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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. The methods used to develop the information presented herein are documented in the *Roadway Safety Design Synthesis*.

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INTERIM ROADWAY SAFETY DESIGN WORKBOOK

by

J. Bonneson, P.E.
Research Engineer
Texas Transportation Institute

K. Zimmerman, P.E.
Assistant Research Engineer
Texas Transportation Institute

and

K. Fitzpatrick, P.E.
Research Engineer
Texas Transportation Institute

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TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

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);
- Ms. Elizabeth Hilton, Project Director (TxDOT);
- Mr. David Bartz (FHWA);
- Mr. Mike Battles (TxDOT);
- Mr. Stan Hall (TxDOT);
- Mr. Richard Harper (TxDOT);
- Ms. Meg Moore (TxDOT); and
- Ms. Joanne Wright (TxDOT).

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Introduction

SAFETY BY DESIGN

PREFACE

The *Interim 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.

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 *Interim Roadway Safety Design Workbook* is to describe 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 review and synthesis of safety information in the literature. The findings from this review are documented in the *Roadway Safety Design Synthesis* (*1*). Users of the *Workbook* are encouraged to consult the *Synthesis* if additional information is desired about the relationships in this *Workbook*.

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. It should be noted that the relationships were neither compared to DPS crash data to confirm the stated trends, nor calibrated to Texas conditions. Nevertheless, they are still useful for evaluating the effect of a change in design or operation in terms of the expected decrease (or increase) in crash frequency.

At this time, quantitative safety relationships are not available for every element of roadway design. The reader is referred to AASHTO's *Highway Safety Design and Operations Guide* (*2*) for a qualitative discussion of safety considerations associated with the various design-related factors for which quantitative information is not available herein.

It is anticipated that research underway at the state and national levels will produce significant new information about the relationship between design components and safety. Hence, this *Workbook* is presented as an interim document, with the intent that it will be updated to include the findings from 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. Also identified is the step in the corresponding development process stage within which they would be conducted. The step numbers show in the table correspond with the step number sequence used in the *Project Development Process Manual* (*3*).

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
- Stopping Sight Distance

The controlling criteria for New Location and Reconstruction Projects (4R) include all of the above criteria plus:

- Cross Slope
- Superelevation
- Vertical Clearance

Additional design elements that may also be considered as "key" because of their known effect on safety include: turn bays at intersections, median treatment, and clear zone (i.e., 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.

The implementation of these tasks will add time to the design process. However, by limiting the evaluation of safety to primarily "key" design elements, it is hoped that the additional time required will 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 will be derived through fewer crashes (by provision of effective features) and lower construction costs (by not over-designing some design elements).

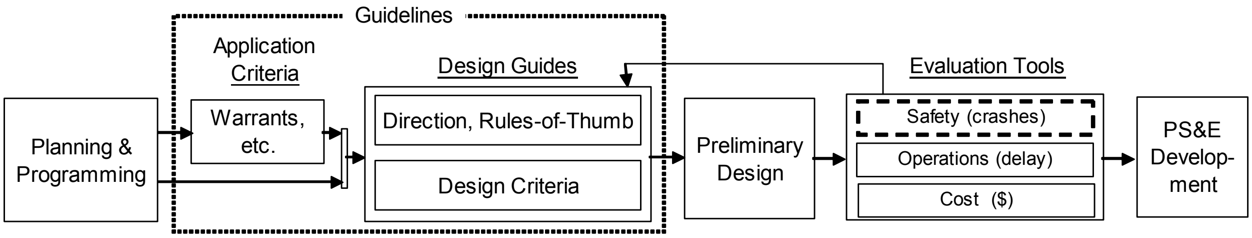


Figure 1-1. Components of the Project Development Process.

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

Stage	Step	Potential Safety-Related Task
Planning and programming	1. Needs identification	• Screen facilities for locations with safety needs.
Preliminary design	1. Preliminary design conference	• Document safety needs. • Identify atypical conditions, complex elements, and high-cost components.
	2. Data collection/preliminary design preparation	• Diagnose safety data to identify crash patterns. • Refine project scope if necessary.
	4. Preliminary schematic	• Perform preliminary level of safety analysis for “key” design elements.[1]
	5. Geometric schematic	• Perform detailed level of safety analysis for “key” design elements.[1]
	6. Value engineering	• Compare cost of specific elements and overall roadway with safety and operational benefits.
	7. Geometric schematic approval	• Document safety of design choices (use results for design exception request, if necessary).
PS&E development	3. Final alignments/profiles	• Re-evaluate alignment, cross section, and roadside design to ensure acceptable level of safety.
	9. 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 streets,
- Interchange ramps,
- 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 annual severe (i.e., injury plus fatal) crash frequency for a roadway segment, ramp, or intersection. These models are based on crash rates derived from the assessment of various safety prediction models reported in the literature. The safety prediction models used for this purpose are described in the *Synthesis*.

The base models are to be used when the crash history for a project location is not available, such as for a new alignment. If the project location has a crash history available, the expected annual severe crash frequency should be estimated as the average severe crash frequency for the most recent three-year period. In this situation, the three-year average should be used instead of the estimate obtained from the base model.

In some instances, the nature of the alternatives analysis requires an estimate of the expected crash frequency for a nonexistent facility. In this situation, the expected annual severe crash frequency should be estimated using the base model for *all* alternatives being considered as well as the existing facility. This technique is necessary to ensure an equitable assessment of the 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 carefully investigated for their applicability, and represent the current best knowledge regarding their relationship to crash frequency. The source of these AMFs is discussed in the *Synthesis* (*1*).

AMFs represent 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.

The AMFs provided herein were developed to have a value of 1.0 when used to evaluate roadways with typical design and traffic characteristics. A table of "base conditions" is provided in each chapter to identify these typical characteristics. A corresponding table of base crash rates is also provided in each chapter. The fact that none of these crash rates equal zero is a reminder that: (1) an AMF value of 1.0 only indicates that conditions are typical and (2) that crashes do occur on roadways considered typical.

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 or median type) as an input. The distributions tabulated herein for these AMFs were obtained from the crash database maintained by the State of Texas, Department of Public Safety (DPS).

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

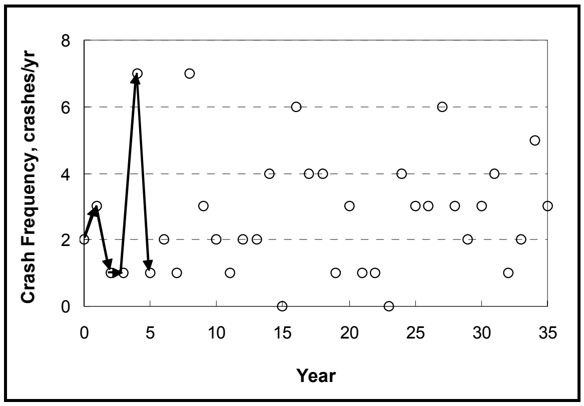
This section examines the variability in crash frequency on one street segment or at one intersection (hereafter, referred to as a "site"). It also explores how long-run averages can be used to reveal the underlying mean crash frequency at a site.

Variability in Crash Frequency

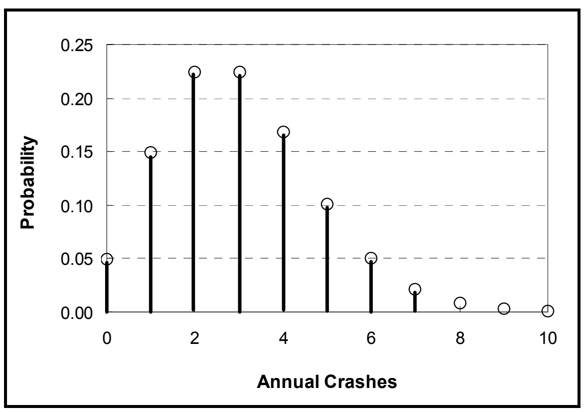
On a year-to-year basis, crash data typically exhibit a large variability in crash frequency. Figure 1-2a illustrates the pattern of crashes at a site for a 35-year period, during which 98 crashes occurred. Traffic growth was negligible at this site and its geometry did not change substantially over the 35 years (a fairly rare occurrence). 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. Yearly Crash Frequency Distribution at a Site.

In year 5, the crash frequency 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 frequency in one year to return to a lower crash frequency (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 frequency, 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 truly problem sites and evaluating treatment effectiveness are described in the safety literature (*4, 5*).

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 frequency in that year. The running average for year 1 represents an average of the reported crash frequency for years 0 and 1. The running average for year 2 represents the average of reported frequencies 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.

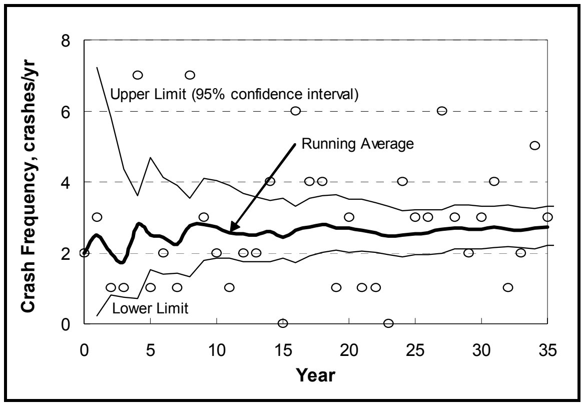


Figure 1-3. Running Average Crash Frequency.

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 (*6*). 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 given 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 Estimating Average Crash Frequency Confidence Interval.

Total Crash Frequency	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 - $N_{upper} = N \times (1 + \textit{Limit Percent}/100)$;
$N_{lower} = N \times (1 - \textit{Limit Percent}/100)$; and
N = average annual 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 (*5*).

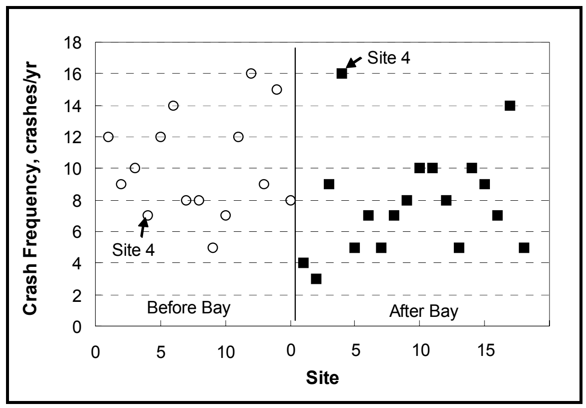
Challenges to Detection. It is generally recognized that some design components are used at a site because of their direct influence on safety. For example, the addition of a left-turn bay at a rural signalized intersection has been found to reduce the expected crash frequency by about 20 percent (*7*). 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 frequency during the "before" period and the solid squares indicate the reported crash frequency during the "after" period.

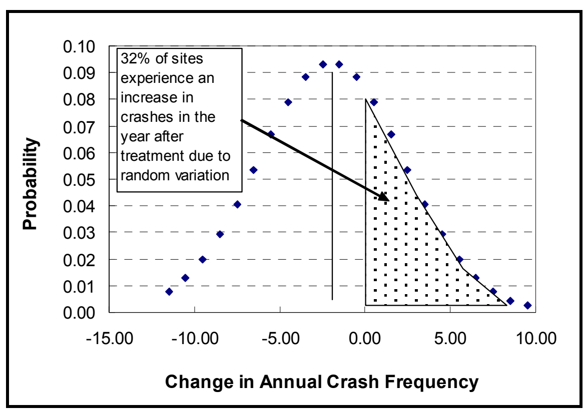
The data in Figure 1-4a indicate that there is a trend toward a decrease in crash frequency at the collective set of sites in the "after" period. However, random variation in the number of crashes makes the trend difficult to see at a given site. In fact, the average reduction of 2 crashes/yr (= 10 − 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 frequency 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. 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 (*5*) developed an equation that can be used to compute the minimum crash frequency 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 frequency represents the total number of crashes reported in the period before the design change.

Table 1-3 lists the minimum crash frequency, as obtained from Hauer's equation. The first nine rows list the crash frequency needed to detect a reduction in mean crash frequency for a specific AMF. The last nine rows list the minimum crash frequency 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 Frequency to Detect the Influence of a Change in Geometry.

AMF	Minimum Crash Frequency 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 frequencies correspond to a 95 percent level of confidence that a change occurred. The time duration for the "before" and "after" periods is the same. Increase the crash frequency by 4.0 to obtain a 95 percent level of confidence in detecting a change equal in magnitude to the AMF listed.

The crash frequencies 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 (*8*). Thus, a minimum crash frequency of about 2056 (= 4.0 ×514) is needed to be reasonably sure that the true mean AMF is 0.90 or less. Also, a minimum crash frequency of about 2272 (= 4.0 ×568) is needed to be reasonably sure that the true mean AMF is 1.1 or more.

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.

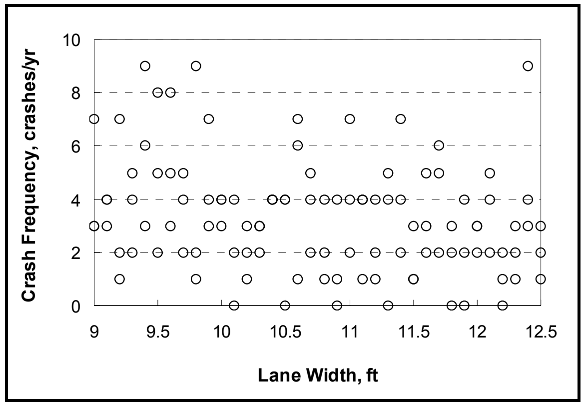
Influence of Different Design Element Sizes

This subsection discusses the effect of crash frequency 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 effect of different design element size 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.

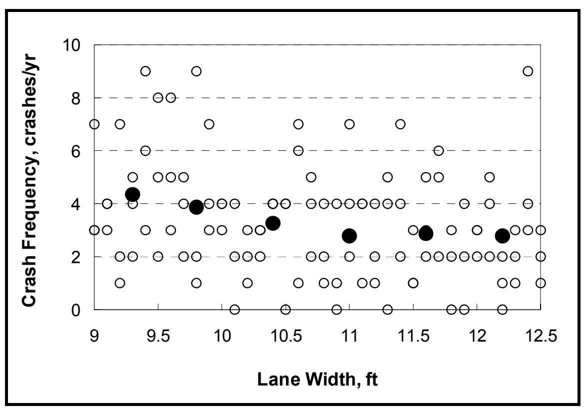
Challenges to Detection. In contrast to the before-after study, a cross section study does not have as strict a requirement for site similarity. Nominal differences in geometry or traffic volume are controlled by including variables in the regression model. Nevertheless, some similarity among the group 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 characterizes 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 frequency 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 frequency 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 frequency needed for regression analysis.

The minimum crash frequencies list in Table 1-4 are partially dependent on the similarity of the sites used for the regression analysis. The frequencies 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 frequency needed to obtain the desired confidence interval will increase.

Table 1-4. Minimum Crash Frequency 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 frequency:	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 frequency:	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. A regression analysis typically considers crash histories that range from three to five years in duration.

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 frequencies 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 section 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 Frequency 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 × (1 + *Limit Percent*/100); AMF_{lower} = AMF × (1 - *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 frequency 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 and thus, 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 (*5*) 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. For reasons cited in the previous subsection, the precision of the AMFs offered in the *Workbook* is difficult to quantify. Whenever possible, the AMFs from multiple studies of a common geometric design element were combined to increase the net total crash frequency underlying the overall average AMF offered. However, the influence of external factors or the extent to which correlations are present can never be fully determined. As a result, the limit percentage corresponding to the AMF for any given study (or that derived from a combination of studies) is difficult to quantify and can sometimes only be estimated using engineering judgment.

In the development of the *Workbook*, the AMFs extracted from the literature were screened such that only those that were obtained from studies determined to be of good quality were used. In this regard, studies of good quality were determined to be those that accounted for most external influences and correlated variables. These studies used databases that included hundreds of crashes. Based on these screening techniques, it is believed that a limit percentage of 5.0 percent or less is applicable to the AMFs in the *Workbook*. A limit percentage of 5.0 percent corresponds to an AMF range of 0.95 to 1.05.

Recommendations

The AMFs offered in the *Workbook* represent the current best estimate of the true mean AMF, regardless of the corresponding limit percentage. However, if a conservative analysis is desired and the 95 percent confidence interval for an AMF is not specifically stated in the *Workbook*, then AMFs in the range of 0.95 to 1.05 can be considered to be not significantly different from 1.0.

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 they 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 frequency observed at the site the year after the improvement is found to have fewer crashes, 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. However, if a conservative analysis is desired and the 95 percent confidence interval for an AMF is not specifically stated in the *Workbook*, then AMFs in the range of 0.95 to 1.05 can be considered to be not significantly different from 1.0.

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 is used to describe 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 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.

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) representing a 33 percent reduction in crashes.

Injury crash is a crash wherein one or more of the persons involved is injured. An injury can be reported as "possible," "probable," or "visible."

Safety (or "substantive 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 crash frequencies reported for a large group of facilities with similar features and conditions.

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 can be derived to include one or more AMFs. Models intended for practical application have one or more empirically based factors that require calibration to local conditions to ensure accurate predictions.

Safety surrogate is any statistic that is directly related to crash frequency or severity (e.g., conflicts) and that quantifies the relative risk of collision or injury.

Severe crash is a crash wherein one or more of the persons involved is injured or killed. An injury can be reported as "possible," "probable," or "visible."

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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 typically have multiple lanes to serve high volumes with associated frequent weaving and lane-changing maneuvers. These attributes can increase the risk of freeway crashes, especially the risk of severe crashes. This risk, coupled with the typically high traffic volume found on freeways, can result in frequent severe crashes. As such, it is especially important to fully evaluate the safety impact of design alternatives during the freeway design process.

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 severe (i.e., injury or 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 the *Roadway Safety Design Synthesis* (*1*). 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 entrance ramp, exit ramp, 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).

Procedures for evaluating interchange ramps and speed-change lanes are described in Chapter 5. These procedures can be used together with the procedure in this chapter to fully 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 a sensitivity to traffic volume, segment length, and the main factors known to be uniquely correlated with crash frequency for the subject freeway. 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. (*2*) in that it predicts *severe* crash frequency (as opposed to total crash frequency). Otherwise, the procedure described herein is similar and shares the same strengths and weaknesses. The reader is referred to the report by Harwood et al. for a discussion of their procedure and its attributes.

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 indicates that crash rates for freeways vary with area type (urban or rural) and the number of lanes in the cross section (*1*). In general, crash rates are higher 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 higher volumes and narrower medians often associated with urban areas.

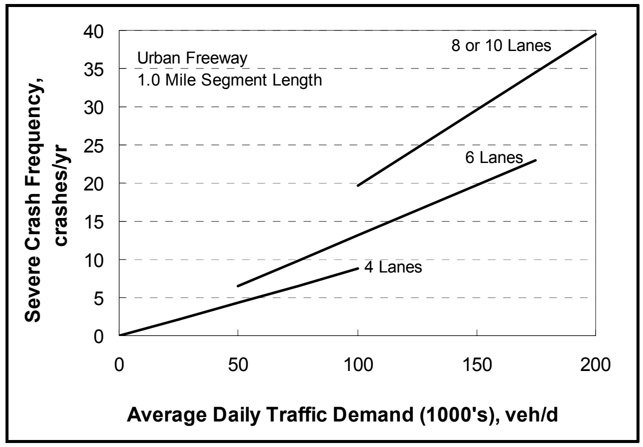
Safety Relationship

The relationship between severe crash frequency and traffic demand for typical freeway conditions is shown in Figure 2-1. The trends shown in this figure apply to freeway segments that are 1 mile long. The crash frequency obtained from the figure can be adjusted for other segment lengths by multiplying it by the actual segment length (in miles).

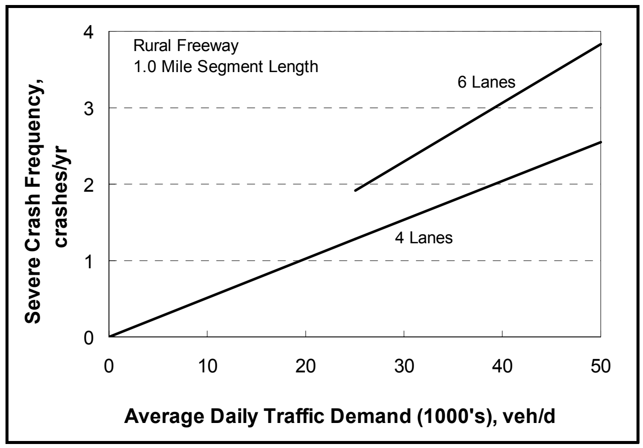
The crash rates that underlie Figure 2-1 are listed in Table 2-1. They can be used with Equation 2-1 to compute the expected severe crash frequency for the typical (i.e., base) condition.

Guidance

The severe crash frequency obtained from Figure 2-1 or Equation 2-1 is applicable to freeways with typical characteristics. These characteristics are identified herein as "base" conditions. The complete set of base conditions are identified in Table 2-2.



a. Urban Freeways.



b. Rural Freeways.

Figure 2-1. Base Crash Rates for Freeways.

$$C_b = 0.000365 \; Base \; ADT \; L \; f \qquad \textbf{(2-1)}$$

where:
C_b = expected severe base crash frequency, crashes/yr;
Base = severe crash rate (see Table 2-1), crashes/mvm;
ADT = average daily traffic volume, veh/d;
L = freeway segment length, mi; and
f = local calibration factor.

Table 2-1. Base Crash Rates for Freeways.

Area Type	Attributes	Base Crash Rate, severe crashes/mvm [1]		
	Through Lanes:	4	6	8 or 10
Urban (includes effect of surfaced median)		0.24	0.36	0.54
Rural (includes effect of depressed median)		0.14	0.21	data not available

Note:
1 - mvm: million vehicle miles.

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

A local calibration factor is identified in Equation 2-1. A default value for this factor is recommended as 1.0. The factor can be used to adjust the predicted base crash frequency so that it is more consistent with typical freeways in the agency's jurisdiction. A procedure for calibrating Equation 2-1 to local conditions is described by Harwood et al. (*2*). If this procedure is used, only severe crashes should be included in the calculations.

Table 2-2. Base Conditions.

Characteristic	Base Condition
Grade	flat (0% grade)
Lane width	12 ft
Outside shoulder width	10 ft
Inside shoulder width	4 ft (4 lane), 10 ft (6 or more lanes)
Median width [1]	24 ft for surfaced median, 76 ft for depressed median
Shoulder rumble strips	not present
Utility pole density and offset	25 poles/mi 30 ft average offset

Note:
1 - Surfaced median: flush-paved median.

Example Application

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

The Facts:

- Through lanes: 4
- Area type: urban
- Segment length: 0.5 mi
- ADT: 80,000 veh/d

The Solution: From Figure 2-1a, find that the typical freeway segment with these characteristics experiences about 7.0 crashes/mi/yr. For this specific freeway segment, the expected severe crash frequency is estimated as 3.5 crashes/yr (= 0.5 × 7.0). The use of Equation 2-1 is illustrated in the box at the right.

$$\begin{aligned} C_b &= 0.000365\ Base\ ADT\ L\ f \\ &= 0.000365 \times 0.24 \times 80{,}000 \times 0.5 \times 1.0 \\ &= 3.5\ crashes/yr \end{aligned} \quad \textbf{(2-2)}$$

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 severe crash frequency. Topics addressed are listed in Table 2-3. The basis for each of these AMFs is described in Chapter 2 of the *Synthesis* (*1*). There are many additional factors, other than those listed in Table 2-3, that are likely to have some effect on severe crash frequency. However, their effect 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 are developed to yield a value of 1.0 when the associated design component or element represents "typical" freeway conditions ("typical" characteristics are defined in the previous section). Deviation from base conditions to a more generous or desirable design condition results in an AMF of less than 1.0. Deviation to a more restricted condition results in an AMF of more than 1.0.

Safety Relationship

The expected severe crash frequency for a specific freeway segment is computed using Equation 2-3. The expected severe crash frequency represents the product of the base severe crash frequency and the various AMFs needed to account for characteristics that are not typical.

Guidance

In application, an AMF is identified for each freeway characteristic that is not typical. All AMFs identified in this manner are then multiplied together. This product is then multiplied by the base severe crash frequency C_b for freeways that are otherwise similar to the subject freeway. The base severe crash frequency can be obtained from Figure 2-1 (or

Table 2-3. AMFs for Freeway Segments.

Application	Accident Modification Factor
Geometric design	Grade Lane width Outside shoulder width Inside shoulder width Median width Shoulder rumble strips
Roadside design	Utility pole offset

$$C = C_b \times AMF_{lw} \times AMF_{mw} \cdots \qquad \textbf{(2-3)}$$

where:
C = expected severe crash frequency, crashes/yr;
C_b = expected severe base crash frequency, crashes/yr;
AMF_{lw} = lane width accident modification factor; and
AMF_{mw} = median width accident modification factor.

Equation 2-1) or estimated from existing crash data. The product of this multiplication represents the expected severe crash frequency for the subject freeway segment.

Example Application

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

The Facts:
- Through lanes: 4
- Area type: urban
- Segment length: 0.5 mi
- ADT: 80,000 veh/d
- Base crash frequency C_b: 3.5 crashes/yr
- Average lane width: 10 ft

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 3.5 \times 1.11 \\ &= 3.9 \; crashes/yr \end{aligned} \tag{2-4}$$

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 1.11. This AMF can be used with Equation 2-3 to estimate the expected severe crash frequency for the subject segment as 3.9 crashes/yr.

Example Application

The Question: What is the expected severe crash frequency for a four-lane urban freeway if the lane width is reduced from 12 to 10 ft?

The Facts:
- Severe crash frequency (3-year average): 9.0 crashes/year
- Existing average lane width: 12 ft
- Proposed lane width: 10 ft

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 9.0 \times 1.11 \\ &= 10 \; crashes/yr \end{aligned} \tag{2-5}$$

The Solution: A three-year crash history is available and considered to be a better estimate of the expected severe crash frequency than the rates in Table 2-1. The AMF for a lane width of 10 ft is 1.11. This AMF can be used with Equation 2-3 to estimate the expected severe crash frequency for the subject segment with 10 ft lanes as 10 crashes/yr. This value represents an increase of 1 crash/yr if the lane width is reduced.

Grade - AMF_g

Discussion

Grade can indirectly influence safety by influencing the speed of the traffic stream. Ascending grades pose a threat to safety due to increased speed differentials. Differences in speed between cars and trucks are most notable. Significant differences in speed among vehicles increases the frequency of lane changes and related crashes. Descending grades also pose a threat to safety due to the natural acceleration of gravity and associated additional demand placed on vehicle braking and maneuverability.

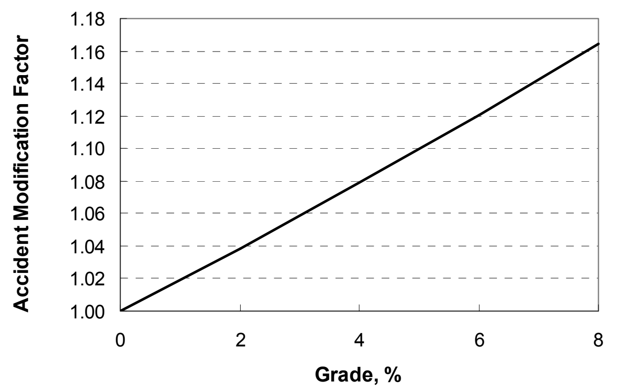


Figure 2-2. Grade AMF.

Safety Relationship

The relationship between grade and severe crash frequency can be estimated using Figure 2-2 or Equation 2-6. 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 flat (i.e., 0 percent grade). In other words, the AMF yields a value of 1.0 when the grade is zero.

$$AMF_g = e^{0.019\,g} \qquad \textbf{(2-6)}$$

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. In this application, the grade used in the AMF should equal the average of the absolute value of the curve entrance and exit grades.

Example Application

The Question: What is the AMF for a crest vertical curve?

The Facts:

- Entrance curve grade: +4 percent
- Exit curve grade: -4 percent

The Solution: The average grade is estimated as 4 percent (= 0.5 × [|+4| + |-4|]). From Figure 2-2, find the AMF of 1.08. This value suggests that 8 percent more crashes will occur on this curve, relative to a flat section.

Lane Width - AMF_{lw}

Discussion

A reduction in lane width for the purpose of increasing the total number of lanes in the cross section is sometimes considered to obtain additional freeway capacity. However, any proposed reduction in lane width should consider the impact on safety. Experience indicates that crashes are more frequent on freeways with lanes narrower than 12 ft. These crashes are particularly frequent when the narrow lanes are accompanied by other design features of minimum dimension.

Safety Relationship

The relationship between lane width and severe crash frequency can be estimated using Figure 2-3 or Equation 2-7. 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 2-7 is used, the proportion of crashes can be 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: 4
- Lane width: 10 ft

The Solution: From Figure 2-3 for "Urban, 4, 6, or 8 lanes," find the AMF of 1.11. This value implies the 10 ft lane width is associated with 11 percent more severe crashes than the 12 ft lane width.

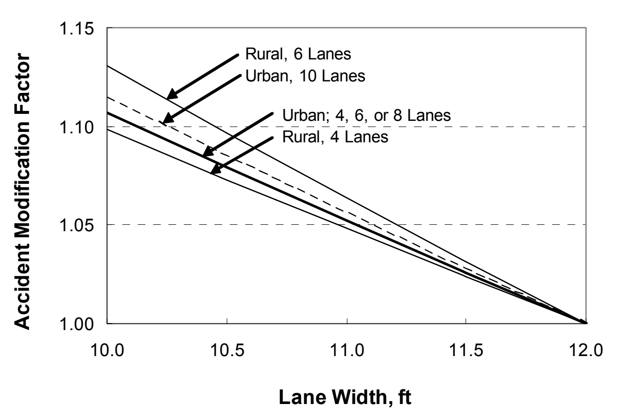


Figure 2-3. Lane Width AMF.

$$AMF_{lw} = (e^{-0.047\,(W_l - 12)} - 1.0)\frac{P_i}{0.37} + 1.0 \qquad \textbf{(2-7)}$$

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	0.37
	6	0.49
Urban	4	0.40
	6	0.40
	8	0.40
	10	0.43

Note:
1 - Single-vehicle run-off-road and same-direction side-swipe crashes.

Outside Shoulder Width - AMF$_{osw}$

Discussion

Shoulders offer numerous safety benefits for freeways. Properly designed shoulders provide space for disabled vehicles and additional room for 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 severe crash frequency can be estimated using Figure 2-4 or Equation 2-8. 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 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 2-8 is used, the proportion of crashes can be obtained from Table 2-5.

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 2-4, find the AMF of 1.05. This value implies that an 8 ft shoulder is likely to be associated with 5 percent more severe crashes than a 10 ft shoulder.

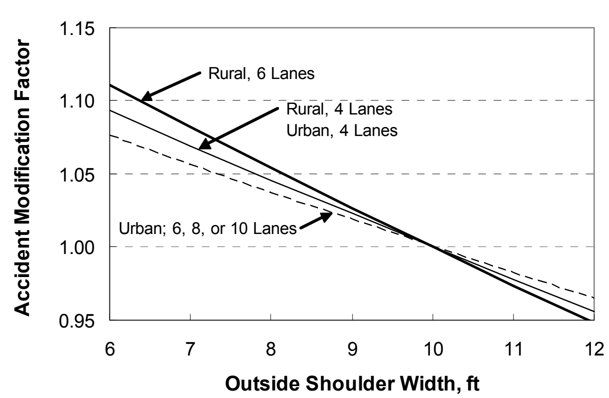


Figure 2-4. Outside Shoulder Width AMF.

$$AMF_{osw} = (e^{-0.021\,(W_s - 10)} - 1.0)\,\frac{P_i}{0.15} + 1.0 \qquad \textbf{(2-8)}$$

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	0.15
	6	0.19
Urban	4	0.16
	6	0.14
	8	0.12
	10	0.13

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

$$AMF_{osw} = (e^{-0.021 \times (8 - 10)} - 1.0)\,\frac{0.16}{0.15} + 1.0 \qquad \textbf{(2-9)}$$
$$= 1.05$$

Inside Shoulder Width - AMF$_{isw}$

Discussion

Inside 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. Inside (i.e., left-hand) shoulders are typically provided on freeways in rural and urban areas because of these safety benefits.

Safety Relationship

The relationship between inside shoulder width and severe crash frequency can be estimated using Figure 2-5 or Equation 2-10. 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 a 4 ft inside shoulder width when there are 4 lanes and a 10 ft inside shoulder width when there are 6 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 2-10 is used, the proportion of crashes can be obtained from Table 2-6. If the shoulder is narrowed to accommodate the addition of an inside high-occupancy-vehicle lane, then $AMF_{isw} \leq 1.04$.

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 2-5 for "Urban, 4 lanes," find the AMF of 0.96. This value implies a 4 percent reduction in severe crashes if a 6 ft shoulder width is used instead of a 4 ft width.

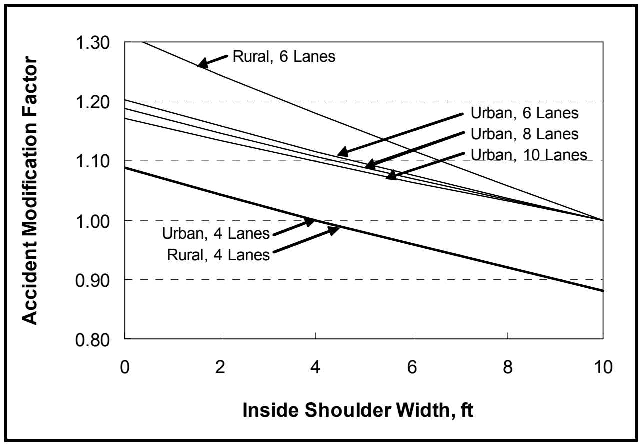


Figure 2-5. Inside Shoulder Width AMF.

$$AMF_{isw} = (e^{-0.021\,(W_{is} - W_{isb})} - 1.0)\frac{P_i}{0.15} + 1.0 \qquad \textbf{(2-10)}$$

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	0.15
	6	0.20
Urban	4	0.15
	6	0.13
	8	0.12
	10	0.11

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

$$AMF_{isw} = (e^{-0.021 \times (6-4)} - 1.0)\frac{0.15}{0.15} + 1.0 \qquad \textbf{(2-11)}$$
$$= 0.96$$

Median Width - AMF$_{mw}$

Discussion

Medians provide several safety benefits including positive separation between oncoming traffic streams and space for errant vehicle recovery. The degree of benefit is correlated with the width of the median such that wider medians are associated with fewer crashes. To provide these safety benefits, the median should have a traversable cross section and be free of fixed objects (with the exception of median barriers or other safety treatments).

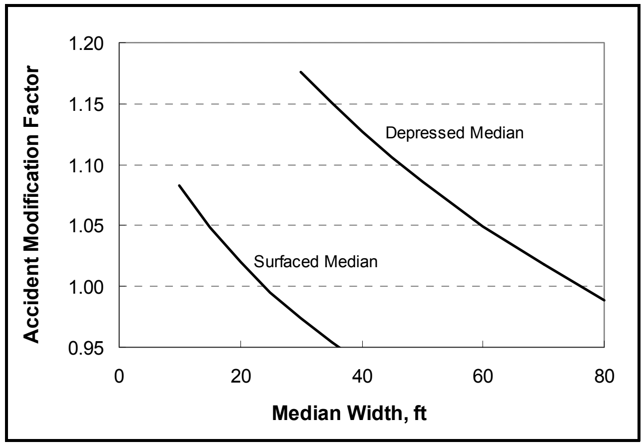


Figure 2-6. Median Width AMF.

Safety Relationship

The relationship between median width and severe crash frequency can be estimated using Figure 2-6, Equation 2-12, or Equation 2-13. 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 24 ft median width for surfaced medians and a 76 ft width for depressed medians.

For depressed medians:

$$AMF_{mw} = e^{-0.050(W_m^{0.5} - 76^{0.5})} \qquad \textbf{(2-12)}$$

For surfaced medians:

$$AMF_{mw} = e^{-0.046(W_m^{0.5} - 24^{0.5})} \qquad \textbf{(2-13)}$$

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

Base Condition: 24 ft width for surfaced medians, 76 ft width for depressed medians

Guidance

This AMF is applicable to freeways with a surfaced median ranging from 10 to 40 ft in width, and to those with a depressed median ranging from 30 to 80 ft in width. Median width is measured between the near edges of the left- and right-side traveled way. As such, it includes the width of the inside shoulders.

Example Application

The Question: How much will severe crashes increase if a 64 ft median is reduced to 48 ft?

The Facts:
- Median type: depressed
- Existing median width: 64 ft
- Proposed median width: 48 ft

The Solution: From Figure 2-6, find the AMF of 1.04 for the 64 ft median and the AMF of 1.09 for the 48 ft median. The ratio of these two AMFs indicates a 4.8 percent increase in severe crashes.

$$\begin{aligned} \%\,Increase &= 100\left(\frac{1.09}{1.04} - 1\right) \\ &= 4.8\,\% \end{aligned} \qquad \textbf{(2-14)}$$

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 severe crash frequency can be estimated using Table 2-7 or Equation 2-15. 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. 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 rumble strips are only placed on the outside shoulders, then the proportion used in Equation 2-15 should be multiplied by 0.50. If there are no shoulder rumble strips, then AMF_{srs} equals 1.0.

Example Application

The Question: What percent reduction in severe crashes will 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 0.96. This value implies that severe crashes will be reduced by 4 percent by the installation of shoulder rumble strips.

Table 2-7. Shoulder Rumble Strip AMF.

Area Type	Through Lanes	AMF_{srs}
Rural	4	0.96
	6	0.95
Urban	4	0.96
	6	0.97
	8	0.97
	10	0.97
Rural or urban	No rumble strips	1.00

$$AMF_{srs} = (0.88 - 1.0)\,P_i + 1.0 \qquad \textbf{(2-15)}$$

where:
AMF_{srs} = shoulder rumble strip 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 Strip AMF.

Area Type	Through Lanes	Proportion of Crashes [1]
Rural	4	0.30
	6	0.39
Urban	4	0.31
	6	0.27
	8	0.24
	10	0.25

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

Utility Pole Offset - AMF_{pd}

Discussion

Utility poles and large sign supports 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 freeway, 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 severe crash frequency can be estimated using Figure 2-7 or Equation 2-16. 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 30 ft and a pole density of 25 poles/mi.

Guidance

This AMF is applicable to pole densities ranging from 20 to 70 poles/mi and pole offsets ranging from 1 to 30 ft. A sensitivity analysis indicates ADT and pole density have a negligible effect on the AMF value. If Equation 2-16 is used, the proportion of crashes can be obtained from Table 2-9.

Example Application

The Question: What percent increase in severe crashes is likely to occur if pole offset is decreased from 30 to 10 ft?

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

The Solution: From Figure 2-7, find the AMF of 1.03. This value suggests that severe crashes will increase by about 3 percent if the utility pole offset is decreased to 10 ft.

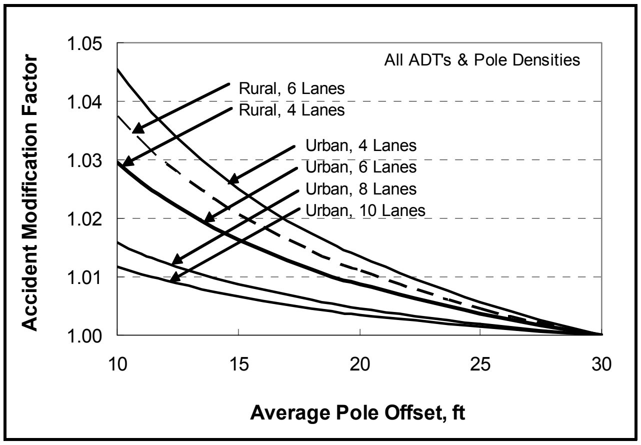


Figure 2-7. Utility Pole Offset AMF.

$$AMF_{pd} = (f_p - 1.0)\,P_i + 1.0 \qquad \textbf{(2-16)}$$

with,

$$f_p = \frac{(0.0000984\,ADT + 0.0354\,D_p)\,W_o^{-0.6} - 0.04}{0.0000128\,ADT + 0.075} \qquad \textbf{(2-17)}$$

where:
AMF_{pd} = utility pole offset accident modification factor;
P_i = proportion of influential crashes of type i (see Table 2-9);
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: 30 ft pole offset and 25 poles/mi

Table 2-9. Crash Distribution for Utility Pole Offset AMF.

Area Type	Through Lanes	Proportion of Crashes [1]
Rural	4	0.030
	6	0.038
Urban	4	0.046
	6	0.029
	8	0.016
	10	0.012

Note:
1 - Single-vehicle-with-pole crashes.

Safety Appurtenances

AMFs for roadside safety appurtenances are not described in this document. A comprehensive procedure for evaluating appurtenances is outlined in a report by Mak and Sicking (*3*) and automated in the *Roadside Safety Analysis Program* (RSAP) (*4*). 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."

REFERENCES

1. *Roadway Safety Design Synthesis*. Texas Transportation Institute, The Texas A&M University System, College Station, Texas, 2005.

2. 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.

3. 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.

4. 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 access to county roads and adjacent property. The combination of high-speed operation and relatively frequent access points can have significant adverse impact on safety and operations. As such, it is important to fully and accurately evaluate the safety impact of highway design alternatives during the design process.

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 or with a proposed design. In this regard, safety is defined as the expected frequency of severe (i.e., injury or 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 the *Roadway Safety Design Synthesis* (*1*). 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 a sensitivity to traffic volume, segment length, and the main factors known to be uniquely correlated with crash frequency for the subject rural highway. 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. (*2*) in that it predicts *severe* crash frequency (as opposed to total crash frequency). Otherwise, the procedure described herein is similar and shares the same strengths and weaknesses. The reader is referred to the report by Harwood et al. for a discussion of their procedure and its attributes.

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 indicates that crash rates for rural highways vary with the number of lanes and the median type used in the cross section (*1*). Each of these factors was found to have an influence on crash frequency. In general, crash rates are higher for highways with many lanes than those with few lanes. Also, crash rates for undivided highways, or those with a surfaced median, tend to be higher than the crash rates for highways with a depressed median. Surfaced medians include two-way left-turn lane (TWLTL) and flush-paved median.

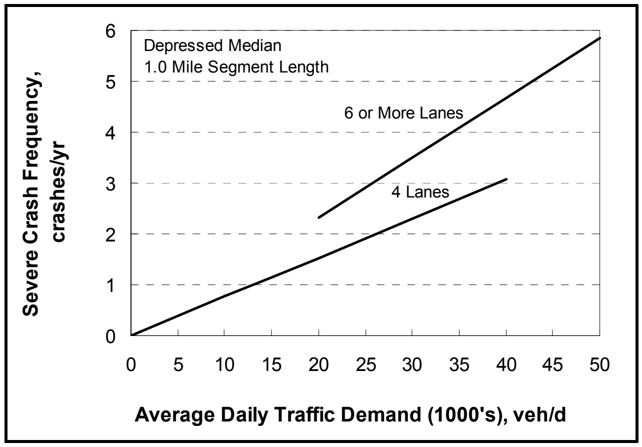
Safety Relationship

The relationship between severe crash frequency and traffic demand for typical rural highway conditions is shown in Figure 3-1. The trends shown in this figure apply to highway segments that are 1 mile long. The crash frequency obtained from the figure can be adjusted for other segment lengths by multiplying it by the actual segment length (in miles).

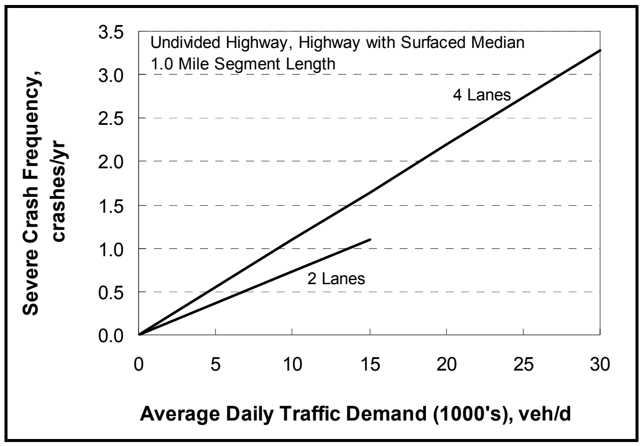
The crash rates that underlie Figure 3-1 are listed in Table 3-1. They can be used with Equation 3-1 to compute the expected severe crash frequency for the typical (i.e., base) condition.

Guidance

The severe crash frequency obtained from Figure 3-1 or Equation 3-1 is applicable to rural highways with typical characteristics. These characteristics are identified herein as "base" conditions. The complete set of base conditions are identified in Table 3-2.



a. Depressed Median.



b. Undivided Highway or Surfaced Median.

Figure 3-1. Base Crash Rates for Rural Highways.

$$C_b = 0.000365 \; Base \; ADT \; L \; f \qquad \textbf{(3-1)}$$

where:
- C_b = expected severe base crash frequency, crashes/yr;
- *Base* = severe crash rate (see Table 3-1), crashes/mvm;
- *ADT* = average daily traffic volume, veh/d;
- *L* = highway segment length, mi; and
- *f* = local calibration factor.

Table 3-1. Base Crash Rates for Rural Highways.

Median Type	Attributes	Base Crash Rate, severe crashes/mvm [1]		
	Through Lanes:	2	4	6
Undivided or Surfaced [2]		0.20	0.30	data not available
Depressed		data not available	0.21	0.32

Notes:
1 - mvm: million vehicle miles.
2 - Surfaced: flush paved or TWLTL. Rates for the TWLTL must be adjusted using the TWLTL median type AMF.

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

A local calibration factor is identified in Equation 3-1. A default value for this factor is recommended as 1.0. The factor can be used to adjust the predicted base crash frequency so that it is more consistent with typical highways in the agency's jurisdiction. A procedure for calibrating Equation 3-1 to local conditions is described by Harwood et al. (*2*). If this procedure is used, only severe crashes should be included in the calculations.

Example Application

The Question: What is the expected severe crash frequency for a typical undivided rural highway segment?

The Facts:

- Through lanes: 4
- Median type: undivided
- Segment length: 0.5 mi
- ADT: 20,000 veh/d

The Solution: From Figure 3-1, find that the typical undivided rural highway segment with these characteristics experiences about 2.2 crashes/mi/yr. For this specific highway segment, the expected severe crash frequency is estimated as 1.1 crashes/yr (= 0.5 × 2.2). The use of Equation 3-1 is illustrated in the box at the right.

Table 3-2. Base Conditions.

Characteristic	Base Condition
Horizontal curve radius	tangent (no curve)
Spiral transition curve	not present
Grade	flat (0% grade)
Lane width	12 ft
Outside shoulder width	8 ft
Inside shoulder width [1]	4 ft
Median width [2]	16 ft for surfaced median 76 ft for depressed median
Shoulder rumble strips	not present
Centerline rumble strip [3]	not present
Superelevation	not deficient
Passing lane [3]	not present
Horizontal clearance	30 ft
Side slope	1V:4H
Utility pole density and offset	25 poles/mi 30 ft average offset
Relative bridge width	12 ft
Driveway density	5 driveways/mi

Notes:
1 - Characteristic applies only to highways with a depressed median type.
2 - Surfaced median: TWLTL or flush-paved median.
3 - Characteristic applies only to undivided highways.

$$\begin{aligned} C_b &= 0.000365\ \mathit{Base}\ \mathit{ADT}\ L\ f \\ &= 0.000365 \times 0.30 \times 20{,}000 \times 0.5 \times 1.0 \\ &= 1.1\ \mathit{crashes/yr} \end{aligned} \qquad \textbf{(3-2)}$$

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 severe crash frequency. Topics addressed are listed in Table 3-3. The basis for each of these AMFs is described in Chapter 3 of the *Synthesis* (*1*). There are many additional factors, other than those listed in Table 3-3, that are likely to have some effect on severe crash frequency. However, their effect 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 are developed to yield a value of 1.0 when the associated design component or element represents "typical" rural highway conditions ("typical" characteristics are defined in the previous section). Deviation from base conditions to a more generous or desirable design condition results in an AMF of less than 1.0. Deviation to a more restricted condition results in an AMF of more than 1.0.

Table 3-3. AMFs for Rural Highway Segments.

Application	Accident Modification Factor
Geometric design	Horizontal curve radius Spiral transition curve Grade Lane width Outside shoulder width Inside shoulder width Median width Shoulder rumble strips Centerline rumble strip TWLTL median type Superelevation Passing lane
Roadside design	Horizontal clearance Side slope Utility pole offset Bridge width
Access control	Driveway density

Safety Relationship

The expected severe crash frequency for a specific rural highway segment is computed using Equation 3-3. The expected severe crash frequency represents the product of the base severe crash frequency and the various AMFs needed to account for characteristics that are not typical.

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

where:
C = expected severe crash frequency, crashes/yr;
C_b = expected severe base crash frequency, crashes/yr;
AMF_{lw} = lane width accident modification factor; and
AMF_{dd} = driveway density accident modification factor.

Guidance

In application, an AMF is identified for each rural highway characteristic that is not typical. All AMFs identified in this manner are then multiplied together. This product is then multiplied by the base severe crash frequency C_b for rural highways that are otherwise similar to the subject highway. The base severe crash frequency can be obtained from Figure 3-1 (or

Equation 3-1) or estimated from existing crash data. The product of this multiplication represents the expected severe crash frequency for the subject highway segment.

Example Application

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

The Facts:
- Through lanes: 4
- Median type: undivided
- Segment length: 0.5 mi
- ADT: 20,000 veh/d
- Base crash frequency C_b: 1.1 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 1.10. This AMF can be used with Equation 3-3 to estimate the expected severe crash frequency for the subject segment as 1.2 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 1.1 \times 1.10 \\ &= 1.2 \ crashes/yr \end{aligned} \tag{3-4}$$

Example Application

The Question: What is the expected severe crash frequency for a four-lane rural highway if the lane width is reduced from 12 to 10 ft?

The Facts:
- Severe crash frequency (3-year average): 3.0 crashes/year
- Existing average lane width: 12 ft
- Proposed lane width: 10 ft

The Solution: A three-year crash history is available and considered to be a better estimate of the expected severe crash frequency than the rates in Table 3-1. The AMF for a lane width of 10 ft is 1.10. This AMF can be used with Equation 3-3 to estimate the expected severe crash frequency for the subject segment with 10 ft lanes as 3.3 crashes/yr. This value represents an increase of 0.3 crashes/yr (3 in 10 years) if the lane width is reduced.

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 3.0 \times 1.10 \\ &= 3.3 \ crashes/yr \end{aligned} \tag{3-5}$$

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 highway ahead and, thereby, more driver sight distance. When curves of near-minimum radius are used, the designer should ensure that both the superelevation and the horizontal clearance to barriers, retaining walls, and embankments are adequate.

Safety Relationship

The relationship between curve radius and severe crash frequency can be estimated using Figure 3-2, Equation 3-6, or Equation 3-7. 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 curves with a radius of 500 ft or more. If spiral transition curves are present, the spiral transition curve AMF should also be used.

Example Application

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

The Facts:

- Curve radius: 1600 ft
- Curve deflection angle: 20 degrees
- No spiral transition

The Solution: From Figure 3-2 for "Deflection Angle = 20 deg," find the AMF of 1.31. This value suggests that 31 percent more severe crashes will occur on this curve, relative to a tangent section.

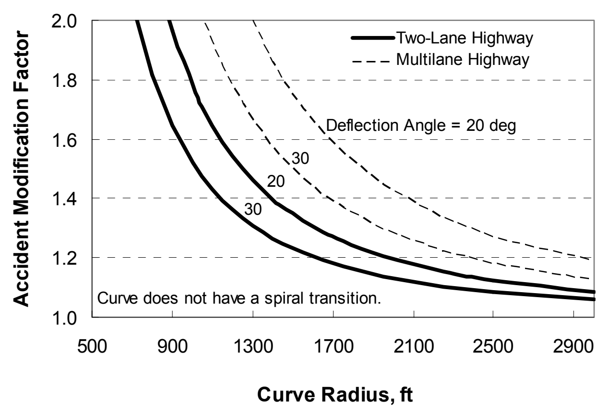


Figure 3-2. Horizontal Curve Radius AMF.

For two-lane highways:

$$AMF_{cr} = \frac{1.55\,L_c + \dfrac{80.2}{R}}{1.55\,L_c} \quad \textbf{(3-6)}$$

For multilane highways:

$$AMF_{cr} = 1 + \frac{1}{I_c}\left(\frac{5800}{R}\right)^2 \quad \textbf{(3-7)}$$

where:
AMF_{cr} = horizontal curve radius accident modification factor;
L_c = horizontal curve length, mi;
L_c = $R\,I_c$ / 5280 / 57.3;
R = curve radius, ft; and
I_c = curve deflection angle, degrees.

Base Condition: tangent section

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 effect a suitable transition path within the limits of a normal lane width for shorter curves or those that are relatively flat.

Safety Relationship

The relationship between spiral presence and severe crash frequency can be estimated using Figure 3-3 or Equation 3-8. 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 that spiral transition curves are not present.

Guidance

This AMF is only applicable to horizontal curves on *two-lane highways* with a radius of 500 ft or more. If a horizontal curve has spiral transitions, both the horizontal curve length AMF and the spiral transition curve AMF should be used. The variable L_c used in the horizontal curve length AMF should equal the length of the circular portion of the curve. If spiral curves are not present, then the spiral transition curve AMF is equal to 1.0.

Example Application

The Question: What is the safety benefit of adding spiral transition curves to a horizontal curve on a two-lane highway?

The Facts:

- Curve radius: 1350 ft
- Curve deflection angle: 20 degrees

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

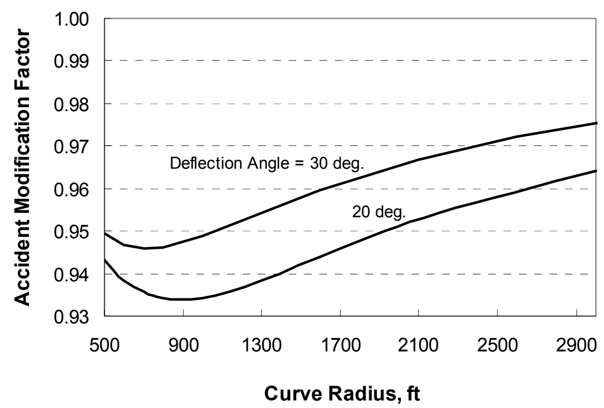


Figure 3-3. Spiral Transition Curve AMF.

$$AMF_{sp} = \frac{1.55\,L_c + \frac{80.2}{R} - 0.012}{1.55\,L_c + \frac{80.2}{R}} \qquad \textbf{(3-8)}$$

where:

- AMF_{sp} = spiral transition curve accident modification factor;
- L_c = horizontal curve length (circular portion), mi;
- L_c = $R\,I_c$ / 5280 / 57.3;
- R = curve radius, ft; and
- I_c = curve deflection angle, degrees.

Base Condition: spiral transition curves not present

Grade - AMF_g

Discussion

Grade can indirectly influence safety by influencing the speed of the traffic stream. Ascending grades pose a threat to safety due to increased speed differentials. Differences in speed between cars and trucks are most notable. Significant differences in speed among vehicles increases the frequency of lane changes and related crashes. Descending grades also pose a threat to safety due to the natural acceleration of gravity and associated additional demand placed on vehicle braking and maneuverability.

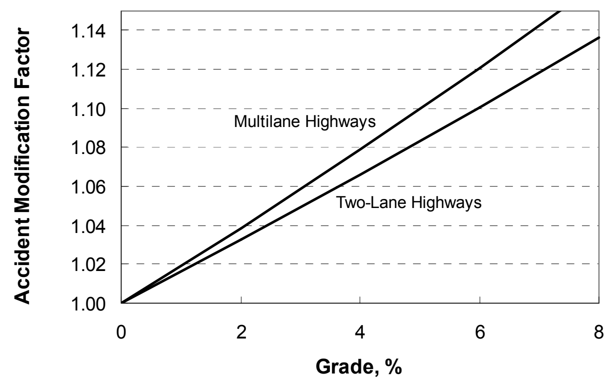


Figure 3-4. Grade AMF.

Safety Relationship

The relationship between grade and severe crash frequency can be estimated using Figure 3-4, Equation 3-9, or Equation 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 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:

$$AMF_g = e^{0.016\,g} \tag{3-9}$$

For multilane highways:

$$AMF_g = e^{0.019\,g} \tag{3-10}$$

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. In this application, the grade used in the AMF should equal the average of the absolute value of the curve entrance and exit grades.

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 3-4, find AMFs of 1.07 and 1.05 for grades of 4.0 and 2.8 percent, respectively. A reduction in severe crashes of 2 percent (= 100 ×[1 − 1.05/1.07]) is likely.

Lane Width (ADT > 2000 veh/d) - AMF_{lw}

Discussion

Occasionally, right-of-way constraints and limited construction costs require consideration of lane and shoulder widths that are narrower than what most guidelines identify as “desirable.” Any proposed reduction in lane width should consider the impact on safety. Experience indicates that crashes are more frequent on highways with lanes narrower than 12 ft. These crashes are particularly frequent when the narrow lanes are accompanied by other design features of minimum dimension.

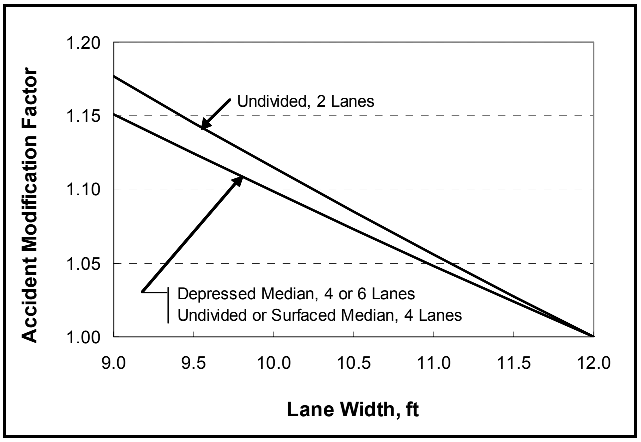


Figure 3-5. Lane Width AMF (ADT>2000 veh/d).

Safety Relationship

The relationship between lane width and severe crash frequency can be estimated using Figure 3-5 or Equation 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 base condition for this AMF is a 12 ft lane width.

$$AMF_{lw} = (e^{-0.047\,(W_l - 12)} - 1.0)\,\frac{P_i}{0.36} + 1.0 \qquad \textbf{(3-11)}$$

where:
AMF_{lw} = lane width accident modification factor;
P_i = proportion of influential crashes of type i (see Table 3-4); and
W_l = lane width, ft.

Base Condition: 12 ft lane width

Guidance

This AMF is applicable when the ADT is greater than 2000 veh/d, and to 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 Equation 3-11 is used, the proportion of crashes can be obtained from Table 3-4.

Table 3-4. Crash Distribution for Lane Width AMF.

Median Type	Through Lanes	Proportion of Crashes
Depressed [1]	4	0.36
	6	0.35
Undivided, TWLTL or flush paved [2]	2	0.42
	4	0.37

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

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 3-5 for “Undivided, 4 lanes” find the AMF of 1.10. This value suggests that 10 ft lanes are associated with a 10 percent increase in severe crashes.

Lane Width (ADT $\leq$ 2000 veh/d) - AMF_{lw}

Discussion

Lane width does not have as significant an influence on the safety of low-volume two-lane highways (i.e., 2000 veh/d or less) as it does for high-volume highways. On low-volume highways, drivers use the full width of the traveled way to their advantage because of infrequent meetings between vehicles from opposing directions. This behavior tends to result in fewer crashes, relative to busier highways.

Safety Relationship

The relationship between lane width and severe crash frequency can be estimated using Figure 3-6. 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. No equation is available for this AMF. The base condition lane for this AMF is a 12 ft lane width.

Guidance

This AMF is applicable only when the ADT is less than or equal to 2000 veh/d, and only to lane widths ranging from 9 to 12 ft. If the lane width is less than 9 ft, then the AMF value for 9 ft should be used. This AMF is only applicable to *two-lane highways*.

Example Application

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

The Facts:
- Average daily traffic: 1000 veh/d
- Lane width: 10 ft

The Solution: From Figure 3-6, find the AMF of 1.04. This value suggests that crash frequency will increase by 4 percent if 10 ft lanes are used instead of 12 ft lanes.

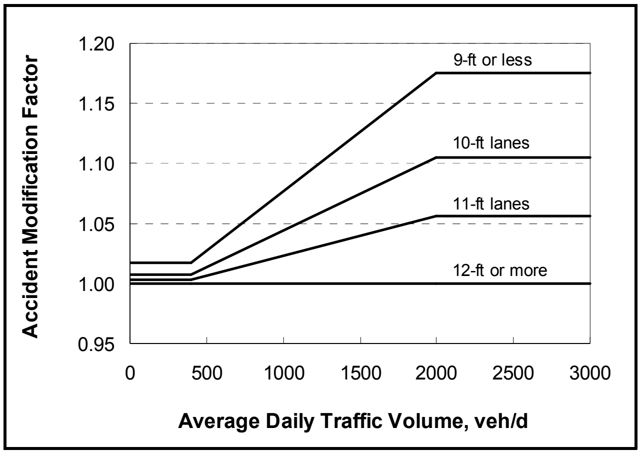


Figure 3-6. Lane Width AMF (ADT $\leq$ 2000 veh/d).

Base Condition: 12 ft lane width

Outside Shoulder Width (ADT > 2000 veh/d)- AMF_{osw}

Discussion

Shoulders offer numerous safety benefits for rural highways. Properly designed shoulders provide space for disabled vehicles and additional room for evasive maneuvers. 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 severe crash frequency can be estimated using Figure 3-7 or Equation 3-12. 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.

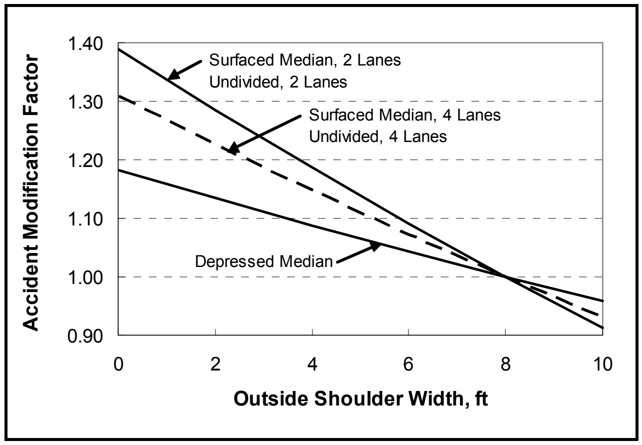


Figure 3-7. Outside Shoulder Width AMF (ADT > 2000 veh/d).

$$AMF_{osw} = (e^{-0.021\,(W_s - 8)} - 1.0)\frac{P_i}{0.16} + 1.0 \qquad \textbf{(3-12)}$$

where:
AMF_{osw} = outside shoulder width accident modification factor;
P_i = proportion of influential crashes of type *i* (see Table 3-5); and
W_s = outside shoulder width, ft.

Base Condition: 8 ft outside shoulder width

Guidance

This AMF is applicable to rural highways with daily volumes greater than 2000 veh/d, having a paved or gravel 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 3-12 is used, the proportion of influential crashes can be obtained from Table 3-5.

Table 3-5. Crash Distribution for Outside Shoulder Width AMF.

Median Type	Through Lanes	Proportion of Crashes
Depressed [1]	4	0.16
	6	0.15
Undivided, TWLTL, or flush paved [2]	2	0.34
	4	0.27

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 3-7, find the AMF of 1.07. This value implies that a 6 ft shoulder width will be associated with 7 percent more severe crashes than an 8 ft shoulder.

$$AMF_{osw} = (e^{-0.021 \times (6-8)} - 1.0)\frac{0.27}{0.16} + 1.0 = 1.07 \qquad \textbf{(3-13)}$$

Outside Shoulder Width (ADT $\leq$ 2000 veh/d) - AMF_{osw}

Discussion

Experience indicates that shoulder width does not have as critical an influence on the safety of low-volume two-lane highways (i.e., 2000 veh/d or less) as it does for high-volume highways. On low-volume highways, drivers can use the full width of the traveled way to their advantage because of relatively infrequent meetings between vehicles from opposing directions. This behavior tends to result in fewer crashes, relative to busier highways.

Safety Relationship

The relationship between outside shoulder width and severe crash frequency can be estimated using Figure 3-8. 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. No equation is available for this AMF. The base condition for this AMF is an 8 ft outside shoulder width.

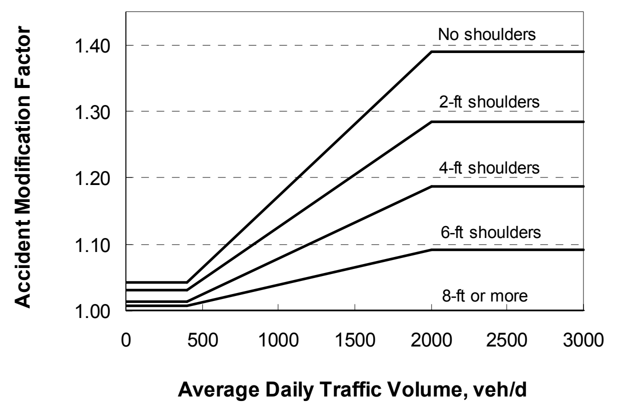


Figure 3-8. Outside Shoulder Width AMF (ADT $\leq$ 2000 veh/d).

Base Condition: 8 ft outside shoulder width

Guidance

This AMF is applicable when the ADT is less than or equal to 2000 veh/d, and the shoulder is paved or gravel ranging from 0 to 8 ft in width. If the shoulder width is greater than 8 ft, then the AMF value for 8 ft should be used. This AMF is only applicable to *two-lane highways*.

Example Application

The Question: What is the expected increase in crashes if a 2 ft shoulder is used instead of an 8 ft shoulder on a two-lane highway?

The Facts:

- Severe crash frequency: 2 in 5 years
- Existing shoulder width: 8 ft
- Average daily traffic: 1500 veh/d

The Solution: From Figure 3-8, find the AMF of 1.20. The expected severe crash frequency with a 2 ft shoulder would be 0.48 crashes/yr (= 2/5 × 1.20). This value equates to less than one additional severe crash in 12 years.

Inside Shoulder Width - AMF_{isw}

Discussion

Inside shoulders offer similar safety benefits for divided rural highways as do outside shoulders. Specifically, they provide storage space for disabled vehicles and additional room for evasive maneuvers. Inside (i.e., left-hand) shoulders are typically provided on divided rural highways because of these safety benefits.

Safety Relationship

The relationship between inside shoulder width and severe crash frequency can be estimated using Figure 3-9 or Equation 3-14. 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. The base condition for this AMF is a 4 ft inside shoulder width.

Guidance

This AMF is applicable to highways with 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 3-14 is used, the proportion of crashes can be obtained from Table 3-6.

Example Application

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

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

The Solution: From Figure 3-9, find the AMF of 0.96. This value implies a 4 percent reduction in severe crashes if a 6 ft shoulder width is used instead of a 4 ft width.

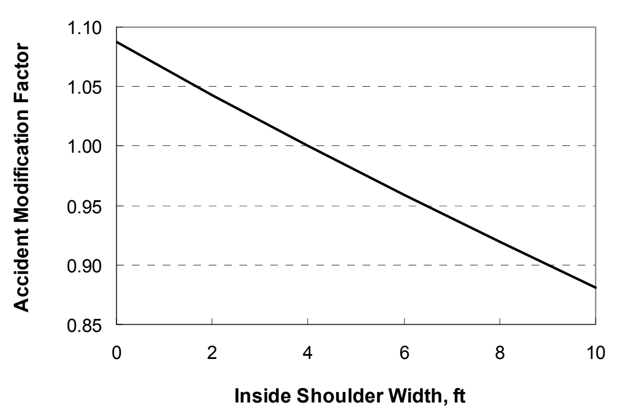


Figure 3-9. Inside Shoulder Width AMF.

$$AMF_{isw} = (e^{-0.021\,(W_{is} - 4)} - 1.0)\,\frac{P_i}{0.16} + 1.0 \qquad \textbf{(3-14)}$$

where:
AMF_{isw} = inside shoulder width accident modification factor;
P_i = proportion of influential crashes of type *i* (see Table 3-6); and
W_{is} = inside shoulder width, ft.

Base Condition: 4 ft inside shoulder width

Table 3-6. Crash Distribution for Inside Shoulder Width AMF.

Median Type	Through Lanes	Proportion of Crashes [1]
Depressed	4	0.16
	6	0.15

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

$$AMF_{isw} = (e^{-0.021 \times (6 - 4)} - 1.0)\,\frac{0.15}{0.16} + 1.0 \qquad \textbf{(3-15)}$$
$$= 0.96$$

Median Width - AMF_{mw}

Discussion

Medians provide several safety benefits including positive separation between oncoming traffic streams, space for left-turn bays, and control of access. The degree of benefit is correlated with the width of the median such that wider medians are associated with fewer crashes. To provide these safety benefits, the median should have a traversable cross section and be free of fixed objects (with the exception of median barriers or other safety treatments).

Safety Relationship

The relationship between median width and severe crash frequency can be estimated using Figure 3-10, Equation 3-16, or 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 16 ft median width for surfaced medians and a 76 ft width for depressed medians.

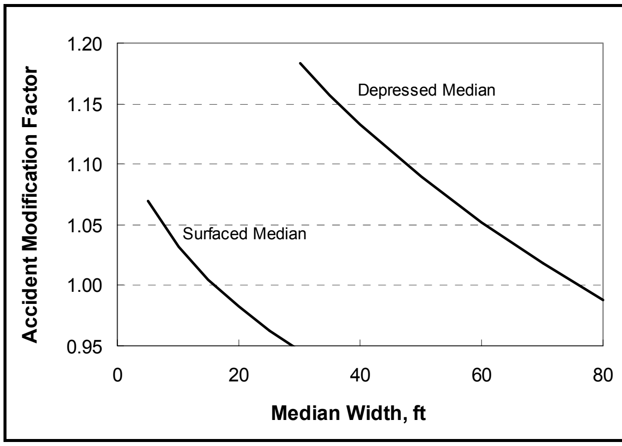


Figure 3-10. Median Width AMF.

For depressed medians:

$$AMF_{mw} = e^{-0.052\,(W_m^{0.5} - 76^{0.5})} \quad \textbf{(3-16)}$$

For surfaced medians:

$$AMF_{mw} = e^{-0.038\,(W_m^{0.5} - 16^{0.5})} \quad \textbf{(3-17)}$$

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

Base Condition: 16 ft width for surfaced medians, 76 ft width for depressed medians

Guidance

This AMF is applicable to highways with a surfaced median ranging from 4 to 30 ft in width, and to those with a depressed median ranging from 30 to 80 ft in width. Median width is measured between the near edges of the left- and right-side traveled way. As such, it includes the width of the inside shoulders. This AMF is not applicable to highways with a TWLTL. Hence, this AMF should not be used with the TWLTL median type AMF.

Example Application

The Question: How much will severe crashes decrease if a 12 ft median is increased to 24 ft?

The Facts:

- Median type: surfaced

The Solution: From Figure 3-10, find AMFs of 1.02 and 0.97 for the 12 and 24 ft median widths, respectively. The ratio of the two AMFs indicates a 4.9 percent decrease in crashes.

$$\begin{aligned} \%\,reduction &= 100\left(1 - \frac{0.97}{1.02}\right) \\ &= 4.9\% \end{aligned} \quad \textbf{(3-18)}$$

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 severe crash frequency can be estimated using Table 3-7 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 no shoulder rumble strips.

Guidance

This AMF is based on the installation of continuous rumble strips along all shoulders. If rumble strips are only placed on the outside shoulders of a divided highway, then the proportion used in Equation 3-19 should be multiplied by 0.50. If there are no shoulder rumble strips, then AMF_{srs} equals 1.0.

Example Application

The Question: What percent increase in severe crashes will result if shoulder rumble strips are removed from both sides of a rural highway?

The Facts:
- Median type: depressed
- Through lanes: 6

The Solution: From Table 3-7, find the AMF of 0.98 for having rumble strips and 1.00 for no rumble strips. The ratio of these two values implies that the removal of rumble strips will increase severe crashes by 2 percent (= 100 ×[1.00/ 0.98 − 1]).

Table 3-7. Shoulder Rumble Strip AMF.

Median Type	Through Lanes	AMF_{srs}
Depressed	4	0.98
	6	0.98
Undivided, TWLTL, or flush paved	2	0.99
	4	0.99
Any	No rumble strips	1.00

$$AMF_{srs} = (0.93 - 1.0)\, P_i + 1.0 \qquad \textbf{(3-19)}$$

where:
AMF_{srs} = shoulder rumble strip accident modification factor; and
P_i = proportion of influential crashes of type *i* (see Table 3-8).

Base Condition: shoulder rumble strips not present

Table 3-8. Crash Distribution for Shoulder Rumble Strip AMF.

Median Type	Through Lanes	Proportion of Crashes [1]
Depressed [1]	4	0.32
	6	0.31
Undivided, TWLTL, or flush paved [2]	2	0.17
	4	0.13

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

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 rumble strip presence and severe 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 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. This AMF is only applicable to *undivided highways*.

Example Application

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

The Facts:
- Through lanes: 2

The Solution: From Table 3-9, find the AMF of 0.86 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 will reduce severe crashes by 14 percent (= 100 ×[1 −0.86/1.00]).

Table 3-9. Centerline Rumble Strip AMF.

Median Type	Through Lanes	AMF_{crs}
Undivided	2	0.86
	4	0.86
Any	No rumble strips	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. In spite of these safety and operational benefits, the TWLTL is generally recognized to be associated with higher crash risk than a depressed median when there are frequent busy driveways.

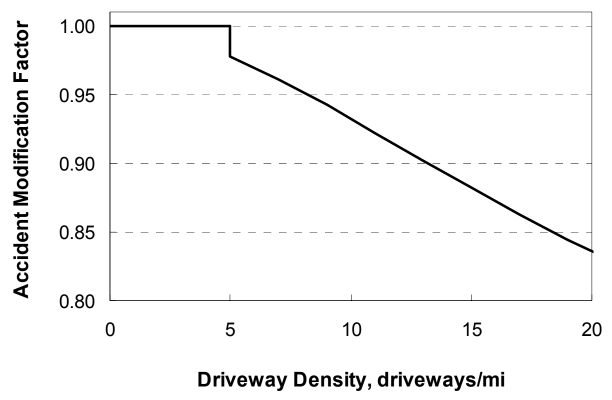


Figure 3-11. TWLTL Median Type AMF.

Safety Relationship

The relationship between TWLTL presence and severe crash frequency can be estimated using Figure 3-11 or 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 no TWLTL (i.e, undivided cross section).

$$AMF_T = 1.0 - 0.35\,P_D \qquad \textbf{(3-20)}$$

with,

$$P_D = \frac{0.0047\,D_d + 0.0024\,D_d^2}{1.199 + 0.0047\,D_d + 0.0024\,D_d^2} \qquad \textbf{(3-21)}$$

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

Base Condition: no TWLTL

Guidance

If the driveway density is less than 5.0 driveways/mi, then AMF_T equals 1.0. This AMF should not be used with the median width AMF.

Example Application

The Question: What percent reduction in crashes will result if a TWLTL is added to an undivided rural highway?

The Facts:
- Driveway density: 10 driveways/mi

The Solution: From Figure 3-11, find the AMF of 0.93. This value implies that crashes will be reduced by 7 percent by the addition of a 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 less than the amount specified by the applicable design guideline, then the risk of a crash increases.

Safety Relationship

The relationship between superelevation deficiency and severe crash frequency can be estimated using Figure 3-12. 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 on a curve is lower than that specified in the applicable design guideline is defined as "superelevation deficiency." The base condition for this AMF is no superelevation deficiency.

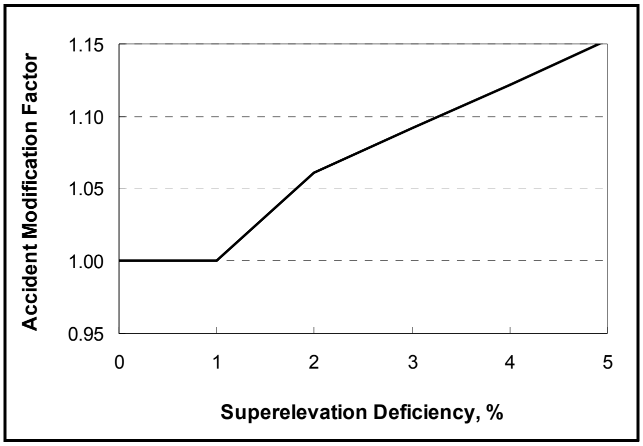


Figure 3-12. Superelevation AMF.

Base Condition: no superelevation deficiency

Guidance

This AMF is applicable to curves with a deficiency of 5 percent or less. If the deficiency exceeds 5 percent, then the value for 5 percent should be used. If the superelevation rate exceeds that specified in the applicable design guideline, 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:

- Severe crash frequency: 0.32 crashes/yr

The Solution: The deficiency is 3 percent (= 5.7 −2.7). From Figure 3-12, find the AMF of 1.09. If 5.7 percent is used, the expected severe crash frequency is 0.29 crashes/yr (= 0.32/1.09). Thus, the improvement yields a reduction of 0.03 severe crashes/yr.

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 severe 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 only 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 will likely 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 severe crashes will 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

Horizontal Clearance - AMF_{hc}

Discussion

Fixed objects along the highway are undesirable because they can cause serious injury if struck by an errant vehicle. For these reasons, maintenance of a nominal horizontal clearance distance along the highway is desirable. However, it is also recognized that right-of-way constraints and other factors make it difficult to provide desirable clearance distances in all cases.

Safety Relationship

The relationship between horizontal clearance and severe crash frequency can be estimated using Figure 3-13 or Equation 3-22. 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.

Guidance

This AMF is applicable to clearance distances ranging from 0 to 30 ft. If Equation 3-22 is used, the proportion of crashes can be obtained from Table 3-11.

Example Application

The Question: How many additional severe crashes will likely occur if horizontal clearance is decreased from 30 to 15 ft?

The Facts:

- Median type: depressed
- Through lanes: 6
- Existing horizontal clearance: 30 ft
- Existing severe crash frequency: 5 cr/yr

The Solution: From Figure 3-13, find the AMF of 1.04. This value suggests that an average of 0.2 additional severe crashes [= 5 ×(1.04 − 1.0)] are likely to occur each year due to the change.

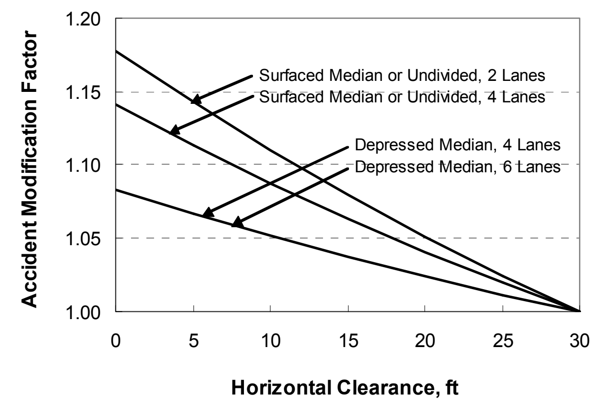


Figure 3-13. Horizontal Clearance AMF.

$$AMF_{hc} = (e^{-0.014\,(W_{hc} - 30)} - 1.0)\,P_i + 1.0 \qquad \textbf{(3-22)}$$

where:
AMF_{hc} = horizontal clearance accident modification factor;
P_i = proportion of influential crashes of type *i* (see Table 3-11); and
W_{hc} = horizontal clearance (average for segment length), ft.

Base Condition: 30 ft horizontal clearance

Table 3-11. Crash Distribution for Horizontal Clearance AMF.

Median Type	Through Lanes	Proportion of Crashes
Depressed [1]	4	0.16
	6	0.15
Undivided, TWLTL, or flush paved [2]	2	0.34
	4	0.27

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

$$\begin{aligned} AMF_{hc} &= (e^{-0.014\,(15 - 30)} - 1.0)\,0.15 + 1.0 \\ &= 1.04 \end{aligned} \qquad \textbf{(3-23)}$$

Side Slope - AMF_{ss}

Discussion

There are several advantages of flatter side slopes. 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 should be graded as flat as practical, but still provide for necessary drainage.

Safety Relationship

The relationship between side slope and severe crash frequency can be estimated using Figure 3-14 or Equation 3-24. 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. 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 3-24 is used, the proportion of crashes can be obtained from Table 3-12.

Example Application

The Question: What percent increase in severe 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 3-14, find the AMF of 1.02. This value suggests that severe crashes will increase 2 percent if a 1:3 side slope is used for the highway instead of a 1:4 slope.

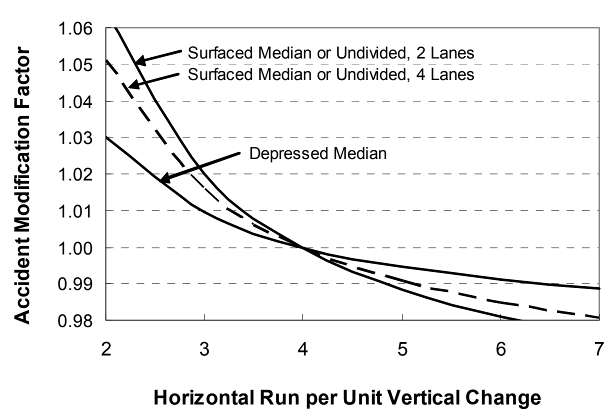


Figure 3-14. Side Slope AMF.

$$AMF_{ss} = (e^{0.69\,(1/S_s - 1/4)} - 1.0)\,P_i + 1.0 \qquad \textbf{(3-24)}$$

where:
AMF_{ss} = side slope accident modification factor;
P_i = proportion of influential crashes of type *i* (see Table 3-12); and
S_s = horizontal run for a 1 ft change in elevation (average for segment length), ft.

Base Condition: 1V:4H

Table 3-12. Crash Distribution for Side Slope AMF.

Median Type	Through Lanes	Proportion of Crashes
Depressed [1]	4	0.16
	6	0.15
Undivided, TWLTL, or flush paved [2]	2	0.34
	4	0.27

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

$$\begin{aligned} AMF_{ss} &= (e^{0.69\,(1/3 - 1/4)} - 1.0)\,0.27 + 1.0 \\ &= 1.02 \end{aligned} \qquad \textbf{(3-25)}$$

Utility Pole Offset - AMF_{pd}

Discussion

Utility poles and large sign supports 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 highway, 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 severe crash frequency can be estimated using Figure 3-15 or 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 for this AMF is a pole offset of 30 ft and a pole density of 25 poles/mi.

Guidance

This AMF is applicable to pole densities ranging from 20 to 70 poles/mi and pole offsets ranging from 1 to 30 ft. A sensitivity analysis indicates traffic volume and pole density have a small effect on the AMF value. If Equation 3-26 is used, the proportion of crashes can be obtained from Table 3-13.

Example Application

The Question: What percent change in severe crashes is likely to be realized if pole offset is decreased from 30 to 5 ft?

The Facts:

- Median type: TWLTL
- Through lanes: 4
- Pole density: 20 poles/mi
- ADT: 5,000 veh/d

The Solution: From Figure 3-15, find the AMF of 1.10. This value suggests that severe crashes will increase by about 10 percent if the utility pole offset is decreased to 5 ft.

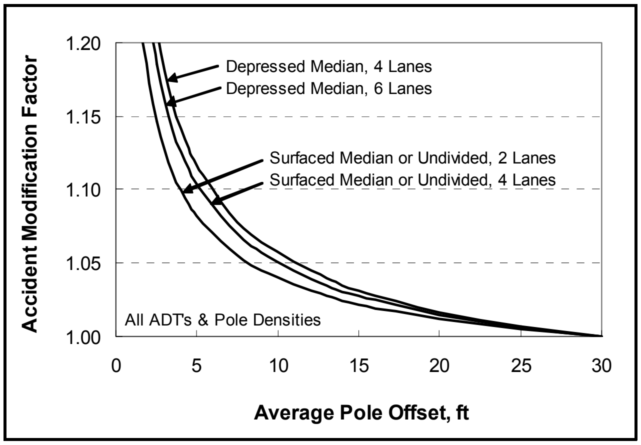


Figure 3-15. Utility Pole Offset AMF.

$$AMF_{pd} = (f_p - 1.0)\, P_i + 1.0 \qquad \textbf{(3-26)}$$

with,

$$f_p = \frac{(0.0000984\, ADT + 0.0354\, D_p)\, W_o^{-0.6} - 0.04}{0.0000128\, ADT + 0.075} \qquad \textbf{(3-27)}$$

where:

AMF_{pd} = utility pole offset accident modification factor;
P_i = proportion of influential crashes of type i (see Table 3-13);
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: 30 ft pole offset and 25 poles/mi

Table 3-13. Crash Distribution for Utility Pole Offset AMF.

Median Type	Through Lanes	Proportion of Crashes [1]
Depressed	4	0.054
	6	0.046
Undivided, TWLTL, or flush paved	2	0.038
	4	0.048

Note:
1 - Single-vehicle-with-pole crashes.

Bridge Width - AMF_{bw}

Discussion

Ideally, bridge railings and approach treatments are located beyond the horizontal clearance distance. However, this ideal is rarely achieved and bridge elements are often located within this distance. In this situation, the more distant these bridge elements are from the edge of traveled way, the greater the safety benefit.

Safety Relationship

The relationship between bridge width and severe crash frequency can be estimated using Figure 3-16 or Equation 3-28. 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 "relative bridge width" of 12 ft. Relative bridge width is defined as the difference between the bridge width and the approach traveled-way width. Bridge width is measured between the face of the bridge rails. Approach traveled-way width is the sum of the lane widths on the approach to the bridge (i.e., it excludes median and shoulder width).

Guidance

This AMF is applicable to relative bridge widths ranging from -6.0 to 14 ft. If Equation 3-28 is used, the proportion of crashes can be obtained from Table 3-14. This AMF is only applicable to *undivided highways*.

Example Application

The Question: What is the AMF for a two-lane highway segment with a bridge on it?

The Facts:
- Through lanes: 2
- Bridge width: 28 ft
- Approach traveled-way width: 24 ft

The Solution: Relative bridge width is 4 ft (= 28 − 24). From Figure 3-16, find the AMF of 1.03. This value implies it is likely that the bridge will be associated with 3 percent more crashes than one with a relative width of 12 ft.

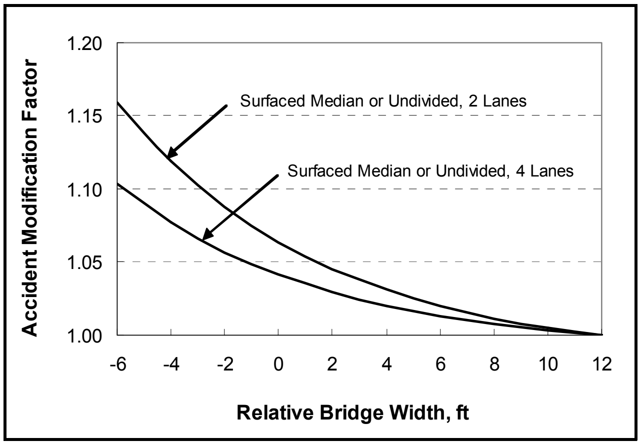


Figure 3-16. Bridge Width AMF.

$$AMF_{bw} = (e^{-0.13\, I_{br}(W_b - 12)} - 1.0)P_i + 1.0 \qquad \textbf{(3-28)}$$

where:
AMF_{bw} = bridge width accident modification factor;
P_i = proportion of influential crashes of type *i* (see Table 3-14);
I_{br} = indicator variable for bridge presence (1 if one or more bridges present, 0 if no bridges); and
W_b = relative bridge width (= bridge width − approach traveled-way width), ft.

Base Condition: 12 ft relative bridge width

Table 3-14. Crash Distribution for Bridge Width AMF.

Median Type	Through Lanes	Proportion of Crashes [1]
Undivided	2	0.017
	4	0.011

Note:
1 - Single-vehicle-with-bridge crashes.

$$AMF_{bw} = (e^{-0.13 \times 1.0\,(4.0 - 12)} - 1.0)0.017 + 1.0 = 1.03 \qquad \textbf{(3-29)}$$

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. In this regard, fewer but wider driveways is a desirable design goal. Access management is an effective technique for guiding the process of locating driveways such that conflicts associated with turning vehicles are minimized. One key element of access management is driveway density.

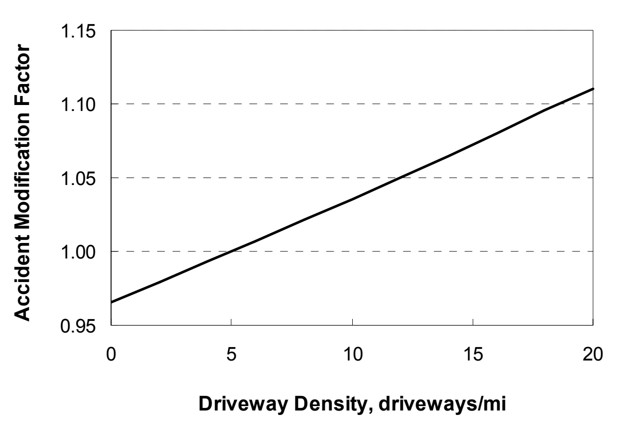


Figure 3-17. Driveway Density AMF.

Safety Relationship

The relationship between driveway density and severe crash frequency can be estimated using Figure 3-17 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 5 driveways/mi.

$$AMF_{dd} = e^{0.007\,(D_d - 5)} \qquad \textbf{(3-30)}$$

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

Base Condition: 5 driveways per mile

Guidance

This AMF is applicable to driveway 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.

Example Application

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

The Facts:
- ADT: 5000 veh/d
- Median type: undivided
- Segment length: 0.5 mi

The Solution: From Figure 3-17, find the AMFs of 0.98 and 1.04 for driveway densities of 2 and 10 driveways/mi, respectively. The percent increase in severe crashes is 6 percent (= 100 × [1.04/0.98 − 1]).

$$\begin{aligned} AMF_{dd} &= e^{0.007\,(10 - 5)} \\ &= 1.04 \end{aligned} \qquad \textbf{(3-31)}$$

Safety Appurtenances

AMFs for roadside safety appurtenances are not described in this document. A comprehensive procedure for evaluating appurtenances is outlined in a report by Mak and Sicking (*3*) and automated in the *Roadside Safety Analysis Program* (RSAP) (*4*). 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."

REFERENCES

1. *Roadway Safety Design Synthesis*. Texas Transportation Institute, The Texas A&M University System, College Station, Texas, 2005.

2. 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.

3. 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.

4. 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 4

Urban Streets

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INTRODUCTION

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

The process of designing an urban 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 urban street facility or with a proposed design. In this regard, safety is defined as the expected frequency of severe (i.e., injury or fatal) crashes. The safety benefit of a proposed design can be obtained by comparing its expected crash frequency with that of the existing facility. Background information about the various equations and constants that comprise the procedure is provided in the *Roadway Safety Design Synthesis* (*1*). 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 street segments. A street 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 an urban street 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 street segment. The base model includes a sensitivity to traffic volume and segment length. AMFs are used to account for factors found to have some correlation with crash frequency. The AMFs are multiplied by the base crash frequency to obtain an expected crash frequency for the subject street segment.

The procedure described herein differs from that developed by Harwood et al. (*2*) in that it predicts *severe* crash frequency (as opposed to total crash frequency). Otherwise, the procedure described herein is similar and shares the same strengths and weaknesses. The reader is referred to the report by Harwood et al. for a discussion of their procedure and its attributes.

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.

The relationships described in this chapter address the occurrence of vehicle-related crashes on urban streets. Relationships that focus on vehicle-pedestrian and vehicle-bicycle crashes on streets will be added in future updates to this chapter.

Base Models

Discussion

An examination of crash trends indicates that crash rates for urban streets vary with the number of lanes, median type, and adjacent land use associated with the street (*1*). In general, crash rates are higher for streets with many lanes than those with few lanes. Also, crash rates for undivided streets or streets with a two-way left-turn lane (TWLTL) tend to be higher than the crash rates for streets with a raised-curb median. Crash rates also tend to be higher in areas with commercial, business, or office land uses; relative to those in residential or industrial areas. This latter influence is likely a reflection of more frequent driveway activity in commercial, business, and office areas.

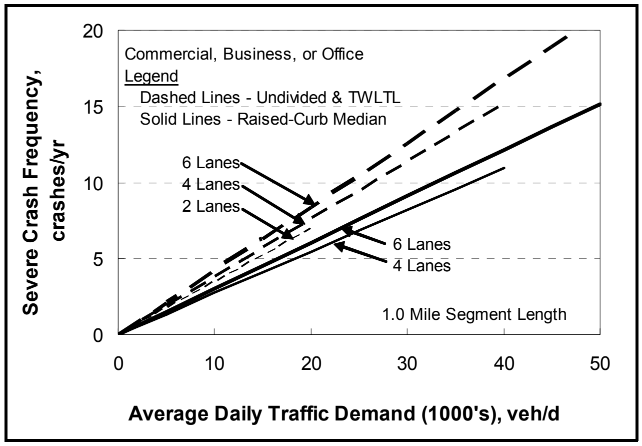
Safety Relationship

The relationship between severe crash frequency and traffic demand for typical urban street conditions is shown in Figure 4-1. The trends shown in this figure apply to street segments that are 1 mile long. The crash frequency obtained from the figure can be adjusted for other segment lengths by multiplying it by the actual segment length (in miles).

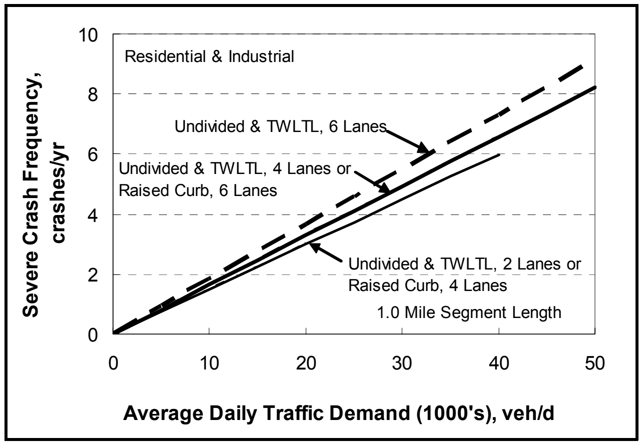
The crash rates that underlie Figure 4-1 are listed in Table 4-1. They can be used with Equation 4-1 to compute the expected severe crash frequency for the typical (i.e., base) condition.

Guidance

The severe crash frequency obtained from Figure 4-1 or Equation 4-1 is applicable to urban streets with typical characteristics. These characteristics are identified herein as "base" conditions. The complete set of base conditions are identified in Table 4-2.



a. Commercial, Business, or Office Land Use.



b. Residential or Industrial Land Use.

Figure 4-1. Base Crash Rates for Urban Streets.

$$C_b = 0.000365\ Base\ ADT\ L\ f \qquad \textbf{(4-1)}$$

where:
C_b = expected severe base crash frequency, crashes/yr;
$Base$ = severe crash rate (see Table 4-1), crashes/mvm;
ADT = average daily traffic volume, veh/d;
L = street segment length, mi; and
f = local calibration factor.

Table 4-1. Base Crash Rates for Urban Streets.

Adjacent Land Use	Attributes	Base Crash Rate, severe crashes/mvm [1]				
	Median Type:	Undivided or TWLTL Median [2]			Raised-Curb Median	
	Through Lanes:	2	4	6	4	6
Commercial, business, or office		0.95	1.04	1.15	0.75	0.83
Residential or industrial		0.41	0.45	0.50	0.41	0.45

Notes:
1 - mvm: million vehicle miles.
2 - Rates for the TWLTL median must be adjusted using the TWLTL median type AMF.

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

A local calibration factor is identified in Equation 4-1. A default value for this factor is recommended as 1.0. The factor can be used to adjust the predicted base crash frequency so that it is more consistent with typical streets in the agency's jurisdiction. A procedure for calibrating Equation 4-1 to local conditions is described by Harwood et al. (*2*). If this procedure is used, only severe crashes should be included in the calculations.

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	16 ft
Curb parking	none
Utility pole density and offset	50 poles/mi 2.0 ft average offset
Driveway density	Undivided - 50 drives/mi TWLTL - 50 drives/mi Raised curb - 25 drives/mi
Truck presence	6% trucks

Note:
1 - Curb-and-gutter is assumed as typical. Width shown is an "effective" shoulder width for curb-and-gutter.

Example Application

The Question: What is the expected severe crash frequency for a typical four-lane street?

The Facts:
- Through lanes: 4
- Median type: undivided
- Adjacent land use: commercial
- Segment length: 0.2 mi
- ADT: 20,000 veh/d

The Solution: From Figure 4-1a, find that the typical street segment with these characteristics experiences about 7.6 crashes/mi/yr. For this specific street segment, the expected severe crash frequency is estimated as 1.5 crashes/yr (= 0.2 × 7.6). The use of Equation 4-1 is illustrated in the box at the right.

$$\begin{aligned} C_b &= 0.000365\ Base\ ADT\ L\ f \\ &= 0.000365 \times 1.04 \times 20{,}000 \times 0.2 \times 1.0 \qquad \textbf{(4-2)} \\ &= 1.5\ crashes/yr \end{aligned}$$

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 severe crash frequency. Topics addressed are listed in Table 4-3. The basis for each of these AMFs is described in Chapter 4 of the *Synthesis* (*1*). There are many additional factors, other than those listed in Table 4-3, that are likely to have some effect on crash frequency. However, their effect has yet to be quantified through research. The list of available AMFs for urban streets is likely to increase as new research in this area is undertaken.

All of the AMFs described in this section are developed to yield a value of 1.0 when the associated design component or element represents "typical" urban street conditions ("typical" characteristics are defined in the previous section). Deviation from base conditions to a more generous or desirable design condition results in an AMF of less than 1.0. Deviation to a more restricted condition results in an AMF of more than 1.0.

Table 4-3. AMFs for Urban Street Segments.

Application	Accident Modification Factor
Geometric design	Horizontal curve radius Lane width Shoulder width Median width TWLTL median type Curb parking
Roadside design	Utility pole offset
Access control	Driveway density
Street environment	Truck presence

Safety Relationship

The expected severe crash frequency for a specific street segment is computed using Equation 4-3. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are not typical.

$$C = C_b \times AMF_{lw} \times AMF_{dd} \ldots \qquad \textbf{(4-3)}$$

where:

C = expected severe crash frequency, crashes/yr;
C_b = expected severe base crash frequency, crashes/yr;
AMF_{lw} = lane width accident modification factor; and
AMF_{dd} = driveway density accident modification factor.

Guidance

In application, an AMF is identified for each street characteristic that is not typical. All AMFs identified in this manner are then multiplied together. This product is then multiplied by the base crash frequency C_b for streets that are otherwise similar to the subject street. The base crash frequency can be obtained from Figure 4-1 (or Equation 4-1) or estimated from existing crash data. The product of this multiplication represents the expected crash frequency for the subject street segment.

Example Application

The Question: What is the expected severe crash frequency for a specific four-lane urban street?

The Facts:

- Through lanes: 4
- Median type: undivided
- Adjacent land use: commercial
- Segment length: 0.2 mi
- ADT: 20,000 veh/d
- Base crash frequency C_b: 1.5 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 4-3 to estimate the expected severe crash frequency for the subject segment as 1.6 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 1.5 \times 1.06 \\ &= 1.6 \; crashes/yr \end{aligned} \tag{4-4}$$

Example Application

The Question: What is the expected severe crash frequency for a four-lane urban street if the lane width is reduced from 12 to 10 ft?

The Facts:

- Severe crash frequency (3-year average): 2.0 crashes/year
- Existing average lane width: 12 ft
- Proposed lane width: 10 ft

The Solution: A three-year crash history is available and considered to be a better estimate of the expected severe crash frequency than the rates in Table 4-1. The AMF for a lane width of 10 ft is 1.06. This AMF can be used with Equation 4-3 to estimate the expected severe crash frequency for the subject segment with 10 ft lanes as 2.1 crashes/yr. This value represents an increase of 0.1 crashes/yr (1 in 10 years) if the lane width is reduced.

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 2.0 \times 1.06 \\ &= 2.1 \; crashes/yr \end{aligned} \tag{4-5}$$

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 street ahead and, thereby, more driver sight distance. When curves of near-minimum radius are used, the designer should ensure that both the superelevation and the horizontal clearance to barriers, retaining walls, and embankments are adequate.

Safety Relationship

The relationship between curve radius and severe crash frequency can be estimated using Figure 4-2 or Equation 4-6. 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 a tangent street 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 curves with a radius of 500 ft or more. If Equation 4-6 is used, the proportion of crashes that occur off the roadway can be obtained from Table 4-4.

Example Application

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

The Facts:
- Median type: undivided
- Through lanes: 4
- Radius: 820 ft (7.0 degrees)

The Solution: From Figure 4-2, find the AMF of 1.11. This value suggests that 11 percent more severe crashes will occur on this curve, relative to a tangent section.

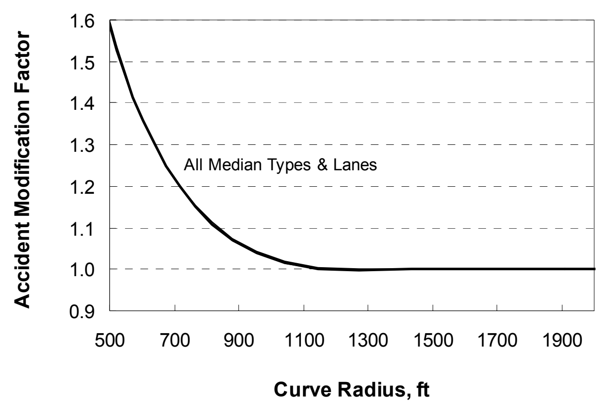


Figure 4-2. Horizontal Curve Radius AMF.

$$AMF_{cr} = \begin{cases} 2.30\,A + 0.781\,B & : \text{if } R \le 1300 \\ 1.00 & : \text{if } R > 1300 \end{cases} \quad \textbf{(4-6)}$$

$$A = \left(e^{-2300/R} + \frac{344}{R} \right)(1 - P_{off-road}) \quad \textbf{(4-7)}$$

$$B = (e^{321/R})\,P_{off-road} \quad \textbf{(4-8)}$$

where:
AMF_{cr} = horizontal curve radius accident modification factor;
$P_{off\text{-}road}$ = proportion of crashes that occur off the roadway (see Table 4-4); and
R = curve radius, ft.

Base Condition: tangent section

Table 4-4. Crash Distribution for Curve Radius AMF.

Median Type	Through Lanes	Proportion of Crashes [1]
Undivided or TWLTL	2	0.17
	4	0.10
	6	0.054
Raised curb	4	0.18
	6	0.17

Note:
1 - Single-vehicle run-off-road crashes.

Lane Width - AMF_{lw}

Discussion

A reduction in lane width for the purpose of increasing the total number of lanes in the cross section is sometimes considered to obtain additional capacity. However, any proposed reduction in lane width should consider the impact on safety. Experience indicates that crashes are more frequent on streets with lanes narrower than 12 ft. These crashes are particularly frequent when the narrow lanes are accompanied by other design features of minimum dimension.

Safety Relationship

The relationship between lane width and severe crash frequency can be estimated using Figure 4-3 or Equation 4-9. 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 lane width for this AMF is 12 ft.

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 4-9 is used, the proportion of crashes can be 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 severe crashes.

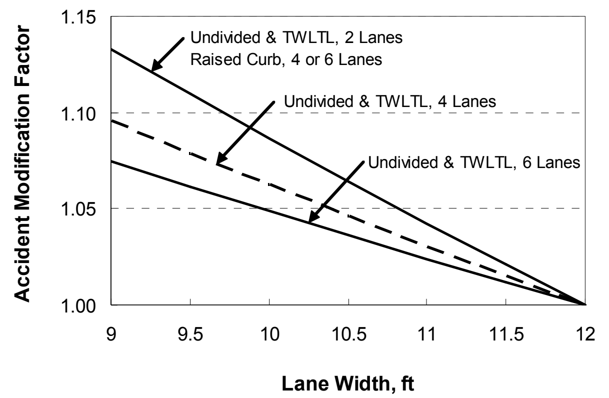


Figure 4-3. Lane Width AMF.

$$AMF_{lw} = (e^{-0.040\,(W_l - 12)} - 1.0)\frac{P_i}{0.24} + 1.0 \qquad \textbf{(4-9)}$$

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
Undivided or TWLTL [1]	2	0.25
	4	0.18
	6	0.14
Raised curb [2]	4	0.24
	6	0.27

Notes:
1 - Single-vehicle run-off-road, same-direction sideswipe, and multiple vehicle opposite direction crashes.
2 - Single-vehicle run-off-road and same-direction side-swipe crashes.

Shoulder Width - AMF_{sw}

Discussion

Shoulders offer numerous safety benefits for urban streets. Properly designed shoulders provide space for disabled vehicles, bicycle traffic, additional room for evasive maneuvers, and, if wide enough, a space within which right-turning vehicles can decelerate. In urban areas, the need to control access and drainage often justifies the use of curb-and-gutter in the cross section. However, the safety trade-offs of curb-and-gutter versus shoulder should be fully evaluated, especially for higher-speed facilities.

Safety Relationship

The relationship between shoulder width and severe crash frequency can be estimated using Figure 4-4 or Equation 4-10. 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 a 1.5 ft shoulder width.

Guidance

This AMF is applicable to outside 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 4-10 is used, the proportion of crashes can be 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 0.96. This value implies that severe crashes will be reduced by 4 percent if a 4 ft shoulder is included in the cross section.

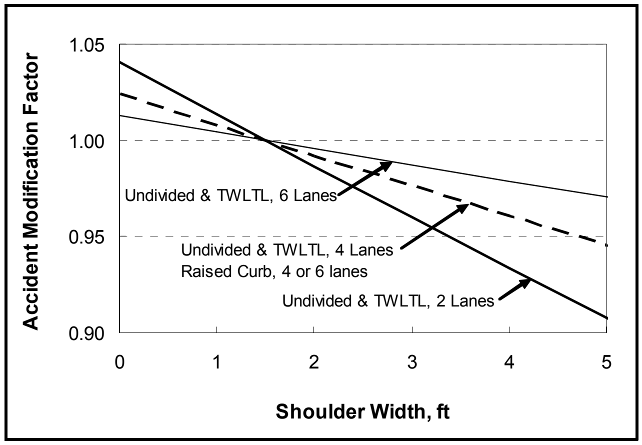


Figure 4-4. Shoulder Width AMF.

$$AMF_{sw} = (e^{-0.014\,(W_s - 1.5)} - 1.0)\,\frac{P_i}{0.088} + 1.0 \qquad \textbf{(4-10)}$$

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 TWLTL [1]	2	0.17
	4	0.10
	6	0.054
Raised curb [2]	4	0.088
	6	0.087

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

$$AMF_{sw} = (e^{-0.014 \times (4 - 1.5)} - 1.0)\,\frac{0.10}{0.088} + 1.0 \qquad \textbf{(4-11)}$$
$$= 0.96$$

Median Width - AMF_{mw}

Discussion

Medians provide several safety benefits including positive separation between opposing traffic streams, space for left-turn bays, refuge for pedestrians, and control of access. The degree of benefit is correlated with the width of the median such that wider medians are associated with fewer crashes. Raised medians can also promote the aesthetics of the street environment; however, aesthetic treatments installed in the median should be designed to allow good visibility and be "forgiving" if impacted by errant vehicles.

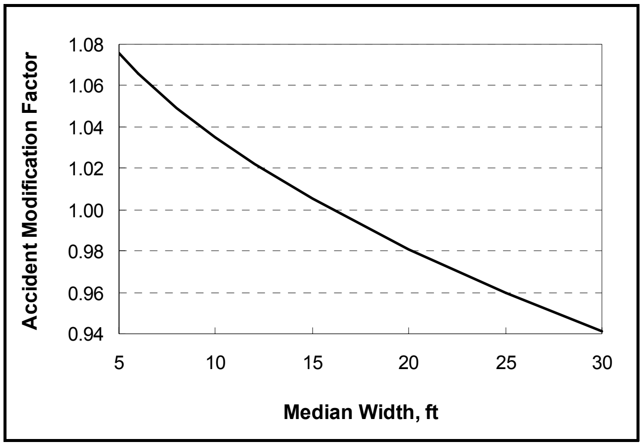


Figure 4-5. Median Width AMF.

Safety Relationship

The relationship between median width and severe crash frequency can be estimated using Figure 4-5 or Equation 4-12. 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 16 ft median width.

$$AMF_{mw} = e^{-0.041\,(W_m^{0.5} - 16^{0.5})} \qquad \textbf{(4-12)}$$

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

Base Condition: 16 ft median width

Guidance

This AMF is applicable to divided streets with median widths ranging from 4 to 30 ft, as measured between the near edges of the left and right-side traveled way. It is not applicable to streets with a TWLTL. Hence, this AMF should not be used with the TWLTL median type AMF.

Example Application

The Question: How much will severe crashes increase if a 24 ft median is reduced to 12 ft?

The Facts:
- Existing median width: 24 ft
- Proposed median width: 12 ft

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 increase in severe crashes.

$$\begin{aligned} \%\ Increase &= 100\left(\frac{1.02}{0.96} - 1\right) \\ &= 6.2\ \% \end{aligned} \qquad \textbf{(4-13)}$$

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. In spite of these safety and operational benefits, the TWLTL is generally recognized to be associated with higher crash risk than a raised-curb median when there are frequent busy driveways.

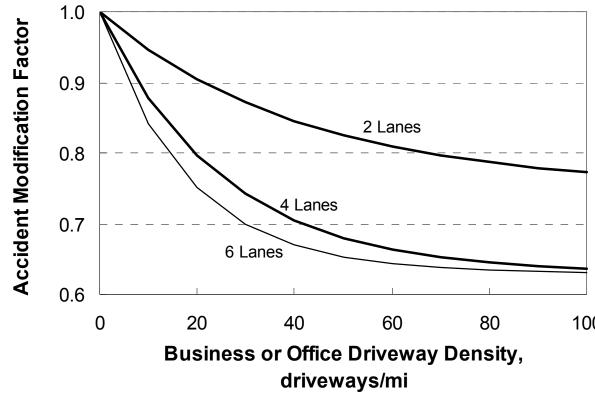


Figure 4-6. TWLTL Median Type AMF.

Safety Relationship

The relationship between TWLTL presence and severe crash frequency can be estimated using Figure 4-6 or Equation 4-14. 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. The base condition for this AMF is no TWLTL (i.e, undivided cross section).

$$AMF_T = (AMF_{target} - 1.0)\,P_{target} + 1.0 \qquad \textbf{(4-14)}$$

with,

$$P_{target} = 1 - e^{-0.008\,D_{d,b/o}\,(n_l + 1)} \qquad \textbf{(4-15)}$$

where:

AMF_T = TWLTL median type accident modification factor;
AMF_{target} = AMF for crash types directly influenced by the addition of a TWLTL (= 0.75 for two through lanes; 0.63 for four or six through lanes);
P_{target} = target crashes as a proportion of total crashes;
$D_{d,\,b/o}$ = density of driveways serving business or office land uses (two-way total), driveways/mi; and
n_l = number of through lanes.

Base Condition: no TWLTL

Guidance

This AMF is based on data for two- and four-lane streets. However, the trends are sufficiently stable that the AMF can be extended to six-lane streets with reasonable confidence. This AMF should not be used with the median width AMF.

Example Application

The Question: What percent reduction in crashes will result if a TWLTL is added to a street in a downtown area?

The Facts:

- Driveway density: 50 driveways/mi
- Through lanes: 4

The Solution: From Figure 4-6, find the AMF of 0.68. This value implies that crashes will be reduced by 32 percent by the addition of a TWLTL.

$$\begin{aligned} AMF_T &= (0.63 - 1.0)\,0.86 + 1.0 \\ &= 0.68 \end{aligned} \qquad \textbf{(4-16)}$$

with,

$$\begin{aligned} P_{target} &= 1 - e^{-0.008 \times 50\,(4 + 1)} \\ &= 0.86 \end{aligned} \qquad \textbf{(4-17)}$$

Curb Parking - AMF_{pk}

Discussion

The provision of parallel or angle curb parking in the cross section is an important consideration in the urban street design. Curb parking offers convenient, and sometimes essential, access to adjacent property. However, parking maneuvers increase the risk of crash, 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, commercial, and office land uses are typically found to be associated with more frequent parking than residential or industrial areas.

Safety Relationship

The relationship between curb parking and severe crash frequency can be estimated using Figure 4-7 or Equation 4-18. 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 percentage of street segment length with curb parking should be based on an assessment of both curb faces. For example, a 0.1 mi segment with parking for the full length of one curb face and for one-half of the other face has 0.15 mi (= 0.10 + 0.05) allocated to curb parking. The percentage of street length with curb parking is 75 percent (= 0.15/[0.1+0.1] × 100).

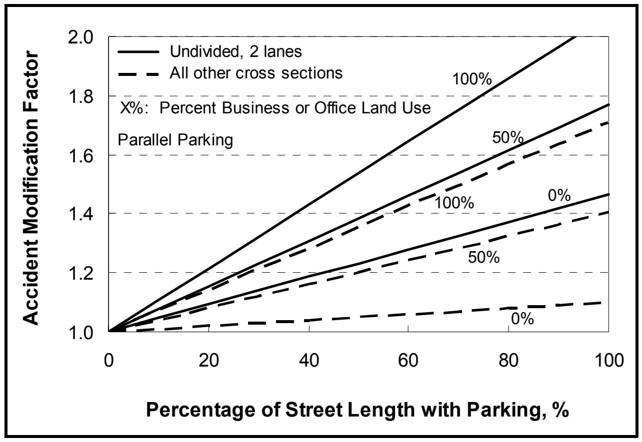
Example Application

The Question: What percent increase in severe crashes will result if a street is widened to add parallel parking?

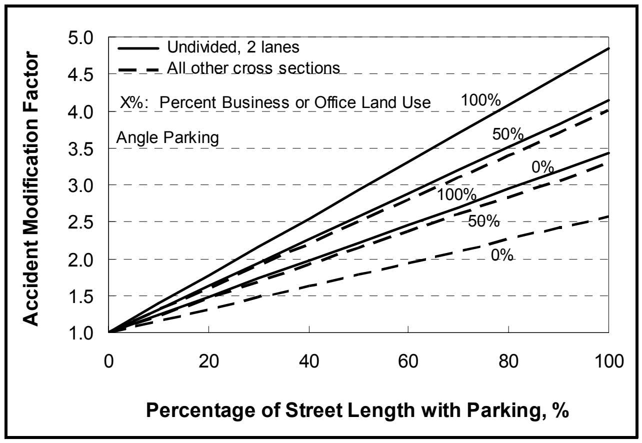
The Facts:

- Land use: 50% office, 50% residential
- Proposed street length with parking: 100%
- Cross section: undivided, 2 lanes

The Solution: From Figure 4-7a, find the AMF of 1.77, which equates to a 77 percent increase.



a. Parallel Parking.



b. Angle Parking.

Figure 4-7. Curb Parking AMF.

Base Condition: no parking

$$AMF_{pk} = 1 + P_{pk}\,(B_{pk} - 1) \qquad \textbf{(4-18)}$$

with,

$$B_{pk} = (1.10 + 0.365\,I_{nl} + 0.609\,P_{b/o})\,[1.34\,P_{ap} + 1.0] \qquad \textbf{(4-19)}$$

where:

AMF_{pk} = curb parking accident modification factor;
P_{pk} = proportion of street segment length with parallel or angle parking (= 0.5 L_{pk} / L);
L_{pk} = curb miles allocated to parking, mi;
I_{nl} = indicator variable for cross section (1 for two-lane street; 0 otherwise);
$P_{b/o}$ = for that part of the street with parking, the proportion that has business or office as an adjacent land use; and
P_{ap} = for that part of the street 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 street, 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 severe crash frequency can be estimated using Figure 4-8 or Equation 4-20. 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 pole offset of 2.0 ft and a pole density of 50 poles/mi.

Guidance

This AMF is applicable to pole densities ranging from 20 to 70 poles/mi and pole offsets ranging from 1 to 30 ft. A sensitivity analysis indicates traffic volume and pole density have a small effect on the AMF value. If Equation 4-20 is used, the proportion of crashes can be obtained from Table 4-7.

Example Application

The Question: What percent reduction in severe 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 4-8, find the AMF of 0.97 (rounded down from 0.974). This value suggests that severe crashes will be reduced about 3 percent if the utility pole offset is increased to 15 ft.

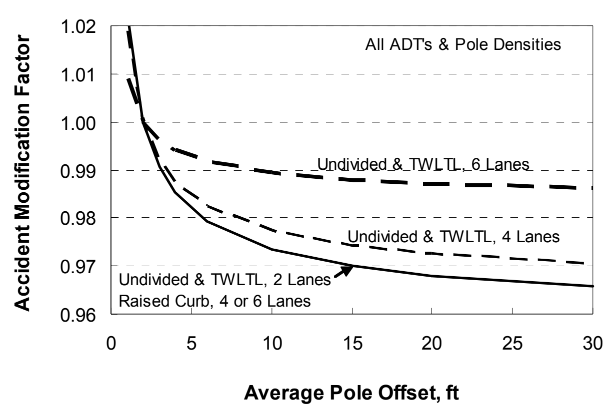


Figure 4-8. Utility Pole Offset AMF.

$$AMF_{pd} = (f_p - 1.0)\,P_i + 1.0 \qquad \textbf{(4-20)}$$

with,

$$f_p = \frac{(0.0000984\,ADT + 0.0354\,D_p)\,W_o^{-0.6} - 0.04}{0.0000649\,ADT + 1.128} \qquad \textbf{(4-21)}$$

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 & 50 poles/mi

Table 4-7. Crash Distribution for Utility Pole Offset AMF.

Median Type	Through Lanes	Proportion of Crashes [1]
Undivided or TWLTL	2	0.042
	4	0.036
	6	0.017
Raised curb	4	0.045
	6	0.046

Note:
1 - Single-vehicle-with-pole crashes.

Driveway Density - AMF_{dd}

Discussion

Uncontrolled access to an urban street creates numerous safety and operational problems. Proper design and spacing of access points can minimize these problems. In this regard, fewer but wider driveways is a desirable design goal. Access management is an effective technique for guiding the process of locating driveways such that conflicts associated with turning vehicles are minimized. One key element of access management is driveway density.

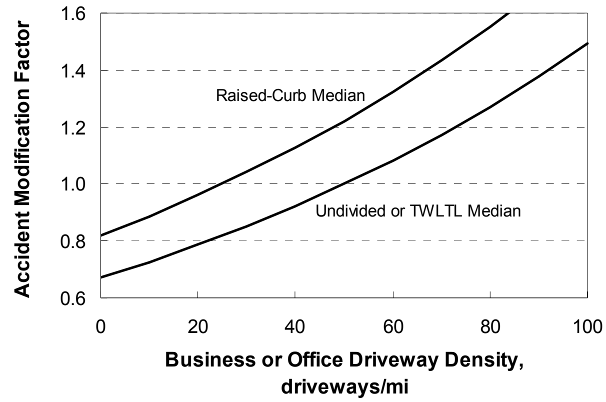


Figure 4-9. Driveway Density AMF.

Safety Relationship

The relationship between driveway density and severe crash frequency can be estimated using Figure 4-9 or Equation 4-22. 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. For undivided and TWLTL medians, the base condition for this AMF is 50 driveways/mi. For raised-curb medians, the base condition is 25 driveways/mi.

$$AMF_{dd} = e^{0.008\,(D_{d,b/o} - D_{base})} \qquad \textbf{(4-22)}$$

where:
AMF_{dd} = driveway density accident modification factor;
$D_{d,\,b/o}$ = density of driveways serving business or office land uses (two-way total); driveways/mi; and
D_{base} = base driveway density; driveways/mi.

Base Condition: 50 drives/mi for undivided and TWLTL, 25 drives/mi for raised curb

Guidance

This AMF requires a determination of the density of driveways serving business or office land uses. These driveways tend to have high volume, relative to residential driveways. Driveway density is the count of driveways on both sides of the segment divided by segment length.

Example Application

The Question: What percent increase in crashes is likely to occur if driveway density is allowed to increase from 20 to 40 driveways/mi?

The Facts:
- Median type: undivided

The Solution: From Figure 4-9, find the AMFs of 0.79 and 0.92 for driveway densities of 20 and 40 driveways/mi, respectively. The percent increase in crashes is 16 percent (= 100 × 0.92/0.79 − 100).

$$\begin{aligned} AMF_{dd} &= e^{0.008\,(40\,-\,50)} \\ &= 0.92 \end{aligned} \qquad \textbf{(4-23)}$$

Truck Presence - AMF_{tk}

Discussion

An analysis of truck crash data indicates that streets with higher truck percentages are associated with fewer crashes. It is likely that more trucks do not make the street safer; rather, this finding probably indicates that drivers are more cautious when there are many trucks in the traffic stream.

Safety Relationship

The relationship between truck presence and severe crash frequency can be estimated using Figure 4-10 or Equation 4-24. 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. The base condition is 6 percent trucks.

Guidance

This AMF is applicable to truck percentages ranging from 0.0 to 20 percent. If Equation 4-24 is used, the proportion of crashes that occur off the roadway can be obtained from Table 4-8.

Example Application

The Question: What is the severe crash frequency for a specific four-lane urban street?

The Facts:

- Base crash frequency C_b: 1.5 crashes/yr
- Truck percentage: 20%
- Through lanes: 4
- Median type: raised curb

The Solution: The segment of interest has typical characteristics with the exception that it has 20 percent trucks. As shown in Figure 4-10, the AMF for 20 percent trucks is 0.70. This value can be used with Equation 4-3 to estimate the expected severe crash frequency for the subject segment as 1.0 crashes/yr.

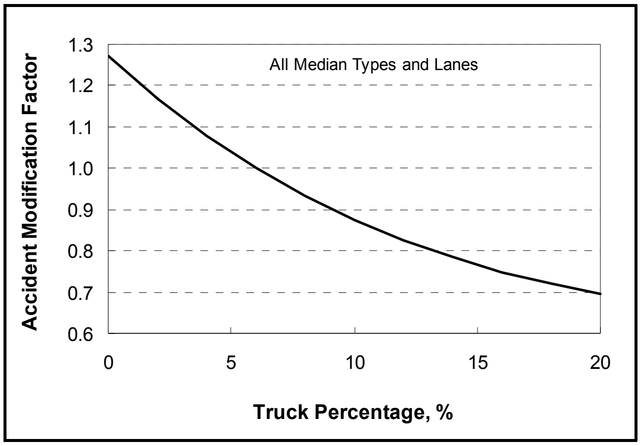


Figure 4-10. Truck Presence AMF.

$$AMF_{tk} = (f_{tk} - 1.0)(1.0 - P_{off\text{-}road}) + 1.0 \qquad \textbf{(4-24)}$$

with,

$$f_{tk} = \frac{2\,e^{-0.059\,P_t} + 0.017\,P_t}{1.506} \qquad \textbf{(4-25)}$$

where:

AMF_{tk} = truck presence accident modification factor;
P_t = percent trucks represented in ADT, %; and
$P_{off\text{-}road}$ = proportion of crashes that occur off the roadway (see Table 4-8).

Base Condition: 6% trucks

Table 4-8. Crash Distribution for Truck Presence AMF.

Median Type	Through Lanes	Proportion of Crashes [1]
Undivided or TWLTL	2	0.17
	4	0.10
	6	0.054
Raised curb	4	0.18
	6	0.17

Note:
1 - Single-vehicle run-off-road crashes.

$$C = C_b \times AMF_{tk} = 1.5 \times 0.70 = 1.0 \; crashes/yr \qquad \textbf{(4-26)}$$

Safety Appurtenances

AMFs for roadside safety appurtenances are not described in this document. A comprehensive procedure for evaluating appurtenances is outlined in a report by Mak and Sicking (*3*) and automated in the *Roadside Safety Analysis Program* (RSAP) (*4*). RSAP can be used to evaluate alternative roadside safety appurtenances on individual street segments. The program accepts as input information about the street 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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1. *Roadway Safety Design Synthesis*. Texas Transportation Institute, The Texas A&M University System, College Station, Texas, 2005.

2. 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.

3. 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.

4. 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 5

Interchange Ramps

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INTRODUCTION

Access to and from grade-separated facilities is achieved using 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 two arterial roadways. Ramps are configured in a variety of shapes to accommodate heavy turn movements and topography. They are associated with more significant crash risk because of the significant speed change that occurs along their length, often coupled with horizontal curves of near-minimum radius and relatively steep grade changes. These attributes complicate the ramp driving task.

The process of designing an interchange ramp can include an evaluation of the operational and safety benefits of various design alternatives, with consideration of overall cost-effectiveness. The level of detail to which these considerations are addressed increases as the available right-of-way is reduced or as access to adjacent properties is impacted.

The procedure described in this chapter can be used to quantify the safety associated with an existing ramp or with a proposed ramp design. In this regard, safety is defined as the expected frequency of severe (i.e., injury or fatal) crashes. The safety benefit of a proposed design can be obtained by comparing its expected crash frequency with that of the existing facility. Background information about the various equations and constants that comprise the procedure is provided in the *Roadway Safety Design Synthesis* (*1*). 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 procedures for evaluating the safety of interchange ramp segments and speed-change lanes. A ramp segment is defined as a length of roadway that supports travel in one direction and connects two grade-separated roadways. The ramp segment addressed herein does not include the intersection of the ramp and the surface street or frontage road. Also, the ramp segment does not include the speed-change lane at the junction of the ramp and the mainlanes. A speed-change lane is defined as an area adjacent to the mainlanes specifically intended for vehicle acceleration or deceleration.

Procedures for evaluating rural and urban ramp intersections are described in Chapters 6 and 7, respectively. These procedures can be used together with the procedures in this chapter to fully evaluate the safety of an interchange.

The procedures described herein are based on the prediction of expected crash frequency. Specifically, the crash frequency for a ramp segment or speed-change lane is computed from a base model. The base model includes a sensitivity to traffic volume, ramp type, and ramp configuration. Currently, no accident modification factors (AMFs) are available for use with this procedure. When AMFs become available for ramps or speed-change lanes, they can be included in the procedure.

The procedures described herein differ from those developed by Harwood et al. (*2*) in that they predicts *severe* crash frequency (as opposed to total crash frequency). Otherwise, the procedures described herein are similar and share the same strengths and weaknesses. The reader is referred to the report by Harwood et al. for a discussion of their procedure and its attributes.

Base crash prediction models for interchange ramps are described in the next section. The section that follows describes base models for speed-change lanes. Example applications are provided throughout this *Workbook* to illustrate the use of the base models.

Base Models–Interchange Ramps

Discussion

A wide range of ramp configurations are used 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.

An examination of the ramp configurations shown in Figure 5-1 indicated that crash rates vary with interchange setting (e.g,, within a frontage road system), type of ramp (i.e., entrance or exit), and the ramp configuration (*1*). In general, crash rates tend to be higher for the various 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 is probably due to the significant deceleration, combined with sharp horizontal curves, often found on exit ramps.

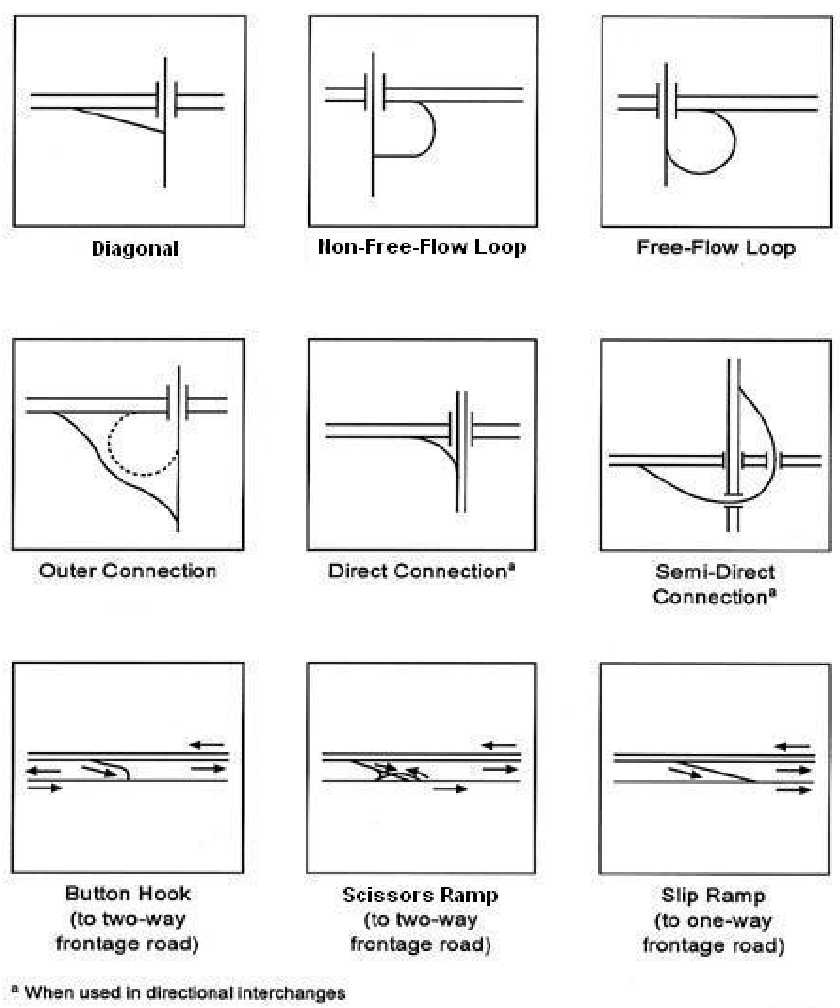

Figure 5-1. Basic Interchange Ramp Configurations.

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. Differences in crash rate among similar ramps in urban and rural areas were not found to be significant; hence, the rates listed in the table are applicable to both urban and rural interchange ramps.

The rates provided in Table 5-1 are in units of severe 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 has negligible effect on crash frequency.

Guidance

Equation 5-1 can be used to estimate the expected severe crash rate for any ramp that meets the criteria shown in Table 5-1. The crash estimate relates to crashes that occur on the ramp segment. It does not include crashes that occur at the ramp terminals (i.e., at the crossroad intersection or the speed-change lane). Crash rates for speed-change lanes are provided in a subsequent section.

A local calibration factor is identified in Equation 5-1. A default value for this factor is recommended as 1.0. The factor can be used to adjust the predicted base crash frequency such that it is more consistent with typical streets in the agency's jurisdiction. A procedure for calibrating Equation 5-1 to local conditions is described by Harwood et al. (*2*). If this procedure is used, only severe crashes should be included in the calculations.

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/mvm: severe crashes per million vehicles.

$$C_b = 0.000365\ Base\ ADT_{ramp}\ f \qquad \textbf{(5-1)}$$

where:
C_b = expected severe base crash frequency, crashes/yr;
$Base$ = severe crash rate (see Table 5-1), crashes/mvm;
ADT_{ramp} = average daily traffic volume on the ramp, veh/d; and
f = local calibration factor.

Example Application

The Question: What is the expected severe 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 severe crash frequency of 1.0 crashes/yr.

$$
\begin{aligned}
C_b &= 0.000365\ Base\ ADT_{ramp}\ f \\
&= 0.000365 \times 0.28 \times 10{,}000 \times 1.0 \qquad \textbf{(5-2)} \\
&= 1.0\ crashes/yr
\end{aligned}
$$

Example Application

The Question: What is the expected severe 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 severe crash frequency of 1.3 crashes/yr.

$$
\begin{aligned}
C_b &= 0.000365\ Base\ ADT_{ramp}\ f \\
&= 0.000365 \times 0.36 \times 10{,}000 \times 1.0 \qquad \textbf{(5-3)} \\
&= 1.3\ crashes/yr
\end{aligned}
$$

Base Models–Speed-Change Lanes

Discussion

An examination of crash trends indicated that crash rates for speed-change lanes are dependent on the area type (urban or rural), the type of speed-change lane (acceleration or deceleration), and, for acceleration lanes, the ratio of ramp volume to mainlane volume (*1*). Each of these factors was found to have a major influence on crash frequency. In general, crash rates are higher for acceleration lanes than deceleration lane, which is likely a result of accelerating vehicles causing more turbulence in the traffic stream as they merge with mainlane vehicles. Also, crash rates are higher for urban speed-change lanes than for rural speed-change lanes. This latter influence is likely a reflection of higher mainlane and ramp volumes.

Safety Relationship

Crash rates for speed-change lanes are provided in Table 5-2. They can be used with Equation 5-4 for acceleration lanes or Equation 5-5 for deceleration lanes to compute the expected severe crash frequency.

Guidance

The severe crash frequency obtained from Equations 5-4 or 5-5 is applicable to any speed-change lane that meets the criteria shown in Table 5-2.

The mainlane freeway volume used with Table 5-2, or Equation 5-4, is the one-way ADT for the mainlanes immediately adjacent to the speed-change lane. A procedure for calibrating Equations 5-4 or 5-5 to local conditions is described by Harwood et al. (*2*). If this procedure is used, only severe crashes should be included in the calculations.

Table 5-2. Base Crash Rates for Speed-Change Lanes.

Speed-Change Lane Type	Area Type	Ratio of Ramp Volume to Mainlane Volume	Base Crash Rate [1]
Acceleration	Urban	0.05	0.0095
		0.10	0.0145
		0.15	0.0183
		0.20	0.0214
		0.25	0.0239
		0.30	0.0260
	Rural	0.04	0.0033
		0.06	0.0043
		0.08	0.0051
		0.10	0.0058
		0.12	0.0064
		0.14	0.0070
Deceleration	Urban	Any	0.075
	Rural	Any	0.021

Notes:
1 - Crash rate unites for acceleration lanes: crashes per million mainlane plus ramp vehicles. Crash rate for deceleration lanes: crashes per million ramp vehicles.

For acceleration lanes:

$$C_b = 0.000365\ Base\ (ADT_{ramp} + ADT_{main})\ f \qquad \textbf{(5-4)}$$

For deceleration lanes:

$$C_b = 0.000365\ Base\ ADT_{ramp}\ f \qquad \textbf{(5-5)}$$

where:
C_b = expected severe crash frequency for speed-change lane, crashes/yr;
$Base$ = severe crash rate (see Table 5-2), crashes/mvm;
ADT_{main} = average one-way daily traffic volume on the freeway mainlanes adjacent to the speed-change lane, veh/d;
ADT_{ramp} = average daily traffic volume on the ramp, veh/d; and
f = local calibration factor.

Example Application

The Question: What is the expected severe crash frequency for an acceleration lane at an entrance ramp?

The Facts:
- Lane type: acceleration
- Area type: urban
- Ramp ADT: 10,000 veh/d
- Mainlane ADT: 100,000 veh/d

The Solution: The ratio of the ramp ADT to the mainlane ADT is 0.10 [= 10,000 /100,000]. From Table 5-2, find that the base crash rate for this ratio is 0.0145 crashes/million-mainlane-plus-ramp-vehicles. This rate yields an expected crash frequency of 0.58 severe crashes/yr.

$$\begin{aligned} C_b &= 0.000365\ Base\ (ADT_{ramp} + ADT_{main})\ f \\ &= 0.000365 \times 0.0145 \times (10{,}000 + 100{,}000) \times 1.0 \\ &= 0.58\ crashes/yr \end{aligned} \qquad \textbf{(5-6)}$$

Example Application

The Question: What is the expected severe crash frequency for a deceleration lane at an exit ramp?

The Facts:
- Lane type: deceleration
- Area type: urban
- Ramp ADT: 10,000 veh/d

The Solution: From Table 5-2, find that the base crash rate for a deceleration lane is 0.075 crashes/million-ramp-vehicles. This rate yields an expected crash frequency of 0.27 severe crashes/yr.

$$\begin{aligned} C_b &= 0.000365\ Base\ ADT_{ramp}\ f \\ &= 0.000365 \times 0.075 \times 10{,}000 \times 1.0 \\ &= 0.27\ crashes/yr \end{aligned} \qquad \textbf{(5-7)}$$

Accident Modification Factors

Discussion

This section is intended to describe AMFs that can be used to evaluate the relationship between a change in ramp or speed-change lane design and a corresponding change in severe crash frequency. At present, there are no documented AMFs for ramps or speed-change lanes. There are many factors that are likely to have some effect on severe crash frequency. However, their effect has yet to be quantified through research. AMFs for ramps and speed-change lanes are likely to be available as new research in this area is undertaken.

Table 5-3. AMFs for Ramps and Speed-Change Lanes.

Application	Accident Modification Factor
Geometric design	none
Roadside design	none

REFERENCES

1. *Roadway Safety Design Synthesis*. Texas Transportation Institute, The Texas A&M University System, College Station, Texas, 2005.

2. 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.

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 severe (i.e., injury or fatal) crashes. The safety benefit of a proposed design can be obtained by comparing its expected crash frequency with that of the existing facility. Background information about the various equations and constants that comprise the procedure is provided in the *Roadway Safety Design Synthesis* (*1*). 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.

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. (*2*) in that it predicts *severe* crash frequency (as opposed to total crash frequency). Otherwise, the procedure described herein is similar and shares the same strengths and weaknesses. The reader is referred to the report by Harwood et al. for a discussion of their procedure and its attributes.

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 (i.e., signalized or unsignalized), 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

Crash rates for various rural intersection types are provided in Tables 6-1 and 6-2. The rates in Table 6-1 are applicable to three-leg intersections; those in Table 6-2 are applicable to four-leg intersections. Within each table, the rates are categorized by intersection control mode, major-road volume, and the ratio of minor-road to major-road volume.

The crash rates in Tables 6-1 and 6-2 are in units of severe crashes per million entering vehicles. They can be used in Equation 6-1 to compute the expected severe crash frequency for a given intersection.

$$C_b = 0.000365\ Base\ (Q_{major} + Q_{minor})\ f \qquad \textbf{(6-1)}$$

where:

C_b = expected severe base crash frequency, crashes/yr;
Base = severe crash rate (see Tables 6-1 and 6-2), crashes/mvm;
Q_{major} = average daily traffic volume on the major road, veh/d;
Q_{minor} = average daily traffic volume on the minor road, veh/d; and
f = local calibration factor.

Guidance

Equation 6-1 is applicable to intersections with "typical" characteristics. These characteristics are identified herein as "base" conditions. The complete set of base conditions are identified in Table 6-3.

Specific major-road volumes and volume ratios were used to develop Tables 6-1 and 6-2. Interpolation may be used for volumes or ratios other than those listed.

Table 6-1. Base Crash Rates for Three-Leg Rural Intersections.

Control Mode	Major-Road Volume, veh/d	Base Crash Rate, severe crashes/mev [1]				
		Ratio of Minor-Road to Major-Road Volume				
		0.05	**0.10**	**0.15**	**0.20**	**0.25**
Unsignalized	5000	0.10	0.14	0.16	0.18	0.19
	10,000	0.13	0.18	0.21	0.23	0.25
	15,000	0.15	0.20	0.24	0.26	0.28
	20,000	0.17	0.23	Intersection very likely to meet signal warrants		
	≥25,000	0.18				
Signalized	5000	0.08	0.11	0.14	0.16	0.17
	10,000	0.10	0.15	0.18	0.20	0.22
	15,000	0.12	0.17	0.21	0.23	0.25
	20,000	0.13	0.19	0.23	0.26	0.28
	25,000	0.14	0.20	0.25	0.28	0.30
	30,000	0.15	0.22	0.26	0.30	0.33
	40,000	0.17	0.24	0.29	0.33	0.36
	≥50,000	0.18	0.26	0.32	0.36	0.39

Note:
1 - mev: million entering vehicles.

Table 6-2. Base Crash Rates for Four-Leg Rural Intersections.

Control Mode	Major-Road Volume, veh/d	Base Crash Rate, severe crashes/mev [1]				
		Ratio of Minor-Road to Major-Road Volume				
		0.10	**0.30**	**0.50**	**0.70**	**0.90**
Unsignalized	5000	0.18	0.26	0.30	0.31	0.32
	10,000	0.20	0.30	0.34	0.36	0.36
	15,000	0.22	0.33	0.37	0.39	0.40
	20,000	0.23	Intersection very likely to meet signal warrants			
	≥25,000	0.25				
Signalized	5000	0.15	0.24	0.28	0.30	0.31
	10,000	0.17	0.28	0.32	0.35	0.36
	15,000	0.18	0.30	0.35	0.38	0.39
	20,000	0.20	0.32	0.37	0.40	0.42
	25,000	0.20	0.33	0.39	0.42	0.44
	30,000	0.21	0.35	0.41	0.44	0.45
	40,000	0.23	0.37	0.43	0.46	0.48
	≥50,000	0.24	0.38	0.45	0.49	0.50

Note:
1 - mev: million entering vehicles.

If a particular intersection has characteristics that differ from the base conditions, 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 Equation 6-1. A default value for this factor is recommended as 1.0. The factor can be used to adjust the predicted base crash frequency such that it is more consistent with typical intersections in the agency's jurisdiction. A procedure for calibrating Equation 6-1 to local conditions is described by Harwood et al. (*2*). If this procedure is used, only severe crashes should be included in the calculations.

Example Application

The Question: What is the expected severe crash frequency for a typical rural signalized intersection?

The Facts:

- Control mode: signalized
- Intersection legs: 3
- Major-road volume: 10,000 veh/d
- Minor-road volume: 2000 veh/d

The Solution: The ratio of minor-road to major-road volume is 0.20 (= 2000/10,000). From Table 6-1, find the severe crash rate of 0.20 crashes/mev. Equation 6-1 is used to compute the expected severe crash frequency of 0.9 crashes/yr. The use of Equation 6-1 is illustrated in the box at the right.

Table 6-3. Base Conditions.

Characteristic	Base Condition
Left-turn lanes on major road	Signalized: present on both major-road approaches Unsignalized: not present on either major-road approach
Right-turn lanes on major road	not present on either major-road approach
Number of lanes on major road	2
Number of lanes on minor road	2
Shoulder width	8 ft
Median presence on major road	Signalized: not applicable Unsignalized: not present
Alignment skew angle	no skew
Sight distance restrictions	none
Driveway frequency	Signalized: 3 driveways Unsignalized: 0 driveways
Truck presence	9% trucks

$$\begin{aligned} C_b &= 0.000365\ Base\ (Q_{major} + Q_{minor})\ f \\ &= 0.000365 \times 0.20 \times (10{,}000 + 2000) \times 1.0 \qquad \textbf{(6-2)} \\ &= 0.9\ crashes/yr \end{aligned}$$

Accident Modification Factors–Signalized Intersections

Discussion

This section describes AMFs that can be used to evaluate the relationship between a signalized intersection design change and the corresponding change in severe crash frequency. AMFs for unsignalized intersections are presented in the next section. Topics addressed are listed in Table 6-4. The basis for each of these AMFs is described in Chapter 6 of the *Synthesis* (*1*). There are many additional factors, other than those listed in Table 6-4, that are likely to have some effect on severe crash frequency. However, their effect 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 are developed to yield a value of 1.0 when the associated design component or element represents "typical" signalized intersection conditions ("typical" characteristics are defined in the previous section). Deviation from base conditions to a more generous or desirable design condition results in an AMF of less than 1.0. Deviation to a more restricted condition results in an AMF of more than 1.0.

Table 6-4. AMFs for Signalized Intersections.

Application	Accident Modification Factor
Geometric design	Left-turn lane Right-turn lane Number of lanes Alignment skew angle
Access control	Driveway frequency
Other	Truck presence

Safety Relationship

The expected severe crash frequency for a specific signalized intersection is computed using Equation 6-3. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are not typical.

$$C = C_b \times AMF_{RT} \times AMF_{nd} \ldots \qquad \textbf{(6-3)}$$

where:
C = expected severe crash frequency, crashes/yr;
C_b = expected severe base crash frequency, crashes/yr;
AMF_{RT} = right-turn lane accident modification factor; and
AMF_{nd} = driveway frequency accident modification factor.

Guidance

In application, an AMF is identified for each intersection characteristic that is not typical. All AMFs identified in this manner are then multiplied together. This product is then multiplied by the base crash frequency C_b for intersections that are otherwise similar to the subject intersection. The base crash frequency can be obtained from Equation 6-1 or estimated from existing crash data. The product of this multiplication represents the expected severe crash frequency for the subject intersection.

Example Application

The Question: What is the expected severe crash frequency for a specific rural signalized intersection?

The Facts:
- Control mode: signalized
- Intersection legs: 3
- Major-road volume: 10,000 veh/d
- Minor-road volume: 2000 veh/d
- Base crash frequency C_b: 0.9 crashes/yr
- Driveway frequency: 4 driveways

The Solution: The intersection of interest has typical characteristics with the exception that its driveway frequency is above average. As described later, the AMF for this frequency is 1.05. This AMF can be used with Equation 6-3 to estimate the expected severe crash frequency for the subject intersection as 0.95 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{nd} \\ &= 0.9 \times 1.05 \\ &= 0.95 \; crashes/yr \end{aligned} \qquad (6\text{-}4)$$

Example Application

The Question: What is the expected severe crash frequency for a signalized intersection if the number of driveways is increased from 3 to 4?

The Facts:
- Control mode: signalized
- Intersection legs: 3
- Severe crash frequency (three-year average): 3.3 crashes/yr
- Existing driveway frequency: 3
- Proposed driveway frequency: 4

The Solution: A three-year crash history is available and considered to be a better estimate of the expected severe crash frequency than the rates in Table 6-1. The AMF for 4 driveways is 1.05. This AMF can be used with Equation 6-3 to estimate the expected severe crash frequency for the subject intersection with 4 driveways as 3.5 crashes/yr. This represents an increase of 0.2 crashes/yr (2 in 10 years) if the additional driveway is allowed.

$$\begin{aligned} C &= C_b \times AMF_{nd} \\ &= 3.3 \times 1.05 \\ &= 3.5 \; crashes/yr \end{aligned} \qquad (6\text{-}5)$$

Left-Turn Lane - AMF_{LT}

Discussion

A 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 lanes. The lack of a lane, or a bay of inadequate length, can cause significant conflict between left-turning and through vehicles, which can lead to increased crash risk as well as poor traffic operations.

Safety Relationship

The relationship between left-turn lane presence and severe crash frequency can be estimated from Table 6-5. 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 left-turn lane of adequate length on both major-road approaches. The AMF equals 1.0 for intersections that satisfy the base condition. The values in Table 6-5 are appropriate when a turn lane is not provided on the major road or when it is provided but is not of adequate length.

Guidance

This AMF is applicable to the major-road approaches at a signalized intersection. At three-leg intersections, the minor road is the discontinuous route. 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 expected percentage increase in severe crashes if a left-turn bay is provided on only one major-road approach?

The Facts:
- Intersection legs: 4

The Solution: From Table 6-5, find the AMF of 1.21. This value suggests that 21 percent more severe crashes are likely to occur because the second approach does not have a turn bay.

Table 6-5. AMF for Excluding a Left-Turn Lane at a Signalized Intersection.

Number of Intersection Legs	Number of Major-Road Approaches without Left-Turn Lanes	
	One Approach	Both Approaches
3	1.16	not applicable [1]
4	1.21	1.45

Note:
1 - Only one major-road left-turn lane is likely at a three-leg intersection.

Base Condition: a left-turn lane (or bay) on both major-road approaches

Right-Turn Lane - AMF_{RT}

Discussion

A 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 cause significant conflict between right-turning and through vehicles, which can lead to increased crash risk as well as poor traffic operations.

Table 6-6. AMF for Adding a Right-Turn Lane at a Signalized Intersection.

Number of Intersection Legs	Number of Major-Road Approaches with Right-Turn Lanes	
	One Approach	Both Approaches
3	0.91	not applicable [1]
4	0.91	0.83

Note:
1 - Only one major-road right-turn lane is likely at a three-leg intersection.

Safety Relationship

The relationship between right-turn lane presence and severe 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 no right-turn lane on either major-road approach. The AMF equals 1.0 for intersections that satisfy the base condition. The values in Table 6-6 are appropriate when a turn lane of adequate length is provided on the major road.

Base Condition: no right-turn lane (or bay) on either major-road approach

Guidance

This AMF is applicable to the major-road approaches at a signalized intersection. At three-leg intersections, the minor road is the discontinuous route. 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 severe crashes will likely be prevented by the addition of right-turn lanes on both major-road approaches?

The Facts:

- Intersection legs: 4
- Base crash frequency C_b: 6.0 crashes/yr

The Solution: From Table 6-6, find the AMF of 0.83. Expected crash frequency after the lanes are installed is 5.0 crashes/yr. Thus, the lanes are likely to prevent 1.0 crash/yr.

$$\begin{aligned} Reduction &= C_b - C_b \times AMF_{RT} \\ &= 6.0 - 6.0 \times 0.83 \\ &= 6.0 - 5.0 \\ &= 1.0 \ crashes/yr \end{aligned} \qquad (6\text{-}6)$$

Number of Lanes - AMF_{lane}

Discussion

Research indicates that the number of lanes at a signalized intersection is correlated with the frequency of severe 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 severe crash frequency can be estimated from Table 6-7. 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

Two AMFs are obtained from Table 6-7–one for the major road and one for the minor road. Both AMFs are multiplied by the base crash frequency to estimate the expected crash frequency. The number of lanes provided at the intersection is often dictated by the cross section of the intersecting roads and by capacity considerations. The AMFs in Table 6-7 should be used primarily to obtain an accurate estimate of the expected crash frequency for a given cross section. These AMFs are not intended to be used to justify changes in cross section.

Example Application

The Question: What is the combined 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 6-7, find the major- and minor-road AMFs of 1.01 and 1.01, respectively. The combined AMF is 1.02 (= 1.01 × 1.01).

Table 6-7. AMF for Number of Through Lanes at a Signalized Intersection.

Road	Number of Through Lanes	AMF_{lane}
Major	3 or fewer	1.00
	4 or 5	1.01
	6 or more	1.03
Minor	3 or fewer	1.00
	4 or more	1.01

Base Condition: 2 lanes on major road, 2 lanes on minor road

Alignment Skew Angle - AMF_{skew}

Discussion

The skew angle of an intersection may have some correlation with signalized intersection safety. Logically, the safety of some turning maneuvers could be adversely affected by large skew angles. However, research has not quantified the relationship between skew angle and crash frequency at signalized intersections. Analysis of crash data indicate that the effect of skew angle is relatively small. In recognition of this finding, it is suggested that the reduction of skew angle (provided this angle is in the range of 0 to 30 degrees) is not a key consideration during the design process.

Safety Relationship

The relationship between skew angle and severe crash frequency at rural signalized intersections is unknown but believed to be negligible for typical skew angles. Thus, a value of 1.0 is recommended for the alignment skew angle AMF.

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.

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 (especially those owners in the vicinity of an intersection). However, traffic movements associated with these driveways add turbulence to the traffic stream and increase the likelihood of collision. Efforts to combine driveways or relocate them away from the intersection tend to result in fewer crashes.

Safety Relationship

The relationship between driveway frequency and crash frequency can be estimated from Figure 6-1 or Equation 6-7. 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 three driveways within 250 ft of the intersection.

Figure 6-1. Driveway Frequency AMF for a Signalized Intersection.

$$AMF_{nd} = e^{0.046\,(d_n - 3)} \qquad \textbf{(6-7)}$$

where:
AMF_{nd} = driveway frequency accident modification factor; and
d_n = number of driveways on the major road within 250 ft of the intersection.

Base Condition: 3 driveways on major road within 250 ft of intersection

Guidance

This AMF applies to driveways on the major-road approaches to the intersection. Driveways on both major-road approaches within 250 ft of the intersection should be counted. If known, 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 severe crashes be expected to decrease if the number of driveways is reduced from 6 to 3?

The Facts:
- Existing number of driveways: 6
- Proposed number of driveways: 3

The Solution: From Figure 6-1, find the AMFs of 1.15 for six driveways and 1.00 for three driveways. These results indicate a 13 percent reduction in severe crashes due to the change.

$$\begin{aligned}\%\ Decrease &= 100\left(1 - \frac{1.00}{1.15}\right) \\ &= 13\ \%\end{aligned} \qquad \textbf{(6-8)}$$

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. Analysis of crash data indicate that intersection crashes increase when more trucks are present.

Safety Relationship

The relationship between truck percentage and severe crash frequency can be estimated from Figure 6-2 or 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. The base condition is 9 percent 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.

Example Application

The Question: If the truck percentage on the major road at a rural signalized intersection is 15 percent, what is the AMF?

The Facts:
- Truck percentage: 15 percent

The Solution: From Figure 6-2, find the AMF of 1.18. This value suggests that severe crashes will be 18 percent higher at this intersection than one just like it but with no trucks.

Figure 6-2. Truck Presence AMF for a Signalized Intersection.

$$AMF_{tk} = e^{0.028\,(P_t - 9)} \quad \textbf{(6-9)}$$

where:
AMF_{tk} = truck presence accident modification factor; and
P_t = percent trucks during the peak hour (average for all intersection movements), %.

Base Condition: 9% trucks

$$\begin{aligned} AMF_{tk} &= e^{0.028\,(15 - 9)} \\ &= 1.18 \end{aligned} \quad \textbf{(6-10)}$$

Accident Modification Factors–Unsignalized Intersections

Discussion

This section describes AMFs that can be used to evaluate the relationship between an unsignalized intersection design change and the corresponding change in severe crash frequency. The AMFs only apply to two-way stop-controlled intersections. Research is needed to develop AMFs for all-way stop control. AMFs for signalized intersections are presented in the previous section.

Topics addressed in this section are listed in Table 6-8. The basis for each of these AMFs is described in Chapter 6 of the *Synthesis* (*1*). There are many additional factors, other than those listed in Table 6-8, that are likely to have some effect on severe crash frequency. However, their effect 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 are developed to yield a value of 1.0 when the associated design component or element represents "typical" unsignalized intersection conditions ("typical" characteristics are defined in the section titled "Base Models"). Deviation from base conditions to a more generous or desirable design condition results in an AMF of less than 1.0. Deviation to a more restricted condition results in an AMF of more than 1.0.

Table 6-8. 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 Intersection sight distance
Access control	Driveway frequency
Other	Truck presence

Note:
1 - Factors listed only apply to two-way stop-controlled intersections.

Safety Relationship

The expected severe crash frequency for a specific unsignalized intersection is computed using Equation 6-3, repeated here as Equation 6-11. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are not typical.

$$C = C_b \times AMF_{RT} \times AMF_{nd} \cdots \qquad \textbf{(6-11)}$$

where:
C = expected severe crash frequency, crashes/yr;
C_b = expected severe base crash frequency, crashes/yr;
AMF_{RT} = right-turn lane accident modification factor; and
AMF_{nd} = driveway frequency accident modification factor.

Guidance

In application, an AMF is identified for each intersection characteristic that is not typical. All AMFs identified in this manner are then multiplied together. This product is then multiplied by the base crash frequency C_b for intersections that are otherwise similar to the subject intersection. The base crash frequency can be obtained from Equation 6-1 or estimated from existing crash data. The product of this multiplication represents the expected severe crash frequency for the subject intersection.

Example Application

The Question: What is the expected severe crash frequency for a specific rural unsignalized intersection?

The Facts:
- Control mode: unsignalized
- Intersection legs: 4
- Major-road volume: 10,000 veh/d
- Minor-road volume: 3000 veh/d
- Base crash frequency C_b: 1.4 crashes/yr
- Average shoulder width: 4 ft

The Solution: The intersection of interest has typical characteristics with the exception that its average shoulder width is 4 ft. The base crash rate is obtained from Table 6-2 as 0.30 crashes/mev. Using this rate, the base crash frequency is computed as 1.4 crashes/yr (= 0.000365 × 0.30 × [10,000 + 3000]).

As described later, the AMF for a shoulder width of 4 ft is 1.13. This AMF can be used with Equation 6-11 to estimate the expected severe crash frequency for the subject intersection as 1.6 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{sw} \\ &= 1.4 \times 1.13 \\ &= 1.6 \ crashes/yr \end{aligned} \qquad \textbf{(6-12)}$$

Left-Turn Lane - AMF_{LT}

Discussion

A 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 lanes. The lack of a lane, or a bay of inadequate length, can cause significant conflict between left-turning and through vehicles, which can lead to increased crash risk as well as poor traffic operations.

Safety Relationship

The relationship between left-turn lane presence and severe crash frequency can be estimated from Table 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. The base condition for this AMF is no left-turn lane on either major-road approach. The AMF equals 1.0 for intersections that satisfy the base condition. The values in Table 6-9 are appropriate when a turn lane of adequate length is provided on the major road.

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. 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 expected percentage reduction in severe crashes if a left-turn bay is provided at a "T" intersection?

The Facts:

- Intersection legs: 3
- Existing left-turn bays on major-road: 0

The Solution: From Table 6-9, find the AMF of 0.45. This value suggests crashes are likely to be reduced 55 percent with the addition of a left-turn bay on the major-road approach.

Table 6-9. AMF for Adding a Left-Turn Lane at an Unsignalized Intersection.

Number of Intersection Legs	Number of Major-Road Approaches with Left-Turn Lanes	
	One Approach	Both Approaches
3	0.45	not applicable [1]
4	0.65	0.42

Note:
1 - Only one major-road left-turn lane is likely at a three-leg intersection.

Base Condition: no left-turn lane (or bay) on either major-road approach

Right-Turn Lane - AMF_{RT}

Discussion

A 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 cause significant conflict between right-turning and through vehicles, which can lead to increased crash risk as well as poor traffic operations.

Safety Relationship

The relationship between right-turn lane presence and severe 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 no right-turn lane on either major-road approach. The AMF equals 1.0 for intersections that satisfy the base condition. The values in Table 6-10 are appropriate when a turn lane of adequate length is provided on the major road.

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. 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 increase in crash frequency if a right-turn bay is removed?

The Facts:

- Intersection legs: 3
- Existing right-turn bays on major road: 1
- Base crash frequency C_b: 3.5 crashes/yr

The Solution: From Table 6-10, find the AMF of 0.77. This value suggests that 4.5 crashes/yr (= 3.5/0.77) will occur after the bay is removed. The net increase is 1.0 crashes/yr.

Table 6-10. AMF for Adding a Right-Turn Lane at an Unsignalized Intersection.

Number of Intersection Legs	Number of Major-Road Approaches with Right-Turn Lanes	
	One Approach	Both Approaches
3	0.77	not applicable [1]
4	0.77	0.59

Note:
1 - Only one major-road right-turn lane is likely at a three-leg intersection.

Base Condition: no right-turn lane (or bay) on either major-road approach

$$\begin{aligned} Increase &= \frac{C_b}{AMF_{RT}} - C_b \\ &= \frac{3.5}{0.77} - 3.5 \\ &= 4.5 - 3.5 \\ &= 1.0 \ crashes/yr \end{aligned} \qquad \textbf{(6-13)}$$

Number of Lanes - AMF_{lane}

Discussion

Research indicates that the number of lanes at an unsignalized intersection is correlated with the frequency of severe 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 crossing and left-turning movements. The result may be fewer crashes due to the redistribution of traffic patterns as intersection size increases.

Safety Relationship

The relationship between the number of through lanes and severe crash frequency can be estimated from Table 6-11. 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 base condition for this AMF is two lanes on the major road and two lanes on the minor road.

Guidance

Two AMFs are obtained from Table 6-11–one for the major road and one for the minor road. The number of lanes provided at the intersection is often dictated by the cross section of the intersecting roads and by capacity considerations. The AMFs in Table 6-11 should be used primarily to obtain an accurate estimate of the expected crash frequency for a given cross section. These AMFs are not intended to be used to justify changes in cross section.

Example Application

The Question: What is the combined 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 6-11, find the major- and minor-road AMFs of 0.83 and 0.83, respectively. The combined AMF is 0.69 (= 0.83 × 0.83)

Table 6-11. AMF for Number of Through Lanes at an Unsignalized Intersection.

Road	Number of Through Lanes	AMF_{lane}
Major	3 or fewer	1.00
	4 or 5	0.83
	6 or more	0.69
Minor	3 or fewer	1.00
	4 or more	0.83

Base Condition: 2 lanes on major road, 2 lanes on minor road

Shoulder Width - AMF_{sw}

Discussion

Shoulders offer numerous safety benefits for rural highways. Properly designed shoulders provide space for disabled vehicles, additional room for evasive maneuvers, and, if wide enough, a space within which right-turning vehicles can decelerate. Right-of-way 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.

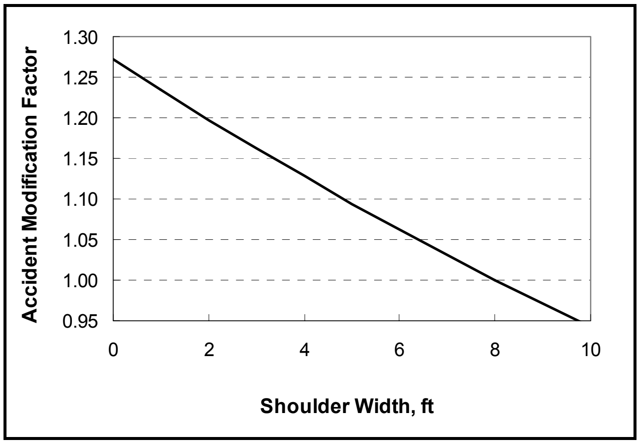

Figure 6-3. Shoulder Width AMF for an Unsignalized Intersection.

Safety Relationship

The relationship between shoulder width and severe crash frequency can be estimated from Figure 6-3 or Equation 6-14. 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. The base condition for this AMF is an 8 ft shoulder.

$$AMF_{sw} = e^{-0.030\,(W_s\,-\,8)} \qquad \textbf{(6-14)}$$

where:
AMF_{sw} = shoulder width accident modification factor; and
W_s = shoulder width, ft.

Base Condition: 8 ft shoulder width

Guidance

The shoulder width used to estimate the AMF is the average width of the outside shoulder on the major-road approaches to the intersection. This AMF is applicable to shoulder widths ranging from 0 to 10 ft.

Example Application

The Question: What is the expected increase in intersection crashes if the major-road shoulder width is reduced?

The Facts:
- Existing shoulder width: 6 ft
- Proposed shoulder width: 2 ft

The Solution: From Figure 6-3, find the AMFs of 1.06 and 1.20 for widths of 6 and 2 ft, respectively. The ratio of these AMFs suggests a 13 percent increase in severe crashes would result from the change.

$$\begin{aligned} AMF_{sw} &= e^{-0.030\,(2\,-\,8)} \\ &= 1.20 \end{aligned} \qquad \textbf{(6-15)}$$

$$\begin{aligned} \%\ Increase &= \left(\frac{1.20}{1.06} - 1\right) \times 100 \\ &= 13\,\% \end{aligned} \qquad \textbf{(6-16)}$$

Median Presence - AMF_{mp}

Discussion

Medians provide several safety benefits including positive separation between opposing traffic streams, a sheltered location for left-turning vehicles, and control of access in the vicinity of the intersection. Research shows that intersections with a median are associated with fewer crashes than intersections without a median. The safety benefit of the median tends to increase with increasing median width.

Safety Relationship

The relationship between median presence and severe crash frequency can be estimated from Equation 6-17, in combination with Figure 6-4 and Table 6-12. 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. This AMF can be used with the left-turn lane AMF.

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:

- Major road median: depressed, with bay

The Solution: From Table 6-12, find $AMF_{mp,base}$ of 1.00. From Figure 6-4, find AMF_{mw} of 0.95. So AMF_{mp} is 0.95 (=1.0 × 0.95). From Table 6-9, find AMF_{LT} of 0.42. The net AMF is 0.40 (= 0.95 ×0.42) for adding both left-turn bays and a median in the vicinity of the intersection.

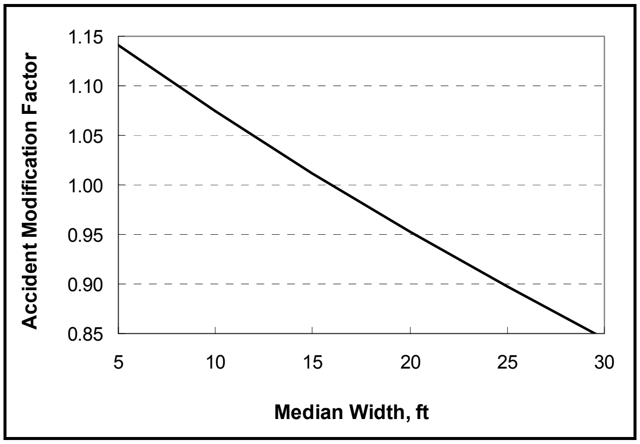

Figure 6-4. Median Width AMF for an Unsignalized Intersection.

Table 6-12. AMF for Base Median Presence at an Unsignalized Intersection.

Median Type on Major Road	$AMF_{mp, base}$
Undivided (may have bay at intersection)	1.00
Two-way left-turn lane, or any median that has a left-turn bay	1.00
Flush paved, or depressed, without left-turn bay [1]	0.73

Note:
1 - Median should be at least 4 ft wide and extend back from the intersection for at least 250 ft. If the median is less than 4 ft wide or extends back less than 250 ft, the AMF value is 1.0.

$$AMF_{mp} = AMF_{mp,base} \times AMF_{mw} \qquad \textbf{(6-17)}$$

with,

$$AMF_{mw} = \begin{cases} e^{-0.012\,(W_m - 16)} & : \textit{if median (or bay) present} \\ 1.0 & : \textit{if undivided} \end{cases} \qquad \textbf{(6-18)}$$

where:
AMF_{mp} = median presence accident modification factor;
$AMF_{mp,\,base}$ = base median presence accident modification factor (see Table 6-12);
AMF_{mw} = median width 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

The skew angle of an intersection can have an adverse effect on unsignalized intersection safety. 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. An analysis of crash data indicates that crashes at unsignalized intersections increase with increasing skew angle.

Safety Relationship

The relationship between alignment skew angle and severe crash frequency can be estimated from Figure 6-5, Equation 6-19, or 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. The base condition skew angle is 0 degrees.

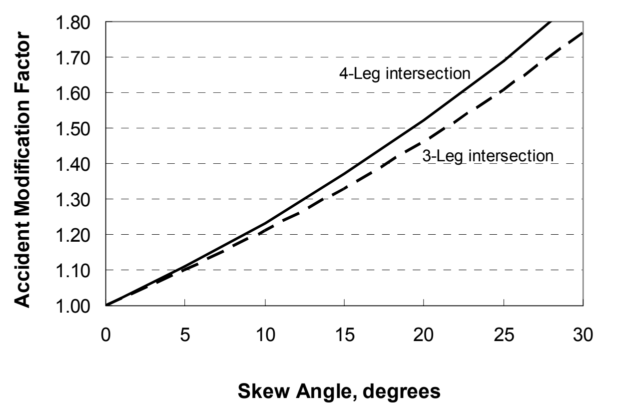

Figure 6-5. Alignment Skew Angle AMF for an Unsignalized Intersection.

For three-leg intersections:

$$AMF_{skew} = e^{0.019\, I_{sk}} \tag{6-19}$$

For four-leg intersections:

$$AMF_{skew} = e^{0.021\, I_{sk}} \tag{6-20}$$

where:

AMF_{skew} = skew angle accident modification factor; and

I_{sk} = skew angle of the intersection (= | intersection angle − 90 |), degrees.

Base Condition: no skew (i.e., 90 degree intersection)

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.

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 6-5, find the AMF of 1.46. This value suggests that the skewed intersection will be associated with 46 percent more severe crashes.

$$\begin{aligned} AMF_{skew} &= e^{0.019 \times 20} \\ &= 1.46 \end{aligned} \tag{6-21}$$

Intersection Sight Distance - AMF_{SD}

Discussion

Research has found that crashes tend to be more frequent when intersection sight distance is limited. Intersection sight distance relates to the length of roadway available to drivers attempting to judge gaps in conflicting traffic movements at the intersection (it is not the minimum sight distance needed to stop a vehicle when presented with an unexpected hazard in the road ahead). Intersection sight distance is typically longer than stopping sight distance.

Safety Relationship

The relationship between sight distance restriction and severe crash frequency can be estimated from Table 6-13. 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 sight distance restriction.

Guidance

A sight distance restriction is defined to occur when the available intersection sight distance is less than the "policy" distance. This distance is that specified by AASHTO policy for a design speed that is 10 mph less than the major-road design speed. This AMF is intended for use when the sight restriction is due to terrain or alignment, not vegetation or man-made objects such as buildings and signs. However, this AMF may be extended with caution to any situation with restricted sight distance.

Example Application

The Question: What is the AMF for an intersection with sight distance restrictions?

The Facts:
- Quadrants with limited sight distance: 3

The Solution: From Table 6-13, find the AMF of 1.15. This value suggests that the sight restrictions are associated with a 15 percent increase in severe crashes.

Table 6-13. AMF for Intersection Sight Distance at an Unsignalized Intersection.

Quadrants with Sight Restriction [1]	AMF_{SD}
0	1.00
1	1.05
2	1.10
3	1.15
4	1.20

Note:
1 - Intersection quadrants with sight distance restriction.

Base Condition: no quadrants with sight distance restriction

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 (especially those owners in the vicinity of an intersection). However, traffic movements associated with these driveways add turbulence to the traffic stream and increase the likelihood of collision. Efforts to combine driveways or relocate them away from the intersection tend to result in fewer crashes.

Safety Relationship

The relationship between driveway frequency and crash frequency can be estimated from Figure 6-6 or Equation 6-22. 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 driveways within 250 ft of the intersection.

Guidance

This AMF applies to driveways on the major-road approaches to the intersection. Driveways on both major-road approaches within 250 ft of the intersection should be counted. If known, 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?

The Facts:
- Number of driveways: 6

The Solution: From Figure 6-6, find the AMF of 1.40 for six driveways. This value implies that the presence of six driveways will likely increase severe crash frequency by 40 percent, compared to an intersection with no driveways.

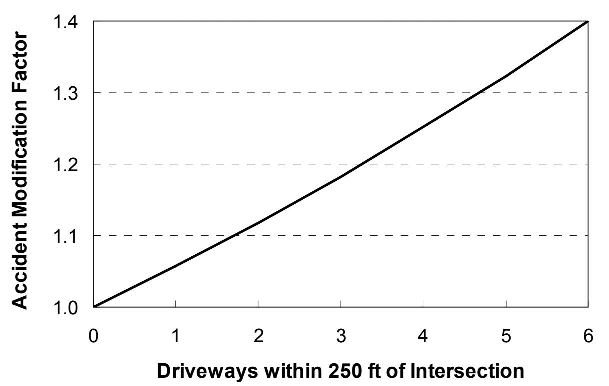

Figure 6-6. Driveway Frequency AMF for an Unsignalized Intersection.

$$AMF_{nd} = e^{0.056\, d_n} \qquad \textbf{(6-22)}$$

where:
AMF_{nd} = driveway frequency accident modification factor; and
d_n = number of driveways on the major road within 250 ft of intersection.

Base Condition: 0 driveways on major road within 250 ft of intersection

$$AMF_{nd} = e^{0.056 \times 6} = 1.40 \qquad \textbf{(6-23)}$$

Truck Presence - AMF_{tk}

Discussion

Analysis of crash data indicate that unsignalized intersections with a higher truck percentage are associated with fewer crashes. It is likely that more trucks do not make the intersection safer; rather, this finding suggests that drivers are more cautious when there are many trucks in the traffic stream.

Safety Relationship

The relationship between truck percentage and severe crash frequency can be estimated from Figure 6-7 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. The base condition is 9 percent 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.

Example Application

The Question: If the truck percentage on the major road at a rural unsignalized intersection is 15 percent, what is the AMF?

The Facts:

- Truck percentage: 15 percent

The Solution: From Figure 6-7, find the AMF of 0.84. This value suggests that severe crashes will be 16 percent lower at this intersection than one just like it but with no trucks.

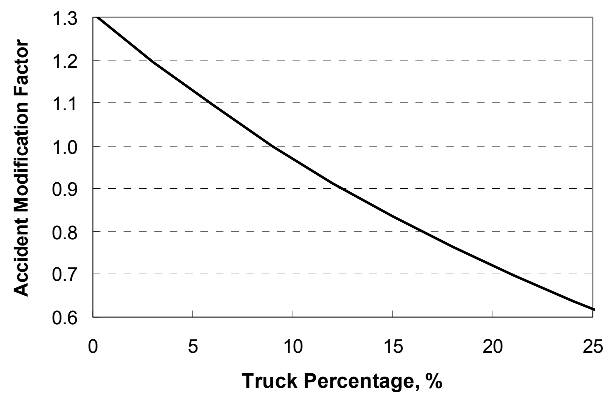

Figure 6-7. Truck Presence AMF for an Unsignalized Intersection.

$$AMF_{tk} = e^{-0.030\,(P_t - 9)} \qquad \textbf{(6-24)}$$

where:
AMF_{tk} = truck presence accident modification factor; and
P_t = percent trucks during the peak hour (average for all intersection movements), %.

Base Condition: 9% trucks

$$AMF_{tk} = e^{-0.030\,(15 - 9)} = 0.84 \qquad \textbf{(6-25)}$$

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2. 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.

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 alternative 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 severe (i.e., injury or fatal) crashes. The safety benefit of a proposed design can be obtained by comparing its expected crash frequency with that of the existing facility. Background information about the various equations and constants that comprise the procedure is provided in the *Roadway Safety Design Synthesis* (*1*). 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.

A procedure for evaluating urban street segments is presented 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, and the number of intersection legs. AMFs are used to account for other factors found to have some correlation with crash frequency. 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. (*2*) with the exception that it predicts *severe* crash frequency (as opposed to total crash frequency). The reader is referred to the report by Harwood et al. for a discussion of their procedure.

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.

The relationships described in this chapter address the occurrence of vehicle-related crashes at urban intersections. Relationships that focus on vehicle-pedestrian and vehicle-bicycle crashes at intersections will be added in future updates to this chapter.

Base Models

Discussion

An examination of crash trends indicates that crash rates for urban intersections are dependent on traffic volume, traffic control mode (i.e., signalized or unsignalized), 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

Crash rates for various urban intersection types are provided in Tables 7-1 and 7-2. The rates in Table 7-1 are applicable to three-leg intersections; those in Table 7-2 are applicable to four-leg intersections. Within each table, the rates are categorized by intersection control mode, major-street volume, and the ratio of minor-street to major-street volume.

The crash rates in Tables 7-1 and 7-2 are in units of severe crashes per million entering vehicles. They can be used in Equation 7-1 to compute the expected severe crash frequency for a given intersection.

$$C_b = 0.000365\ Base\ (Q_{major} + Q_{minor})\ f \qquad \textbf{(7-1)}$$

where:
C_b = expected severe base crash frequency, crashes/yr;
$Base$ = severe crash rate (see Tables 7-1 and 7-2), crashes/mvm;
Q_{major} = average daily traffic volume on the major street, veh/d;
Q_{minor} = average daily traffic volume on the minor street, veh/d; and
f = local calibration factor.

Guidance

Equation 7-1 is applicable to intersections with “typical” characteristics. These characteristics are identified herein as “base” conditions. The complete set of base conditions are identified in Table 7-3.

Specific major-street volumes and volume ratios were used to develop Tables 7-1 and 7-2. Interpolation may be used for volumes or ratios other than those listed.

Table 7-1. Base Crash Rates for Three-Leg Urban Intersections.

Control Mode	Base Crash Rate, severe crashes/mev [1]				
	Ratio of Minor-Street to Major-Street Volume				
	0.05	**0.10**	**0.15**	**0.20**	**0.25**
Unsignalized	0.18	0.21	0.22	0.22	0.23
Signalized	0.12	0.15	0.17	0.18	0.19

Note:
1 - mev: million entering vehicles.

Table 7-2. Base Crash Rates for Four-Leg Urban Intersections.

Control Mode	Major-Street Volume, veh/d	Base Crash Rate, severe crashes/mev [1]				
		Ratio of Minor-Street to Major-Street Volume				
		0.10	**0.30**	**0.50**	**0.70**	**0.90**
Unsignalized	5000	0.25	0.29	0.28	0.27	0.26
	10,000	0.23	0.26	0.26	0.25	0.24
	15,000	0.22	0.24	0.24	0.23	0.22
	20,000	0.21	Intersection very likely to meet signal warrants			
	≥25,000	0.20				
Signalized	5000	0.19	0.24	0.26	0.26	0.26
	10,000	0.17	0.22	0.23	0.23	0.23
	15,000	0.16	0.21	0.22	0.22	0.22
	20,000	0.15	0.20	0.21	0.21	0.21
	25,000	0.15	0.19	0.20	0.21	0.20
	30,000	0.14	0.19	0.20	0.20	0.20
	40,000	0.14	0.18	0.19	0.19	0.19
	≥50,000	0.13	0.17	0.18	0.19	0.18

Note:
1 - mev: million entering vehicles.

If a particular intersection has characteristics that differ from the base conditions, 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 Equation 7-1. A default value for this factor is recommended as 1.0. The factor can be used to adjust the predicted base crash frequency such that it is more consistent with typical intersections in the agency's jurisdiction. A procedure for calibrating Equation 7-1 to local conditions is described by Harwood et al. (*2*). If this procedure is used, only severe crashes should be included in the calculations.

Table 7-3. Base Conditions.

Characteristic	Base Condition
Left-turn lanes on major street	present on both major-street approaches
Right-turn lanes on major street	not present on either major-street approach
Number of lanes on major street	4
Number of lanes on minor street	2
Lane width	12 ft
Shoulder width [1]	1.5 ft
Median presence on major street	Signalized: not applicable Unsignalized: not present

Note:
1 - "Curb-and-gutter" section is assumed as typical with an equivalent shoulder width of 1.5 ft.

Example Application

The Question: What is the expected severe crash frequency for a typical urban signalized intersection?

The Facts:
- Control mode: signalized
- Intersection legs: 3
- Major-street volume: 20,000 veh/d
- Minor-street volume: 4000 veh/d

The Solution: The ratio of minor-street to major-street volume is 0.20 (= 4000/20,000). From Table 7-1, find the severe crash rate of 0.18 crashes/mev. Equation 7-1 is used to compute the expected severe crash frequency of 1.6 crashes/yr. The use of Equation 7-1 is illustrated in the box at the right.

$$\begin{aligned} C_b &= 0.000365\ Base\ (Q_{major} + Q_{minor})\ f \\ &= 0.000365 \times 0.18 \times (20{,}000 + 4000) \times 1.0 \\ &= 1.6\ crashes/yr \end{aligned} \qquad \textbf{(7-2)}$$

Accident Modification Factors–Signalized Intersections

Discussion

This section describes AMFs that can be used to evaluate the relationship between a signalized intersection design change and the corresponding change in severe crash frequency. AMFs for unsignalized intersections are presented in the next section. Topics addressed are listed in Table 7-4. The basis for each of these AMFs is described in Chapter 7 of the *Synthesis* (*1*). There are many additional factors, other than those listed in Table 7-4, that are likely to have some effect on severe crash frequency. However, their effect 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 are developed to yield a value of 1.0 when the associated design component or element represents "typical" signalized intersection conditions ("typical" characteristics are defined in the previous section). Deviation from base conditions to a more generous or desirable design condition results in an AMF of less than 1.0. Deviation to a more restricted condition results in an AMF of more than 1.0.

Table 7-4. AMFs for Signalized Intersections.

Application	Accident Modification Factor
Geometric design	Left-turn lane Right-turn lane Number of lanes Lane width

Safety Relationship

The expected severe crash frequency for a specific signalized intersection is computed using Equation 7-3. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are not typical.

$$C = C_b \times AMF_{lw} \times AMF_{RT} \ldots \tag{7-3}$$

where:
C = expected severe crash frequency, crashes/yr;
C_b = expected severe base crash frequency, crashes/yr;
AMF_{lw} = lane width accident modification factor; and
AMF_{RT} = right-turn lane accident modification factor.

Guidance

In application, an AMF is identified for each intersection characteristic that is not typical. All AMFs identified in this manner are then multiplied together. This product is then multiplied by the base crash frequency C_b for intersections that are otherwise similar to the subject intersection. The base crash frequency can be obtained from Equation 7-1 or estimated from existing crash data. The product of this multiplication represents the expected severe crash frequency for the subject intersection.

Example Application

The Question: What is the expected severe crash frequency for a specific urban signalized intersection?

The Facts:
- Control mode: signalized
- Intersection legs: 3
- Major-street volume: 20,000 veh/d
- Minor-street volume: 4000 veh/d
- Base crash frequency C_b: 1.6 crashes/yr
- Average lane width: 10 ft

The Solution: The intersection 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 1.11. This AMF can be used with Equation 7-3 to estimate the expected severe crash frequency for the subject intersection as 1.8 crashes/yr.

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 1.6 \times 1.11 \\ &= 1.8 \ crashes/yr \end{aligned} \quad \textbf{(7-4)}$$

Example Application

The Question: What is the expected severe crash frequency for a signalized intersection if the lane width is reduced from 12 to 10 ft?

The Facts:
- Control mode: signalized
- Intersection legs: 3
- Severe crash frequency (three-year average): 8.0 crashes/year
- Existing average lane width: 12 ft
- Proposed lane width: 10 ft

The Solution: A three-year crash history is available and considered to be a better estimate of the expected severe crash frequency than the rates in Table 7-1. The AMF for a lane width of 10 ft is 1.11. This AMF can be used with Equation 7-3 to estimate the expected severe crash frequency for the subject intersection with 10 ft lanes as 8.9 crashes/yr. This represents an increase of 0.9 crashes/yr (9 in 10 years) if the lane width is reduced.

$$\begin{aligned} C &= C_b \times AMF_{lw} \\ &= 8.0 \times 1.11 \\ &= 8.9 \ crashes/yr \end{aligned} \quad \textbf{(7-5)}$$

Left-Turn Lane - AMF_{LT}

Discussion

A left-turn lane (or bay) at an intersection provides a length of roadway within which left-turning vehicles can store without disrupting the smooth flow of traffic in the adjacent through lanes. The lack of a lane, or a bay of inadequate length, can cause significant conflict between left-turning and through vehicles, which can lead to increased crash risk as well as poor traffic operations.

Safety Relationship

The relationship between left-turn lane presence and severe crash frequency can be estimated from Table 7-5. 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 left-turn lane of adequate length on both major-street approaches. The AMF equals 1.0 for intersections that satisfy the base condition. The values in Table 7-5 are appropriate when a turn lane is not provided on the major street or when it is provided but is not of adequate length.

Guidance

This AMF is applicable to the major-street approaches at a signalized intersection. At three-leg intersections, the minor street is the discontinuous route. A turn 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 severe crashes if a left-turn bay is removed at a "T" intersection?

The Facts:

- Intersection legs: 3

The Solution: From Table 7-5, find the AMF of 1.06. This value suggests that 6 percent more severe crashes will occur if the left-turn bay is removed from the major-street approach.

Table 7-5. AMF for Excluding a Left-Turn Lane at a Signalized Intersection.

Number of Intersection Legs	Number of Major-Street Approaches without Left-Turn Lanes	
	One Approach	Both Approaches
3	1.06	not applicable [1]
4	1.10	1.21

Note:
1 - Only one major-street left-turn lane is likely at a three-leg intersection.

Base Condition: a left-turn lane (or bay) on both major-street approaches

Right-Turn Lane - AMF_{RT}

Discussion

A 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 cause significant conflict between right-turning and through vehicles, which can lead to increased crash risk as well as poor traffic operations.

Safety Relationship

The relationship between right-turn lane presence and severe crash frequency can be estimated from Table 7-6. 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 no right-turn lane on either major-street approach. The AMF equals 1.0 for intersections that satisfy the base condition. The values in Table 7-6 are appropriate when a turn lane of adequate length is provided on the major street.

Guidance

This AMF is applicable to the major-street approaches at a signalized intersection. At three-leg intersections, the minor street is the discontinuous route. 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 severe crashes will likely be prevented by the addition of a right-turn lane on one major-street approach?

The Facts:
- Intersection legs: 4
- Base crash frequency C_b: 6.7 crashes/yr

The Solution: From Table 7-6, find the AMF of 0.91. Expected crash frequency after the one lane is installed is 6.0 crashes/yr. Thus, the bay is likely to prevent about 0.7 crashes/yr.

Table 7-6. AMF for Adding a Right-Turn Lane at a Signalized Intersection.

Number of Intersection Legs	Number of Major-Street Approaches with Right-Turn Lanes	
	One Approach	Both Approaches
3	0.91	not applicable [1]
4	0.91	0.83

Note:
1 - Only one major-street right-turn lane is likely at a three-leg intersection.

Base Condition: no right-turn lane (or bay) on either major-street approach

$$\begin{aligned} Reduction &= C_b - C_b \times AMF_{RT} \\ &= 6.7 - 6.7 \times 0.91 \\ &= 6.7 - 6.0 \\ &= 0.7\ crashes/yr \end{aligned} \qquad \textbf{(7-6)}$$

Number of Lanes - AMF_{lane}

Discussion

Research indicates that the number of lanes at a signalized intersection is correlated with the frequency of severe 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.

Table 7-7. AMF for Number of Through Lanes at a Signalized Intersection.

Street	Number of Through Lanes	AMF_{lane}
Major	3 or fewer	0.91
	4 or 5	1.00
	6 or more	1.01
Minor	3 or fewer	1.00
	4 or more	1.01

Safety Relationship

The relationship between the number of through lanes and severe crash frequency can be estimated from Table 7-7. 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.

Base Condition: 4 lanes on major street, 2 lanes on minor street

Guidance

Two AMFs are obtained from Table 7-7–one for the major street and one for the minor street. Both AMFs are multiplied by the base crash frequency to estimate the expected crash frequency. The number of lanes provided at the intersection is often dictated by the cross section of the intersecting streets and by capacity considerations. The AMFs in Table 7-7 should be used primarily to obtain an accurate estimate of the expected crash frequency for a given cross section. These AMFs are not intended to be used to justify changes in cross section.

Example Application

The Question: What are the AMFs for the intersection of two, four-lane streets?

The Facts:

- Major-street through lanes: 4
- Minor-street through lanes: 4

The Solution: From Table 7-7, find the AMFs of 1.00 for the major street and 1.01 for the minor street.

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. These crashes are particularly frequent when the narrow lanes are accompanied by other design features of minimum dimension.

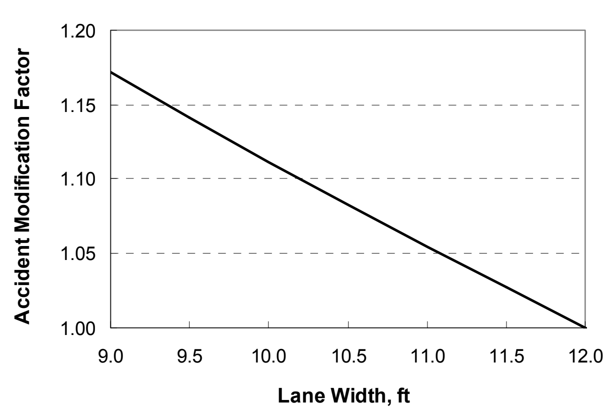

Figure 7-1. Lane Width AMF for a Signalized Intersection.

Safety Relationship

The relationship between lane width and severe crash frequency can be estimated from Figure 7-1 or Equation 7-7. 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 width for this AMF is 12 ft.

$$AMF_{lw} = e^{-0.053\,(W_l\,-\,12)} \qquad \textbf{(7-7)}$$

where:
AMF_{lw} = lane width accident modification factor; and
W_l = lane width, ft.

Base Condition: 12 ft lane width

Guidance

The lane width used to estimate the AMF is the average width of all major-street through lanes. The width of turn lanes or through lanes on the minor-street 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

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 Figure 7-1, find the AMF of 1.11.

$$AMF_{lw} = e^{-0.053\,(10\,-\,12)} = 1.11 \qquad \textbf{(7-8)}$$

Accident Modification Factors–Unsignalized Intersections

Discussion

This section describes AMFs that can be used to evaluate the relationship between an unsignalized intersection design change and the corresponding change in severe crash frequency. The AMFs only apply to two-way stop-controlled intersections. Research is needed to develop AMFs for all-way stop control. AMFs for signalized intersections are presented in a previous section.

Topics addressed in this section are listed in Table 7-8. The basis for each of these AMFs is described in Chapter 7 of the *Synthesis* (*1*). There are many additional factors, other than those listed in Table 7-8, that are likely to have some effect on severe crash frequency. However, their effect 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 are developed to yield a value of 1.0 when the associated design component or element represents "typical" unsignalized intersection conditions ("typical" characteristics are defined in the section titled "Base Models"). Deviation from base conditions to a more generous or desirable design condition results in an AMF of less than 1.0. Deviation to a more restricted condition results in an AMF of more than 1.0.

Safety Relationship

The expected severe crash frequency for a specific unsignalized intersection is computed using Equation 7-3, repeated here as Equation 7-9. The expected crash frequency represents the product of the base crash frequency and the various AMFs needed to account for characteristics that are not typical.

Table 7-8. AMFs for Unsignalized Intersections.

Application	Accident Modification Factor[1]
Geometric design	Left-turn lane Right-turn lane Number of lanes Lane width Shoulder width Median presence

Note:
1 - Factors listed only apply to two-way stop-controlled intersections.

$$C = C_b \times AMF_{lw} \times AMF_{RT} \ldots \qquad \textbf{(7-9)}$$

where:
C = expected severe crash frequency, crashes/yr;
C_b = expected severe base crash frequency, crashes/yr;
AMF_{lw} = lane width accident modification factor; and
AMF_{RT} = right-turn lane accident modification factor.

Guidance

In application, an AMF is identified for each intersection characteristic that is not typical. All AMFs identified in this manner are then multiplied together. This product is then multiplied by the base crash frequency C_b for intersections that are otherwise similar to the subject intersection. The base crash frequency can be obtained from Equation 7-1 or estimated from existing crash data. The product of this multiplication represents the expected severe crash frequency for the subject intersection.

Example Application

The Question: What is the expected severe crash frequency for an unsignalized intersection if the lane width is reduced from 12 to 10 ft?

The Facts:
- Control mode: unsignalized
- Intersection legs: 3
- Severe crash frequency (three-year average): 3.0 crashes/year
- Existing average lane width: 12 ft
- Proposed lane width: 10 ft

The Solution: A three-year crash history is available and considered to be a better estimate of the expected severe crash frequency than the rates in Table 7-1. The AMF for a lane width of 10 ft is 1.12. This AMF can be used with Equation 7-9 to estimate the expected severe crash frequency for the subject segment with 10 ft lanes as 3.4 crashes/yr. This represents an increase of 0.4 crashes/yr (4 in 10 years) if the lane width is reduced.

$$
\begin{aligned}
C &= C_b \times AMF_{lw} \\
&= 3.0 \times 1.12 \\
&= 3.4 \ \textit{crashes/yr}
\end{aligned}
\qquad \textbf{(7-10)}
$$

Left-Turn Lane - AMF_{LT}

Discussion

A left-turn lane (or bay) at an intersection provides a length of roadway within which left-turning vehicles can store without disrupting the smooth flow of traffic in the adjacent through lanes. The lack of a lane, or a bay of inadequate length, can cause significant conflict between left-turning and through vehicles, which can lead to increased crash risk as well as poor traffic operations.

Safety Relationship

The relationship between left-turn lane presence and severe crash frequency can be estimated from Table 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. The base condition for this AMF is a left-turn lane of adequate length on both major-street approaches. The AMF equals 1.0 for intersections that satisfy the base condition. The values in Table 7-9 are appropriate when a turn lane is not provided on the major street or when it is provided but is not of adequate length.

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. 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 severe crashes if the left-turn bays on the major street are removed?

The Facts:
- Intersection legs: 4

The Solution: From Table 7-9, find the AMF of 1.98. This value suggests that 98 percent more severe crashes will likely occur at this intersection if the left-turn bays are removed.

Table 7-9. AMF for Excluding a Left-Turn Lane at an Unsignalized Intersection.

Number of Intersection Legs	Number of Major-Street Approaches without Left-Turn Lanes	
	One Approach	Both Approaches
3	1.53	not applicable [1]
4	1.41	1.98

Note:
1 - Only one major-street left-turn lane is likely at a three-leg intersection.

Base Condition: a left-turn lane (or bay) on both major-street approaches

Right-Turn Lane - AMF_{RT}

Discussion

A 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 cause significant conflict between right-turning and through vehicles, which can lead to increased crash risk as well as poor traffic operations.

Safety Relationship

The relationship between right-turn lane presence and severe crash frequency can be estimated from Table 7-10. 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 no right-turn lane on either major-street approach. The AMF equals 1.0 for intersections that satisfy the base condition. The values in Table 7-10 are appropriate when a turn lane of adequate length is provided on the major street.

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. 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
- Base crash frequency C_b: 2.3 crashes/yr

The Solution: From Table 7-10, find the AMF of 0.59. This value suggests that 1.4 crashes/yr will occur after installation of the right-turn bay (i.e., a reduction of 0.9 crashes/yr).

Table 7-10. AMF for Adding a Right-Turn Lane at an Unsignalized Intersection.

Number of Intersection Legs	Number of Major-Street Approaches with Right-Turn Lanes	
	One Approach	**Both Approaches**
3	0.77	not applicable [1]
4	0.77	0.59

Note:
1 - Only one major-street right-turn lane is likely at a three-leg intersection.

Base Condition: no right-turn lane (or bay) on either major-street approach

$$\begin{aligned} Reduction &= C - C \times AMF_{RT} \\ &= 2.3 - 2.3 \times 0.59 \\ &= 2.3 - 1.4 \\ &= 0.9 \ crashes/yr \end{aligned} \qquad \textbf{(7-11)}$$

Number of Lanes - AMF_{lane}

Discussion

Research indicates that the number of lanes at an unsignalized intersection is correlated with the frequency of severe 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 crossing and left-turning movements. The result may be fewer crashes due to the redistribution of traffic patterns as intersection size increases.

Safety Relationship

The relationship between the number of through lanes and severe crash frequency can be estimated from Table 7-11. 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 base condition for this AMF is four lanes on the major street and two lanes on the minor street.

Guidance

Two AMFs are obtained from Table 7-11–one for the major street and one for the minor street. The number of lanes provided at the intersection is often dictated by the cross section of the intersecting streets and by capacity considerations. The AMFs in Table 7-11 should be used primarily to obtain an accurate estimate of the expected crash frequency for a given cross section. These AMFs are not intended to be used to justify changes in cross section.

Example Application

The Question: What are the AMFs for the intersection of two, four-lane streets?

The Facts:
- Major-street through lanes: 4
- Minor-street through lanes: 4

The Solution: From Table 7-11, find the AMF for the major street as 1.00; that for the minor street is 0.83.

Table 7-11. AMF for Number of Through Lanes at an Unsignalized Intersection.

Street	Number of Through Lanes	AMF_{lane}
Major	3 or fewer	1.20
	4 or 5	1.00
	6 or more	0.83
Minor	3 or fewer	1.00
	4 or more	0.83

Base Condition: 4 lanes on major street, 2 lanes on minor street

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. These crashes are particularly frequent when the narrow lanes are accompanied by other design features of minimum dimension.

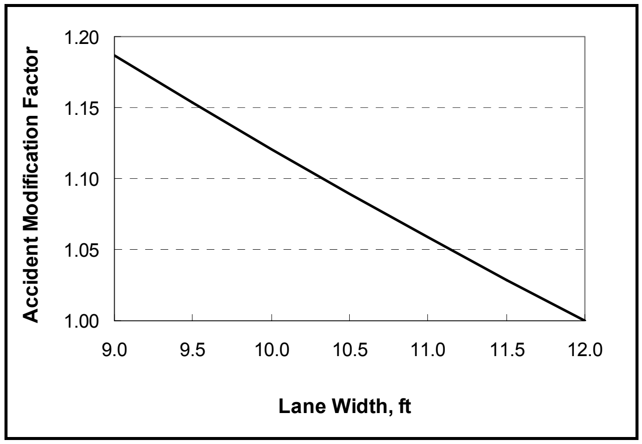

Figure 7-2. Lane Width AMF for an Unsignalized Intersection.

Safety Relationship

The relationship between lane width and severe crash frequency can be estimated from Figure 7-2 or Equation 7-12. 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 width for this AMF is 12 ft.

$$AMF_{lw} = e^{-0.057\,(W_l\, -\, 12)} \qquad \textbf{(7-12)}$$

where:
AMF_{lw} = lane width accident modification factor; and
W_l = lane width, ft.

Base Condition: 12 ft lane width

Guidance

The lane width used to estimate the AMF is the average width of all major-street through lanes. The width of turn lanes or through lanes on the minor-street 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:
- Intersection legs: 3
- Major-street lane widths (left to right, in feet): 10, 14 bay, 10

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 Figure 7-2, find the AMF of 1.12.

$$AMF_{lw} = e^{-0.057\,(10\, -\, 12)} = 1.12 \qquad \textbf{(7-13)}$$

Shoulder Width - AMF_{sw}

Discussion

Shoulders offer several safety benefits for urban intersections. Properly designed shoulders provide space for disabled vehicles, bicycle traffic, additional room for evasive maneuvers, and, if wide enough, a space within which right-turning vehicles can decelerate. In urban areas, the need to control access and drainage often justifies the use of curb-and-gutter in the cross section. However, the safety trade-offs of curb-and-gutter versus shoulder should be fully evaluated, especially for higher-speed facilities.

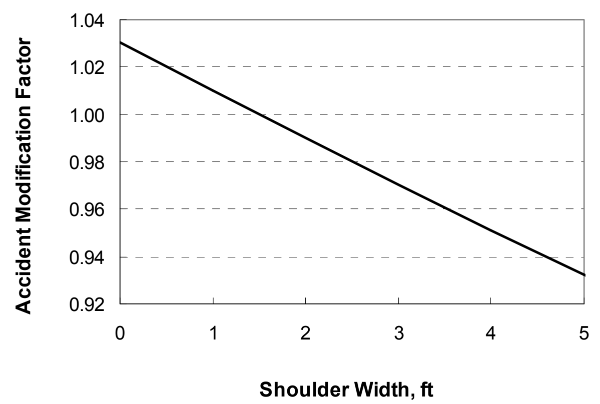

Figure 7-3. Shoulder Width AMF for an Unsignalized Intersection.

Safety Relationship

The relationship between shoulder width and severe crash frequency can be estimated from Figure 7-3 or 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. The base condition for this AMF is a 1.5 ft effective shoulder width, as obtained from a curb-and-gutter cross section.

$$AMF_{sw} = e^{-0.020\,(W_s\,-\,1.5)} \qquad \textbf{(7-14)}$$

where:
AMF_{sw} = shoulder width accident modification factor; and
W_s = shoulder width, ft.

Base Condition: 1.5 ft effective shoulder width (i.e., curb-and-gutter cross section)

Guidance

The shoulder width used to estimate the AMF is the average width of the outside shoulder on the major-street approaches to the intersection. This AMF is applicable to shoulder widths ranging from 0 to 5 ft.

Example Application

The Question: What is the expected reduction in intersection crashes if the major-street shoulder width is increased?

The Facts:
- Existing shoulder width: 1.5 ft
- Proposed shoulder width: 4 ft

The Solution: From Figure 7-3, find the AMFs of 1.00 and 0.95 for widths of 1.5 and 4 ft. The ratio of these AMFs suggests a 5.0 percent reduction in severe crashes would result from the change.

$$\begin{aligned} AMF_{sw} &= e^{-0.020\,(4\,-\,1.5)} \\ &= 0.95 \end{aligned} \qquad \textbf{(7-15)}$$

$$\begin{aligned} \%\ Reduction &= \left(1 - \frac{0.95}{1.00}\right) \times 100 \\ &= 5.0\ \% \end{aligned} \qquad \textbf{(7-16)}$$

Median Presence - AMF_{mp}

Discussion

Medians provide several safety benefits including positive separation between opposing traffic streams, space for left-turn bays, refuge for pedestrians, and control of access in the vicinity of the intersection. However, in urban and suburban areas, the median roadway associated with wide medians tends to be improperly used by drivers and consistently demonstrates an increase in intersection crashes with increasing median width.

Safety Relationship

The relationship between median presence and severe crash frequency can be estimated from Equation 7-17, in combination with Figure 7-4 and Table 7-12. 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. This AMF can be used with the left-turn lane AMF.

Example Application

The Question: What percent reduction in crashes should occur after installing a 20 ft median in the vicinity of a 3-leg intersection?

The Facts:

- Major street median: raised curb, w/bay
- Left-turn bay: present prior to median

The Solution: From Table 7-12, find the $AMF_{mp,base}$ of 1.00. From Figure 7-4, find AMF_{mw} of 1.03. The resulting AMF_{mp} is 1.03 (= 1.00 × 1.03). It represents a 3 percent increase in intersection crashes.

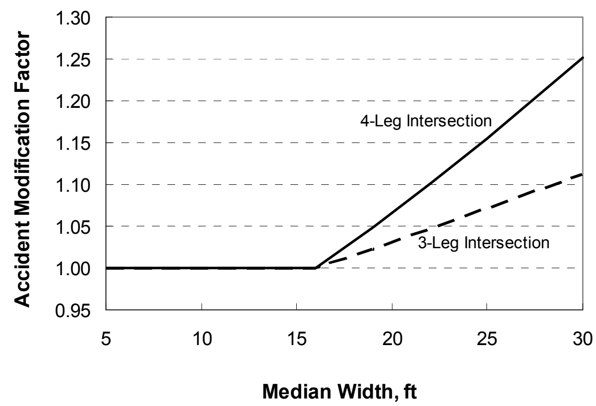

Figure 7-4. Median Width AMF for an Unsignalized Intersection.

Table 7-12. AMF for Base Median Presence at an Unsignalized Intersection.

Median Type on Major Street	$AMF_{mp, base}$
Undivided (may have bay at intersection)	1.00
Two-way left-turn lane, or any median that has a left-turn bay	1.00
Raised curb without left-turn bay [1]	0.83

Note:
1 - Median should be at least 4 ft wide and extend back from the intersection for at least 250 ft. If the median is less than 4 ft wide or extends back less than 250 ft, the AMF value is 1.0.

$$AMF_{mp} = AMF_{mp,base} \times AMF_{mw} \quad \textbf{(7-17)}$$

with,

$$AMF_{mw} = \begin{cases} e^{0.0076\,(W_m - 16)} & : \textit{if } W_m > 16 \textit{ ft and 3 legs} \\ e^{0.0160\,(W_m - 16)} & : \textit{if } W_m > 16 \textit{ ft and 4 legs} \\ 1.0 & : \textit{if undivided or } W_m \le 16 \textit{ ft} \end{cases} \quad \textbf{(7-18)}$$

where:
AMF_{mp} = median presence accident modification factor;
$AMF_{mp,\,base}$ = base median presence accident modification factor (see Table 7-12);
AMF_{mw} = median width accident modification factor; and
W_m = median width (including bay, if present), ft.

Base Condition: no median on major street

REFERENCES

1. *Roadway Safety Design Synthesis*. Texas Transportation Institute, The Texas A&M University System, College Station, Texas, 2005.

2. 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.

Appendix A

Worksheets

TABLE OF CONTENTS

Freeway Worksheet (1 of 2)

General Information	***Site Information***
Analyst: ____________	Freeway number: ____________
Agency: ____________	Roadway segment: ____________
Date performed: ____________	District: ____________
Location:	Analysis year:

Input Data			
Basic Roadway Data			
Frequency of severe crashes (injury & fatal), crashes/yr:			
Number of through lanes:			
Area type:	___ Urban	___ Rural	
Segment length (L), mi:			
Traffic Data			
Average daily traffic (ADT), veh/d:			
Geometric Data			
Grade (g), percent:			
Cross Section Data			
Lane width (W_l), ft:			
Outside shoulder width (W_s), ft:			
Inside shoulder width (W_{is}), ft:			
Median type:	___ Surfaced	___ Depressed	
Median width (W_m), ft:			
Presence of shoulder rumble strips:	___ Outside	___ Inside	___ None
Roadside Data			
Utility pole density (D_p), poles/mi:			
Utility pole offset (W_o), ft:			

Freeway Worksheet (2 of 2)

Base Crash Frequency Information for Segment		
Crash Data		
Base crash rate (*Base*), severe crashes/mvm: Table 2-1		
Local calibration factor (*f*):		
Accident Modification Factors (AMF) for Segment		
Grade (AMF_g): Equation 2-6		
Lane width (AMF_{lw}): Equation 2-7		
Outside shoulder width (AMF_{osw}): Equation 2-8		
Inside shoulder width (AMF_{isw}): Equation 2-10		
Median width (AMF_{mw}): Equations 2-12 & 2-13		
Shoulder rumble strips (AMF_{srs}): Equation 2-15		
Utility pole offset (AMF_{pd}): Equation 2-16		
Combined AMF (product of all AMFs above) ($AMF_{combined}$):		Multiply all AMFs evaluated, disregard others.

Expected Crash Frequency for Segment		
Expected severe base crash frequency (C_b), crashes/yr:	$C_b = 0.000365 \times Base \times ADT \times L \times f$	
Expected severe crash frequency for segment (C), crashes/yr:	$C = C_b \times AMF_{combined}$	

Rural Highway Worksheet (1 of 2)

General Information		Site Information	
Analyst:	___	Highway number:	___
Agency:	___	Roadway segment:	___
Date performed:	___	District:	___
Location:		Analysis year:	

Input Data			
Basic Roadway Data			
Frequency of severe crashes (injury & fatal), crashes/yr:			
Number of through lanes:			
Segment length (L), mi:			
Traffic Data			
Average daily traffic (ADT), veh/d:			
Geometric Data			
Presence of horizontal curve:	___ Yes	___ No	
Presence of spiral transition curves:	___ Yes	___ No	
Curve radius (R), ft:			
Curve length (L_c), ft:			
Superelevation rate specified by design guidelines (e_d), %:			
Superelevation rate (e), %:			
Grade (g) (absolute value), percent:			
Presence of a passing or climbing lane:	___ One direction	___ Both directions	
Cross Section Data			
Lane width (W_l), ft:			
Outside shoulder width (W_s), ft:			
Inside shoulder width (W_{is}), ft:			
Median type:	___ Undivided, TWLTL, or flush paved		___ Depressed
Median width (W_m), ft:			
Presence of shoulder rumble strips:	___ Outside	___ Inside	___ None
Presence of centerline rumble strip:	___ Yes	___ No	
Access Control Data			
Driveway density (D_d) (two-way total), driveways/mi:			
Roadside Data			
Horizontal clearance (W_{hc}), ft:			
Side slope (S_s) (horiz. change for 1 ft change in vertical), ft:	1V:___H		
Utility pole density (D_p), poles/mi:			
Utility pole offset (W_o), ft:			
Bridge Data			
Presence of bridges (I_{br})	___ Yes	___ No	
Relative bridge width (W_b), ft:			

Rural Highway Worksheet (2 of 2)

Base Crash Frequency Information for Segment			
Crash Data			
Base crash rate (*Base*), severe crashes/mvm:	Table 3-1		
Local calibration factor (f):			
Accident Modification Factors (AMF) for Segment			
Curve deflection angle (I_c)	I_c = 5280 (57.3 L_c)/R		
Horizontal curve radius (AMF_{cr}):	Equation 3-6		
Spiral transition curve (AMF_{sp}):	Equation 3-8		
Grade (AMF_g):	Equations 3-9 & 3-10		
Lane width (AMF_{lw}):	Equation 3-11 or Figure 3-6		
Outside shoulder width (AMF_{osw}):	Equation 3-12 or Figure 3-8		
Inside shoulder width (AMF_{isw}):	Equation 3-14		
Median width (AMF_{mw}):	Equation 3-16 & 3-17		
Shoulder rumble strips (AMF_{srs}):	Equation 3-19		
Centerline rumble strip (AMF_{crs}):	Table 3-9		
TWLTL median type (AMF_T):	Equation 3-20		
Superelevation (AMF_e):	Figure 3-12		
Passing lane (AMF_{pass}):	Table 3-10		
Horizontal clearance (AMF_{hc}):	Equation 3-22		
Side slope (AMF_{ss}):	Equation 3-24		
Utility pole offset (AMF_{pd}):	Equation 3-26		
Bridge width (AMF_{bw}):	Equation 3-28		
Driveway density (AMF_{dd}):	Equation 3-30		
Combined AMF (product of all AMFs above) ($AMF_{combined}$):			Multiply all AMFs evaluated, disregard others.
Expected Crash Frequency for Segment			
Expected severe base crash frequency (C_b), crashes/yr:	$C_b = 0.000365 \times Base \times ADT \times L \times f$		
Expected severe crash frequency for segment (C), crashes/yr:	$C = C_b \times AMF_{combined}$		

Urban Street Worksheet (1 of 2)

General Information	**Site Information**
Analyst: ____________	Street number or name: ____________
Agency: ____________	Street segment: ____________
Date performed: ____________	District: ____________
Location:	Analysis year:

Input Data			
Basic Roadway Data			
Frequency of severe crashes (injury & fatal), crashes/yr:			
Number of through lanes (n_l):			
Development type:	___ Commercial, Business, or Office		___ Residential or Industrial
Segment length (L), mi:			
Traffic Data			
Average daily traffic (ADT), veh/d:			
Percent trucks represented in ADT (P_t), percent:			
Geometric Data			
Presence of horizontal curve:	___ Yes	___ No	
Curve radius (R), ft:			
Cross Section Data			
Lane width (W_l), ft:			
Shoulder width (W_s), ft:			
Median type:	___ Undivided	___ TWLTL	___ Raised- curb
Median width (W_m), ft:			
Parking and Access Control Data			
Density of driveways serving business or office land uses ($D_{d,\ b/o}$), driveways/mi:			
Percent of right-side segment length with parking (P_{pkr}), %:			
Percent of left-side segment length with parking (P_{pkl}), %:			
Percent of parking adjacent to business land use ($P_{b/o}$), %:			
Percent of curb parking that is angle parking (P_{ap}), %:			
Roadside Data			
Utility pole density (D_p), poles/mi:			
Utility pole offset (W_o), ft:			

Urban Street Worksheet (2 of 2)

Base Crash Frequency Information for Segment			
Crash Data			
Base crash rate (*Base*), severe crashes/mvm:	Table 4-1		
Base driveway density (D_{base}), driveways/mile:			
Proportion of off-road crashes ($P_{off\text{-}road}$):			
Local calibration factor (*f*):			
Accident Modification Factors (AMF) for Segment			
Horizontal curve radius (AMF_{cr}):	Equation 4-6		
Lane width (AMF_{lw}):	Equation 4-9		
Shoulder width (AMF_{sw}):	Equation 4-10		
Median width (AMF_{mw}):	Equation 4-12		
TWLTL median type (AMF_{T}):	Equation 4-14		
Curb parking (AMF_{pk}):	Equation 4-18		
Utility pole offset (AMF_{pd}):	Equation 4-20		
Driveway density (AMF_{dd}):	Equation 4-22		
Truck presence (AMF_{tk}):	Equation 4-24		
Combined AMF (product of all AMFs above) ($AMF_{combined}$):			Multiply all AMFs evaluated, disregard others.

Expected Crash Frequency for Segment		
Expected severe base crash frequency (C_b), crashes/yr:	$C_b = 0.000365 \times Base \times ADT \times L \times f$	
Expected severe crash frequency for segment (*C*), crashes/yr:	$C = C_b \times AMF_{combined}$	

Interchange Ramp Worksheet (1 of 1)

General Information		***Site Information***	
Analyst:	____________	Highway number:	____________
Agency:	____________	Intersecting highway:	____________
Date performed:	____________	District:	____________
Location:		Analysis year:	

Input Data			
Basic Roadway Data			
Frequency of severe crashes (injury & fatal), crashes/yr:			
Area type:	___ Urban	___ Rural	
Traffic Data			
Average daily traffic volume on ramp (ADT_{ramp}), veh/d:			
Average one-way daily traffic on the adjacent mainlanes (ADT_{main}), 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	
Speed-change lane type:	___ Acceleration	___ Deceleration	

Base Crash Frequency Information for Interchange Ramp		
Crash Data		
Base crash rate (*Base*), severe crashes/mv: Table 5-1		
Local calibration factor (*f*):		

Expected Crash Frequency for Interchange Ramp		
Expected severe base crash frequency (C_b), crashes/yr:	$C_b = 0.000365 \times Base \times ADT_{ramp}$	

Base Crash Frequency Information for Speed-Change Lane		
Crash Data		
Base crash rate (*Base*), severe crashes/mv: Table 5-2		
Local calibration factor (*f*):		

Expected Crash Frequency for Speed-Change Lane		
Expected severe base crash frequency for acceleration lane (C_b), crashes/yr:	$C_b = 0.000365 \times Base \times (ADT_{ramp} + ADT_{main}) \times f$	
Expected severe base crash frequency for deceleration lane (C_b), crashes/yr:	$C_b = 0.000365 \times Base \times ADT_{ramp} \times f$	

Rural Intersection Worksheet (1 of 2)

General Information	**Site Information**
Analyst: ____________	Highway Number: ____________
Agency: ____________	Intersecting Highway: ____________
Date Performed: ____________	District: ____________
Location:	Analysis Year:

Input Data				
Basic Intersection Data				
Frequency of severe crashes (injury & fatal), crashes/yr:				
Number of through lanes on major road:				
Number of through lanes on minor road:				
Number of intersection legs:	___ 3	___ 4		
Intersection control mode:	___ Signalized	___ Two-way stop (unsignalized)		
Traffic Data				
Average daily traffic of the major road (Q_{major}), veh/d:				
Average daily traffic of the minor road (Q_{minor}), veh/d:				
Percent trucks in traffic stream (P_t), %:				
Cross Section Data				
Major-road approaches with left-turn lanes:	___ 1	___ 2		
Major-road approaches with right-turn lanes:	___ 1	___ 2		
Shoulder width on major road (W_s), ft:				
Median type on major road in vicinity of intersection:	___ Undivided	___ TWLTL	___ Depressed	
Median width (W_m), ft:				
Alignment skew angle (I_{sk}), degrees:				
Intersection Sight Distance				
Quadrants with sight distance restrictions:	___ 1	___ 2	___ 3	___ 4
Access Data				
Number of driveways within 250 ft along the major road (d_n):				

Rural Intersection Worksheet (2 of 2)

Base Crash Frequency Information for Rural Intersection		
Crash Data		
Base crash rate (*Base*), severe crashes/mev: Tables 6-1 & 6-2		
Local calibration factor (*f*):		

Accident Modification Factors (AMF) for Rural Intersection					
	Signal Control		Two-Way Stop Control		
Left-turn lane (AMF_{LT}):	Table 6-5		Table 6-9		
Right-turn lane (AMF_{RT}):	Table 6-6		Table 6-10		
Number of lanes (AMF_{lane}), major road:	Table 6-7		Table 6-11		
Number of lanes (AMF_{lane}), minor road:	Table 6-7		Table 6-11		
Shoulder width (AMF_{sw}):			Equation 6-14		
Median width (AMF_{mw}):			Equation 6-18		
Base median presence ($AMF_{mp,\ base}$):			Table 6-12		
Alignment skew angle (AMF_{skew}):	--	1.00	Equ. 6-19, 6-20		
Intersection sight distance (AMF_{SD}):			Table 6-13		
Driveway frequency (AMF_{nd}):	Equation 6-7		Equation 6-22		
Truck presence (AMF_{tk}):	Equation 6-9		Equation 6-24		
Combined AMF (product of all AMFs above) ($AMF_{combined}$):					Multiply all AMFs evaluated, disregard others.

Expected Crash Frequency for Rural Intersection		
Expected severe base crash frequency (C_b), crashes/yr:	$C_b = 0.000365 \times Base \times (Q_{major} + Q_{minor}) \times f$	
Expected severe crash frequency for segment (C), crashes/yr:	$C = C_b \times AMF_{combined}$	

Urban 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			
Basic Intersection Data			
Frequency of severe crashes (injury & fatal), crashes/yr:			
Number of through lanes on major street:			
Number of through lanes on minor street:			
Number of intersection legs:	___ 3	___ 4	
Intersection control mode:	___ Signalized	___ Two-way stop (unsignalized)	
Traffic Data			
Average daily traffic of the major street (Q_{major}), veh/d:			
Average daily traffic of the minor street (Q_{minor}), veh/d:			
Cross Section Data			
Major-street approaches with left-turn lanes:	___ 1	___ 2	
Major-street approaches with right-turn lanes:	___ 1	___ 2	
Through movement lane width on major street (W_l), ft:			
Shoulder width on major street (W_s), ft:			
Median type on major street in vicinity of intersection:	___ Undivided	___ TWLTL	___ Raised curb
Median width on major street (W_m), ft:			

Urban Intersection Worksheet (2 of 2)

Base Crash Frequency Information for Urban Intersection					
Crash Data					
Base crash rate (*Base*), severe crashes/mev: Table 7-1 & 7-2					
Local calibration factor (*f*):					
Accident Modification Factors (AMF) for Urban Intersection					
	Signal Control		Two-Way Stop Control		
Left-turn lane (AMF_{LT}):	Table 7-5		Table 7-9		
Right-turn lane (AMF_{RT}):	Table 7-6		Table 7-10		
Number of lanes (AMF_{lane}), major street:	Table 7-7		Table 7-11		
Number of lanes (AMF_{lane}), minor street:	Table 7-7		Table 7-11		
Lane width (AMF_{lw}):	Equation 7-7		Equation 7-12		
Shoulder width (AMF_{sw}):			Equation 7-14		
Median width (AMF_{mw}):			Equation 7-18		
Base median presence ($AMF_{mp,\ base}$):			Table 7-12		
Combined AMF (product of all AMFs above) ($AMF_{combined}$):					Multiply all AMFs evaluated, disregard others.
Expected Crash Frequency for Urban Intersection					
Expected severe base crash frequency (C_b), crashes/yr:	$C_b = 0.000365 \times Base \times (Q_{major} + Q_{minor}) \times f$				
Expected severe crash frequency for segment (C), crashes/yr:	$C = C_b \times AMF_{combined}$				