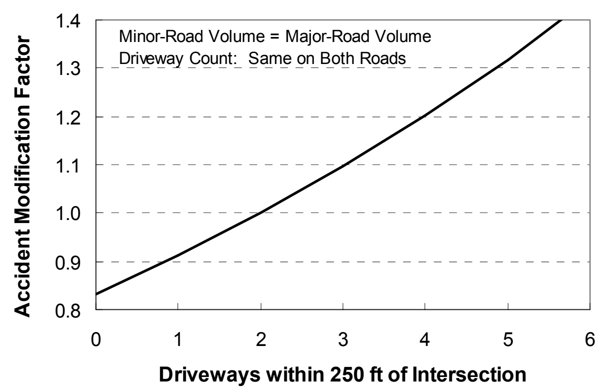


Figure 6-2. Driveway Frequency AMF for a Signalized Intersection.

$$AMF_{nd} = AMF_{nd,major} \times AMF_{nd,minor} \qquad \textbf{(6-20)}$$

$$AMF_{nd,major} = e^{0.046\,(d_{n,major} - 2)} \qquad \textbf{(6-21)}$$

$$AMF_{nd,minor} = \left(e^{0.046\,(d_{n,minor} - 2)} - 1\right)\frac{ADT_{minor}}{ADT_{major}} + 1.0 \qquad \textbf{(6-22)}$$

where:
AMF_{nd} = driveway frequency accident modification factor;
AMF_{major} = average daily traffic volume on the major road, veh/d;
AMF_{minor} = average daily traffic volume on the minor road, veh/d;
$d_{n,\,major}$ = number of driveways on the major road within 250 ft of the intersection; and
$d_{n,\,minor}$ = number of driveways on the minor road within 250 ft of the intersection.

Base Condition:
major road: 2 driveways within 250 ft of intersection
minor road: 2 driveways within 250 ft of intersection

Guidance

This AMF applies to driveways on the major- and minor-road approaches to the intersection. When estimating d_n for a given road, driveways on both approaches within 250 ft of the intersection should be counted. The count should only include *active* driveways (i.e., those driveways with an average daily volume of 10 veh/d or more). Public highway intersection approaches should not be included in the count of driveways.

Example Application

The Question: By what percentage would crashes decrease if the number of driveways on the major road is reduced from three to two?

The Facts:

- Minor-road driveways: 2
- Major-road volume: 5000 veh/d
- Minor-road volume: 5000 veh/d

The Solution: From Equation 6-20, find the AMFs of 1.05 for three driveways and 1.00 for two driveways. These results indicate a 4.8 percent reduction in crashes due to the change.

$$\begin{aligned} \%\,Decrease &= 100\left(1 - \frac{1.00}{1.05}\right) \\ &= 4.8\,\% \end{aligned} \qquad \textbf{(6-23)}$$

Truck Presence - AMF$_{tk}$

Discussion

The number of trucks traveling through an intersection can affect both its safety and operation. Trucks are slower to accelerate and decelerate than automobiles, and they physically occupy more space on the intersection approach. These effects may be more pronounced at signalized intersections where trucks may frequently be required to stop.

Safety Relationship

The relationship between truck percentage and injury (plus fatal) crash frequency can be estimated from Figure 6-3 or Equation 6-24. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Figure 6-3. Truck Presence AMF for a Signalized Intersection.

Guidance

The percent trucks variable used to estimate this AMF is computed as the total truck volume for all traffic movements at the intersection divided by the total volume of these movements. The volumes used should represent the peak (or design) hour.

$$AMF_{tk} = e^{0.028\,(P_t\,-\,11)} \qquad \textbf{(6-24)}$$

where:
AMF_{tk} = truck presence accident modification factor; and
P_t = percent trucks during the peak hour (average for all intersection movements), %.

Base Condition: 11% trucks

This AMF is appropriate for truck percentages ranging from 0 to 25 percent. It should not be used as a basis for design decisions regarding truck percentage. Rather, it should be used as a means of adjusting the base crash frequency to accurately reflect the presence of trucks at the subject intersection.

Example Application

The Question: If the truck percentage at a rural signalized intersection is 15 percent, what is the AMF?

The Facts:
- Truck percentage: 15 percent

The Solution: From Figure 6-3, find the AMF of 1.12. This value suggests that crashes will be 12 percent higher at this intersection than one just like it but with 11 percent trucks.

$$\begin{aligned} AMF_{tk} &= e^{0.028\,(15\,-\,11)} \\ &= 1.12 \end{aligned} \qquad \textbf{(6-25)}$$

Accident Modification Factors – Unsignalized Intersections

Discussion

This section describes AMFs that can be used to evaluate the relationship between a design change and the corresponding change in injury (plus fatal) crash frequency. The AMFs that apply to unsignalized intersections are listed in Table 6-7. AMFs for signalized intersections are presented in the previous section.

There are many additional factors, other than those listed in Table 6-8, that are likely to have some influence on crash frequency. However, their relationship has yet to be quantified through research. The list of available AMFs for unsignalized intersections is likely to increase as new research in this area is undertaken.

All of the AMFs described in this section yield a value of 1.0 when the associated design component or element represents base conditions. A condition that is more generous (i.e., desirable) than the base condition results in an AMF of less than 1.0. A condition that is less generous will result in an AMF of more than 1.0.

Safety Relationship

The expected injury (plus fatal) crash frequency for a specific unsignalized intersection is computed using Equation 6-7, repeated here as Equation 6-26. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are different from base conditions.

Guidance

In application, all applicable AMFs should be quantified for the subject intersection and then multiplied together. The base crash frequency C_b for unsignalized intersections is obtained from Equation 6-1 or 6-2. The product of the AMFs and C_b represents the expected injury

Table 6-7. AMFs for Unsignalized Intersections.

Application	Accident Modification Factor [1]
Geometric design	Left-turn lane Right-turn lane Number of lanes Shoulder width Median presence Alignment skew angle
Access control	Driveway frequency
Other	Truck presence

Note:
1 - Factors listed only apply to intersections that have an uncontrolled major road and a stop-controlled minor road.

$$C = C_b \times AMF_{RT} \times AMF_{nd} \cdots \tag{6-26}$$

where:
C = expected injury (plus fatal) crash frequency, crashes/yr;
C_b = base injury (plus fatal) crash frequency, crashes/yr;
AMF_{RT} = right-turn lane accident modification factor; and
AMF_{nd} = driveway frequency accident modification factor.

(plus fatal) crash frequency for the subject intersection.

Example Application

The Question: What is the expected crash frequency for a specific rural unsignalized intersection?

The Facts:

- Control mode: unsignalized
- Intersection legs: 4
- Major-road volume: 8000 veh/d
- Minor-road volume: 800 veh/d
- Base crash frequency C_b: 0.88 crashes/yr
- Major-road shoulder width: 8 ft
- Minor-road shoulder width: 4 ft

The Solution: The intersection of interest has typical characteristics with the exception that its major-road shoulder width is 8 ft. As described later, the AMF for this shoulder width combination is 0.887. This AMF can be used with Equation 6-7 to estimate the expected crash frequency for the subject intersection as 0.78 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{sw} \\ &= 0.88 \times 0.887 \\ &= 0.78 \; crashes/yr \end{aligned} \tag{6-27}$$

Left-Turn Lane - AMF_{LT}

Discussion

An exclusive left-turn lane (or bay) at an intersection provides a length of roadway within which left-turning vehicles can decelerate and store without disrupting the smooth flow of traffic in the adjacent through lane. The lack of a lane, or a bay of inadequate length, can lead to conflict between left-turning and through vehicles as well as poor traffic operations.

Safety Relationship

The relationship between left-turn lane presence and injury (plus fatal) crash frequency is shown in Table 6-8. The AMF value should be estimated using Equation 6-28. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF is applicable to the major-road approaches at an unsignalized intersection. At three-leg intersections, the minor road is the discontinuous route.

The values from Equation 6-28 are appropriate when an exclusive turn lane of adequate length is provided. A turn lane is of adequate length if turning vehicles decelerate and store in it without impeding the flow of through traffic.

Example Application

The Question: What is the AMF_{LT} if a left-turn bay is provided on the major road?

The Facts:

- Intersection legs: 3
- Major-road leg volume: 8000 veh/d
- Minor-road leg volume: 800 veh/d
- Existing left-turn bays on major road: 0

The Solution: Table 6-8 applies for this volume combination. It indicates that AMF_{LT} equals 0.70. This value corresponds to a 30-percent reduction in crashes.

Table 6-8. AMF for Left-Turn Lanes at an Unsignalized Intersection.

Number of Intersection Legs	Approach Type	AMF Based on Number of Legs with a Left-Turn Lane [1]	
		One	Both
3	Major	0.70	not applicable [2]
4	Major	0.71	0.50

Notes:
1 - Values based on minor-road volume equal to one-tenth of the major-road volume.
2 - Only one major-road left-turn lane is likely at a three-leg intersection.

For four-leg intersections:

$$AMF_{LT} = AMF_1 \times AMF_2 \qquad (6\text{-}28)$$

Evaluate Rule 1 for each major-road leg i (i = 1, 2).

Rule 1: If leg i has a turn lane, then:

$$AMF_i = 0.36\, P_{leg,i} + 1.0\,(1 - P_{leg,i}) \qquad (6\text{-}29)$$

otherwise, if leg i does *not* have a turn lane, then AMF_i = 1.0.

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3 + ADT_4} \qquad (6\text{-}30)$$

where:
AMF_{LT} = left-turn lane accident modification factor;
AMF_i = leg i accident modification factor;
ADT_i = average daily traffic volume on leg i, veh/d; and
$P_{leg,i}$ = proportion of average daily traffic volume on leg i.

For three-leg intersections:

$$AMF_{LT} = AMF_1 \qquad (6\text{-}31)$$

Evaluate Rule 1 for the major-road leg i (i = 1).

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3} \qquad (6\text{-}32)$$

Base Condition:
major-road legs: left-turn lane (or bay) not present

Right-Turn Lane - AMF_{RT}

Discussion

An exclusive right-turn lane (or bay) at an intersection provides a length of roadway within which right-turning vehicles can decelerate without disrupting the smooth flow of traffic in the adjacent through lane. The lack of a lane, or a bay of inadequate length, can lead to conflict between right-turning and through vehicles as well as poor traffic operations.

Safety Relationship

The relationship between right-turn lane presence and injury (plus fatal) crash frequency is shown in Table 6-9. The AMF value should be estimated using Equation 6-33. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF is applicable to the major-road approaches at an unsignalized intersection. At three-leg intersections, the minor road is the discontinuous route.

The values from Equation 6-33 are appropriate when an exclusive turn lane of adequate length is provided. A turn lane is of adequate length if turning vehicles can decelerate in it without impeding the flow of through traffic.

Example Application

The Question: What is the likely decrease in crash frequency if a right-turn bay is added to each major-road leg?

The Facts:

- Intersection legs: 4
- Major-road leg volume: 8000 veh/d
- Minor-road leg volume: 800 veh/d
- Base crash frequency C_b: 0.88 crashes/yr

The Solution: Table 6-9 applies for this volume combination. It indicates that AMF_{RT} is 0.79. The expected crash frequency with the lanes installed is 0.70 crashes/yr (= 0.88 ×0.79), which equates to a reduction of 0.18 crashes/yr.

Table 6-9. AMF for Right-Turn Lanes at an Unsignalized Intersection.

Number of Intersection Legs	Approach Type	AMF Based on Number of Legs with a Right-Turn Lane[1]	
		One	Both
3	Major	0.89	not applicable[2]
4	Major	0.89	0.79

Notes:
1 - Values based on minor-road volume equal to one-tenth of the major-road volume.
2 - Only one major-road right-turn lane is likely at a three-leg intersection.

For four-leg intersections:

$$AMF_{RT} = AMF_1 \times AMF_2 \quad \textbf{(6-33)}$$

Evaluate Rule 1 for each major-road leg i (i = 1, 2).

Rule 1: If leg i has a turn lane, then:

$$AMF_i = 0.76\,P_{leg,i} + 1.0\,(1 - P_{leg,i}) \quad \textbf{(6-34)}$$

otherwise, if leg i does *not* have a turn lane, then AMF_i = 1.0.

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3 + ADT_4} \quad \textbf{(6-35)}$$

where:
AMF_{RT} = right-turn lane accident modification factor;
AMF_i = leg i accident modification factor;
ADT_i = average daily traffic volume on leg i, veh/d; and
$P_{leg,i}$ = proportion of average daily traffic volume on leg i.

For three-leg intersections:

$$AMF_{RT} = AMF_1 \quad \textbf{(6-36)}$$

Evaluate Rule 1 for the major-road leg i (i = 1).

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3} \quad \textbf{(6-37)}$$

Base Condition:
major-road legs: right-turn lane (or bay) not present

Number of Lanes - AMF_{lane}

Discussion

Research indicates that the number of lanes at an unsignalized intersection is correlated with the frequency of crashes. The trend is one of fewer crashes with an increase in the number of lanes. More traffic on the major road, which typically coincides with more lanes, is likely to discourage minor-road crossing and left-turning movements. The resulting redistribution of traffic patterns may explain the aforementioned trend.

Safety Relationship

The relationship between the number of through lanes and injury (plus fatal) crash frequency can be estimated from Table 6-10. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is two lanes on the major road and two lanes on the minor road.

Table 6-10. AMF for Number of Through Lanes at an Unsignalized Intersection.

Number of Major-Road Through Lanes	AMF Based on Number of Minor-Road Through Lanes		
	2	3	4
2	1.00		
3	1.00	1.00	
4	0.83	0.83	0.69
5	0.83	0.83	0.69
6	0.69	0.69	0.57

Base Condition:
major road: 2 lanes
minor road: 2 lanes

Guidance

The number of through lanes is counted at the intersection, regardless of whether the lanes are added or dropped away from the intersection. The number of lanes provided at the intersection is often dictated by capacity considerations. The AMF from Table 6-10 should be used to obtain an accurate estimate of the expected crash frequency for a given cross section. This AMF is not intended to be used to justify a change in cross section.

Example Application

The Question: What is the AMF for the intersection of two, four-lane roads?

The Facts:
- Major-road through lanes: 4
- Minor-road through lanes: 4

The Solution: From Table 6-10, find the combined AMF of 0.69. This intersection typically has about 31 percent fewer crashes than one with two or three lanes on each road.

Shoulder Width - AMF_{sw}

Discussion

Shoulders offer numerous safety benefits for rural intersections. Depending on their width, shoulders may provide space for disabled vehicles, evasive maneuvers, and space within which right-turning vehicles can decelerate. Right-of-way availability can pose some constraint in intersection areas where additional lanes are needed for capacity and a reduction in shoulder width is sometimes considered. In these situations, both the safety and operational trade-offs should be considered.

Safety Relationship

The relationship between shoulder width and injury (plus fatal) crash frequency is shown in Figure 6-4. The AMF value should be estimated using Equation 6-38. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF applies to the shoulders on the major- and minor-road approaches to the intersection. The shoulder width used to estimate the AMF is the average width of the outside shoulders on each leg. This AMF is applicable to shoulder widths ranging from 0 to 10 ft.

Example Application

The Question: What is the expected change in crashes if the shoulder is eliminated on one side to add a right-turn bay (as shown in the figure)?

The Facts:
- Major-road shoulder width: 8 ft
- Minor-road shoulder width: 4 ft

The Solution: The existing condition has an average shoulder width of 8 ft. From Equation 6-38, find an AMF_{sw} of 0.887. The proposed change has an average width of 6 ft (= [8 +8 +8+0]/4) and an AMF_{sw} of 0.942. Table 6-9 indicates that AMF_{RT} is 0.89 for the bay. The ratio of these AMFs suggests a likely 6.0 percent decrease in crashes due to the change.

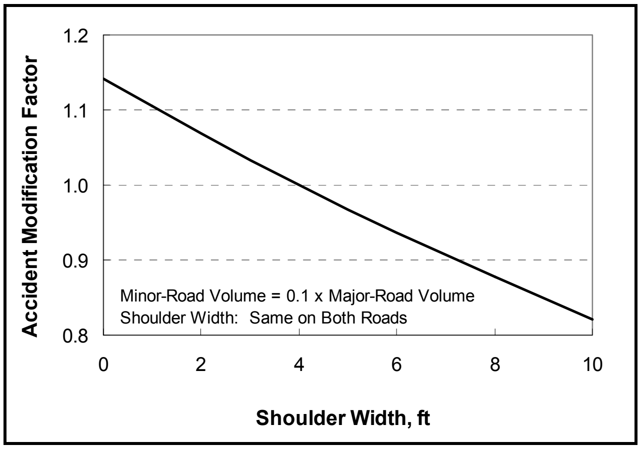


Figure 6-4. Shoulder Width AMF for an Unsignalized Intersection.

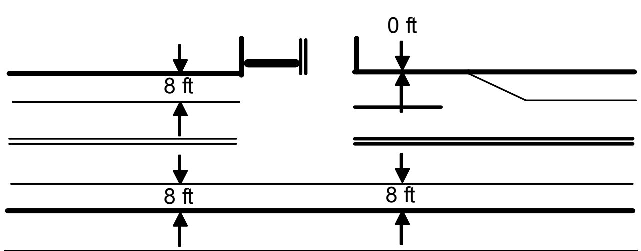
$$AMF_{sw} = AMF_{sw,major} \times AMF_{sw,minor} \quad \textbf{(6-38)}$$

$$AMF_{sw,major} = e^{-0.030\,(W_{s,major} - 4)} \quad \textbf{(6-39)}$$

$$AMF_{sw,minor} = \left(e^{-0.030\,(W_{s,minor} - 4)} - 1\right)\frac{ADT_{minor}}{ADT_{major}} + 1.0 \quad \textbf{(6-40)}$$

where:
AMF_{sw} = shoulder width accident modification factor;
AMF_{major} = average daily traffic volume on the major road, veh/d;
AMF_{minor} = average daily traffic volume on the minor road, veh/d;
$W_{s,\,major}$ = shoulder width on the major road, ft; and
$W_{s,\,minor}$ = shoulder width on the minor road, ft.

Base Condition:
major road: 4-ft shoulder width
minor road: 4-ft shoulder width



$$\%\ Decrease = \left(1 - \frac{0.942 \times 0.89}{0.887}\right) \times 100 \quad \textbf{(6-41)}$$
$$= 6.0\%$$

Median Presence - AMF_{mp}

Discussion

A median provides several functions including positive separation between opposing traffic streams, a sheltered location for left-turning vehicles, and control of access in the vicinity of the intersection. The benefits derived from these functions tend to increase with wider medians.

Safety Relationship

The relationship between median presence and injury (plus fatal) crash frequency can be estimated using Figure 6-5 or Equation 6-42. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is an undivided major road (i.e., AMF_{mp} = 1.0).

Guidance

This AMF applies to medians on the major road. The presence of a median on the minor road is not addressed by this AMF. The median should extend back from the stop line for a distance of 250 ft or more. The median should also be at least 4 ft in width.

Example Application

The Question: What percent reduction in crashes should occur after installing a left-turn bay on both major-road approaches at a four-leg intersection along with a 20 ft median?

The Facts:

- Intersection legs: 4
- Major-road leg volume: 8000 veh/d
- Minor-road leg volume: 800 veh/d

The Solution: From Figure 6-5, find AMF_{mp} of 0.95. From Table 6-8, find AMF_{LT} of 0.50. The combined AMF is 0.48 (= 0.95×0.50) for adding both left-turn bays and a median in the vicinity of the intersection. This AMF corresponds to a 52 percent reduction in crash frequency.

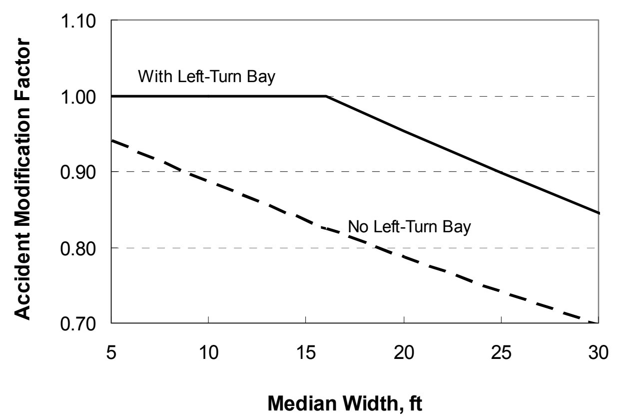


Figure 6-5. Median Presence AMF for an Unsignalized Intersection.

$$AMF_{mp} = AMF_1 \times AMF_2 \quad \textbf{(6-42)}$$

Evaluate Rules 1 and 2.

Rule 1: If a left-turn lane is present and the median is 16 ft or more in width, then:

$$AMF_1 = e^{-0.012\,(W_m - 16)} \quad \textbf{(6-43)}$$

otherwise, AMF_1 = 1.0.

Rule 2: If a left-turn lane is *not* present and a median is present, then:

$$AMF_2 = e^{-0.012\,W_m} \quad \textbf{(6-44)}$$

otherwise, AMF_2 = 1.0.

where:

AMF_{mp} = median presence accident modification factor; and

W_m = median width (including bay, if present), ft.

Base Condition: no median on major road

Alignment Skew Angle - AMF_{skew}

Discussion

Severe skew can make it more difficult for drivers stopped on the minor road to judge gaps in the conflicting traffic stream, especially when the skew causes them to have to look back over their shoulder to see conflicting vehicles. Also, the turn maneuver may take a longer time with increasing skew angle.

Safety Relationship

The relationship between alignment skew angle and injury (plus fatal) crash frequency can be estimated from Figure 6-6, Equation 6-45, or Equation 6-46. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

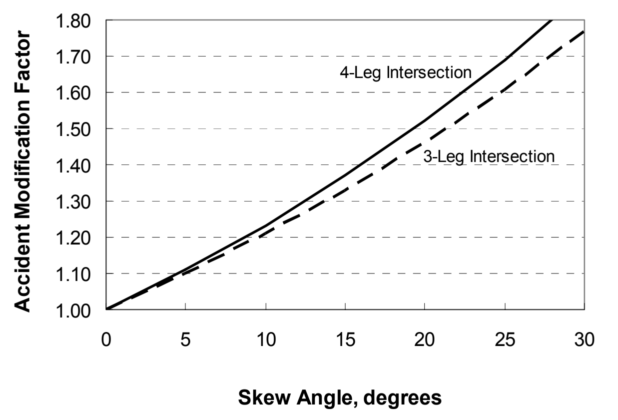


Figure 6-6. Alignment Skew Angle AMF for an Unsignalized Intersection.

For three-leg intersections:

$$AMF_{skew} = e^{0.019\, I_{sk}} \tag{6-45}$$

For four-leg intersections:

$$AMF_{skew} = e^{0.021\, I_{sk}} \tag{6-46}$$

where:
AMF_{skew} = skew angle accident modification factor; and
I_{sk} = skew angle of the intersection (= | intersection angle − 90 |), degrees.

Guidance

This AMF is applicable to alignment skew angles in the range of 0 to 30 degrees. Skew angle is computed as the absolute value of the difference between the intersection angle and 90 degrees. If the minor legs of a four-leg intersection intersect the major road at different angles from each other, then use the average skew angle.

Base Condition: no skew (i.e., 90 degree intersection)

Example Application

The Question: What is the AMF for an intersection angle of 70 degrees?

The Facts:

- Intersection legs: 3

The Solution: The skew angle is computed as 20 degrees (= |70 − 90|). From Figure 6-6, find the AMF of 1.46. This value suggests that the skewed intersection will be associated with 46 percent more crashes.

$$\begin{aligned} AMF_{skew} &= e^{0.019 \times 20} \\ &= 1.46 \end{aligned} \tag{6-47}$$

Driveway Frequency - AMF_{nd}

Discussion

For most rural highways, provision of driveway access is consistent with the highway's function and essential to adjacent property owners. However, traffic movements associated with these driveways add turbulence to the traffic stream as vehicles enter or exit the highway. This turbulence can be more pronounced when the intersection is close to a busy intersection.

Safety Relationship

The relationship between driveway frequency and injury (plus fatal) crash frequency is shown in Figure 6-7. The AMF value should be estimated using Equation 6-48. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF applies to driveways on the major- and minor-road approaches to the intersection. When estimating d_n for a given road, driveways on both approaches within 250 ft of the intersection should be counted. The count should only include *active* driveways (i.e., those driveways with an average daily volume of 10 veh/d or more). Public highway intersection approaches should not be included in the count of driveways.

Example Application

The Question: What is the AMF for an intersection with six driveways on the major road?

The Facts:

- Minor-road driveways: 0
- Major-road volume: 8000 veh/d
- Minor-road volume: 800 veh/d

The Solution: From Equation 6-48, find the AMF of 1.32 for six driveways. This value implies that the six driveways will likely increase crash frequency by 32 percent, when compared to one driveway on the major road.

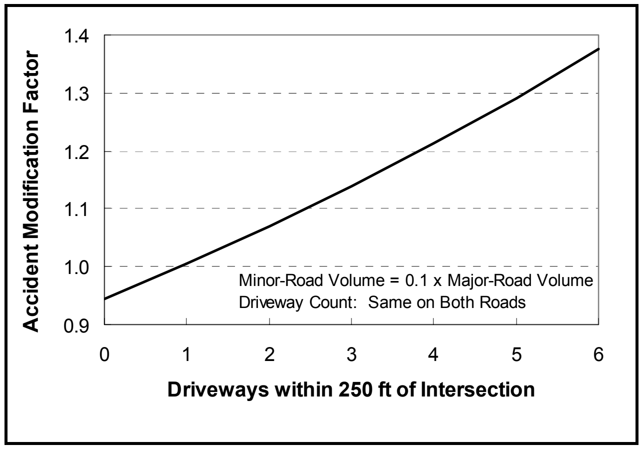


Figure 6-7. Driveway Frequency AMF for an Unsignalized Intersection.

$$AMF_{nd} = AMF_{nd,major} \times AMF_{nd,minor} \quad \textbf{(6-48)}$$

$$AMF_{nd,major} = e^{0.056\,(d_{n,major} - 1)} \quad \textbf{(6-49)}$$

$$AMF_{nd,minor} = \left(e^{0.056\,d_{n,minor}} - 1\right)\frac{ADT_{minor}}{ADT_{major}} + 1.0 \quad \textbf{(6-50)}$$

where:
AMF_{nd} = driveway frequency accident modification factor;
AMF_{major} = average daily traffic volume on the major road, veh/d;
AMF_{minor} = average daily traffic volume on the minor road, veh/d;
$d_{n,\,major}$ = number of driveways on the major road within 250 ft of the intersection; and
$d_{n,\,minor}$ = number of driveways on the minor road within 250 ft of the intersection.

Base Condition:
major road: 1 driveway within 250 ft of intersection
minor road: 0 driveways within 250 ft of intersection

Truck Presence - AMF_{tk}

Discussion

An analysis of crash data indicates that unsignalized intersections with a higher truck percentage are associated with fewer crashes. This trend suggests that drivers may be more cautious when there are many trucks in the traffic stream.

Safety Relationship

The relationship between truck percentage and injury (plus fatal) crash frequency can be estimated from Figure 6-8 or Equation 6-51. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

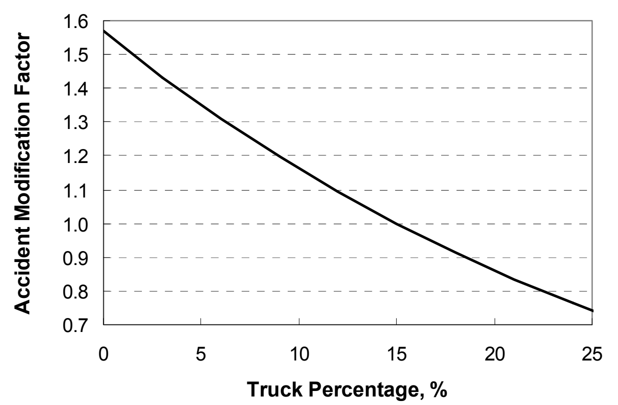


Figure 6-8. Truck Presence AMF for an Unsignalized Intersection.

$$AMF_{tk} = e^{-0.030\,(P_t\,-\,15)} \tag{6-51}$$

where:
AMF_{tk} = truck presence accident modification factor; and
P_t = percent trucks during the peak hour (average for all intersection movements), %.

Base Condition: 15% trucks

Guidance

The percent trucks variable used to estimate this AMF is computed as the total truck volume for all traffic movements at the intersection divided by the total volume of these movements. The volumes used should represent the peak (or design) hour.

This AMF is appropriate for truck percentages ranging from 0 to 25 percent. It should not be used as a basis for design decisions regarding truck percentage. Rather, it should be used as a means of adjusting the base crash frequency to accurately reflect the presence of trucks at the subject intersection.

Example Application

The Question: If the truck percentage at a rural unsignalized intersection is 6 percent, what is the AMF?

The Facts:

- Truck percentage: 6 percent

The Solution: From Figure 6-8, find the AMF of 1.31. This value suggests that crashes will be 31 percent higher at this intersection than one just like it but with 15 percent trucks.

$$\begin{aligned} AMF_{tk} &= e^{-0.030\,(6\,-\,15)} \\ &= 1.31 \end{aligned} \tag{6-52}$$

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Chapter 7

Urban Intersections

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INTRODUCTION

In Texas, about one-half of all crashes in urban areas occur at intersections. Intersections are a necessary consequence of a surface street system. They represent the point where two streets (and their traffic streams) cross and therefore are potential sources of traffic conflict. The design of the intersection can have a significant impact on its safety and operation. In addition, the accommodation of automobile, truck, pedestrian, and bicycle travel modes presents unique design challenges in the urban environment, and especially at intersections. Design elements that provide positive separation between turning movements and between travel modes tend to provide the greatest safety benefit.

The process of designing an urban intersection can include an evaluation of the operational and safety benefits associated with various design alternatives, with consideration to the overall cost-effectiveness of each alternative. The importance of this evaluation increases when right-of-way is more constrained, or when access to adjacent properties is adversely impacted.

The procedure described in this chapter can be used to quantify the safety associated with an existing urban intersection or with a proposed design. In this regard, safety is defined as the expected frequency of injury (plus fatal) crashes. The safety benefit of a proposed design can be obtained by comparing its expected crash frequency with that of the existing facility or of another alternative. Background information about the various equations and constants that comprise the procedure is provided in references *1, 2, 3,* and *4*. Procedures for estimating the operational or other impacts of a design alternative are beyond the scope of this *Workbook*.

PROCEDURE

This part of the chapter describes a procedure for evaluating the safety of urban intersections. An intersection is defined to be the pavement area common to two or more crossing public streets, plus a length of each street 250 ft back from the point of crossing. Intersection crashes include all crashes classified as "at intersection" or "intersection-related;" all other crashes are segment crashes.

A procedure for evaluating urban street segments is described in Chapter 4. This procedure can be used together with the procedure in this chapter to evaluate an urban street and its intersections.

The procedure described herein is based on the prediction of expected crash frequency. Specifically, the crash frequency for the typical intersection is computed from a base model. This frequency is then adjusted using various accident modification factors (AMFs) to tailor the resulting estimate to a specific intersection. The base model includes a sensitivity to traffic volume, traffic control mode, the number of intersection legs, and the main factors known to be uniquely correlated with crash frequency for the subject intersection. AMFs are used to account for other factors found to have some correlation with crash frequency, typically of a more subtle nature than the main factors. The AMFs are multiplied by the base crash frequency to obtain an expected crash frequency for the subject intersection.

The procedure described herein is similar to that developed by Harwood et al. (*5*) because this procedure predicts injury (plus fatal) crash frequency, as opposed to total crash frequency. Otherwise, the procedure described herein is similar and shares the same strengths and weaknesses.

Base crash prediction models are described in the next section. The two sections that follow describe the AMFs to be used with these models. Example applications are provided throughout this *Workbook* to illustrate the use of the base models and the AMFs.

Base Models

Discussion

An examination of crash trends indicates that crash rates for urban intersections are dependent on traffic volume, traffic control mode, and the number of intersection approach legs (*1*). In general, crash rates tend to be lower for lower volume intersections. Also, crash rates are typically higher at signalized intersections than two-way stop-controlled intersections, for the same volume levels. Crash rates at intersections with three legs are often lower than those at intersections with four legs. This latter influence is likely a reflection of the fewer number of conflict points at a three-leg intersection, compared to a four-leg intersection.

Safety Relationship

The relationship between crash frequency and traffic demand for base intersection conditions is shown in Figure 7-1. The trends shown in this figure apply to intersections at which the minor street volume equals one-half of the major street volume. Equations 7-1 through 7-4 should be used for other volume conditions.

Equations 7-1 and 7-2 apply to unsignalized intersections that have an uncontrolled major street and a stop-controlled minor street. Equations 7-3 and 7-4 apply to signalized intersections. Equations are not available for four-way stop-controlled intersections.

Table 7-1 lists the over-dispersion parameter k for each equation. The use of this parameter is described in reference *6*.

Guidance

The crash frequency obtained from a base model is applicable to intersections having base conditions. These conditions generally represent uncomplicated geometry, straight alignment, and typical cross section elements. The complete set of base conditions is identified in Table 7-2.

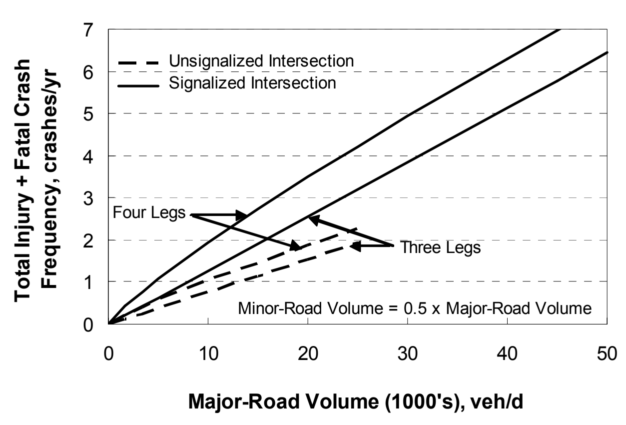


Figure 7-1. Illustrative Intersection Crash Trends.

For three-leg, unsignalized intersections:

$$C_{b,3u} = 0.0877\left(\frac{ADT_{major}}{1000}\right)^{0.766}\left(\frac{ADT_{minor}}{1000}\right)^{0.248} f_{3u} \qquad \textbf{(7-1)}$$

where:

C_b = base injury (plus fatal) crash frequency, crashes/yr;
ADT_{major} = average daily traffic volume on the major street, veh/d;
ADT_{minor} = average daily traffic volume on the minor street, veh/d; and
f = local calibration factor.

For four-leg, unsignalized intersections:

$$C_{b,4u} = 0.172\left(\frac{ADT_{major}}{1000}\right)^{0.596}\left(\frac{ADT_{minor}}{1000}\right)^{0.260} f_{4u} \qquad \textbf{(7-2)}$$

For three-leg, signalized intersections:

$$C_{b,3s} = 0.159\left(\frac{ADT_{major}}{1000}\right)^{0.629}\left(\frac{ADT_{minor}}{1000}\right)^{0.385} f_{3s} \qquad \textbf{(7-3)}$$

For four-leg, signalized intersections:

$$C_{b,4s} = 0.353\left(\frac{ADT_{major}}{1000}\right)^{0.459}\left(\frac{ADT_{minor}}{1000}\right)^{0.397} f_{4s} \qquad \textbf{(7-4)}$$

If a particular intersection has characteristics that differ from the base conditions, then the AMFs described in the next two sections can be used to obtain a more accurate estimate of intersection crash frequency.

A local calibration factor is identified in Equations 7-1 through 7-4. The factor can be used to adjust the computed value so that it is more consistent with typical intersections in the agency's jurisdiction. A calibration procedure is identified in reference *4*. A calibration factor of 1.0 should be used unless a local calibration indicates another value is more appropriate.

Example Application

The Question: What is the expected crash frequency for an urban signalized intersection?

The Facts:
- Control mode: signalized
- Intersection legs: 4
- Major-street volume: 20,000 veh/d
- Minor-street volume: 10,000 veh/d

The Solution: Equation 7-4 is used to compute the expected crash frequency of 3.48 crashes/yr. The use of this equation is illustrated in the box at the right.

Example Application

The Question: What is the expected crash frequency for an urban unsignalized intersection?

The Facts:
- Control mode: unsignalized
- Intersection legs: 4
- Major-street volume: 16,000 veh/d
- Minor-street volume: 1600 veh/d

The Solution: Equation 7-2 is used to compute the expected crash frequency of 1.01 crashes/yr.

Table 7-1. Over-Dispersion Parameters.

Control Mode	Number of Intersection Legs	Over-Disp. Parameter (k)
Unsignalized[1]	3	1.20
	4	2.07
Signalized	3	2.89
	4	3.53

Note:
1 - Unsignalized intersections have an uncontrolled major street and a stop-controlled minor street.

Table 7-2. Base Conditions.

Characteristic	Approach Type	Base Condition by Control Type	
		Signalized	Unsignalized
Left-turn lanes	Major	both legs	both legs
	Minor	both legs	
Right-turn lanes	Major	none	none
	Minor	none	
Number of lanes	Major	4	4
	Minor	2	2
Right-turn channelization	Major	none	none
	Minor	none	none
Lane width	Major	12 ft	12 ft
	Minor	12 ft	12 ft
Shoulder width[1]	Major		1.5 ft
	Minor		1.5 ft
Median presence	Major		not present
	Minor		

Note:
1 - "Curb-and-gutter" section is assumed as typical with an equivalent shoulder width of 1.5 ft.

$$C_{b,4s} = 0.353 \left(\frac{20{,}000}{1000}\right)^{0.459} \left(\frac{10{,}000}{1000}\right)^{0.397} \times 1.0 \qquad \textbf{(7-5)}$$

$$= 3.48 \; crashes/yr$$

Accident Modification Factors – Signalized Intersections

Discussion

This section describes AMFs that can be used to evaluate the relationship between a design change and the corresponding change in injury (plus fatal) crash frequency. The AMFs that apply to signalized intersections are listed in Table 7-3. AMFs applicable to unsignalized intersections are presented in the next section.

There are many additional factors, other than those listed in Table 7-3, that are likely to have some influence on crash frequency. However, their relationship has yet to be quantified through research. The list of available AMFs for signalized intersections is likely to increase as new research in this area is undertaken.

All of the AMFs described in this section yield a value of 1.0 when the associated design component or element represents base conditions. A condition that is more generous (i.e., desirable) than the base condition results in an AMF of less than 1.0. A condition that is less generous will result in an AMF of more than 1.0.

Table 7-3. AMFs for Signalized Intersections.

Application	Accident Modification Factor
Geometric design	Left-turn lane Right-turn lane Number of lanes Right-turn channelization Lane width

Safety Relationship

The expected injury (plus fatal) crash frequency for a specific signalized intersection is computed using Equation 7-6. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are different from base conditions.

$$C = C_b \times AMF_{lw} \times AMF_{RT} \cdots \qquad \textbf{(7-6)}$$

where:
C = expected injury (plus fatal) crash frequency, crashes/yr;
C_b = base injury (plus fatal) crash frequency, crashes/yr;
AMF_{lw} = lane width accident modification factor; and
AMF_{RT} = right-turn lane accident modification factor.

Guidance

In application, all applicable AMFs should be identified for the subject intersection and then multiplied together. The base crash frequency C_b for signalized intersections is obtained from Equation 7-3 or 7-4. The product of the AMFs and C_b represents the expected injury (plus fatal) crash frequency for the subject intersection.

If the crash history is available for the intersection, then the over-dispersion parameters in Table 7-1 can be used with the empirical Bayes adjustment procedure described in reference *6* to increase the accuracy of the expected crash frequency (over that obtained from Equation 7-6).

Example Application

The Question: What is the expected crash frequency for a specific urban signalized intersection?

The Facts:
- Control mode: signalized
- Intersection legs: 4
- Major-street volume: 20,000 veh/d
- Minor-street volume: 10,000 veh/d
- Base crash frequency C_b: 3.48 crashes/yr
- Major-street lane width: 10 ft
- Minor-street lane width: 12 ft

The Solution: The intersection of interest has typical characteristics with the exception that its major-street lane width is 10 ft. As described later, the AMF for a lane width of 10 ft is 1.11. This AMF can be used with Equation 7-6 to estimate the expected crash frequency for the subject intersection as 3.86 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 3.48 \times 1.11 \\ &= 3.86 \textit{ crashes/yr} \end{aligned} \qquad (7\text{-}7)$$

Example Application

The Question: What is the expected change in crash frequency for the previous example intersection if a right-turn bay is added to each of the major-street approaches?

The Facts:
- Control mode: signalized
- Intersection legs: 4

The Solution: As described later, the AMF for adding two right-turn lanes at this intersection is 0.85. This AMF can be used with Equation 7-6 to estimate the expected crash frequency for the subject intersection as 3.28 crashes/yr. This represents a decrease of 0.58 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{RT} \\ &= 3.86 \times 0.85 \\ &= 3.28 \textit{ crashes/yr} \end{aligned} \qquad (7\text{-}8)$$

Left-Turn Lane - AMF_{LT}

Discussion

An exclusive left-turn lane (or bay) at an intersection provides a length of roadway within which left-turning vehicles can decelerate and store without disrupting the smooth flow of traffic in the adjacent through lane. The lack of a lane, or a bay of inadequate length, can lead to conflict between left-turning and through vehicles as well as poor traffic operations.

Safety Relationship

The relationship between left-turn lane presence and injury (plus fatal) crash frequency is shown in Table 7-4. The AMF value should be estimated using Equation 7-9. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF is applicable to the major- and minor-street approaches at a signalized intersection. At three-leg intersections, the minor street is the discontinuous route. This AMF is based on the concurrent provision of a left-turn phase with the left-turn lane.

The values from Equation 7-9 are appropriate when an exclusive turn lane is not provided or when it is provided but is not of adequate length. A turn lane is of adequate length if turning vehicles decelerate and store in it without impeding the flow of through traffic.

Example Application

The Question: What is AMF_{LT} when only one bay is provided on the major street?

The Facts:

- Intersection legs: 4
- Major-street leg volume: 20,000 veh/d
- Minor-street leg volume: 10,000 veh/d

The Solution: Table 7-4 applies for this volume combination. It indicates that AMF_{LT} equals 1.18 (i.e., an 18 percent increase in crashes).

Table 7-4. AMF for Left-Turn Lanes at a Signalized Intersection.

Number of Intersection Legs	Approach Type	AMF Based on Number of Legs without a Left-Turn Lane [1]	
		One	**Both**
3	Major	1.22	not applicable [2]
4	Major	1.18	1.39
	Minor	1.09	1.19

Notes:
1 - Values based on minor-street volume equal to one-half of the major-street volume.
2 - Only one left-turn lane per roadway is likely at a three-leg intersection.

For four-leg intersections:

$$AMF_{LT} = AMF_1 \times AMF_2 \times AMF_3 \times AMF_4 \qquad \textbf{(7-9)}$$

Evaluate Rule 1 for each intersection leg i (i = 1, 2, 3, 4).

Rule 1: If leg i does *not* have a turn lane, then:

$$AMF_i = 1.54\, P_{leg,i} + 1.0\,(1 - P_{leg,i}) \qquad \textbf{(7-10)}$$

otherwise, if leg i has a turn lane, then AMF_i = 1.0.

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3 + ADT_4} \qquad \textbf{(7-11)}$$

where:
AMF_{LT} = left-turn lane accident modification factor;
AMF_i = leg i accident modification factor;
ADT_i = average daily traffic volume on leg i, veh/d; and
$P_{leg,i}$ = proportion of average daily traffic volume on leg i.

For three-leg intersections:

$$AMF_{LT} = AMF_1 \qquad \textbf{(7-12)}$$

Evaluate Rule 1 for the major-street leg i (i = 1).

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3} \qquad \textbf{(7-13)}$$

Base Condition:
major-street legs: left-turn lane (or bay) on both legs
minor-street legs: left-turn lane (or bay) on both legs

Right-Turn Lane - AMF_{RT}

Discussion

An exclusive right-turn lane (or bay) at an intersection provides a length of roadway within which right-turning vehicles can decelerate and store without disrupting the smooth flow of traffic in the adjacent through lane. The lack of a lane, or a bay of inadequate length, can lead to conflict between right-turning and through vehicles as well as poor traffic operations.

Safety Relationship

The relationship between right-turn lane presence and injury (plus fatal) crash frequency is shown in Table 7-5. The AMF value should be estimated using Equation 7-14. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF is applicable to the major- and minor-street approaches at a signalized intersection. At three-leg intersections, the minor street is the discontinuous route.

The value from Equation 7-14 is appropriate when an exclusive turn lane of adequate length is provided. A turn lane is of adequate length if turning vehicles can decelerate in it without impeding the flow of through traffic.

Example Application

The Question: How many crashes will likely be prevented by the addition of a right-turn lane on one major-street approach?

The Facts:

- Intersection legs: 4
- Major-street leg volume: 20,000 veh/d
- Minor-street leg volume: 10,000 veh/d
- Base crash frequency C_b: 3.48 crashes/yr

The Solution: From Table 7-5, find the AMF of 0.92. Expected crash frequency after the turn lane is installed is 3.20 crashes/yr. Thus, the change is likely to prevent about 0.28 crashes/yr.

Table 7-5. AMF for Right-Turn Lanes at a Signalized Intersection.

Number of Intersection Legs	Approach Type	AMF Based on Number of Legs with a Right-Turn Lane [1]	
		One	Both
3	Major	0.90	not applicable [2]
4	Major	0.92	0.85
	Minor	0.96	0.92

Notes:
1 - Values based on minor-street volume equal to one-half of the major-street volume.
2 - Only one right-turn lane per roadway is likely at a three-leg intersection.

For four-leg intersections:

$$AMF_{RT} = AMF_1 \times AMF_2 \times AMF_3 \times AMF_4 \qquad \textbf{(7-14)}$$

Evaluate Rule 1 for each intersection leg i (i = 1, 2, 3, 4).

Rule 1: If leg i has a turn lane, then:

$$AMF_i = 0.76\, P_{leg,i} + 1.0\,(1 - P_{leg,i}) \qquad \textbf{(7-15)}$$

otherwise, if leg i does *not* have a turn lane, then AMF_i = 1.0.

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3 + ADT_4} \qquad \textbf{(7-16)}$$

where:
AMF_{RT} = right-turn lane accident modification factor;
AMF_i = leg i accident modification factor;
ADT_i = average daily traffic volume on leg i, veh/d; and
$P_{leg,i}$ = proportion of average daily traffic volume on leg i.

For three-leg intersections:

$$AMF_{RT} = AMF_1 \qquad \textbf{(7-17)}$$

Evaluate Rule 1 for the major-street leg i (i = 1).

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3} \qquad \textbf{(7-18)}$$

Base Condition:
major-street legs: right-turn lane (or bay) not present
minor-street legs: right-turn lane (or bay) not present

Number of Lanes - AMF_{lane}

Discussion

Research indicates that the number of lanes at a signalized intersection is correlated with the frequency of crashes. The trend is one of more crashes with an increase in the number of lanes. The number of lanes in the cross section tends to increase the size of the intersection conflict area, which could increase the exposure of vehicles to conflict with crossing movements.

Safety Relationship

The relationship between the number of through lanes and injury (plus fatal) crash frequency is shown in Table 7-6. The AMF value should be estimated using Equation 7-19. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

The number of through lanes is counted at the intersection, regardless of whether the lanes are added or dropped away from the intersection. The number of lanes provided at the intersection is often dictated by capacity considerations. The AMF from Equation 7-19 should be used to obtain an accurate estimate of the expected crash frequency for a given cross section. This AMF is not intended to be used to justify a change in cross section.

Example Application

The Question: What is the AMF for the intersection of two, four-lane streets?

The Facts:

- Major-street through lanes: 4
- Minor-street through lanes: 4
- Major-street volume: 20,000 veh/d
- Minor-street volume: 10,000 veh/d

The Solution: Table 7-6 applies for this volume condition. It indicates that AMF_{lane} equals 1.16 for this lane arrangement.

Table 7-6. AMF for Number of Through Lanes at a Signalized Intersection.

Number of Major-Street Through Lanes	AMF Based on Number of Minor-Street Through Lanes [1]				
	2	3	4	5	6
2	0.78				
3	0.88	0.94			
4	1.00	1.07	1.16		
5	1.15	1.23	1.33	1.45	
6	1.32	1.42	1.53	1.68	1.85

Note:
1 - Values based on minor-street volume equal to one-half of the major-street volume.

$$AMF_{lane} = AMF_{major} \times AMF_{minor} \qquad (7\text{-}19)$$

with,

$$AMF_{major} = e^{0.197(N_{major} - 4)}\, P_{major} + 1.0\,(1 - P_{major}) \qquad (7\text{-}20)$$

$$P_{major} = \frac{ADT_{major}}{ADT_{major} + ADT_{minor}} \qquad (7\text{-}21)$$

$$AMF_{minor} = e^{0.197(N_{minor} - 2)}\, P_{minor} + 1.0\,(1 - P_{minor}) \qquad (7\text{-}22)$$

$$P_{minor} = \frac{ADT_{minor}}{ADT_{major} + ADT_{minor}} \qquad (7\text{-}23)$$

where:
AMF_{lane} = number-of-lanes lane accident modification factor;
ADT_{major} = average daily traffic volume on the major street, veh/d;
N_{major} = number of through lanes on the major street;
P_{major} = proportion of average daily traffic volume on the major street;
ADT_{minor} = average daily traffic volume on the minor street, veh/d;
N_{minor} = number of through lanes on the minor street; and
P_{minor} = proportion of average daily traffic volume on the minor street.

Base Condition:
major street: 4 lanes
minor street: 2 lanes

Right-Turn Channelization - AMF_{CH}

Discussion

A channelized right-turn lane operates as a turning roadway that provides for the free (or possibly yield-controlled) flow of right-turning vehicles. It may be preceded by a right-turn bay, and it may be followed by an auxiliary lane or added through lane. The higher speed and circular path of the turning roadway complicate the driving task. Adequate sight lines are needed so drivers can be attentive to pedestrians crossing the roadway and to unexpected stops by the turning vehicle ahead.

Safety Relationship

The relationship between the presence of a channelized right-turn lane and injury (plus fatal) crash frequency is shown in Table 7-7. The AMF value should be estimated using Equation 7-24. This equation is based on the presence of a channelizing island, and represents the long-run average of an unknown mix of both raised-curb and flush islands. The estimate can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF is applicable to the major- and minor-street approaches at a signalized intersection. At three-leg intersections, the minor street is the discontinuous route. The channelizing island is delineated with pavement markings or a curb.

Example Application

The Question: What is the likely increase in crash frequency if right-turn channelization is added to one major-street approach?

The Facts:

- Intersection legs: 4
- Major-street leg volume: 20,000 veh/d
- Minor-street leg volume: 10,000 veh/d
- Base crash frequency C_b: 3.48 crashes/yr

The Solution: From Table 7-5, find the AMF of 1.09. Expected crash frequency after the change is 3.81 crashes/yr. Thus, the change is likely to increase crashes by 0.33 crashes/yr.

Table 7-7. AMF for Right-Turn Channelization at a Signalized Intersection.

Number of Intersection Legs	Approach Type	AMF Based on Number of Legs with Right-Turn Channelization [1]	
		One	Both
3	Major	1.11	not applicable [2]
	Minor	1.06	
4	Major	1.09	1.20
	Minor	1.05	1.10

Notes:
1 - Values based on minor-street volume equal to one-half of the major-street volume.
2 - Only one right-turn movement per roadway is likely at a three-leg intersection.

For four-leg intersections:

$$AMF_{CH} = AMF_1 \times AMF_2 \times AMF_3 \times AMF_4 \qquad (7\text{-}24)$$

Evaluate Rule 1 for each intersection leg i (i = 1, 2, 3, 4).

Rule 1: If leg i has right-turn channelization, then:

$$AMF_i = 1.28\, P_{leg,i} + 1.0\,(1 - P_{leg,i}) \qquad (7\text{-}25)$$

otherwise, if leg i does *not* have right-turn channelization, then AMF_i = 1.0.

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3 + ADT_4} \qquad (7\text{-}26)$$

where:
AMF_{CH} = right-turn channelization accident modification factor;
AMF_i = leg i accident modification factor;
ADT_i = average daily traffic volume on leg i, veh/d; and
$P_{leg,i}$ = proportion of average daily traffic volume on leg i.

For three-leg intersections:

$$AMF_{CH} = AMF_1 \times AMF_3 \qquad (7\text{-}27)$$

Evaluate Rule 1 for the major-street leg i (i = 1).
Evaluate Rule 1 for the minor-street leg i (i = 3).

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3} \qquad (7\text{-}28)$$

Base Condition:
major-street legs: right-turn channelization not present
minor-street legs: right-turn channelization not present

Lane Width - AMF_{lw}

Discussion

The width of the traffic lane at an intersection has a recognized influence on capacity. Narrow lanes tend to operate less efficiently because drivers are concerned about impact with adjacent vehicles and roadside objects. For these same reasons, a narrow lane is likely to be associated with more crashes. In fact, research indicates that crashes are more frequent at intersections with lanes narrower than 12 ft.

Safety Relationship

The relationship between lane width and injury (plus fatal) crash frequency is shown in Figure 7-2. The AMF value should be estimated with Equation 7-29. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

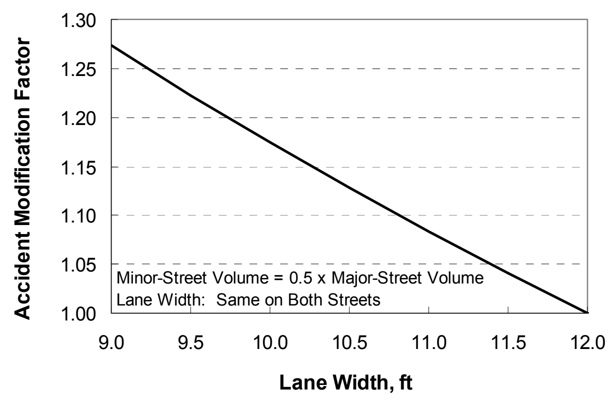


Figure 7-2. Lane Width AMF for a Signalized Intersection.

$$AMF_{lw} = AMF_{lw,major} \times AMF_{lw,minor} \qquad \textbf{(7-29)}$$

$$AMF_{lw,major} = e^{-0.053\,(W_{l,major} - 12)} \qquad \textbf{(7-30)}$$

$$AMF_{lw,minor} = \left(e^{-0.053\,(W_{l,minor} - 12)} - 1\right)\frac{ADT_{minor}}{ADT_{major}} + 1.0 \qquad \textbf{(7-31)}$$

where:
AMF_{lw} = lane width accident modification factor;
AMF_{major} = average daily traffic volume on the major street, veh/d;
AMF_{minor} = average daily traffic volume on the minor street, veh/d;
$W_{l,\,major}$ = lane width on the major street, ft; and
$W_{l,\,minor}$ = lane width on the minor street, ft.

Base Condition:
major street: 12-ft lane width
minor street: 12-ft lane width

Guidance

The lane width used to estimate the AMF is the average width of all major- and minor-street through lanes. The width of turn lanes is not considered. This AMF is applicable to lane widths ranging from 9 to 12 ft.

Example Application

The Question: What is the AMF for an intersection with a mix of lane widths?

The Facts:

- Major-street lane widths (left to right, in feet): 10.5, 9.5, 12 bay, 9.5, 10.5
- Minor-street lane width: 12 ft
- Major-street leg volume: 20,000 veh/d
- Minor-street leg volume: 10,000 veh/d

The Solution: The average through lane width is 10 ft (= [10.5+9.5+9.5+10.5]/4). The 12-ft left-turn bay width is not included in the average. From Equation 7-29, find the AMF value of 1.11.

Accident Modification Factors – Unsignalized Intersections

Discussion

This section describes AMFs that can be used to evaluate the relationship between a design change and the corresponding change in injury (plus fatal) crash frequency. The AMFs that apply to unsignalized intersections are listed in Table 7-8. AMFs for signalized intersections are presented in the previous section.

There are many additional factors, other than those listed in Table 7-8, that are likely to have some influence on crash frequency. However, their relationship has yet to be quantified through research. The list of available AMFs for unsignalized intersections is likely to increase as new research in this area is undertaken.

All of the AMFs described in this section yield a value of 1.0 when the associated design component or element represents base conditions. A condition that is more generous (i.e., desirable) than the base condition results in an AMF of less than 1.0. A condition that is less generous will result in an AMF of more than 1.0.

Table 7-8. AMFs for Unsignalized Intersections.

Application	Accident Modification Factor [1]
Geometric design	Left-turn lane Right-turn lane Number of lanes Right-turn channelization Lane width Shoulder width Median presence

Note:
1 - Factors listed only apply to intersections that have an uncontrolled major street and a stop-controlled minor street.

Safety Relationship

The expected injury (plus fatal) crash frequency for a specific unsignalized intersection is computed using Equation 7-6, repeated here as Equation 7-32. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are different from base conditions.

$$C = C_b \times AMF_{lw} \times AMF_{RT} \ldots \qquad \textbf{(7-32)}$$

where:
C = expected injury (plus fatal) crash frequency, crashes/yr;
C_b = base injury (plus fatal) crash frequency, crashes/yr;
AMF_{lw} = lane width accident modification factor; and
AMF_{RT} = right-turn lane accident modification factor.

Guidance

In application, all applicable AMFs should be quantified for the subject intersection and then multiplied together. The base crash frequency C_b for unsignalized intersections is obtained from Equation 7-1 or 7-2. The product of the AMFs and C_b represents the expected injury (plus fatal) crash frequency for the subject intersection.

Example Application

The Question: What is the expected crash frequency for a specific urban unsignalized intersection?

The Facts:

- Control mode: unsignalized
- Intersection legs: 4
- Major-street volume: 16,000 veh/d
- Minor-street volume: 1600 veh/d
- Base crash frequency C_b: 1.01 crashes/yr
- Major-street lane width: 10 ft
- Minor-street lane width: 12 ft

The Solution: The intersection of interest has typical characteristics with the exception that its major-street lane width is 10 ft. The AMF for a lane width of 10 ft is 1.12. This AMF can be used with Equation 7-6 to estimate the expected crash frequency for the subject intersection as 1.13 crashes/yr. This finding suggests an additional 0.12 crashes/yr at this intersection due to the 10-ft lane width (relative to an intersection with a 12-ft lane width).

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 1.01 \times 1.12 \\ &= 1.13 \ crashes/yr \end{aligned} \tag{7-33}$$

Left-Turn Lane - AMF_{LT}

Discussion

An exclusive left-turn lane (or bay) at an intersection provides a length of roadway within which left-turning vehicles can decelerate and store without disrupting the smooth flow of traffic in the adjacent through lane. The lack of a lane, or a bay of inadequate length, can lead to conflict between left-turning and through vehicles as well as poor traffic operations.

Safety Relationship

The relationship between left-turn lane presence and injury (plus fatal) crash frequency is shown in Table 7-9. The AMF value should be estimated using Equation 7-34. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF is applicable to the major-street approaches at an unsignalized intersection. At three-leg intersections, the minor street is the discontinuous route.

The values from Equation 7-34 are appropriate when an exclusive turn lane is not provided or when it is provided but is not of adequate length. A lane is of adequate length if turning vehicles store in it without impeding the flow of through traffic.

Example Application

The Question: What is the expected percentage increase in crashes if the left-turn bays on the major street are removed?

The Facts:

- Intersection legs: 4
- Major-street leg volume: 16,000 veh/d
- Minor-street leg volume: 1600 veh/d

The Solution: Table 7-9 applies for this volume combination. It indicates that AMF_{LT} is 1.73. This value suggests that crashes will increase 73 percent if the bays are removed.

Table 7-9. AMF for Left-Turn Lanes at an Unsignalized Intersection.

Number of Intersection Legs	Approach Type	AMF Based on Number of Legs without a Left-Turn Lane [1]	
		One	Both
3	Major	1.33	not applicable [2]
4	Major	1.31	1.73

Notes:
1 - Values based on minor-street volume equal to one-tenth of the major-street volume.
2 - Only one major-street left-turn lane is likely at a three-leg intersection.

For four-leg intersections:

$$AMF_{LT} = AMF_1 \times AMF_2 \qquad (7\text{-}34)$$

Evaluate Rule 1 for each major-street leg *i* (*i* = 1, 2).

Rule 1: If leg *i* does *not* have a turn lane, then:

$$AMF_i = 1.69\, P_{leg,i} + 1.0\,(1 - P_{leg,i}) \qquad (7\text{-}35)$$

otherwise, if leg *i* has a turn lane, then $AMF_i = 1.0$.

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3 + ADT_4} \qquad (7\text{-}36)$$

where:
AMF_{LT} = left-turn lane accident modification factor;
AMF_i = leg *i* accident modification factor;
ADT_i = average daily traffic volume on leg *i*, veh/d; and
$P_{leg,i}$ = proportion of average daily traffic volume on leg *i*.

For three-leg intersections:

$$AMF_{LT} = AMF_1 \qquad (7\text{-}37)$$

Evaluate Rule 1 for the major-street leg *i* (*i* = 1).

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3} \qquad (7\text{-}38)$$

Base Condition:
major-street legs: left-turn lane (or bay) on both legs

Right-Turn Lane - AMF_{RT}

Discussion

An exclusive right-turn lane (or bay) at an intersection provides a length of roadway within which right-turning vehicles can decelerate without disrupting the smooth flow of traffic in the adjacent through lanes. The lack of a lane, or a bay of inadequate length, can lead to conflict between right-turning and through vehicles as well as poor traffic operations.

Safety Relationship

The relationship between right-turn lane presence and injury (plus fatal) crash frequency is shown in Table 7-10. The AMF value should be estimated using Equation 7-39. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF is applicable to the major-street approaches at an unsignalized intersection. At three-leg intersections, the minor street is the discontinuous route.

The values from Equation 7-39 are appropriate when an exclusive turn lane of adequate length is provided. A turn lane is of adequate length if turning vehicles can decelerate in it without impeding the flow of through traffic.

Example Application

The Question: If right-turn bays are installed on both major-street approaches, how many crashes will be prevented?

The Facts:

- Intersection legs: 4
- Major-street leg volume: 16,000 veh/d
- Minor-street leg volume: 1600 veh/d
- Base crash frequency C_b: 1.01 crashes/yr

The Solution: From Table 7-10, find the AMF of 0.89. When used in Equation 7-6, the result is 0.89 crashes/yr (= 1.01 ×0.89). Thus, the installation of the right-turn bay equates to a reduction of 0.12 crashes/yr.

Table 7-10. AMF for Right-Turn Lanes at an Unsignalized Intersection.

Number of Intersection Legs	Approach Type	AMF Based on Number of Legs with a Right-Turn Lane [1]	
		One	Both
3	Major	0.94	not applicable [2]
4	Major	0.94	0.89

Notes:
1 - Values based on minor-street volume equal to one-tenth of the major-street volume.
2 - Only one major-street right-turn lane is likely at a three-leg intersection.

For four-leg intersections:

$$AMF_{RT} = AMF_1 \times AMF_2 \quad \textbf{(7-39)}$$

Evaluate Rule 1 for each major-street leg *i* (*i* = 1, 2).

Rule 1: If leg *i* has a turn lane, then:

$$AMF_i = 0.87\, P_{leg,i} + 1.0\,(1 - P_{leg,i}) \quad \textbf{(7-40)}$$

otherwise, if leg *i* does *not* have a turn lane, then AMF_i = 1.0.

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3 + ADT_4} \quad \textbf{(7-41)}$$

where:
AMF_{RT} = right-turn lane accident modification factor;
AMF_i = leg *i* accident modification factor;
ADT_i = average daily traffic volume on leg *i*, veh/d; and
$P_{leg,i}$ = proportion of average daily traffic volume on leg *i*.

For three-leg intersections:

$$AMF_{RT} = AMF_1 \quad \textbf{(7-42)}$$

Evaluate Rule 1 for the major-street leg *i* (*i* = 1).

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3} \quad \textbf{(7-43)}$$

Base Condition:
major-street legs: right-turn lane (or bay) not present

Number of Lanes - AMF_{lane}

Discussion

Research indicates that the number of lanes at an unsignalized intersection is correlated with the frequency of crashes. The trend is one of fewer crashes with an increase in the number of lanes. More traffic on the major street, which typically coincides with more lanes, is likely to discourage minor-street crossing and left-turning movements. The resulting redistribution of traffic patterns may explain the aforementioned trend.

Safety Relationship

The relationship between the number of through lanes and injury (plus fatal) crash frequency is shown in Table 7-11. The AMF value should be estimated using Equation 7-44. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is four lanes on the major street and two lanes on the minor street.

Guidance

The number of through lanes is counted at the intersection, regardless of whether the lanes are added or dropped away from the intersection. The number of lanes provided at the intersection is often dictated by capacity considerations. The AMF from Equation 7-44 should be used to obtain an accurate estimate of the expected crash frequency for a given cross section. This AMF is not intended to be used to justify a change in cross section.

Example Application

The Question: What is the AMF for the intersection of two, four-lane streets?

The Facts:

- Major-street through lanes: 4
- Minor-street through lanes: 4
- Major-street volume: 16,000 veh/d
- Minor-street volume: 1600 veh/d

The Solution: Table 7-11 applies for this volume condition. It indicates that AMF_{lane} equals 0.98 for this lane arrangement.

Table 7-11. AMF for Number of Through Lanes at an Unsignalized Intersection.

Number of Major-Street Through Lanes	AMF Based on Number of Minor-Street Through Lanes [1]				
	2	3	4	5	6
2	1.28				
3	1.13	1.12			
4	1.00	0.99	0.98		
5	0.89	0.88	0.87	0.86	
6	0.78	0.78	0.77	0.76	0.76

Note:
1 - Values based on minor-street volume equal to one-tenth of the major-street volume.

$$AMF_{lane} = AMF_{major} \times AMF_{minor} \quad (7\text{-}44)$$

with,

$$AMF_{major} = e^{-0.135(N_{major}-4)} P_{major} + 1.0\,(1 - P_{major}) \quad (7\text{-}45)$$

$$P_{major} = \frac{ADT_{major}}{ADT_{major} + ADT_{minor}} \quad (7\text{-}46)$$

$$AMF_{minor} = e^{-0.135(N_{minor}-2)} P_{minor} + 1.0\,(1 - P_{minor}) \quad (7\text{-}47)$$

$$P_{minor} = \frac{ADT_{minor}}{ADT_{major} + ADT_{minor}} \quad (7\text{-}48)$$

where:
AMF_{lane} = number-of-lanes lane accident modification factor;
ADT_{major} = average daily traffic volume on the major street, veh/d;
N_{major} = number of through lanes on the major street;
P_{major} = proportion of average daily traffic volume on the major street;
ADT_{minor} = average daily traffic volume on the minor street, veh/d;
N_{minor} = number of through lanes on the minor street; and
P_{minor} = proportion of average daily traffic volume on the minor street.

Base Condition:
major street: 4 lanes
minor street: 2 lanes

Right-Turn Channelization - AMF_{CH}

Discussion

A channelized right-turn lane operates as a turning roadway that provides for the free (or possibly Yield-controlled) flow of right-turning vehicles. It may be preceded by a right-turn bay and it may be followed by an auxiliary lane or added through lane. The higher speed and circular path of the turning roadway complicate the driving task. Adequate sight lines are needed so drivers can be attentive to pedestrians crossing the roadway and to unexpected stops by the turning vehicle ahead.

Safety Relationship

The relationship between the presence of a channelized right-turn lane and injury (plus fatal) crash frequency is shown in Table 7-7. The AMF value should be estimated using Equation 7-49. This equation is based on the presence of a channelizing island, and represents the long-run average of an unknown mix of both raised-curb and flush islands. The estimate can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

Guidance

This AMF is applicable to the major- and minor-street approaches at an unsignalized intersection. At three-leg intersections, the minor street is the discontinuous route. The channelizing island is delineated using pavement markings or a curb.

Example Application

The Question: What is the likely increase in crash frequency if right-turn channelization is added to one major-street approach?

The Facts:

- Intersection legs: 4
- Major-street leg volume: 16,000 veh/d
- Minor-street leg volume: 1600 veh/d
- Base crash frequency C_b: 1.01 crashes/yr

The Solution: From Table 7-5, find the AMF of 1.70. Expected crash frequency after the change is 1.72 crashes/yr. Thus, the change is likely to increase crashes by 0.71 crashes/yr.

Table 7-12. AMF for Right-Turn Channelization at an Unsignalized Intersection.

Number of Intersection Legs	Approach Type	AMF Based on Number of Legs with Right-Turn Channelization [1]	
		One	Both
3	Major	1.74	not applicable [2]
	Minor	1.07	
4	Major	1.70	2.91
	Minor	1.07	1.15

Notes:
1 - Values based on minor-street volume equal to one-tenth of the major-street volume.
2 - Only one right-turn movement per roadway is likely at a three-leg intersection.

For four-leg intersections:

$$AMF_{CH} = AMF_1 \times AMF_2 \times AMF_3 \times AMF_4 \quad \textbf{(7-49)}$$

Evaluate Rule 1 for each intersection leg i (i = 1, 2, 3, 4).

Rule 1: If leg i has right-turn channelization, then:

$$AMF_i = 2.55\, P_{leg,i} + 1.0\,(1 - P_{leg,i}) \quad \textbf{(7-50)}$$

otherwise, if leg i does *not* have right-turn channelization, then AMF_i = 1.0.

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3 + ADT_4} \quad \textbf{(7-51)}$$

where:
AMF_{CH} = right-turn channelization accident modification factor;
AMF_i = leg i accident modification factor;
ADT_i = average daily traffic volume on leg i, veh/d; and
$P_{leg,i}$ = proportion of average daily traffic volume on leg i.

For three-leg intersections:

$$AMF_{CH} = AMF_1 \times AMF_3 \quad \textbf{(7-52)}$$

Evaluate Rule 1 for the major-street leg i (i = 1).
Evaluate Rule 1 for the minor-street leg i (i = 3).

with,

$$P_{leg,i} = \frac{ADT_i}{ADT_1 + ADT_2 + ADT_3} \quad \textbf{(7-53)}$$

Base Condition:
major-street legs: right-turn channelization not present
minor-street legs: right-turn channelization not present

Lane Width - AMF_{lw}

Discussion

The width of the traffic lane at an intersection has a recognized influence on capacity. Narrow lanes tend to operate less efficiently because drivers are concerned about impact with adjacent vehicles and roadside objects. For these same reasons, a narrow lane is likely to be associated with more crashes. In fact, research indicates that crashes are more frequent at intersections with lanes narrower than 12 ft.

Safety Relationship

The relationship between lane width and injury (plus fatal) crash frequency is shown in Figure 7-3. The AMF value should be estimated with Equation 7-54. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1.

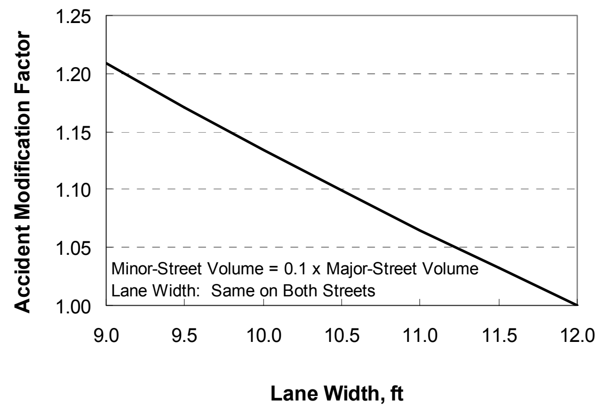


Figure 7-3. Lane Width AMF for an Unsignalized Intersection.

$$AMF_{lw} = AMF_{lw,major} \times AMF_{lw,minor} \qquad \textbf{(7-54)}$$

$$AMF_{lw,major} = e^{-0.057\,(W_{l,major} - 12)} \qquad \textbf{(7-55)}$$

$$AMF_{lw,minor} = \left(e^{-0.057\,(W_{l,minor} - 12)} - 1\right)\frac{ADT_{minor}}{ADT_{major}} + 1.0 \qquad \textbf{(7-56)}$$

where:

AMF_{lw} = lane width accident modification factor;
AMF_{major} = average daily traffic volume on the major street, veh/d;
AMF_{minor} = average daily traffic volume on the minor street, veh/d;
$W_{l,\,major}$ = lane width on the major street, ft; and
$W_{l,\,minor}$ = lane width on the minor street, ft.

Guidance

The lane width used to estimate the AMF is the average width of all major- and minor-street through lanes. The width of turn lanes is not considered. This AMF is applicable to lane widths ranging from 9 to 12 ft.

Base Condition:
major street: 12-ft lane width
minor street: 12-ft lane width

Example Application

The Question: What is the AMF for an intersection with a mix of lane widths?

The Facts:

- Major-street lane widths (left to right, in feet): 10, 14 bay, 10
- Minor-street lane width: 12 ft
- Major-street leg volume: 16,000 veh/d
- Minor-street leg volume: 1600 veh/d

The Solution: The average through lane width is 10 ft (= [10+10]/2). The 14-ft left-turn bay width is not included in the average. From Equation 7-54, find the AMF value of 1.12.

Shoulder Width - AMF_{sw}

Discussion

Shoulders offer several safety benefits for urban intersections. Depending on their width, shoulders may provide space for disabled vehicles, bicycle traffic, evasive maneuvers, and space within which right-turning vehicles can decelerate. In urban areas, curbing is often combined with the shoulder to control access and drainage.

Safety Relationship

The relationship between shoulder width and injury (plus fatal) crash frequency is shown in Figure 7-4. The AMF value should be estimated using Equation 7-57. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is a 1.5-ft effective shoulder width, as obtained from a curb-and-gutter cross section.

Guidance

This AMF applies to the shoulders on the major- and minor-street approaches to the intersection. The shoulder width used to estimate the AMF is the average width of the outside shoulders on each leg. This AMF is applicable to shoulder widths ranging from 0 to 5 ft.

Example Application

The Question: What is the shoulder width AMF for the intersection shown at right?

The Facts:

- Minor-street shoulder width: 1.5 ft
- Major-street volume: 16,000 veh/d
- Minor-street volume: 1600 veh/d

The Solution: The average shoulder width is 3.0 ft (= [2 + 2 + 4 + 4]/4). From Equation 7-57, find the AMF of 0.97. This AMF suggests this intersection will have 3.0 percent fewer crashes than an intersection with a 1.5-ft shoulder width.

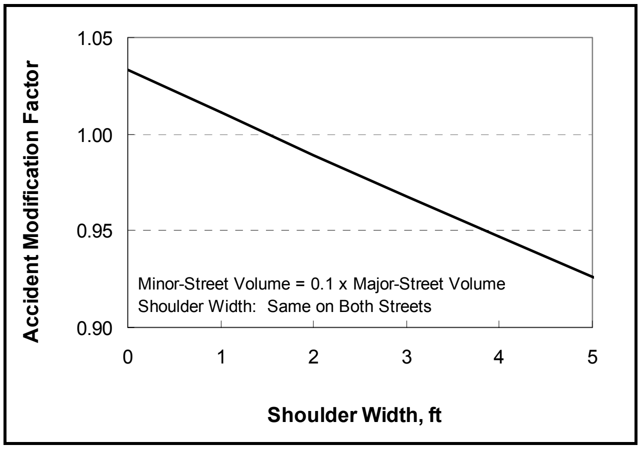


Figure 7-4. Shoulder Width AMF for an Unsignalized Intersection.

$$AMF_{sw} = AMF_{sw,major} \times AMF_{sw,minor} \qquad (7\text{-}57)$$

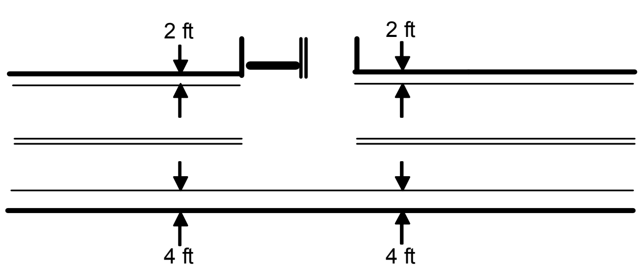
$$AMF_{sw,major} = e^{-0.020\,(W_{s,major} - 1.5)} \qquad (7\text{-}58)$$

$$AMF_{sw,minor} = \left(e^{-0.020\,(W_{s,minor} - 1.5)} - 1\right)\frac{ADT_{minor}}{ADT_{major}} + 1.0 \qquad (7\text{-}59)$$

where:

AMF_{sw} = shoulder width accident modification factor;
AMF_{major} = average daily traffic volume on the major street, veh/d;
AMF_{minor} = average daily traffic volume on the minor street, veh/d;
$W_{s,\,major}$ = shoulder width on the major street, ft; and
$W_{s,\,minor}$ = shoulder width on the minor street, ft.

Base Condition:
major street: 1.5-ft shoulder width
minor street: 1.5-ft shoulder width



Median Presence - AMF_{mp}

Discussion

A median provides several functions including positive separation between opposing traffic streams, a sheltered location for left-turning vehicles, refuge for pedestrians, and control of access in the vicinity of the intersection. However, in areas with high volume turning and crossing traffic, a wider median can present some additional driving challenges. Opposing left-turn vehicles tend to block the driver's view of oncoming vehicles. Notably wide medians tend to require a secondary stop in the median by crossing or turning vehicles.

Safety Relationship

The relationship between median presence and injury (plus fatal) crash frequency can be estimated using Figure 7-5 or Equation 7-60. The estimate represents the long-run average of many sites. It can vary for any single site. More discussion of AMF variability is provided in Chapter 1. The base condition for this AMF is an undivided major street (i.e., $AMF_{mp} = 1.0$).

Guidance

This AMF applies to medians on the major street. The presence of a median on the minor street is not addressed by this AMF. The median should extend back from the stop line for a distance of 250 ft or more. The median should also be at least 4 ft in width.

Example Application

The Question: What percent change in crashes should occur after installing a 20-ft median in the vicinity of a three-leg intersection?

The Facts:

- Intersection legs: 3
- Left-turn bay: present prior to median
- Existing condition: no median

The Solution: From Figure 7-5, find AMF_{mp} of 1.03. It suggests that crashes will increase about 3 percent due to the median addition.

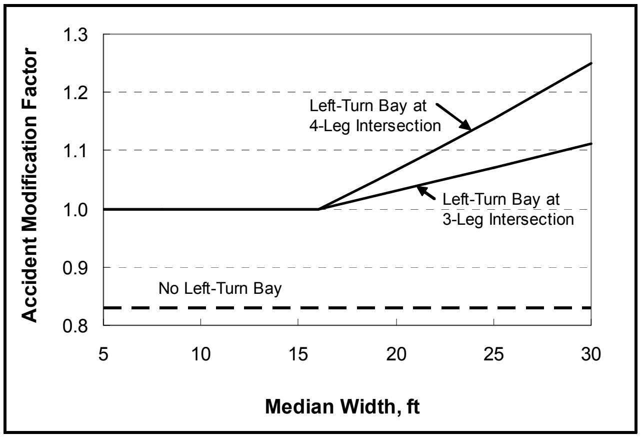


Figure 7-5. Median Presence AMF for an Unsignalized Intersection.

$$AMF_{mp} = AMF_1 \times AMF_2 \quad (7\text{-}60)$$

Evaluate Rules 1 and 2.

Rule 1: If a left-turn lane is present and the median is 16 ft or more in width, then:

$$AMF_1 = \begin{cases} e^{0.0076\,(W_m - 16)} & : \textit{if intersection has 3 legs} \\ e^{0.0160\,(W_m - 16)} & : \textit{if intersection has 4 legs} \end{cases} \quad (7\text{-}61)$$

otherwise, $AMF_1 = 1.0$.

Rule 2: If a left-turn lane is *not* present and a median is present, then:

$$AMF_2 = 0.83 \quad (7\text{-}62)$$

otherwise, $AMF_2 = 1.0$.

where:
AMF_{mp} = median presence accident modification factor; and
W_m = median width (including bay, if present), ft.

Base Condition: no median on major street

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Appendix A

Worksheets

TABLE OF CONTENTS

Freeways Worksheet (1 of 3)

General Information		**Site Information**	
Analyst:	____________	Freeway number:	____________
Agency:	____________	Freeway segment:	____________
Date performed:	____________	District:	____________
Location:		Analysis year:	

Input Data			
Crash Data	**Crash Period**	**Analysis Year**	If crash data are not available or the number of through lanes are different for the crash period and analysis year, then complete only the Analysis Year column.
Crash data time period:			_____ Start date / _____ End date
Count of injury + fatal crashes (X), crashes:	____ Multiple-vehicle (non-ramp) ____ Single-vehicle	____ Ramp-exit- related ____ Ramp-entrance-related	
Basic Roadway Data			
Number of through lanes:			Same value for crash period and analysis year.
Area type:	___ Urban	___ Rural	Same type for crash period and analysis year.
Segment length (L), mi:			
Number of ramp entrances (n_{enr}):			
Number of ramp exits (n_{exr}):			
Traffic Data			
Speed limit (V), mph:			
Percent trucks represented in ADT (P_t), %:			
Average daily traffic (ADT), veh/d:			
Geometric Data			
Presence of horizontal curve:	___ Y/N	___ Y/N	
Curve radius (R), ft:			
Curve length (L_c), mi:			
Percent grade (g), %:			
Cross Section Data			
Lane width (W_l), ft:			
Outside shoulder width (W_s), ft:			
Inside shoulder width (W_{is}), ft:			
Median width (W_m), ft:			
Presence of short lengths of barrier in median:	___ Y/N	___ Y/N	If Yes, fill out barrier worksheet.
Presence of continuous median barrier:	___ Y/N	___ Y/N	
Presence of shoulder rumble strips:	___ Y/N	___ Y/N	
Roadside Data			
Horizontal clearance (W_{hc}), ft:			
Presence of short lengths of barrier on roadside:	___ Y/N	___ Y/N	If Yes, fill out barrier worksheet.
Presence of continuous roadside barrier:	___ Y/N	___ Y/N	
Access Data			
Presence of one or more ramp entrances:	___ Y/N	___ Y/N	If Yes, fill out the ramp entrance worksheet.
Presence of one or more weaving sections:	___ Y/N	___ Y/N	If Yes, fill out the weaving section worksheet.

Freeways Worksheet (2 of 3)

Accident Modification Factors (AMF)				
		Crash Period	***Analysis Year***	If crash data are not available, then complete only the Analysis Year column.
Horizontal curve radius (*AMF_{cr}*):	Equation 2-25			
Grade (*AMF_{g}*):	Equation 2-26			
Lane width (*AMF_{lw}*):	Equation 2-27			
Outside shoulder width (*AMF_{osw}*):	Equation 2-28			
Inside shoulder width (*AMF_{isw}*):	Equation 2-30			
Median width (no barrier) (*AMF_{mwnb}*):	Equation 2-32			
Median width (some barrier) (*AMF_{mwsb}*):	Equation 2-33			
Median width (full barrier) (*AMF_{mwfb}*):	Equation 2-38 or 2-39			
Shoulder rumble strips (*AMF_{srs}*):	Equation 2-42			
Outside clearance (no barrier) (*AMF_{ocnb}*):	Equation 2-43			
Outside clearance (some barrier) (*AMF_{ocsb}*):	Equation 2-44			
Outside clearance (full barrier) (*AMF_{ocfb}*):	Equation 2-49			
Aggregated ramp entrance (*$AMF_{enr\|agg}$*):	Equation 2-53			
Aggregated weaving section (*$AMF_{wev\|agg}$*):	Equation 2-56			
Truck presence (*AMF_{tk}*):	Equation 2-59			
Combined AMF (product of all AMFs above) (*$AMF_{combined}$*):				Multiply all AMFs evaluated, disregard others.

Freeways Worksheet (3 of 3)

Expected Crash Frequency				
Calibration factor (f):				
Multiple-Vehicle Crash Analysis				
Over-dispersion parameter (k):	Table 2-1			
Base crash frequency (C_b), crashes/yr:	Equation 2-3, 2-9, 2-14, or 2-19			Multiply by 0.860 if area type is rural.
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1+ (C\ y)/(k\ L)]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C_{mv}): If crash data are available, then: If crash data are NOT available, then:	 $C_{mv} = C_x\ C_{analysis\ year}/C_{crash\ period}$ $C_{mv} = C_{analysis\ year}$			
Single-Vehicle Crash Analysis				
Over-dispersion parameter (k):	Table 2-1			
Base crash frequency (C_b), crashes/yr:	Equation 2-4, 2-10, 2-15, or 2-20			Multiply by 0.991 if area type is rural.
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1+ (C\ y)/(k\ L)]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C_{sv}): If crash data are available, then: If crash data are NOT available, then:	 $C_{sv} = C_x\ C_{analysis\ year}/C_{crash\ period}$ $C_{sv} = C_{analysis\ year}$			
Ramp Entrance Crash Analysis				
Over-dispersion parameter (k):	Table 2-1			
Base crash frequency (C_b), crashes/yr:	Equation 2-5, 2-11, 2-16, or 2-21			Multiply by 0.638 if area type is rural.
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1+ (C\ y)/k]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C_{enr}): If crash data are available, then: If crash data are NOT available, then:	 $C_{enr} = C_x\ C_{analysis\ year}/C_{crash\ period}$ $C_{enr} = C_{analysis\ year}$			
Ramp Exit Crash Analysis				
Over-dispersion parameter (k):	Table 2-1			
Base crash frequency (C_b), crashes/yr:	Equation 2-6, 2-12, 2-17, or 2-22			Multiply by 3.51 if area type is rural.
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1+ (C\ y)/k]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C_{exr}): If crash data are available, then: If crash data are NOT available, then:	 $C_{exr} = C_x\ C_{analysis\ year}/C_{crash\ period}$ $C_{exr} = C_{analysis\ year}$			
Total expected crash freq., crashes/yr:	$C = (C_{mv} + C_{sv} + C_{enr} + C_{exr})\ f$			

Barrier Worksheet

Inside Barrier			
Segment length (L), mi:		Inside shoulder width (W_{is}), ft:	
Median width (W_m), ft:		Inside barrier width (W_{ib}), ft:	
Barrier Location	**Length ($L_{ib,off}$), mi**	**Width from Edge of Traveled Way to Face of Barrier (W_{off}), ft**	**Ratio ($L_{ib,off}$/[W_{off}-W_{is}])**
1.			
2.			
3.			
4.			
5.			
6.			
7.			
8.			
Sum1:		**Sum2:**	
Median Width (some barrier)			
Proportion of segment length with barrier in median P_{ib} = [Sum1] / [2 L]:		Width from edge of shoulder to barrier face W_{icb} = [Sum1] / [Sum2], ft:	
Median Width (full barrier)			
Width from edge of shoulder to barrier face (W_{icb}), ft: For barrier in center of median W_{icb} = [2 L] / [Sum2 + 2 (2 L - Sum1) / (W_m - 2 W_{is} - W_{ib})] For barrier adjacent to one roadbed: W_{icb} = [2 L] / [L /2 + Sum2 + (L - Sum1) / (W_m - 2 W_{is} - W_{ib} - 2.0)]			
Outside Barrier			
Segment length (L), mi:		Outside shoulder width (W_s), ft:	
Barrier Location	**Length ($L_{ob,off}$), mi**	**Width from Edge of Traveled Way to Face of Barrier (W_{off}), ft**	**Ratio ($L_{ob,off}$/[W_{off}-W_s])**
1.			
2.			
3.			
4.			
5.			
6.			
7.			
8.			
Sum1:		**Sum2:**	
Outside Clearance (some barrier)			
Proportion of segment length with barrier on roadside P_{ob} = [Sum1] / [2 L]:		Width from edge of shoulder to barrier face W_{ocb} = [Sum1] / [Sum2], ft:	
Outside Clearance (full barrier)			
Width from edge of shoulder to barrier face W_{ocb} = 2 / [1/($W_{off,1}$ - W_s) + 1/($W_{off,2}$ - W_s)], ft:			

Ramp Entrance and Weaving Section Worksheet

Ramp Entrance			
Segment length (L), mi:			
Ramp Entrance Location	**Length of Ramp Entrance in Segment ($L_{enr,seg}$), mi**	**Length of Ramp Entrance (L_{enr}), mi**	**Ratio ($L_{enr,seg}/L_{enr}$)**
1.			
2.			
3.			
4.			
5.			
6.			
7.			
8.			
Sum1:		**Sum2:**	
Proportion of segment length adjacent to a ramp entrance P_{enr} = [Sum1] / [2 L] :		Average ramp entrance length l_{enr} = 5280 [Sum1] / [Sum2], ft:	

Weaving Section			
Segment length (L), mi:			
Weaving Section Location	**Length of Weaving Section in Segment ($L_{wev,seg}$), mi**	**Length of Weaving Section (L_{wev}), mi**	**Ratio ($L_{wev,seg}/L_{wev}$)**
1.			
2.			
3.			
4.			
5.			
6.			
7.			
8.			
Sum1:		**Sum2:**	
Proportion of segment length adjacent to a weaving section P_{wev} = [Sum1] / [2 L]:		Average weaving section length l_{wev} = 5280 [Sum1] / [Sum2], ft:	

Rural Two-Lane Highways Worksheet (1 of 2)

General Information		***Site Information***	
Analyst:	________	Highway number:	________
Agency:	________	Roadway segment:	________
Date performed:	________	District:	________
Location:		Analysis year:	

Input Data					
Crash Data	***Crash Period***	***Analysis Year***	If crash data are not available, then complete only the Analysis Year column.		
Crash data time period:			_____ Start date	_____ End date	
Count of injury + fatal crashes (X), crashes:					
Basic Roadway Data					
Segment length (L), mi:					
Number of driveways, (n_d):			Two-way total. Driveway density (D_d) = n_d /L		
Traffic Data					
Speed limit (V), mph:					
Average daily traffic (ADT), veh/d:					
Geometric Data					
Presence of horizontal curve:	___ Y/N	___ Y/N			
Presence of spiral transition curves:	___ Y/N	___ Y/N			
Curve radius (R), ft:					
Curve length (L_c), mi:					
Guideline superelevation rate (e_d), %:			Rate specified by design guidelines.		
Superelevation rate (e), %:					
Percent grade (g), %:					
Presence of a passing or climbing lane:	___ Y/N	___ Y/N	If present, indicate number of directions (1 or 2).		
Cross Section Data					
Lane width (W_l), ft:					
Outside shoulder width (W_s), ft:					
Median type:	___ U/T	___ U/T	U - undivided; T - TWLTL		
Presence of shoulder rumble strips:	___ Y/N	___ Y/N			
Presence of centerline rumble strip:	___ Y/N	___ Y/N			
Roadside Data					
Horizontal clearance (W_{hc}), ft:					
Presence of short lengths of barrier on roadside:	___ Y/N	___ Y/N	If Yes, fill out barrier worksheet.		
Presence of continuous roadside barrier:	___ Y/N	___ Y/N			
Side slope (S_s), ft:	1V:___H	1V:___H			

Rural Two-Lane Highways Worksheet (2 of 2)

Accident Modification Factors (AMF)				
		Crash Period	***Analysis Year***	If crash data are not available, then complete only the Analysis Year column.
Horizontal curve radius (AMF_{cr}):	Equation 3-17			
Grade (AMF_{g}):	Equation 3-18			
Outside clearance (no barrier) (AMF_{ocnb}):	Equation 3-20			
Outside clearance (some barrier) (AMF_{ocsb}):	Equation 3-21			
Outside clearance (full barrier) (AMF_{ocfb}):	Equation 3-26			
Side slope (AMF_{ss}):	Equation 3-30			
Spiral transition curve (AMF_{sp}):	Equation 3-34			
Lane and shoulder width ($AMF_{lw,sw}$):	Equation 3-35			
Shoulder rumble strips (AMF_{srs}):	Equation 3-42			
Centerline rumble strip (AMF_{crs}):	Table 3-9			
TWLTL median type (AMF_{T}):	Equation 3-43			
Superelevation (AMF_{e}):	Figure 3-11			
Passing lane (AMF_{pass}):	Table 3-10			
Driveway density (AMF_{dd}):	Equation 3-45			
Combined AMF (product of all AMFs above) ($AMF_{combined}$):				Multiply all AMFs evaluated, disregard others.

Expected Crash Frequency				
Calibration factor (f):				
Over-dispersion parameter (k):	Table 3-1			
Base crash frequency (C_b), crashes/yr:	Equation 3-1			
Expected crash frequency (C), crashes/yr:	$C = C_b \, AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1 + (C\,y)/(k\,L)]$			
Adjusted crash frequency given X (C_x):	$C_X = C\,w + X/y\,(1 - w)$			
Expected crash frequency (C): If crash data are available, then: If crash data are NOT available, then:	 $C = C_x \, C_{analysis\ year} / C_{crash\ period}$ $C = C_{analysis\ year}$			

Rural Multilane Highways Worksheet (1 of 3)

General Information	**Site Information**
Analyst: ____________	Highway number: ____________
Agency: ____________	Roadway segment: ____________
Date performed: ____________	District: ____________
Location:	Analysis year:

Input Data			
Crash Data	**Crash Period**	**Analysis Year**	If crash data are not available or the number of through lanes is different for the crash period and analysis year, then complete only the Analysis Year column.
Crash data time period:			_____ Start date _____ End date
Count of injury + fatal crashes (X), crashes:	____ Multiple-veh. (non-driveway) ____ Single-vehicle		____ Driveway-related
Basic Roadway Data			
Segment length (L), mi:			
Number of residential driveways (n_{res}):			Two-way total.
Number of industrial driveways (n_{ind}):			Two-way total.
Number of business driveways (n_{bus}):			Two-way total.
Number of office driveways (n_{off}):			Two-way total.
Traffic Data			
Speed limit (V), mph:			
Percent trucks represented in ADT (P_t), %:			
Average daily traffic (ADT), veh/d:			
Geometric Data			
Presence of horizontal curve:	___ Y/N	___ Y/N	
Curve radius (R), ft:			
Curve length (L_c), mi:			
Percent grade (g), %:			
Cross Section Data			
Lane width (W_l), ft:			
Outside shoulder width (W_s), ft:			
Inside shoulder width (W_{is}), ft:			
Median type:	___ U/N/R	___ U/N/R	U - undivided; N - nonrestrictive; R - restrictive
Median width (W_m), ft:			
Presence of short lengths of barrier in median:	___ Y/N	___ Y/N	If Yes, fill out barrier worksheet.
Presence of continuous median barrier:	___ Y/N	___ Y/N	
Roadside Data			
Horizontal clearance (W_{hc}), ft:			
Presence of short lengths of barrier on roadside:	___ Y/N	___ Y/N	If Yes, fill out barrier worksheet.
Presence of continuous roadside barrier:	___ Y/N	___ Y/N	
Side slope (S_s), ft:	1V:___H	1V:___H	

Rural Multilane Highways Worksheet (2 of 3)

Accident Modification Factors (AMF)				
		Crash Period	***Analysis Year***	If crash data are not available, then complete only the Analysis Year column.
Horizontal curve radius (AMF_{cr}):	Equation 3-17			
Grade (AMF_{g}):	Equation 3-19			
Outside clearance (no barrier) (AMF_{ocnb}):	Equation 3-20			
Outside clearance (some barrier) (AMF_{ocsb}):	Equation 3-21			
Outside clearance (full barrier) (AMF_{ocfb}):	Equation 3-26			
Side slope (AMF_{ss}):	Equation 3-30			
Lane width (AMF_{lw}):	Equation 3-49			
Outside shoulder width (AMF_{osw}):	Equation 3-50			
Inside shoulder width (AMF_{isw}):	Equation 3-52			
Median width (no barrier) (AMF_{mwnb}):	Equation 3-54 or 3-55			
Median width (some barrier) (AMF_{mwsb}):	Equation 3-56			
Median width (full barrier) (AMF_{mwfb}):	Equation 3-60			
Truck presence (AMF_{tk}):	Equation 3-63			
Combined AMF (product of all AMFs above) ($AMF_{combined}$):				Multiply all AMFs evaluated, disregard others.

Rural Multilane Highways Worksheet (3 of 3)

Expected Crash Frequency				
Calibration factor (f):				
Multiple-Vehicle Crash Analysis				
Over-dispersion parameter (k):	Table 3-1			
Base crash frequency (C_b), crashes/yr:	Equation 3-3, 3-8, or 3-12			
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1 + (C\ y)/(k\ L)]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C_{mv}): If crash data are available, then: If crash data are NOT available, then:	 $C_{mv} = C_x\ C_{analysis\ year}/C_{crash\ period}$ $C_{mv} = C_{analysis\ year}$			
Single-Vehicle Crash Analysis				
Over-dispersion parameter (k):	Table 3-1			
Base crash frequency (C_b), crashes/yr:	Equation 3-4, 3-9, or 3-13			
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1 + (C\ y)/(k\ L)]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C_{sv}): If crash data are available, then: If crash data are NOT available, then:	 $C_{sv} = C_x\ C_{analysis\ year}/C_{crash\ period}$ $C_{sv} = C_{analysis\ year}$			
Driveway Crash Analysis				
Over-dispersion parameter (k):	Table 3-1			
No. of equivalent residential driveways (n_e):	Equation 3-6			
Base crash frequency (C_b), crashes/yr:	Equation 3-5, 3-10, or 3-14			
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1 + (C\ y)/k]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C_{dw}): If crash data are available, then: If crash data are NOT available, then:	 $C_{dw} = C_x\ C_{analysis\ year}/C_{crash\ period}$ $C_{dw} = C_{analysis\ year}$			
Total expected crash freq., crashes/yr:	$C = (C_{mv} + C_{sv} + C_{dw})\ f$			

Urban and Suburban Arterials Worksheet (1 of 3)

General Information		Site Information	
Analyst:	____________	Street number or name:	____________
Agency:	____________	Street segment:	____________
Date performed:	____________	District:	____________
Location:		Analysis year:	

Input Data			
Crash Data	***Crash Period***	***Analysis Year***	If crash data are not available or the number of through lanes is different for the crash period and analysis year, then complete only the Analysis Year column.
Crash data time period:			_____ Start date / _____ End date
Count of injury + fatal crashes (X), crashes:	____ Multiple-veh. (non-driveway) ____ Single-vehicle	____ Driveway-related	
Basic Roadway Data			
Number of through lanes:			Same value for crash period and analysis year.
Segment length (L), mi:			
Number of residential driveways (n_{res}):			Two-way total.
Number of industrial driveways (n_{ind}):			Two-way total.
Number of business driveways (n_{bus}):			Two-way total.
Number of office driveways (n_{off}):			Two-way total.
Curb miles next to residential land use (L_{res}), mi:			L_{res} + L_{ind} + L_{bus} + L_{off} must total to twice the length of the segment for the crash period. They must also total to twice the segment length for the analysis year.
Curb miles next to industrial land use (L_{ind}), mi:			
Curb miles next to business land use (L_{bus}), mi:			
Curb miles next to office land use (L_{off}), mi:			
Traffic Data			
Speed limit (V), mph:			
Percent trucks represented in ADT (P_t), %:			
Average daily traffic (ADT), veh/d:			
Geometric Data			
Presence of horizontal curve:	___ Y/N	___ Y/N	
Curve radius (R), ft:			
Curve length (L_c), mi:			
Cross Section Data			
Lane width (W_l), ft:			
Shoulder width (W_s), ft:			
Median type:	___ U/N/R	___ U/N/R	U - undivided; N - nonrestrictive; R - restrictive
Median width (W_m), ft:			
Curb Parking Data			
Presence of curb parking:	___ Y/N	___ Y/N	If Yes, fill out curb parking worksheet.
Roadside Data			
Number of utility poles along street (n_p), poles:			Two-way total. Pole density (D_p) = n_p / L
Utility pole offset (W_o), ft:			Average of both travel directions.

Urban and Suburban Arterials Worksheet (2 of 3)

Accident Modification Factors (AMF)				
		Crash Period	***Analysis Year***	If crash data are not available, then complete only the Analysis Year column.
Horizontal curve radius (AMF_{cr}):	Equation 4-33			
Lane width (AMF_{lw}):	Equation 4-34			
Shoulder width (AMF_{sw}):	Equation 4-35			
Median width (AMF_{mw}):	Equation 4-37 or 4-38			
Curb parking (AMF_{pk}):	Equation 4-39			
Utility pole offset (AMF_{pd}):	Equation 4-41			
Truck presence (AMF_{tk}):	Equation 4-43			
Combined AMF (product of all AMFs above) ($AMF_{combined}$):				Multiply all AMFs evaluated, disregard others.

Urban and Suburban Arterials Worksheet (3 of 3)

Expected Crash Frequency				
Calibration factor (f):				
Land use adjustment factor (F_{lu}):	Equation 4-6			
Multiple-Vehicle Crash Analysis				
Over-dispersion parameter (k):	Table 4-1			
Base crash frequency (C_b), crashes/yr:	Equation 4-2, 4-8, 4-12, 4-16, 4-20, 4-24, or 4-28			
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1+ (C\ y)/(k\ L)]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C_{mv}): If crash data are available, then: If crash data are NOT available, then:	 $C_{mv} = C_x\ C_{analysis\ year}/C_{crash\ period}$ $C_{mv} = C_{analysis\ year}$			
Single-Vehicle Crash Analysis				
Over-dispersion parameter (k):	Table 4-1			
Base crash frequency (C_b), crashes/yr:	Equation 4-3, 4-9, 4-13, 4-17, 4-21, 4-25, or 4-29			
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1+ (C\ y)/(k\ L)]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C_{sv}): If crash data are available, then: If crash data are NOT available, then:	 $C_{sv} = C_x\ C_{analysis\ year}/C_{crash\ period}$ $C_{sv} = C_{analysis\ year}$			
Driveway Crash Analysis				
Over-dispersion parameter (k):	Table 4-1			
Driveway spacing (S_d), mi/driveway:	$S_d = 2\ L / (n_{res} + n_{ind} + n_{bus} + n_{off} + 1)$			
No. of equivalent residential driveways (n_e):	Equation 4-5			
Base crash frequency (C_b), crashes/yr:	Equation 4-4, 4-10, 4-14, 4-18, 4-22, 4-26, or 4-30			
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1+ (C\ y)/k]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C_{dw}): If crash data are available, then: If crash data are NOT available, then:	 $C_{dw} = C_x\ C_{analysis\ year}/C_{crash\ period}$ $C_{dw} = C_{analysis\ year}$			
Total expected crash freq., crashes/yr:	$C = (C_{mv} + C_{sv} + C_{dw})\ f$			

Curb Parking Worksheet

Residential and Industrial Parking			
Parking on LEFT Side of Segment by Type		**Parking on RIGHT Side of Segment by Type**	
Parallel Parking Length, ft[1]	**Angle Parking Length, ft[1]**	**Parallel Parking Length, ft[1]**	**Angle Parking Length, ft[1]**
1.	1.		
2.	2.		
3.	3.		
4.	4.		
5.	5.		
6.	6.		
7.	7.		
8.	8.		
Sum1:	**Sum2:**	**Sum3:**	**Sum4:**
Business and Office Parking			
Parking on LEFT Side of Segment by Type		**Parking on RIGHT Side of Segment by Type**	
Parallel Parking Length, ft[1]	**Angle Parking Length, ft[1]**	**Parallel Parking Length, ft[1]**	**Angle Parking Length, ft[1]**
1.	1.		
2.	2.		
3.	3.		
4.	4.		
5.	5.		
6.	6.		
7.	7.		
8.	8.		
Sum5:	**Sum6:**	**Sum7:**	**Sum8:**
Analysis			
Segment length (L), mi:			
TOTAL Parking on LEFT Side of Segment by Type		**TOTAL Parking on RIGHT Side of Segment by Type**	
Parallel Parking Length, mi	**Angle Parking Length, mi**	**Parallel Parking Length, mi**	**Angle Parking Length, mi**
Sum15: = (Sum1+Sum5)/5280	Sum26: = (Sum2 + Sum6)/5280	Sum37: = (Sum3 + Sum7)/5280	Sum48: = (Sum4 + Sum8)/5280
Proportion of segment length with parking (P_{pk}):		P_{pk} = (Sum15 + Sum26 + Sum37 + Sum48) / (2 L) = ________	
Proportion of parking adjacent to business land use ($P_{b/o}$):		$P_{b/o}$ = (Sum5 +Sum6 +Sum7 +Sum8) / (10560 L P_{pk}) = _______	
Proportion of curb parking that is angle parking (P_{ap}):		P_{ap} = (Sum26 + Sum48) / (2 L P_{pk}) = ________	

Note:

1 - Record in each numbered row the length of one contiguous curb parking zone, as measured along the curb and only for the actual length of the stall(s) in the zone.

Interchange Ramp Worksheet

General Information		Site Information	
Analyst:	______	Highway number:	______
Agency:	______	Intersecting highway:	______
Date performed:	______	District:	______
Ramp location:		Analysis year:	

Input Data			
Traffic Data			
Average daily traffic volume on ramp (ADT_{ramp}), veh/d:			
Geometric Data			
Ramp type:	___ Exit	___ Entrance	
Ramp configuration:	___ Diagonal	___ Non-free-flow loop	
	___ Free-flow loop	___ Outer connection	
	___ Semi-direct connection	___ Direct connection	
	___ Button hook	___ Scissor	
	___ Slip	___ Other	

Expected Crash Frequency			
Base crash rate (*Base*), crashes/mv:	Table 5-1		
Local calibration factor (*f*):			
Base crash frequency (C_b), crashes/yr:	Equation 5-1		
Expected crash frequency (C), crashes/yr:	$C = C_b$		

Frontage Roads Worksheet

General Information		Site Information	
Analyst:		Highway number:	
Agency:		Roadway segment:	
Date performed:		District:	
Location:		Analysis year:	

Input Data			
Crash Data	**Crash Period**	**Analysis Year**	If crash data are not available, then complete only the Analysis Year column.
Crash data time period:			_____ Start date | _____ End date
Count of injury + fatal crashes (X), crashes:			
Basic Roadway Data			
Segment length (L), mi:			
Traffic Data			
Average daily traffic (ADT), veh/d:			
Cross Section Data			
Lane width (W_l), ft:			
Right-side paved shoulder width ($W_{s,r}$), ft:			
Left-side paved shoulder width ($W_{s,l}$), ft:			

Accident Modification Factors (AMF)		**Crash Period**	**Analysis Year**	If crash data are not available, then complete only the Analysis Year column.
Lane width (AMF_{lw}):	Equation 5-8			
Shoulder width (AMF_{sw}):	Equation 5-9			
Combined AMF (product of all AMFs above) ($AMF_{combined}$):				Multiply all AMFs evaluated, disregard others.

Expected Crash Frequency				
Calibration factor (f):				
Over-dispersion parameter (k):	Table 5-3			
Base crash frequency (C_b), crashes/yr:	Equation 5-4			
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1+ (C\ y)/(k\ L)]$			
Adjusted crash frequency given X (C_x):	$C_x = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C): If crash data are available, then: If crash data are NOT available, then:	 $C = C_x\ C_{analysis\ year} / C_{crash\ period}$ $C = C_{analysis\ year}$			

Rural Signalized Intersection Worksheet (1 of 2)

General Information	***Site Information***
Analyst: ________	Highway number: ________
Agency: ________	Intersecting highway: ________
Date performed: ________	District: ________
Location:	Analysis year:

Input Data				
Crash Data		***Crash Period***	***Analysis Year***	If crash data are not available, the number of legs are different for the crash period and analysis year, or the number of lanes are different for the crash period and analysis year, then complete only the Analysis Year column.
Crash data time period:				_____ Start date / _____ End date
Count of injury + fatal crashes (X), crashes:				
Basic Intersection Data				
Number of through lanes:	Major			Same value for crash period and analysis year.
	Minor			Same value for crash period and analysis year.
Number of intersection legs:		___ 3 legs	___ 4 legs	Same value for crash period and analysis year.
Traffic Data				
Average daily traffic (ADT), veh/d: (two-way volume by leg)	Major 1			Major-road leg 1 ADT.
	Major 2			Major-road leg 2 ADT.
	Minor 1			Minor-road leg 1 ADT.
	Minor 2			Minor-road leg 2 ADT (not applicable for 3 leg).
Percent trucks in peak hour (P_t), %:				
Cross Section Data				
Legs with a left-turn lane:	Major	___ 1 or 2	___ 1 or 2	
	Minor	___ 1 or 2	___ 1 or 2	
Legs with a right-turn lane:	Major	___ 1 or 2	___ 1 or 2	
	Minor	___ 1 or 2	___ 1 or 2	
Access Data				
Number of driveways (d_n):	Major			
	Minor			

Rural Signalized Intersection Worksheet (2 of 2)

ADT Distribution				
		Crash Period	***Analysis Year***	If crash data are not available, then complete only the Analysis Year column.
Proportion of ADT on leg (P_{leg}):	Major 1			
	Major 2			
	Minor 1			
	Minor 2			
Average ADT on road, veh/d:	Major			
	Minor			
Accident Modification Factors (AMF)				
		Crash Period	***Analysis Year***	If crash data are not available, then complete only the Analysis Year column.
Left-turn lane (AMF_{LT}):	Equation 6-9			
Right-turn lane (AMF_{RT}):	Equation 6-15			
Number of lanes (AMF_{lane}):	Table 6-6			
Driveway frequency (AMF_{nd}):	Equation 6-20			
Truck presence (AMF_{tk}):	Equation 6-24			
Combined AMF (product of all AMFs above) ($AMF_{combined}$):				Multiply all AMFs evaluated, disregard others.
Expected Crash Frequency				
Calibration factor (f):				
Over-dispersion parameter (k):	Table 6-1			
Base crash frequency (C_b), crashes/yr:	Equation 6-3 or 6-4			
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1+ (C\ y)/k]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C): If crash data are available, then: If crash data are NOT available, then:	 $C = C_x\ C_{analysis\ year} / C_{crash\ period}$ $C = C_{analysis\ year}$			

Rural Unsignalized Intersection Worksheet (1 of 2)

General Information		Site Information	
Analyst:	______________	Highway number:	______________
Agency:	______________	Intersecting highway:	______________
Date performed:	______________	District:	______________
Location:		Analysis year:	

Input Data					
Crash Data		***Crash Period***	***Analysis Year***	If crash data are not available, the number of legs are different for the crash period and analysis year, or the number of lanes are different for the crash period and analysis year, then complete only the Analysis Year column.	
Crash data time period:				_____ Start date	_____ End date
Count of injury + fatal crashes (X), crashes:					
Basic Intersection Data					
Number of through lanes:	Major			Same value for crash period and analysis year.	
	Minor			Same value for crash period and analysis year.	
Number of intersection legs:		___ 3 legs	___ 4 legs	Same value for crash period and analysis year.	
Traffic Data					
Average daily traffic (ADT), veh/d: (two-way volume by leg)	Major 1			Major-road leg 1 ADT.	
	Major 2			Major-road leg 2 ADT.	
	Minor 1			Minor-road leg 1 ADT.	
	Minor 2			Minor-road leg 2 ADT (not applicable for 3 leg).	
Percent trucks in peak hour (P_t), %:					
Cross Section Data					
Legs with a left-turn lane:	Major	___ 1 or 2	___ 1 or 2		
Legs with a right-turn lane:	Major	___ 1 or 2	___ 1 or 2		
Shoulder width (W_s): ft:	Major				
	Minor				
Median width (W_m), ft:	Major				
Alignment skew angle (I_{sk}), degrees:					
Access Data					
Number of driveways (d_n):	Major				
	Minor				

Rural Unsignalized Intersection Worksheet (2 of 2)

ADT Distribution				
		Crash Period	***Analysis Year***	If crash data are not available, then complete only the Analysis Year column.
Proportion of ADT on leg (P_{leg}):	Major 1			
	Major 2			
	Minor 1			
	Minor 2			
Average ADT on road, veh/d:	Major			
	Minor			

Accident Modification Factors (AMF)				
		Crash Period	***Analysis Year***	If crash data are not available, then complete only the Analysis Year column.
Left-turn lane (AMF_{LT}):	Equation 6-28			
Right-turn lane (AMF_{RT}):	Equation 6-33			
Number of lanes (AMF_{lane}):	Table 6-10			
Shoulder width (AMF_{sw}):	Equation 6-38			
Median presence (AMF_{mp}):	Equation 6-42			
Alignment skew angle (AMF_{skew}):	Equation 6-45 or 6-46			
Driveway frequency (AMF_{nd}):	Equation 6-48			
Truck presence (AMF_{tk}):	Equation 6-51			
Combined AMF (product of all AMFs above) ($AMF_{combined}$):				Multiply all AMFs evaluated, disregard others.

Expected Crash Frequency				
Calibration factor (f):				
Over-dispersion parameter (k):	Table 6-1			
Base crash frequency (C_b), crashes/yr:	Equation 6-1 or 6-2			
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1+ (C\ y)/k]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C): If crash data are available, then: If crash data are NOT available, then:	 $C = C_x\ C_{analysis\ year}/C_{crash\ period}$ $C = C_{analysis\ year}$			

Urban Signalized Intersection Worksheet (1 of 2)

General Information	**Site Information**
Analyst: ______________	Highway number or street: ______________
Agency: ______________	Intersecting street: ______________
Date performed: ______________	District: ______________
Location:	Analysis year:

Input Data					
Crash Data		***Crash Period***	***Analysis Year***	If crash data are not available, the number of legs are different for the crash period and analysis year, or the number of lanes are different for the crash period and analysis year, then complete only the Analysis Year column.	
Crash data time period:				_____ Start date	_____ End date
Count of injury + fatal crashes (X), crashes:					
Basic Intersection Data					
Number of through lanes:	Major			Same value for crash period and analysis year.	
	Minor			Same value for crash period and analysis year.	
Number of intersection legs:		___ 3 legs	___ 4 legs	Same value for crash period and analysis year.	
Traffic Data					
Average daily traffic (ADT), veh/d: (two-way volume by leg)	Major 1			Major-street leg 1 ADT.	
	Major 2			Major-street leg 2 ADT.	
	Minor 1			Minor-street leg 1 ADT.	
	Minor 2			Minor-street leg 2 ADT (not applicable for 3 leg).	
Cross Section Data					
Legs with a left-turn lane:	Major	___ 1 or 2	___ 1 or 2		
	Minor	___ 1 or 2	___ 1 or 2		
Legs with a right-turn lane:	Major	___ 1 or 2	___ 1 or 2		
	Minor	___ 1 or 2	___ 1 or 2		
Legs with right-turn channelization:	Major	___ 1 or 2	___ 1 or 2		
	Minor	___ 1 or 2	___ 1 or 2		
Lane width (W_l): ft:	Major				
	Minor				

Urban Signalized Intersection Worksheet (2 of 2)

ADT Distribution				
		Crash Period	**Analysis Year**	If crash data are not available, then complete only the Analysis Year column.
Proportion of ADT on leg (P_{leg}):	Major 1			
	Major 2			
	Minor 1			
	Minor 2			
Average ADT on street, veh/d:	Major			
	Minor			

Accident Modification Factors (AMF)				
		Crash Period	**Analysis Year**	If crash data are not available, then complete only the Analysis Year column.
Left-turn lane (AMF_{LT}):	Equation 7-9			
Right-turn lane (AMF_{RT}):	Equation 7-14			
Number of lanes (AMF_{lane}):	Equation 7-19			
Right-turn channelization (AMF_{CH}):	Equation 7-24			
Lane width (AMF_{lw}):	Equation 7-29			
Combined AMF (product of all AMFs above) ($AMF_{combined}$):				Multiply all AMFs evaluated, disregard others.

Expected Crash Frequency				
Calibration factor (f):				
Over-dispersion parameter (k):	Table 7-1			
Base crash frequency (C_b), crashes/yr:	Equation 7-3 or 7-4			
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1+ (C\ y)/k]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X\ /y\ (1 - w)$			
Expected crash frequency (C): If crash data are available, then: If crash data are NOT available, then:	 $C = C_x\ C_{analysis\ year}\ /C_{crash\ period}$ $C = C_{analysis\ year}$			

Urban Unsignalized Intersection Worksheet (1 of 2)

General Information	**Site Information**
Analyst: ______	Highway number or street: ______
Agency: ______	Intersecting street: ______
Date performed: ______	District: ______
Location:	Analysis year:

Input Data					
Crash Data		***Crash Period***	***Analysis Year***	If crash data are not available, the number of legs are different for the crash period and analysis year, or the number of lanes are different for the crash period and analysis year, then complete only the Analysis Year column.	
Crash data time period:				_____ Start date	_____ End date
Count of injury + fatal crashes (X), crashes:					
Basic Intersection Data					
Number of through lanes:	Major			Same value for crash period and analysis year.	
	Minor			Same value for crash period and analysis year.	
Number of intersection legs:		___ 3 legs	___ 4 legs	Same value for crash period and analysis year.	
Traffic Data					
Average daily traffic (ADT), veh/d: (two-way volume by leg)	Major 1			Major-street leg 1 ADT.	
	Major 2			Major-street leg 2 ADT.	
	Minor 1			Minor-street leg 1 ADT.	
	Minor 2			Minor-street leg 2 ADT (not applicable for 3 leg).	
Cross Section Data					
Legs with a left-turn lane:	Major	___ 1 or 2	___ 1 or 2		
Legs with a right-turn lane:	Major	___ 1 or 2	___ 1 or 2		
Legs with right-turn channelization:	Major	___ 1 or 2	___ 1 or 2		
	Minor	___ 1 or 2	___ 1 or 2		
Lane width (W_l): ft:	Major				
	Minor				
Shoulder width (W_s): ft:	Major				
	Minor				
Median width (W_m), ft:	Major				

Urban Unsignalized Intersection Worksheet (2 of 2)

ADT Distribution				
		Crash Period	***Analysis Year***	If crash data are not available, then complete only the Analysis Year column.
Proportion of ADT on leg (P_{leg}):	Major 1			
	Major 2			
	Minor 1			
	Minor 2			
Average ADT on street, veh/d:	Major			
	Minor			

Accident Modification Factors (AMF)				
		Crash Period	***Analysis Year***	If crash data are not available, then complete only the Analysis Year column.
Left-turn lane (AMF_{LT}):	Equation 7-34			
Right-turn lane (AMF_{RT}):	Equation 7-39			
Number of lanes (AMF_{lane}):	Equation 7-44			
Right-turn channelization (AMF_{CH}):	Equation 7-49			
Lane width (AMF_{lw}):	Equation 7-54			
Shoulder width (AMF_{sw}):	Equation 7-57			
Median presence (AMF_{mp}):	Equation 7-60			
Combined AMF (product of all AMFs above) ($AMF_{combined}$):				Multiply all AMFs evaluated, disregard others.

Expected Crash Frequency				
Calibration factor (f):				
Over-dispersion parameter (k):	Table 7-1			
Base crash frequency (C_b), crashes/yr:	Equation 7-1 or 7-2			
Expected crash frequency (C), crashes/yr:	$C = C_b\ AMF_{combined}$			$= C_{crash\ period}$ and $C_{analysis\ year}$
Crash data time period (y), yr:	y = end − start date			
Weight associated with C (w):	$w = 1/[1+ (C\ y)/k]$			
Adjusted crash frequency given X (C_x):	$C_X = C\ w + X/y\ (1 - w)$			
Expected crash frequency (C): If crash data are available, then: If crash data are NOT available, then:	 $C = C_x\ C_{analysis\ year}/C_{crash\ period}$ $C = C_{analysis\ year}$			

Appendix B

Change History

TABLE OF CONTENTS

OVERVIEW

This appendix provides a summary of the changes made since the publication of the *Interim Roadway Safety Design Workbook* in 2006 and reflected in this final edition of the Workbook.

Chapter 2: Freeways

Current Base Model	Previous Base Model [1]	Change	Comment
Urban, 4 lanes	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Urban, 6 lanes	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Urban, 8 lanes	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Urban, 10 lanes	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Rural, 4 lanes	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Rural, 6 lanes	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).

Note:
1 - Base models listed in the *Interim Roadway Safety Design Workbook* (*2*).

Current AMF	Previous AMF [1]	Change	Comment
Horizontal curve radius	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Grade	Same name	No change	--
Lane width	Same name	Changed	Crash distribution proportions updated to include multiple vehicle opposite direction crashes and additional years of crash data. Lane width coefficient changed due to changes in proportions (see Table 3-7 of Reference *1*).
Outside shoulder width	Same name	Changed	See Lane Width AMF
Inside shoulder width	Same name	Changed	See Lane Width AMF
Median width (no barrier)	Median width	Changed	Development documented in Report No. 0-4703-5 (*4*).
Median width (some barrier)	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Median width (full barrier)	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Shoulder rumble strips	Same name	Changed	Crash distribution proportions updated to include additional years of crash data.
Outside clearance (no barrier)	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Outside clearance (some barrier)	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Outside clearance (full barrier)	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Aggregated ramp entrance	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Aggregated weaving section	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Truck presence	None	Added	Development documented in Report No. 0-4703-5 (*4*).
--	Utility pole offset	Discontinued	Redundant to Outside Clearance (no barrier) AMF.

Note:
1 - AMFs listed in the *Interim Roadway Safety Design Workbook* (*2*).

Chapter 3: Rural Highways (1 of 2)

Current Base Model	Previous Base Model [1]	Change	Comment
Undivided, 2 lanes	Same name	Changed	Development documented in Report No. 0-4703-4 (*3*).
Undivided, 4 lanes	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
--	Surfaced, 2 lanes	Discontinued	Data were not available to calibrate model (*4*).
Nonrestrictive median, 4 lanes	Surfaced, 4 lanes	Changed	Development documented in Report No. 0-4703-5 (*4*).
Restrictive median, 4 lanes	Depressed, 4 lanes	Changed	Development documented in Report No. 0-4703-5 (*4*).
--	Depressed, 6 lanes	Discontinued	Data were not available to calibrate model (*4*).

Note:
1 - Base models listed in the *Interim Roadway Safety Design Workbook* (*2*).

Current AMF	Previous AMF [1]	Change	Comment
Horizontal curve radius	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Spiral transition curve	Same name	No change	--
Grade	Same name	No change	--
Lane width	Lane width (ADT > 2000 veh/d)	Changed	Restriction to multilane highways is added. Crash distribution proportions updated to include multiple vehicle opposite direction crashes and additional years of crash data. Lane width coefficient changed due to changes in proportions (see Table 3-7 of Reference *1*).
--	Lane width (ADT < 2000 veh/d)	Discontinued	Effect of ADT is incorporated in Lane and Shoulder Width AMF.
Outside shoulder width	Outside shoulder width (ADT > 2000 veh/d)	Changed	See Lane Width AMF.
--	Outside shoulder width (ADT < 2000 veh/d)	Discontinued	Effect of ADT is incorporated in Lane and Shoulder Width AMF.
Lane and shoulder width	None	Added	Restriction to two-lane highways. Development documented in Report No. 0-4703-4 (*3*). Effect of ADT on lane width and shoulder width converted into one continuous function using Equation 3-36.
Inside shoulder width	Same name	Changed	See Lane Width AMF.
Median width (no barrier)	Median width	Changed	Development documented in Report No. 0-4703-5 (*4*).
Median width (some barrier)	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Median width (full barrier)	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Shoulder rumble strips	Same name	Changed	Restriction to only two-lane highways is added. It is based on guidance in NCHRP Report 617 (*6*). The value cited for two-lane highways is based on research by Patel et al. (*7*). Crash distribution proportions updated to include additional years of crash data
Centerline rumble strip	Same name	Changed	Restriction to only two-lane highways is added. It is based on guidance in NCHRP Report 617 (*6*). The value cited is changed based on recommendation in Report 617.
TWLTL median type	Same name	Changed	Restriction to only two-lane highways is added. Effect of two-way left-turn lane (TWLTL) on four-lane highways is addressed in the current base models.

Note:
1 - AMFs listed in the *Interim Roadway Safety Design Workbook* (*2*).

Chapter 3: Rural Highways (2 of 2)

Current AMF	Previous AMF [1]	Change	Comment
Superelevation	Same name	Changed	Restriction to only two-lane highways is added. It is based on guidance in NCHRP Report 617 (*6*). The value cited is changed based on recommendation in Report 617.
Passing lane	Same name	No change	--
Outside clearance (no barrier)	Horizontal clearance	Changed	Modified equation to include a sensitivity to outside shoulder width.
Outside clearance (some barrier)	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Outside clearance (full barrier)	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Side slope	Same name	Changed	Crash distribution proportions updated to include additional years of crash data.
Driveway density	Same name	No change	Restriction to only two-lane highways. The effect of driveway presence on multilane highways is reflected in current base models.
Truck presence	None	Added	Development documented in Report No. 0-4703-5 (*4*).
--	Utility pole offset	Discontinued	Redundant to Outside Clearance (no barrier) AMF.
--	Bridge width	Discontinued	Redundant to Outside Clearance (full barrier) AMF.

Note:
1 - AMFs listed in the *Interim Roadway Safety Design Workbook* (*2*).

Chapter 4: Urban and Suburban Arterials

Current Base Model	Previous Base Model [1]	Change	Comment
Undivided, 2 lanes	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Undivided, 4 lanes	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
--	Undivided, 6 lanes	Discontinued	Data were not available to calibrate model (*4*).
Nonrestrictive median, 2 lanes	TWLTL, 2 lanes	Changed	Development documented in Report No. 0-4703-5 (*4*).
Nonrestrictive median, 4 lanes	TWLTL, 4 lanes	Changed	Development documented in Report No. 0-4703-5 (*4*).
Nonrestrictive median, 6 lanes	TWLTL, 6 lanes	Changed	Development documented in Report No. 0-4703-5 (*4*).
Restrictive median, 4 lanes	Raised-curb, 4 lanes	Changed	Development documented in Report No. 0-4703-5 (*4*).
Restrictive median, 6 lanes	Raised-curb, 6 lanes	Changed	Development documented in Report No. 0-4703-5 (*4*).

Note:
1 - Base models listed in the *Interim Roadway Safety Design Workbook* (*2*).

Current AMF	Previous AMF [1]	Change	Comment
Horizontal curve radius	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Lane width	Same name	Changed	Crash distribution proportions updated to include additional years of crash data. Lane width coefficient changed due to changes in proportions (see Table 3-7 of Reference *1*).
Shoulder width	Same name	Changed	Crash distribution proportions updated to include additional years of crash data. Development documented in Report No. 0-4703-5 (*4*).
Median width	Same name	Changed	AMF for nonrestrictive median width is added. Its development is documented in Report No. 0-4703-5 (*4*).
--	TWLTL median type	Discontinued	Effect of two-way left-turn lane (TWLTL) is addressed in the current base models.
Curb parking	Same name	No change	--
Utility pole offset	Same name	Changed	Crash distribution proportions updated to include additional years of crash data.
--	Driveway density	Discontinued	Effect of driveway presence is addressed in the current base models.
Truck presence	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).

Note:
1 - AMFs listed in the *Interim Roadway Safety Design Workbook* (*2*).

Chapter 5: Interchange Ramps and Frontage Roads

Interchange Ramps

Current Base Model	Previous Base Model [1]	Change	Comment
Non-frontage road, exit, diagonal	Same name	No change	--
Non-frontage road, exit, non-free-flow loop	Same name	No change	--
Non-frontage road, exit, free-flow loop	Same name	No change	--
Non-frontage road, exit, outer connection	Same name	No change	--
Non-frontage road, exit, semi-directional	Same name	No change	--
Non-frontage road, exit, direct connection	Same name	No change	--
Non-frontage road, entrance, diagonal	Same name	No change	--
Non-frontage road, entrance, non-free-flow loop	Same name	No change	--
Non-frontage road, entrance, free-flow loop	Same name	No change	--
Non-frontage road, entrance, outer connection	Same name	No change	--
Non-frontage road, entrance, semi-directional	Same name	No change	--
Non-frontage road, entrance, direct connection	Same name	No change	--
Frontage road, exit, button hook	Same name	No change	--
Frontage road, exit, scissor	Same name	No change	--
Frontage road, exit, slip	Same name	No change	--
Frontage road, entrance, button hook	Same name	No change	--
Frontage road, entrance, scissor	Same name	No change	--
Frontage road, entrance, slip	Same name	No change	--
--	Speed-change lane, acceleration, urban	Discontinued	AMF for ramp entrance provided in Chapter 2.
--	Speed-change lane, acceleration, rural		
--	Speed-change lane, deceleration, urban	Discontinued	AMF for ramp exit provided in Chapter 2.
--	Speed-change lane, deceleration, rural		

Note:
1 - Base models listed in the *Interim Roadway Safety Design Workbook* (*2*).

Frontage Roads

Current Base Model	Previous Base Model [1]	Change	Comment
Rural, two-lane	None	Added	Development documented in Report No. 0-4703-4 (*3*). Also described in Report No. 0-4703-P5 (*5*).

Note:
1 - Base models listed in the *Interim Roadway Safety Design Workbook* (*2*).

Current AMF	Previous AMF [1]	Change	Comment
Lane width	None	Added	Development documented in Report No. 0-4703-4 (*3*). Also described in Report No. 0-4703-P5 (*5*).
Shoulder width	None	Added	Development documented in Report No. 0-4703-4 (*3*). Also described in Report No. 0-4703-P5 (*5*).

Note:
1 - AMFs listed in the *Interim Roadway Safety Design Workbook* (*2*).

Chapter 6: Rural Intersections

Current Base Model	Previous Base Model [1]	Change	Comment
Three-leg, unsignalized	Same name	Changed	Development documented in Report No. 0-4703-4 (3).
Four-leg, unsignalized	Same name	Changed	Development documented in Report No. 0-4703-4 (3).
Three-leg, signalized	Same name	Changed	Development documented in Report No. 0-4703-4 (3).
Four-leg, signalized	Same name	Changed	Development documented in Report No. 0-4703-4 (3).

Note:
1 - Base models listed in the *Interim Roadway Safety Design Workbook* (2).

Signalized Intersections

Current AMF	Previous AMF [1]	Change	Comment
Left-turn lane	Same name	Changed	Development documented in Report No. 0-4703-5 (4).
Right-turn lane	Same name	Changed	Development documented in Report No. 0-4703-5 (4).
Number of lanes	Same name	No change	Presentation altered to show combined AMF.
--	Alignment skew angle	Discontinued	Re-review of literature did not identify any research quantifying the effect of skew on safety.
Driveway frequency	Same name	Changed	AMF for major-road unchanged. Equation used to extend AMF to minor-road is documented in Report 0-4703-5 (4). Base condition changed from 3 driveways on major road to 2 driveways on major road based on data documented in Report No. 0-4703-4 (3).
Traffic presence	Same name	No change	Base condition changed from 9 percent trucks to 11 percent trucks based on data documented in Report No. 0-4703-4 (3).

Note:
1 - AMFs listed in the *Interim Roadway Safety Design Workbook* (2).

Unsignalized Intersections

Current AMF	Previous AMF [1]	Change	Comment
Left-turn lane	Same name	Changed	Development documented in Report No. 0-4703-5 (4).
Right-turn lane	Same name	Changed	Development documented in Report No. 0-4703-5 (4).
Number of lanes	Same name	No change	Presentation altered to show combined AMF.
Shoulder width	Same name	Changed	AMF for major-road unchanged. Equation used to extend AMF to minor-road is documented in Report 0-4703-5 (4). Base condition changed from 8-ft to 4-ft shoulder width.
Median presence	Same name	No change	AMF is unchanged. Presentation is changed to simplify.
Alignment skew angle	Same name	No change	--
--	Intersection sight distance	Discontinued	AMF is not recognized in NCHRP Report 617 (6).
Driveway frequency	Same name	Changed	AMF for major-road unchanged. Equation used to extend AMF to minor-road is documented in Report 0-4703-5 (4). Base condition changed from 0 driveways on major road to 1 driveway on major road based on data documented in Report No. 0-4703-4 (3).
Traffic presence	Same name	No change	Base condition changed from 9 percent trucks to 15 percent trucks based on data documented in Report No. 0-4703-4 (3).

Note:
1 - AMFs listed in the *Interim Roadway Safety Design Workbook* (2).

Chapter 7: Urban Intersections

Current Base Model	Previous Base Model [1]	Change	Comment
Three-leg, unsignalized	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Four-leg, unsignalized	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Three-leg, signalized	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Four-leg, signalized	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).

Note:
1 - Base models listed in the *Interim Roadway Safety Design Workbook* (*2*).

Signalized Intersections

Current AMF	Previous AMF [1]	Change	Comment
Left-turn lane	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Right-turn lane	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Number of lanes	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Right-turn channelization	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Lane width	Same name	Changed	AMF for major-street unchanged. Equation used to extend AMF to minor-street is documented in Report 0-4703-5 (*4*).

Note:
1 - AMFs listed in the *Interim Roadway Safety Design Workbook* (*2*).

Unsignalized Intersections

Current AMF	Previous AMF [1]	Change	Comment
Left-turn lane	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Right-turn lane	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Number of lanes	Same name	Changed	Development documented in Report No. 0-4703-5 (*4*).
Right-turn channelization	None	Added	Development documented in Report No. 0-4703-5 (*4*).
Lane width	Same name	Changed	AMF for major-street unchanged. Equation used to extend AMF to minor-street is documented in Report 0-4703-5 (*4*).
Shoulder width	Same name	Changed	AMF for major-street unchanged. Equation used to extend AMF to minor-street is documented in Report 0-4703-5 (*4*).
Median presence	Same name	No change	AMF is unchanged. Presentation is changed to simplify.

Note:
1 - AMFs listed in the *Interim Roadway Safety Design Workbook* (*2*).

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4. Bonneson, J., and M. Pratt. *Calibration Factors Handbook: Safety Prediction Models Calibrated with Texas Highway System Data.* Report No. FHWA/TX-08/0-4703-5. Texas Department of Transportation, Austin, Texas, October 2008.

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