Memorandum

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Re: Advancing Application of Data Driven Safety Analysis Task 2: Explore the Validity of Combining Predictive Methods

Introduction

The purpose of this memorandum is to present current practices for implementing the empirical Bayes (EB) method within the context of predictive safety analysis principles outlined in the first edition of the *Highway Safety Manual* (*HSM*) and to provide recommendations for filling gaps in applicability of this methodology. The primary objectives include the following:

1. Exploring a combined predictive method that accounts for a project alternatives analysis with and without an EB adjustment.
2. Investigating the appropriate traffic volumes to use in alternatives analysis.
3. Evaluating the need for calibrating prediction models.

This memo focuses on the first edition of the *HSM* as the second edition is currently under development and it is unclear at this point how and if the topics in this memo will be addressed. This memo also includes several case studies illustrating issues identified with the current *HSM* approach and how the proposed methodology more broadly applies for project alternatives analysis with and without the EB method. The first section provides an overview of terminology used throughout this memorandum.

Terminology and Key Considerations

Terminology

This section introduces terminology needed to understand the project alternative analysis process and supporting methods for each step. Most terminology is consistent with that found in the HSM; however, this section provides specific definitions used throughout this memorandum. The following are commonly referenced terms throughout this document:

- **Average annual crash frequency** – A measure of the long-term average number of crashes per year for a particular type of roadway or intersection. The *HSM* glossary uses the term expected crashes.[1] Average annual crash frequency can be determined based on observed crash frequency, predicted crash frequency, or expected crash frequency.

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- **Observed crash frequency** – The number of crashes reported to have occurred at a study site, obtained using crash data. Annualized, this produces an average annual crash frequency estimated with observed crash history.
- **Predicted crash frequency** – The number of crashes predicted to occur at a study site based on a prediction made using a safety performance function (SPF) or other crash prediction model (CPM) developed using similar sites. Annualized, this produces an average annual crash frequency estimated with predictive models.
- **Expected crash frequency** - The number of crashes expected to occur at a site, calculated using the EB method. Conceptually, this represents a weighted average of observed and predicted crash frequencies. The *HSM* defines this as the expected average crash frequency. Annualized, this produces an average annual crash frequency estimated with the EB method.
- **Crash severity** – The maximum injury severity reported for all persons involved in a crash. Crash severity is commonly categorized on crash reports using the KABCO scale, where:
  - K is a fatal injury.
  - A is a suspected serious injury.
  - B is a suspected minor injury.
  - C is a possible injury.
  - O is no apparent injury, commonly referred to as a property damage only (PDO) crash.
- **Crash modification factor (CMF)** – A factor used to compute the average annual crash frequency after implementing a given countermeasure at a site relative to the average annual crash frequency without installation.
- **Pseudo-CMF** – A CMF based on a ratio of predicted crash frequency for an alternative compared to a baseline no-build condition.
- **Crash prediction model** – Also, known as a predictive model, a CPM is a general term for a model used to predict average annual crash frequency based on traffic volume and roadway and operational characteristics.
- **Safety performance function** – An equation used to predict average annual crash frequency per year at a site as a function of traffic volume and, in some cases, other roadway and operational characteristics. SPFs are often derived from CPMs but may be derived from a traffic-volume-only SPF and SPF adjustment factors (AFs).
- **SPF adjustment factors** – Project design-level SPF CMFs developed specifically for use with base condition SPFs found in the *HSM* Part C predictive method. SPF AFs work in combination and are unique to each facility type included in the *HSM*.
- **HSM Predictive Method** – The methodology used in Part C of the *HSM* to estimate the average annual crash frequency (including by crash severity and collision type) of a site, facility, or roadway under given geometric conditions, traffic volumes, and period of time. This applies to existing facilities, project alternatives, or proposed new roadways.

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Key Considerations

In addition to terminology, it is important for readers of this memo to have background on key considerations related to selecting and applying SPFs as part of a predictive method. The following subsections provide an overview of SPF sources, calibration, and SPF considerations.

SPF Sources

There are two basic sources for project design-level SPFs supporting alternative analysis:

- *HSM* base model SPFs and their associated SPF AFs.
- Jurisdiction-specific SPFs.

The *HSM* Part C CPM uses three components to estimate predicted crash frequency, shown as Figure 1.

$$N_{predicted} = N_{spf,x} \times \left(\prod AF_{n,x}\right) \times C_x$$

Figure 1. *HSM* Part C CPM.[1]

where:

$N_{predicted,x}$ = predicted crash frequency for site of interest.

$N_{spf,x}$ = predicted crash frequency for SPF base conditions for site type x.

$AF_{n,x}$ = SPF AF for factor n for site type x, reflecting the conditions at the site of interest.

C_x = Calibration factor for site type x.

Each SPF, associated SPF AFs, and associated calibration factor is specific to a site type, also known as a reference population. The *HSM* defines a reference population as a grouping of sites with similar characteristics.[1] Reference populations are commonly grouped into types of segments or intersections and are alternatively described as facility types. An example segment facility type is a rural, two-lane, two-way segment and an example intersection facility type is a four-leg intersection with minor road stop control on rural two-lane, two-way highways.

Each facility type CPM includes an SPF for base conditions, one or more SPF AFs, and a local calibration factor. The base condition predicts crash frequency for a segment using only traffic volume and segment length (or major and minor road traffic volumes for intersections) assuming the site consists of a specified set of base conditions for roadway and operational features. The set of SPF AFs adjust the base condition prediction such that the CPM can provide more reliable estimates for sites that do not match all base conditions. One base condition value is specified for each variable represented in an SPF AF. These variables are referred to herein as "base variables." The set of specified values are referred to as "base condition values" for the SPF. Each Part C chapter lists the base variables and the base condition values associated with the CPMs in that chapter.

The predictive method in the *HSM* Part C uses a site-based approach for safety analysis. With this approach, the road facility of interest is separated into homogenous road segment sites and intersection sites. The predictive method is used to estimate the average crash frequency for each site. The estimates for all sites are combined to obtain an estimate of the average crash frequency for the facility. A homogenous segment is one that has key safety-influential characteristics (e.g., AADT, lane width) that are consistent for the length of the segment.

The calibration factor (discussed in greater detail in the calibration subsection) adjusts the national SPF to local conditions.

As an alternative to using the *HSM* SPFs or CPMs, jurisdictions may choose to develop their own SPFs or CPMs. Jurisdiction-specific SPFs typically replace the base condition SPF in the *HSM* predictive method while jurisdiction-specific CPMs replace both the base condition SPF and SPF AFs.

Srinivasan et al. developed a decision-making process for deciding whether to calibrate national SPFs or develop jurisdiction-specific SPFs.[2] The process generally consists of the following steps:

1. Identify existing SPF(s).
2. Consider sample size necessary for calibrating SPFs.
3. Consider required and desired data attributes necessary for calibration.
4. Calibrate SPFs.
5. Assess calibration factor reliability.
6. Consider sample size and additional data necessary for developing SPFs.
7. Develop SPFs.
8. Assess SPF reliability.

In general, researchers have found that:

1. Calibrated *HSM* SPFs outperform uncalibrated SPFs.
2. Calibration factors grouped by AADT range and calibration functions outperform single calibration factors.
3. Jurisdiction-specific SPFs can further improve reliability.

However, jurisdictions should weigh the additional benefit against the cost to undertake the extra effort involved.[3]

SPF Calibration

The need for SPF calibration was briefly highlighted in the SPF Sources section; however, this section provides more details on the need for calibration and key considerations.

The *User's Guide to Develop Highway Safety Manual Safety Performance Function Calibration Factors* states that "The use of predictive models in any jurisdiction calls for calibration of the *HSM* Part C's SPFs, and for replacement of default crash distribution tables and AFs to local conditions."[4] The reasoning for this is twofold:

1. *HSM* SPFs were developed using data from two or three jurisdictions, often in different parts of the country. Calibration can account for differences in climate, driving population, animal population, crash reporting thresholds, and crash reporting systems.
2. Data used to develop the SPFs came from different time periods, for which predictions are likely to be biased.

The guide notes that the purpose of calibration is to reduce bias and improve reliability in predicted crash frequency to inform decision-making. The guide goes on to highlight that unadjusted *HSM* predictive models over-estimate crashes in Oregon from 25 to 80 percent, depending on the facility type. Additionally, States have found that calibration factors differ, sometimes vastly, from one facility type to another for *HSM* SPFs.

Prior to release of the first edition of the *HSM*, the prediction models included in Part C were calibrated to support practitioners utilizing SPFs for different facility types for comparative analysis.[5] Of particular interest was the need to standardize the underlying SPFs for the CPMs in the *HSM* Chapters 10, 11, and 12, since these used data from different States. The difference in predicted crash frequency is likely the result of differences among States as much as the difference is due to differences in facility types. The intent of the single-state calibration was to calibrate the predictive methods to one State, to remove or reduce this bias. Intersection SPFs were calibrated using Highway Safety Information System (HSIS) data from California while segment SPFs were calibrated using HSIS data from Washington. Notably, freeway SPFs were not calibrated with either States' data, having been released as a supplemental chapter in 2014. However, the freeway SPFs contain data from Washington and California.

Although a single-State calibration was considered to support analysis across facility types, it is unclear how well even a relative assessment using the HSM will work from one state to another, particularly given the differences States have found across facility types in calibration factors. Having calibration factors for all facility types of interest provides the only solution for reliably estimating average annual crash frequency from predicted crashes when considering SPFs from different facility types.

Background

Overview of the Highway Safety Manual and Part C Predictive Method

The first edition of the *HSM* presents a formal approach for applying highway safety principles to transportation planning and design projects.[1] The manual consists of the following sections:

Part A: Introduction, Human Factors, and Fundamentals.

Part B: Roadway Safety Management Process.

Part C: Predictive Method.

Part D: Crash Modification Factors.

Information presented in Part C within the context of project-level analysis forms the framework for this memo.

Part C of the *HSM* provides a predictive method comprised of 18 steps to estimate crashes by severity and crash type based on different road characteristics such as facility type, site type, and geometric conditions. Figure 2 provides the framework of the HSM predictive method.[1] The purpose of the *HSM* predictive method is to provide a methodology for assessing the average crash frequency for segments or intersections given the existing or proposed conditions and to provide for analysis of design option tradeoffs during project development. This method provides predicted crash frequency independent of observed crash frequency.

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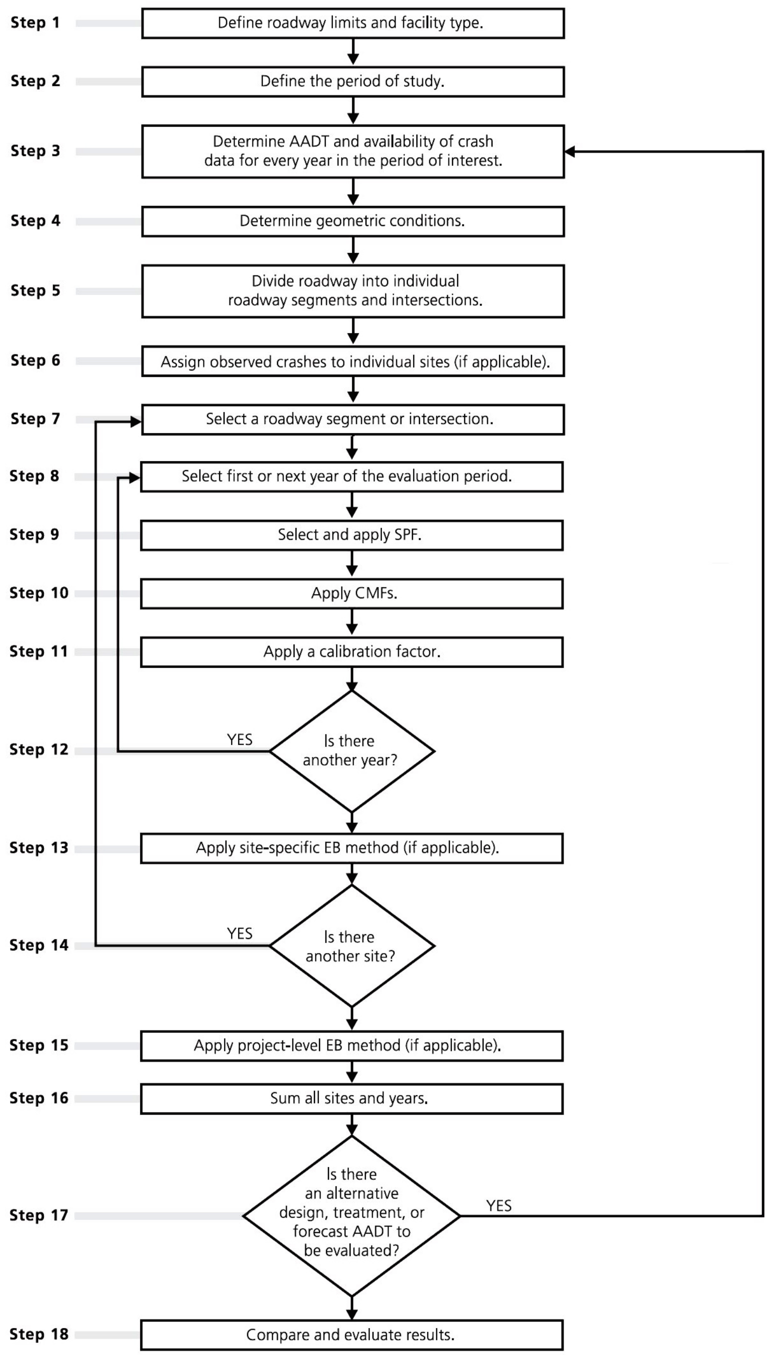

Figure 2. Chart. The HSM Predictive Method.[1]

The predictive method consists of a CPM allowing users to estimate average annual crash frequency using SPFs and SPF AFs. As noted in the Terminology section, SPFs are regression models used to predict average crash frequency for a segment or intersection, in this context assuming a set of base conditions. The SPF AFs account for roadway and geometric conditions that differ from base conditions and have been shown to have a significant relationship with crash frequency or severity. SPFs are developed by facility type and represent the average crash frequency for a segment or intersection, given the traffic volume and geometric features of the facility. SPFs can be used to estimate predicted crash frequency for existing conditions, modifications to an existing roadway, or a proposed new roadway. The *HSM* provides SPFs, and associated AFs, based on data from multiple states for a variety of facility types.

Observed crash frequency on a roadway segment or at an intersection often fluctuates year-over-year, and short-term trends are not necessarily indicative of long-term averages. Furthermore, effects are magnified for locations with low average crash frequencies because any increase in crashes represents a large variation from the long-term average. Statistically speaking, a period of unusually high crash frequency is likely to be followed by a period of lower crash frequency (assuming site conditions remain constant), and vice versa. This tendency is referred to as regression-to-the-mean (RTM), and the effects should be accounted for to avoid biasing project or countermeasure selection toward sites with recently observed crash frequencies that are abnormally higher or lower than long-term averages. Additionally, accounting for RTM bias avoids overestimating the effects of proposed countermeasures or changes to roadway geometric features.

The *HSM* predictive method uses the EB method to address RTM bias. The EB method estimates a weighted average of the observed crash history at the site of interest and the predicted crashes from an applicable SPF for a given facility type. The weight is based on the SPF prediction and the statistical variance of the SPF (via the overdispersion parameter, which accounts for the model's reliability). Figure 3 provides the equation for estimating expected crash frequency based on predicted crash frequency and observed crash frequency.[1]

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

Figure 3. Equation. EB method for calculating expected crash frequency.[1]

where:

$N_{expected}$ = estimate of expected average crash frequency for a study period.

$N_{predicted}$ = predicted average crash frequency for a study period.

$N_{observed}$ = observed average crash frequency for a study period.

w = weight adjustment for predicted crash frequency.

Figure 4 provides the equation used to estimate the weight factor.[1]

$$w = \frac{1}{1 + k \times \left(\Sigma_{study\ years} N_{predicted}\right)}$$

Figure 4. Equation. Calculation for EB method weight adjustment.[1]

where:

k = overdispersion parameter from the associated SPF.

The EB method accounts for RTM bias by pulling the observed crash frequency toward the average (in this case predicted) crash frequency based on the reliability of the SPF and the level of predicted crash frequency. The EB method is either applied on a site-specific basis or at the project level depending on whether observed crash data can accurately be assigned to specific locations within the study area. Researchers have determined that expected crash frequency is a better metric for evaluating project alternatives than observed or predicted crash frequency alone.[1]

EB Method Applicability

Practitioners use Step 3 of the *HSM* predictive method to determine whether the EB method is applicable for a project evaluating an existing site or alternative condition to an existing site. Practitioners can apply the EB method at the site-specific level during Step 13 or at the project-level during Step 15 depending on whether observed crash data can be assigned to individual roadway segments or intersections. If observed crash data are not available, or not applicable, then the expected average crash frequency is limited to the predicted average crash frequency.

In Section C.5, the *HSM* recommends using at least two years of observed crash frequency data to improve the reliability of observed crash frequency.[1] The *HSM* Section A.2.1 also suggests the applicability of the EB method to a project depends both on the type of analysis and the type of future project being planned. Section A.2.1 of the *HSM* states verbatim that the EB method is applicable for the following project types[1]:

- Sites at which the roadway geometrics and traffic control are not being changed (e.g., the "do-nothing" alternative).
- Projects in which the roadway cross section is modified but the basic number of through lanes remains the same (this would include, for example, projects for which lanes or shoulders were widened or the roadside was improved, but the roadway remained a rural two-lane highway).
- Projects in which minor changes in alignment are made, such as flattening individual horizontal curves while leaving most of the alignment intact.
- Projects in which a passing lane or a short four-lane section is added to a rural two-lane highway to increase passing opportunities.
- Any combination of the above improvements.

If the EB method is applied to individual roadway segments and intersections, and some roadway segments and intersections within the project limits will not be affected by the major geometric improvement, it is acceptable to apply the EB method to those unaffected segments and intersections.

HSM Section A.2.1 further states verbatim that the EB method is not applicable for the following types of improvements[1]:

- Projects in which a new alignment is developed for a substantial proportion of the project length.

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- Intersections at which the basic number of intersection legs or type of traffic control is changed as part of a project.

In general, the *HSM* does not recommend the EB method be used for project alternative analysis when the observed crash data for the previous period is not relevant to or indicative of future crash frequencies given the nature of the geometric and/or operational changes being considered. For example, if a roadway segment is being widened from two to four through lanes (and not just a short four-lane section), the observed crash history is not relevant to the segment's expected performance and thus the EB method should not be applied. The project team further explored the merits of using the EB method for project alternative analysis when the facility type does not change (see the *Highway Safety Manual User Guide* case study example in the Additional Considerations section).

In cases where the EB method does not apply to one or more alternatives considered, the *HSM Supplement*, Section B.2.1 recommends not using the EB method within the Part C predictive method by skipping the associated steps in the process and relying solely on predicted crashes for comparing alternatives.[6] This section specifically states that "the EB method will need to be consistently applied to all alternatives being evaluated. If the EB method cannot be consistently applied to all alternatives, then it should not be used for any alternatives.[6]" This recognizes a potential bias the EB method can introduce when comparing predicted crash frequency for one alternative to expected crash frequency for another alternative. The *HSM* recommends using the predictive method without EB adjustment in this case.

It is important to distinguish that the *HSM* guidance focuses on analyzing alternatives during the same period in which the historic crash data may apply for one alternative but not for others. When considering the baseline condition as being the present period with no site changes, or a future period with no site changes, the historic crash data would apply. If a CMF is applied to the current no-build or future no-build, then the EB expected crash frequency would provide the most reliable estimate (provided predicted crash frequency comes from a calibrated *HSM* SPF or from a jurisdiction-specific SPF). The EB method would not be appropriate to directly compare average annual crash frequency from expected crashes (e.g., for the future no-build condition) to the average annual crash frequency from predicted crashes (e.g., for a future alternative condition) if the historic crash data were not applicable in the alternative. However, the EB method would be appropriate if the comparison were made using the future no-build condition as a baseline and a CMF were applied or a pseudo-CMF were applied (so long as the pseudo-CMF is deemed to be reliable). The EB method is appropriate in this case since a baseline condition is used and a relative change is assessed rather than a direct comparison between average annual crash frequency based on expected crash frequency and predicted crash frequency.

There are several gaps that must be addressed when using the predictive method to evaluate the safety performance for an existing project, particularly when analyzing project alternatives. These gaps are addressed in the next subsection.

Gaps in HSM Predictive Method Application

The first edition of the *HSM* focuses on straightforward applications of the predictive method and the EB method; however, the *HSM* Part C provides no examples and little context on how to handle project alternative analysis. Based on practitioner feedback, there is a need for guidance and support on the following topics related to the application of the *HSM* predictive method.

1. **Establishing a uniform and consistent approach** for where and when to apply the EB method for project alternatives. There are several scenarios for which questions can arise on the best approach to undertake specific project circumstances.

    a. **The project results in a change in facility type**. Agencies may have access to calibrated *HSM* SPFs or jurisdiction-specific SPFs for none, some, or all of the alternatives under consideration. The *HSM* suggests not using the EB method when the project results in a change in facility type, but a comparison of predicted crash frequencies for all alternatives may not provide reliable results due to issues of calibration, or due to ignoring site specific characteristics related to safety for which the location was selected for a safety project. If a project location has an unusually high or low historic crash count, there may be underlying reasons. If the predictive method does not capture those reasons, then ignoring the site-specific crash history will bias the results of the analysis. This is particularly true when benefit-cost ratios are calculated. Additionally, the EB method should not be applied, in general, if the agency does not have access to calibrated *HSM* SPFs or jurisdiction-specific SPFs.

    b. **The *HSM* does not include a CPM for a facility type included in the project alternatives**. For example, CPMs are not included in the *HSM* for three-leg signalized intersections on rural two-lane highways. Practitioners can use additional guidance on how to select the appropriate method for analyzing safety effects of proposed countermeasures or treatments when a predictive method is not directly applicable This guidance could be added to the *HSM* Part C introduction, in section C.7 Methods for Estimating the Safety Effectiveness of a Proposed Project as an expanded overall section.

    c. **Questions on the applicability of historic crash data given proposed alternatives do not change the facility type**. What the *HSM* defines as minor changes to roadway alignment or cross-section can still have impacts on whether the historic crash data are valid for a future period. Using the EB method for some alternatives may result in differences in expected crash frequency that underestimate the effects of the proposed alternative(s).

2. **Providing a consistent message on appropriate traffic volumes** for use in the predictive method when analyzing the future conditions during alternatives analysis. For example, an alternative widening from two lanes to four lanes may require using substantially higher predicted traffic volumes based on the increased capacity. Moreover, when the expected traffic volume changes among alternatives, there is no guidance provided for when additional analyses should be conducted, including for other nearby facilities, if traffic is expected to be diverted to or from another route.

3. **Conducting economic analysis based on predicted crash frequency** for alternatives when local calibration factors are not available. Practitioners have focused on *HSM* language suggesting a comparison of relative changes in crash frequency, but this does not provide for an estimate of the expected safety benefit for economic analysis. Additionally, while the *HSM* predictive method is calibrated to a single-State (California for intersections and Washington for segments), the relationship among relative crash frequencies for different

facility types may be unique for each jurisdiction. Additionally, agencies may have jurisdiction-specific SPFs or calibration factors for some but not all facility types. Therefore, a comparison of relative safety performance using national models may not be appropriate.

The project team reviewed the literature and State practices to examine how State transportation agencies and researchers are addressing these issues. The following section provides a summary of the findings of the literature and State practices review.

Literature and State Practices Review

The Appendix to this memo provides the complete literature and State practices review. The literature and State practices review covered three main topics: methods for estimating expected crash frequency for project alternatives; applicable traffic volumes for project alternatives; and SPF calibration needs.

Estimating Expected Crash Frequency

Since publication of the first edition of the *HSM*, expected crash frequency has become synonymous with crash frequency estimated from the EB method. However, the *HSM* defines expected crashes as "an estimate of long-range average number of crashes per year for a particular type of roadway or intersection."[1] Expected crash frequency may be based on the EB method, SPF prediction, observed crash history, or other means based on engineering judgment. As noted in the terminology section, this memorandum uses average annual crash frequency, which may be based on observed, predicted, or expected crash frequency (based on the EB method).

The literature and State practices review uncovered a basic approach for estimating average annual crash frequency when the EB method cannot be used for all project alternatives during trade-off analysis. The Massachusetts Department of Transportation (MassDOT) *Safety Alternatives Analysis Guide* breaks this approach into two distinct stages[7]:

A. In the first stage, MassDOT provides guidance based on the availability of a calibrated SPF and reliable historic crash data. Depending on their availability, the MassDOT Guide provides three methods to establish the estimated number of crashes for the no-build condition in the design year. The estimated number of crashes for the no-build condition in the design year is the baseline against which alternatives are measured and comes from one of three methods:

   1. Expected crash frequency via the EB method.
   2. Predicted crash frequency.
   3. Observed crash frequency.

B. In the second stage, MassDOT recommends practitioners multiply the baseline expected crash frequency for the no-build condition by CMF to account for the change in safety performance. The State provides a preferred CMF list and recommends no more than two CMFs be combined for an alternative.

This analytical approach results in alternatives with numeric changes in expected crash frequency which can be carried into economic analysis. Even without a calibrated predictive method, practitioners are not limited to a relative comparison of safety performance (e.g., the result suggesting a reduction from 20 crashes per year to 16 crashes per year versus only suggesting a 20 percent reduction in average crash frequency).

The literature review showed that researchers, such as Persaud et al., have used a similar approach, with subtle differences, during before-after analysis to evaluate the safety effectiveness of converting a stop-controlled intersection to a signalized intersection.[8] The Persaud et al. study proposed the following approach for analysis:

1. Estimate the number of crashes that would be expected if the intersection was stop-controlled. Persaud et al, developed jurisdiction-specific SPFs for stop-controlled intersections for use in this step of the EB Method along with expected traffic volumes and recent crash counts.
2. Estimate the number of crashes that would be predicted if the intersection was signalized. Persaud et al. developed jurisdiction-specific SPFs for signalized intersections for use here along with expected traffic volumes.
3. Estimate the expected change in safety as the difference between estimates 1 and 2.

Rodegerdts et al. applied the method from Persaud et al. in an example to calculate the change in crashes due to conversion from a two-way stop-controlled intersection to a roundabout.[9] They first calculated the number of expected crashes using the EB method if the intersection was not converted to a roundabout. The study then calculated the predicted number of crashes for the roundabout using the roundabout crash prediction models developed as a part of the study, which were specific to the local jurisdictions. The change in crashes due to the intersection conversion was calculated using the expected crashes from the stop-controlled intersection and the predicted crashes for the roundabout. They also illustrated an alternative method for cases where a roundabout crash prediction model may not be available. In this, a CMF for conversion from stop-controlled to roundabout operation is applied to the number of expected crashes using the EB Method if the intersection was not converted to a roundabout.

In both the Persaud and Rodegerdts CMF evaluation approaches, predicted crash frequency for the alternative is compared to expected crash frequency for the baseline condition.[8,9] Both researcher teams developed jurisdiction-specific SPFs for application of predicted crash frequency, even when comparing EB expected crash frequency for one condition to predicted crash frequency for the alternative condition. In general, to utilize this approach, calibrated *HSM* SPFs or jurisdiction-specific SPFs are required. Additionally, the analytical approach compares expected crash frequency for the no-build condition to predicted crash frequency for the alternative, which introduces bias as noted by the *HSM*.

The key distinction with this approach is that direct measures of crash frequency are compared for safety analysis. In this case, expected crash frequency and predicted crash frequency. Expected crash frequency considers crash history while predicted crash frequency does not. An alternative approach, as noted previously, would be to estimate the average annual crash frequency using expected crash frequency, predicted crash frequency, or observed crash history and then compare alternatives using a relative assessment, such as a CMF or pseudo-CMF approach. This reduces bias by providing a common baseline estimate and then addressing the change in safety through a relative comparison. However, when using a pseudo-CMF method, analysts need to consider that the relative safety performance may be assessed using calibrated *HSM* SPFs, uncalibrated SPFs, or jurisdiction-specific SPF. Using calibrated SPFs or jurisdiction-specific SPFs will reduce bias in a pseudo-CMF approach.

Appropriate Traffic Volumes

The second topic studied in the literature and State practices review concerned the selection of appropriate traffic volumes for alternatives analysis. The primary research question focused on whether alternative-specific design-year traffic volumes should be used or if existing no-build volumes should be used during analysis of project alternatives.

Research has shown that, in general, the average crash frequency increases as traffic volume increases. This increase is not typically linear, which is why the *HSM* recommends SPFs for estimating safety performance as a function of AADT rather than crash rates. Consequently, the resulting number of expected crashes will generally be higher for alternatives with higher anticipated traffic volumes.

The *HSM*, in Section 3.5.2, states that to apply an SPF, AADT forecast estimates for future periods is necessary.[1] Additionally, Section C.1 highlights that the predictive method provides a quantitative measure of average annual crash frequency under existing conditions and conditions which have not yet occurred. This includes predicting safety performance for existing facilities under past or future traffic volumes, alternative designs for an existing facility under past or future traffic volumes or designs for a new facility under forecast traffic volumes. In Section C.4, the *HSM* states that the predictive method is used to estimate the average annual crash frequency for a given time period during which the geometric design and traffic control features are unchanged and traffic volumes are known or forecast.[1]

In a sample application of the predictive method, the Florida Department of Transportation (FDOT) *Safety Analysis Guidebook for PD&E Studies* provides a comparative assessment for a no-build condition of a rural two-lane highway versus an alternative condition of a four-lane divided highway.[10] The forecasted AADT for the no-build condition is 17,800 vehicles per day and 20,000 vehicles per day for the 2042 analysis year. The guide notes the EB method is not applicable due to the change in facility type. The FDOT guide highlights the importance of considering the forecasted difference in exposure in the design year, where the alternative condition observes 2,200 more vehicles per day. The guide notes the no-build condition may under-represent crash frequency, assuming the 2,200 vehicles per day would be distributed in other locations along the highway network. The guide concludes that the crash reduction benefit for the four-lane alternative may be greater than what is shown for this reason.

In general, language in the *HSM* focuses on using observed AADTs for estimating safety performance for past periods and using forecast AADTs for analysis of future periods. While not explicitly stated, *HSM* guidance suggests using alternative-specific traffic volume forecasts during analysis of project alternatives. However, the *Highway Safety Manual User Guide* includes an example alternatives analysis considering a rural highway which does not apply this methodology.[11] One of the project alternatives includes a substantial change to the roadway, entailing changing the existing rural two-lane road to a four-lane divided road. When predicting crashes for all alternatives, the example uses the same future year AADT for all alternatives, regardless of the conversion from two lanes to four lanes under one project alternative. The example provided in the *HSM User Guide* is overly simplistic and does not provide practitioners with realistic considerations, leading to confusion on application. Future editions of the *HSM* and accompanying implementation guidance should strive to avoid overly simplified examples. Providing realistic examples, presenting real world issues and methods to overcome those will help practitioners to better understand the kinds of decisions and supporting data that can be used in application of the predictive method for alternative analysis.

An Introduction to the Highway Safety Manual also includes an alternatives analysis example; however, AADT is treated differently than in the *HSM User Guide* example.[12] The example in *An Introduction to the Highway Safety Manual* includes a no-build option and two alternatives. Alternative 1 is a mix of a three- and five-lane road and Alternative 2 is a completely five-lane road. To calculate the predicted crash frequency for each alternative, the example uses the same forecasted AADT for the no-build option and Alternative 1 but used a higher forecasted AADT for Alternative 2, which results in a higher predicted crash frequency for Alternative 2, despite increasing the number of through lanes. This example is consistent with the language included in the first edition of the *HSM*. In this case, accounting for the anticipated change in traffic volume allowed the analyst to identify the potential for an increase in crash frequency despite the increase in number of lanes. However, as noted by Council and Stewart, a comprehensive approach to analyzing safety in this case may seek to determine if an increase in traffic volume (and thereby predicted crash frequency) will be offset by a decrease in AADT on adjacent segments, or if the resulting increase in traffic volume is due to induced demand.[13]

Predictive Model Calibration

The literature and State practices review included documentation related to calibrating *HSM* models in the predictive method. The *HSM* recommends jurisdictions calibrate SPFs for local conditions using a minimum of 30 to 50 sites with at least 100 crashes per year.[1] Researchers have examined sample sizes needed for calibration, transferability of SPFs from one jurisdiction to another, and calibration from one time-period to another. Results for transferability studies have varied, leading many State agencies to develop calibration factors for predictive models by facility type. More recently, State agencies and researchers have identified that calibration factors may not sufficiently reflect the differences between State-specific trends and national SPFs. Therefore, agencies have explored developing calibration functions, allowing calibration factors to vary as a function of one or more input variables (for example, AADT). Agencies have further considered developing jurisdiction-specific SPFs and CPMs to improve reliability over relative calibration factors and functions.

While the vast majority of research has focused on developing calibration factors and establishing calibration factor sufficiency, there is little research on application of the predictive method with uncalibrated models. Farid et al. proposed a modified EB measure to provide for segment-specific calibration factors transferring SPFs to local conditions.[14] The researchers found this approach to be more reliable than the *HSM* calibration procedure and it allows agencies to use numeric values for economic evaluation rather than relative assessments from uncalibrated models. The *Highway Safety Manual User Guide* directly indicates that users can make relative assessments within the same facility and control type.[11] The User Guide expressly states that the output from an *HSM* SPF cannot be used to describe an actual prediction if it is not calibrated to local conditions. Many practitioners have taken this guidance to suggest the use of relative changes when calibrated SPFs are not available, including applications comparing the results of project alternatives resulting in different facility types. Since the relative safety performance from one facility type to another would be expected to differ from agency to agency (i.e., calibration factors would not be consistent from one facility type to another), it is unclear if this approach produces reliable results.

A further extension of calibrated SPFs is their application in the EB method. As noted in Figure 3, predicted crash frequency is a weighted component of EB expected crash frequency. Using an uncalibrated model will result in a predicted crash frequency that may not reliably estimate the average crash frequency for a site with those conditions in that specific jurisdiction. Using historic crash data will support bringing the expected crash frequency closer to historic crash data, but the difference may introduce a bias since it is no longer only correcting for RTM bias, but also bias in the predicted crash frequency. Ahmed et al. identified that the SPF prediction plays a pivotal role in the EB method.[15] Their results concluded that more accurate, calibrated models provide more accurate results when evaluating the safety effectiveness of a treatment. Furthermore, Bahar and Hauer noted that "recent traffic and safety data for any project corridor or site are sufficient for a sound analysis when using relevant SPF and the EB method."[4]

Additional Considerations

This section focuses on additional considerations of a prescribed approach for conducting alternatives analysis using the *HSM* predictive approach. Beyond those found in the literature and State practices review, *HSM* pooled-fund members provided examples of projects and supporting calculations for which the *HSM* predictive method could not be applied in a straightforward manner and adjustments to the method needed to be considered.

The North Carolina Department of Transportation (NCDOT) provided the first example, where no direct *HSM* predictive method is available to support quantification of project alternatives. The Wisconsin Department of Transportation provided the second example, where the predictive method provides a predicted crash frequency (from calibrated SPFs) that is not in line with historic crash frequency for a given project location. The *Highway Safety Manual User Guide* provides the third example, which demonstrates potential issues with including historic crash data for a project even when project alternatives include only minor changes to the cross-section and roadside.

Note that each of these case study examples are revisited using the recommended project alternative analysis approach from this memo in the Case Studies Revisited section. The case studies provided in this section are intended to demonstrate methodological difficulties, challenges, and inconsistencies identified. They are provided as demonstrations of how guidance can further support consistent and reliable application of *HSM* methods for alternative analysis.

NCDOT Example: Three-Leg Signalized Intersection

Overview

NCDOT conducted a predictive safety analysis for an existing three-leg intersection with stop control on the minor road approach. The project alternatives included installing a standard traffic signal and left-turn lanes at the intersection or installing a Continuous Green-T signalized intersection. Since the *HSM* does not provide SPFs for three-leg signalized intersections or for Continuous Green-T intersections, NCDOT could not use the predictive method for analyzing the safety effects of the two alternatives. Additionally, the existing stop-controlled intersection has a convenience store located along the intersection, making this intersection different from a standard three-leg intersection. Based on this, NCDOT used the most recent five years of crash data as a baseline value and opted to use CMFs to estimate the change in safety performance for each alternative. This approach was chosen after analysis revealed the observed crash frequency for this intersection was approximately four times higher than the crash frequency observed at intersections similar to the study intersection. Figure 5 provides the collision diagram developed by NCDOT for the study intersection.

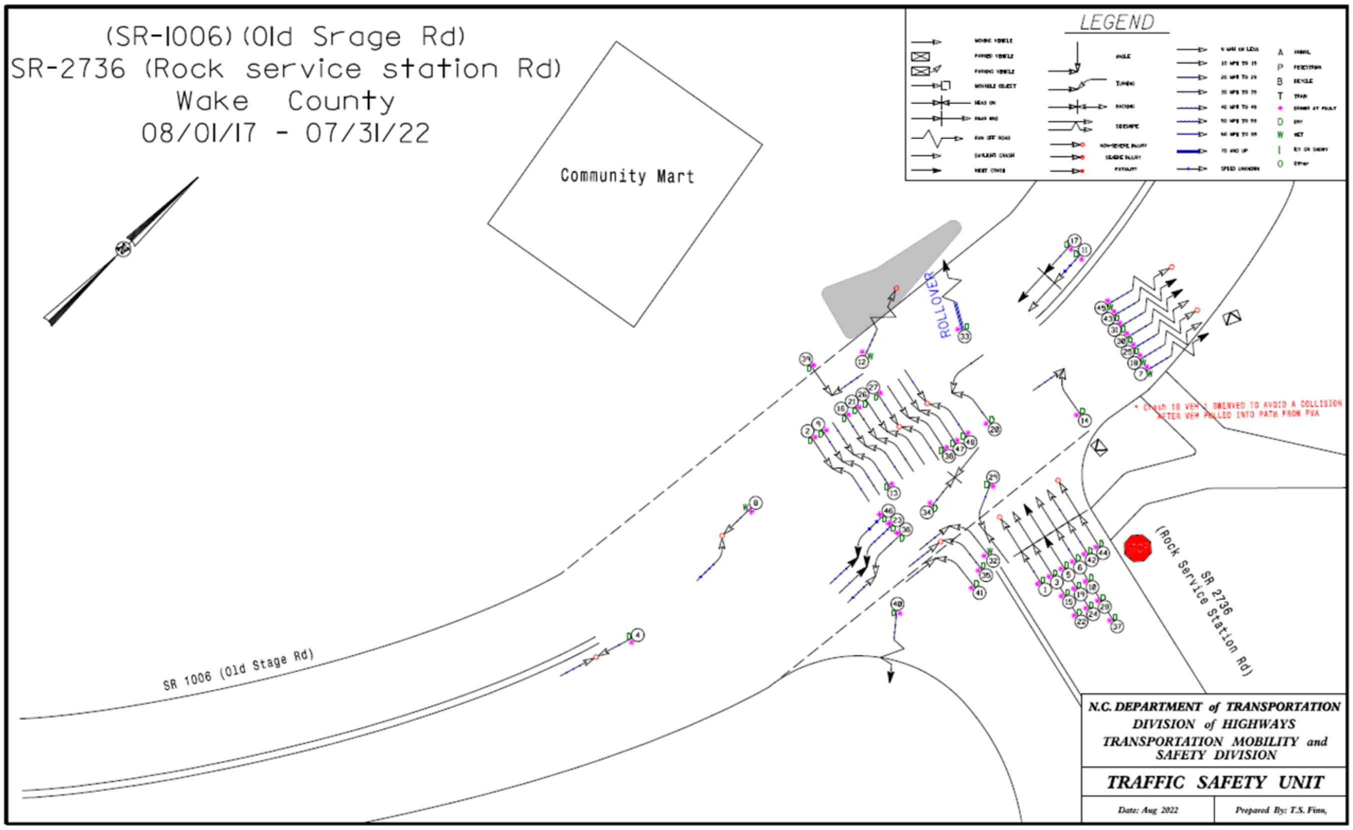

Figure 5. Graphic. NCDOT Collision Diagram.

Methodology

NCDOT first disaggregated the 5-year observed crash history consisting of 48 total crashes of various types and severities. Table 1 provides a breakdown of crash frequency by crash type and severity based on the five-years of data. For example, the following provide for the calculation of five-year crash frequency:

- 48 total crashes ÷ 5 years = 9.6 crashes per year.
- 10 fatal and injury crashes ÷ 5 years = 2.0 fatal and injury crashes per year.
- 1 angle crash ÷ 5 years = 0.2 angle crashes per year.
- 18 left-turn crashes ÷ 5 years = 3.6 left-turn crashes per year.
- 15 rear-end crashes ÷ 5 years = 3.0 rear-end crashes per year.
- 11 gas station-related crashes ÷ 5 years = 2.2 gas station-related crashes per year.

Table 1. Crash type and severity distribution for NCDOT example.

Crash Severity or Type	Average Observed Frequency	Proportion of Total Crashes
Total crashes	9.6 crashes/year	1.00
Fatal crashes	0 crashes/year	0.00
Fatal and injury crashes	2.0 crashes/year	0.21
Angle crashes	0.2 crashes/year	0.02
Left-turn crashes	3.6 crashes/year	0.38
Rear-end crashes	3.0 crashes/year	0.31
Crashes related to gas station	2.2 crashes/year	0.23

The proportion of total crashes were calculated as follows:

- Total crashes = 9.6 ÷ 9.6 total crashes = 1.00.
- Fatal crashes = 0 ÷ 9.6 total crashes = 0.00.
- Fatal and injury crashes = 2.0 ÷ 9.6 total crashes = 0.21.
- Angle crashes = 0.2 ÷ 9.6 total crashes = 0.02.
- Left-turn crashes = 3.6 ÷ 9.6 total crashes = 0.38.
- Rear-end crashes = 3.0 ÷ 9.6 total crashes = 0.31.
- Gas station-related crashes = 2.2 ÷ 9.6 total crashes = 0.23.

The average observed crash frequencies provided in the second column of Table 1 serve as the baseline condition for the existing intersection.

Table 2 provides the CMFs NCDOT identified for both alternatives for use in a comparative analysis. NCDOT drew these CMFs from their State list of preferred CMFs for safety analysis. Table 2 also includes the estimate of average annual crash frequency for each alternative given current traffic volume.

Table 2. NCDOT project alternative analysis.

Alternative	Target Crash Type	Baseline Crash Frequency	CMF	Resulting Crash Frequency
1	Angle and left-turn	3.80	0.39	1.48
1	Rear-end	3.00	0.71	2.13
1	Remainder	2.80	1.00	2.80
1	Total	9.60	0.67	6.41
2	Angle and left-turn	3.80	0.39	1.48
2	Rear-end	3.00	0.71	2.13
2	Remainder	2.80	1.00	2.80
2	Fatal and injury	1.34	0.85	1.14
2	Property damage only	5.07	1.00	5.07
2	Total	6.41	0.97	6.21

Memorandum

For Alternative 1 (installing a traffic signal and left-turn lanes), NCDOT identified CMFs of 0.39 and 0.71 for angle and left-turn crashes and rear-end crashes, respectively. For the remaining crashes, NCDOT assumed a CMF of 1.00 (i.e., no change in crash frequency). NCDOT calculated the following to determine the estimated average annual crashes for Alternative 1:

- Angle and left-turn crash frequency = (3.60 crashes/year + 0.20 crashes/year) × 0.39 = 1.48 crashes/year.
- Rear-end crash frequency = 3.00 crashes/year × 0.71 = 2.13 crashes/year.
- Remaining crash frequency = 2.80 crashes/year × 1.00 = 2.80 crashes/year.
- Total Alternative 1 estimated crash frequency = 1.48 + 2.13 + 2.80 = 6.41 crashes/year.

The average resulting CMF was computed as the following:

- Average CMF = 6.41 crashes/year ÷ 9.60 crashes/year = 0.67.

For Alternative 2 (Continuous Green-T), NCDOT first computed the expected change assuming a traffic signal installation. To estimate the safety performance of the traffic signal base condition (Alternative 1), NCDOT used the total estimated crashes from above (6.41 crashes per year) and applied the crash severity proportions (from Table 1) to estimate the fatal and injury crashes (proportion = 0.21) as follows.

- Alternative 1 estimated fatal and injury crash frequency = 6.41 crashes/year × 0.21 = 1.34 fatal and injury crashes/year.

This assumes the fatal and injury crash proportion stays the same from the existing minor road stop control condition to a traffic signal condition. This assumption was made due to the unique nature of this intersection and the lack of a typical crash severity proportion for a three-leg rural signalized intersection (with a high-volume driveway at the intersection).

NCDOT then used a CMF (0.85) to estimate the change in fatal and injury crashes associated with converting the intersection from a signal (Alternative 1) to a Continuous Green-T intersection (Alternative 2). NCDOT applied the CMF for a Continuous Green-T intersection as follows:

- Alternative 2 estimated fatal and injury crash frequency = 1.34 fatal and injury crashes/year × 0.85 = 1.14 fatal and injury crashes/year.

The remaining proportion of crashes (property damage only crashes) was assumed to be unaffected by the Continuous Green-T design. NCDOT calculated the following to determine the estimated average annual property damage only crashes for Alternative 1:

- Total crash frequency = fatal and injury crash frequency + property damage only crash frequency.
- Property damage only crash frequency = total crash frequency – fatal and injury crash frequency.
- Property damage only crash frequency = 6.41 total crashes/year – 1.34 fatal and injury crashes/year = 5.07 crashes/year.

Finally, NCDOT calculated the following to determine the estimated average annual total crashes for Alternative 2:

- Total crash frequency = fatal and injury crash frequency + property damage only crash frequency.
- Total crash frequency = 1.14 fatal and injury crashes/year + 5.07 property damage only crashes/year = 6.21 crashes/year.

The result was an estimated 6.21 total annual crashes after conversion to a Continuous Green-T intersection. The resulting CMF for conversion from a standard signal to a Continuous Green-T intersection was calculated as follows:

- Average CMF = 6.21 crashes/year ÷ 6.41 crashes/year = 0.97.

NCDOT then estimated the 2045 safety performance by scaling the total crash frequency based on a linear estimate of the change in forecasted traffic volumes. NCDOT estimated the traffic volume increase to 2045 to be 188 percent of current volumes, yielding a multiplier of 1.88. NCDOT estimated 2045 crash frequency to be the following:

- No build: 9.6 crashes/year ×1.88 = 18.05 crashes/year.
- Alternative 1: 6.41 crashes/year × 1.88 = 12.05 crashes/year.
- Alternative 2: 6.21 crashes/year × 1.88 = 11.67 crashes/year.

Discussion

NCDOT noted that the five-year crash history shows a large number of crashes (23 percent) are related to movements in and out of the gas station. Both project alternatives involve redesigning the intersection to change the major road to be Rock Service Station and the north leg of Old Stage Road. NCDOT added that the new configuration should reduce crashes involving the gas station since many of these movements would now become right-turns; however, no safety benefits were added based on crashes involving vehicles from the gas station. Additionally, due to selection for treatment based on a high crash frequency at the intersection, RTM bias is possible. Due to the lack of a predictive method, observed crash history was used since the EB method could not be applied. Since there was no applicable SPF (to determine an alternative relationship between traffic volume and crash frequency) or crash severity distribution available for a rural three-leg signalized intersection, NCDOT worked within the constraints of the information available to them. As such, NCDOT assumed the proportion of fatal and injury crashes would be the same for the signalized intersection and used a linear growth rate in crashes based on the change in forecasted traffic volumes.

Wisconsin DOT (WisDOT) Example: Four-Leg Roundabout

Overview

WisDOT conducted a predictive analysis of an existing four-leg unsignalized intersection identified as a site with promise based on the State’s network screening analysis using 2014 through 2017 data. Crash data analysis revealed most crashes were right-angle, failure to yield crashes. WisDOT evaluated a roundabout alternative based on the crash data. Table 3 provides the updated historic crash data identified for the predictive analysis. The crash data reveal that observed crash frequency remained high after accounting for two years of newer data (lessening the likelihood of RTM bias) and that the proportion of fatal and injury crashes (65 percent) is much higher than the assumed proportion for a four-leg stop control intersection (31 percent) in Wisconsin.

Memorandum

Table 3. Historic crash data for example four-leg unsignalized intersection.

Year	Total Observed	Fatal and Injury Crashes	Property Damage Only Crashes
2015	4	2	2
2016	7	3	4
2017	3	3	0
2018	6	4	2
2019	3	3	0
Total	23	15	8
Average Frequency	4.6	3.0	1.6

Methodology

WisDOT conducted an alternatives and economic analysis based on predicted crash frequency since the alternative resulted in a change in facility type. Using this analysis approach, historic crash data could not be accounted for, even in the baseline condition, to reduce potential bias in the analysis results. WisDOT employed the Wisconsin-calibrated SPFs incorporated into FHWA's Interactive Highway Safety Design Model (IHSDM) for comparative analysis and economic evaluation. WisDOT completed all steps using the IHSDM.

WisDOT predicted crash frequency for the 10-year horizon period from 2026 through 2035 for each alternative. The analysis for the no-build condition indicated a predicted 0.64 fatal and injury crashes per year and 1.43 property damage only crashes per year. The following shows the calculations for the no-build predicted crash frequency over the 10-year period:

- Fatal and injury crashes = 0.64 crashes/year × 10 years = 6.40 fatal and injury crashes.
- Property damage only crashes = 1.43 crashes/year × 10 years = 14.30 property damage only crashes.

The analysis for the roundabout alternative indicated a predicted 0.11 fatal and injury crashes per year and 1.54 property damage only crashes per year. The following shows the calculations for the roundabout alternative predicted crashes frequency over the 10-year period:

- Fatal and injury crashes = 0.11 crashes/year × 10 years = 1.10 fatal and injury crashes.
- Property damage only crashes = 1.54 crashes/year × 10 years = 15.40 property damage only crashes.

WisDOT conducted an evaluation of the roundabout alternative assuming the crash costs provided in Table 4. Using a discount rate of 5 percent, the economic analysis indicated a total present value of crash cost of $4,511,630 for the no-build alternative and $448,920 for the roundabout alternative. The present value of the project costs for the roundabout alternative is $1,909,000. The benefit-cost ratio for the roundabout alternative was calculated as the following:

- Benefit-cost ratio = ($4,511,630 - $448,920) ÷ $1,909,000 = 2.13.

Table 4. Wisconsin crash cost data.

Injury Severity	Assumed 2021 Crash Costs
K	$13,021,489
A	$698,010
B	$220,717
C	$125,983
O	$16,034

Discussion

WisDOT noted that this analysis estimated a benefit-cost ratio greater than 1.0 for the roundabout alternative, justifying the project. However, WisDOT further demonstrated that the predicted crash frequency for the no-build condition greatly undervalues the potential benefits of the project. When accounting for the observed crash history, the EB method indicates the following:

- 3.0 expected fatal and injury crashes/year.
- 2.2 expected property damage only crashes/year.

Over 10 years, this translates to the following:

- 30.3 expected fatal and injury crashes.
- 21.7 expected property damage only crashes.

As indicated previously, predicted crash frequency over the 10-year period included the following:

- 6.40 predicted fatal and injury crashes.
- 14.30 predicted property damage only crashes.

This difference, particularly in fatal and injury crashes, would have led to a much higher benefit-cost ratio when incorporating historic crash frequency and severity data. For similar projects, comparing only predicted crash frequency may result in economic analysis that suggests the project is not economically justified, ignoring the potential benefits suggested from a history of higher-injury severity due to site-specific conditions.

Highway Safety Manual User Guide Example: EB Method Application for Alternatives within a Facility Type

Overview

The *Highway Safety Manual User Guide* provides an example project alternatives analysis for rural two-lane highway segments where two alternatives are considered to improve safety performance. The study corridor consists of three roadway segments. Table 5 provides an overview of the geometric characteristics, calibration factor for the local jurisdiction, and observed crash frequency for each segment.[11] Segments 1 and 3 are tangents and Segment 2 is a horizontal curve. The guide provides sample calculations to establish the predicted crash frequency for each segment, using the predictive method (i.e., base SPF, SPF AFs, and calibration factor). Table 5 provides the predicted crash frequency for each segment.

Table 5. *Highway Safety Manual User Guide* example roadway segment data.[11]

Characteristics	Segment 1	Segment 2	Segment 3
Length (miles)	1.17	0.78	1.95
AADT (vpd)	9,000	9,000	9,000
Lane width (feet)	12	12	12
Shoulder width (feet)	1	1	1
Shoulder type	Paved	Paved	Paved
Curve length (miles)	0	0.78	0
Curve radius (feet)	0	2,650	0
Spiral transition	Not present	Not present	Not present
Superelevation variance	0	0.02	0
Grade (percent)	2	2	2
Driveway density	1.7	0	4.5
Centerline rumble strips	Not present	Not present	Not present
Passing lanes	Not present	Not present	Not present
Two-way left-turn lane	Not present	Not present	Not present
Roadside hazard rating	5	5	5
Segment lighting	Not present	Not present	Not present
Automated speed enforcement	Not present	Not present	Not present
Calibration factor	1.30	1.30	1.30
Observed crash data	11	40	11
Predicted crash frequency	4.94	3.58	8.24

Using the equations provided in Figure 3 and Figure 4, Table 6 provides the expected crash frequency for the existing conditions for all three segments based on the predicted and observed crashes.

Table 6. *Highway Safety Manual User Guide* example predicted and expected crash frequency calculations.[11]

Segment	Predicted Crash Frequency	Observed Crash Frequency	Overdispersion Parameter	Weighted Adjustment	Expected Crash Frequency
1	4.94	11	0.202	0.167	9.99
2	3.58	40	0.303	0.156	34.32
3	8.24	11	0.121	0.167	10.54

Based on these findings, the agency considered two project alternatives to improve safety performance for the three roadway segments analyzed. The project alternatives consist of the following changes:

1. Shoulder widening from 1 foot to 6 feet in each direction.

2. Shoulder widening from 1 foot to 6 feet in each direction, installing roadway lighting, improving roadside hazard rating, and implementing automated speed enforcement.

For both alternatives, the guide assumes AADT remains the same and the road does not attract any additional traffic.

Methodology

Since the project alternatives consist of one or more geometric changes included as SPF AFs in the *HSM* predictive method, the guide estimated the effects of multiple changes in combination. Since the facility type did not change, the authors recalculated the weighted adjustment based on the new SPF predictions and calculated expected crash frequency for each segment for each alternative. Table 7 provides a summary of the alternatives analysis results by segment for each alternative. The results suggest a safety improvement for each segment for both Alternative 1 and Alternative 2, with a larger decrease in expected crash frequency in Alternative 2.

Table 7. *Highway Safety Manual User Guide* example alternative analysis results summary.[11]

Alternative	Segment	Predicted Crash Frequency	Observed Crash Frequency	Overdispersion Parameter	Weighted Adjustment	Expected Crash Frequency
No Build	1	4.94	11	0.202	0.167	9.99
No Build	2	3.58	40	0.303	0.156	34.32
No Build	3	8.24	11	0.121	0.167	10.54
No Build	Total	16.76	62	N/A	N/A	54.85
1	1	4.02	11	0.202	0.198	9.62
1	2	2.91	40	0.303	0.185	33.14
1	3	6.70	11	0.121	0.198	10.15
1	Total	13.63	62	N/A	N/A	52.91
2	1	3.01	11	0.202	0.248	9.02
2	2	2.18	40	0.303	0.232	31.21
2	3	5.02	11	0.121	0.248	9.52
2	Total	10.21	62	N/A	N/A	49.75

Discussion

Using the EB method for each of the alternatives, the historic crash data are expected to be applicable under the new cross-sectional and improvement conditions. In theory, this suggests (under Alternative 2) that increased shoulder width, improved roadside hazard rating, installation of roadway lighting, and automated speed enforcement would have no bearing on the observed crash frequency component of expected crash frequency.

The following demonstrates the change in predicted and expected crash frequency for Alternative 2:

- Change in predicted crash frequency = 100 × (1 – (10.21 ÷ 16.76)) = 39 percent decrease.
- Change in expected crash frequency = 100 × (1 – (49.75 ÷ 54.85)) = 9 percent decrease.

. For Alternative 1, the following demonstrates the change in predicted and expected crash frequency:

- Change in predicted crash frequency = 100 × (1 – (13.63 ÷ 16.76)) = 19 percent decrease.
- Change in expected crash frequency = 100 × (1 – (52.91 ÷ 54.85)) = 4 percent decrease.

By assuming the historic crash frequency holds under the alternatives, this evaluation methodology likely underestimates the potential crash reduction for the proposed improvements. The *Highway Safety Manual User Guide* approach is inconsistent with approaches for establishing a baseline average annual crash frequency for a no-build condition and then applying the change in crash frequency as a CMF or combination of CMFs.

Recommended Project Alternative Analysis Approach

Based on the review of literature and State practices and case study examples, the project team identified a generalized practical approach for conducting project alternatives analysis. The framework builds on the process MassDOT included in its *Safety Alternatives Analysis Guide*. The process includes five primary steps:

1. Establishing the baseline estimated average crash frequency for the future no-build condition.
2. Determining alternative-specific baseline average crash frequency.
3. Identifying the applicable method for estimating the safety effectiveness of project alternatives.
4. Calculating the project alternative estimated crash frequency.
5. Calculating expected change in crash frequency.

The methods provided in Step 1 are consistent with those found in the MassDOT *Safety Alternatives Analysis Guide*. Steps 2 and 3 are adapted from the MassDOT process but are generalized to a broader HSM-based approach. Figure 6 provides an overview of the recommended approach.

For Step 3, this memorandum provides several options for estimating the safety effectiveness of project alternatives. The intention of this memorandum is to include methods for agencies to use based on data availability and based on questions being answered by the alternative analysis. This memorandum does not include a hierarchy or preference for one method over another. The methods for estimating the safety effectiveness of project alternatives include the following:

- **Preferred CMF Method**. This method involves using an agency-preferred CMF (or CMFs) or CMF from a resource such as FHWA's CMF Clearinghouse to estimate a change in average annual crash frequency based on one or more countermeasures or design options.
- **Pseudo-CMF Method**. This method involves applying the predictive method directly to estimate a change in average annual crash frequency within a facility type or across facility types.
- **Surrogate Safety Measure Method**. This method involves estimating a CMF from safety surrogates or other measures as is deemed practical.

Each of these methods is explained in further detail in Step 3. Step 3 also provides information on the advantages and disadvantages of each approach.

Memorandum

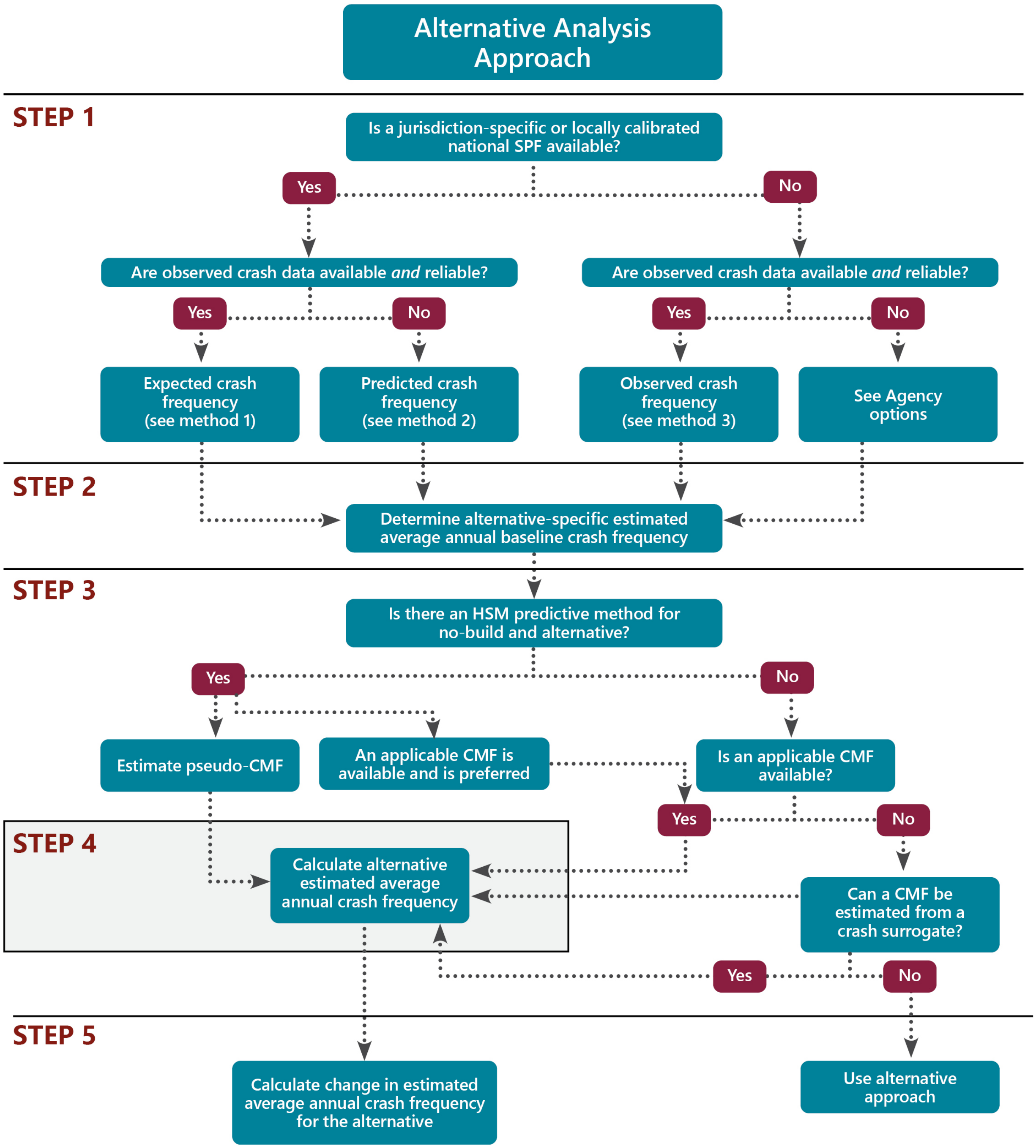

Figure 6. Flowchart. Recommended alternatives analysis approach.

Step 1: Determine Future No-Build Baseline Safety Performance

The future safety performance of a study segment or intersection, assuming no changes, serves as the baseline for alternative comparison. There are three common methods practitioners use for estimating the baseline average annual crash frequency, provided in order of reliability according to the *HSM*:

1. Expected crash frequency calculated using the EB method. This method requires both observed crash frequency and predicted crash frequency. To apply the EB method, the predicted crash frequency must come from a locally calibrated national SPF (such as those included in the *HSM*) or from a jurisdiction-specific SPF.

2. Predicted crash frequency calculated using a locally calibrated national SPF (such as those included in the *HSM*) or from a jurisdiction-specific SPF.

3. Observed crash frequency.

Additionally, it is possible that a segment or intersection does not have a locally calibrated national SPF (such as those included in the *HSM*), a jurisdiction-specific SPF, or reliable observed crash data. For example, one city may not collect property damage only crashes or may not provide the same level of completeness and accuracy as other reporting agencies within an urbanized area or State. In cases where a State or local agency does not have locally calibrated SPFs, jurisdiction-specific SPFs, or reliable observed crash data, the agency should provide guidance on a preferred method for estimating baseline crash frequency. The following are not specific recommendations, but include example options for estimating baseline crash frequency that are not proven at this time:

- Identifying a group of similar locations with reliable observed crash data and using the average observed crash frequency as the baseline for the current site conditions.

- Using a predictive method for a similar facility type and using predicted crash frequency as the baseline for current conditions. Under this approach, assumptions should be documented and adjustments to the predicted crash frequency may be considered using external CMFs or engineering judgment consistent with agency recommendations and suggested practices.

The following sections provide a brief overview of the three primary methods for estimating baseline average crash frequency.

Method 1 – Expected Crash Frequency

The expected crash frequency, calculated using the EB method as documented by Hauer et al., is a statistically weighted average of predicted and observed crash frequency (as described in Figure 3).[16] The *Highway Safety Improvement Program (HSIP) Evaluation Guide* and *A Guide to Developing Quality Crash Modification Factors* provide further resources on calculating expected crash frequency using the EB method.[17,18]The *HSM* recommends including a minimum of two years of observed crash data; however, three to five years of crash and traffic data are commonly used in this method. The method uses locally calibrated national SPFs or jurisdiction-specific SPFs to predict average crash frequency for a site. Uncalibrated national SPFs can bias the expected crash frequency as they may not capture jurisdiction-specific crash patterns.

For each year of the study period, practitioners should collect traffic volume information and observed crash frequency data by crash type and severity included in the analysis. Practitioners should estimate traffic volume data for the design year, when available, using traffic growth rates or a regional traffic model.

Practitioners should then predict crash frequency for each year of the study period using the applicable SPF, existing geometry, and appropriate annual traffic volumes. Moreover, practitioners should predict crash frequency for the design year of the project.

Using the study period predicted crash frequency, observed crash data, and SPF overdispersion parameter, practitioners can utilize the equations provided in Figure 3 and Figure 4 to obtain the expected crash frequency for the study period. The final step of this method includes estimating baseline average annual crash frequency for the design year. Figure 7 provides the calculation for estimated baseline average annual crash frequency from expected crash frequency. This is completed by multiplying the expected crash frequency for the study period by the ratio of the predicted crash frequency for the design year relative to the predicted crash frequency for the study period.

$$N_{baseline,design,nobuild} = N_{exp,study} \times \frac{N_{pr,design}}{N_{pr,study}}$$

Figure 7. Equation. Calculation of estimated baseline average annual crash frequency from Method 1.

Where:

$N_{baseline,design,nobuild}$ = estimated baseline average annual crash frequency in the design year.

$N_{exp,study}$ = expected crash frequency during the study period.

$N_{pr,design}$ = predicted crash frequency in the design year.

$N_{pr,study}$ = predicted crash frequency during the study period.

The resulting estimated baseline average annual crash frequency in the design year under no-build conditions serves as the baseline against which the alternatives are measured.

Method 2 – Predicted Crash Frequency

In some circumstances, practitioners may have access to locally calibrated national SPFs (such as those included in the *HSM*) or jurisdiction-specific SPFs but not have access to reliable crash data or crash data may not represent the existing conditions at the project location. In these cases, the expected crash frequency cannot be used for the baseline average annual crash frequency in the design year under no-build conditions. In this case, the predicted crash frequency from the locally calibrated national SPF or jurisdiction-specific SPF represents the baseline average annual crash frequency in the design year under no-build conditions. As with method 1, method 2 requires the practitioner to obtain existing geometric conditions and no-build traffic volumes for the study period and design year. The practitioner can use the existing roadway geometry information and design year traffic volumes to directly compute the predicted crash frequency for the design year under no-build conditions.

The resulting estimated baseline average annual crash frequency in the design year under no-build conditions serves as the baseline against which the alternatives are measured.

Method 3 – Observed Crash Frequency

In some cases, national SPFs may not exist for certain facility types or jurisdictions may not have access to locally calibrated national SPFs or jurisdiction-specific SPFs for some or all facility types. It may be that some locations are unique or are not common enough to justify SPF development or calibration. For these locations, practitioners can use observed crash history to represent the estimated baseline average annual crash frequency in the design year under no-build conditions. To improve the reliability of this approach, practitioners should strive to include as many years of crash data as possible to improve the estimate of long-term crash frequency. However, practitioners should avoid using too many years of data when it is possible other changes at the location impacting safety performance may have been implemented. When practical, including a study period of least five years of historic crash data for existing site conditions will improve the reliability of the estimated average crash frequency.

The final step of this method is estimating baseline average annual crash frequency in the design year under no-build conditions. Figure 8 and Figure 9 provide the calculation for estimating baseline average annual crash frequency from observed crash frequency for segments and intersections, respectively.

$$N_{baseline,design,nobuild} = N_{obs,study} \times \frac{AADT_{design}^{a}}{AADT_{study}^{a}}$$

Figure 8. Equation. Calculation of estimated baseline average annual crash frequency from Method 3 for segments.

$$N_{baseline,design,nobuild} = N_{obs,study} \times \frac{AADT_{maj,design}^{b} \times AADT_{min,design}^{c}}{AADT_{maj,study}^{b} \times AADT_{min,study}^{c}}$$

Figure 9. Equation. Calculation of estimated baseline average annual crash frequency from Method 3 for intersections.

Where:

$N_{obs,study}$ = observed crash frequency during the study period.

$AADT_{design}$ = AADT representative of the design year.

$AADT_{study}$ = AADT representative of the study period.

$AADT_{maj,design}$ = major road AADT representative of the design year.

$AADT_{min,design}$ = minor road AADT representative of the design year.

$AADT_{maj,study}$ = major road AADT representative of the study period.

$AADT_{min,study}$ = major road AADT representative of the study period.

a, b, c = SPF AADT parameters.

The *HSM* includes SPF AADT parameter estimates for segments and intersections by facility type, crash type, and crash severity. Table 8 provides the SPF AADT parameters by facility type from the *HSM*, which can be applied in Figure 8 and Figure 9. For facility types included in the *HSM*, an analyst can use the AADT coefficients in the appropriate equation to account for the non-linear relationship between AADT and crash frequency. For agencies that have jurisdiction-specific SPFs, the SPF parameters can be used in place of those from the *HSM*. If an *HSM* or jurisdiction-specific SPF is not available for the facility type of interest, an analyst could use SPF parameters from a similar facility type or consider using a crash rate approach (i.e., substituting 1.0 for the SPF AADT parameters).

Table 8. SPF Coefficients from the *HSM* by facility type, crash severity, and crash type.

Facility Type	Total			Fatal and Injury			Property Damage Only		
	AADT (a)	$AADT_{maj}$ (b)	$AADT_{min}$ (c)	AADT (a)	$AADT_{maj}$ (b)	$AADT_{min}$ (c)	AADT (a)	$AADT_{maj}$ (b)	$AADT_{min}$ (c)
Rural 2-lane undivided	1.000								
Rural 2-lane, 3-leg stop control		0.790	0.490						
Rural 2-lane, 4-leg stop control		0.600	0.610						
Rural 2-lane, 4-leg signalized		0.600	0.200						
Rural 4-lane undivided	1.176			1.094					
Rural 4-lane divided	1.049			0.958					
Rural 4-lane, 3-leg stop control		1.204	0.236		1.107	0.236			
Rural 4-lane, 4-leg stop control		0.848	0.448		0.888	0.525			
Rural 4-lane, 4-leg signalized		0.722	0.337		0.638	0.232			
Urban 2-lane undivided MV	1.680			1.660			1.690		
Urban 3-lane TWLTL MV	1.410			1.690			1.330		
Urban 4-lane undivided MV	1.330			1.250			1.380		
Urban 4-lane divided MV	1.360			1.280			1.380		
Urban 5-lane TWLTL MV	1.170			1.120			1.170		
Urban 2-lane undivided SV	0.560			0.230			0.640		
Urban 3-lane TWLTL SV	0.540			0.470			0.560		
Urban 4-lane undivided SV	0.810			0.610			0.840		
Urban 4-lane divided SV	0.470			0.660			0.450		
Urban 5-lane TWLTL SV	0.540			0.350			0.610		
Urban 3-leg stop control MV		1.110	0.410		1.160	0.300		1.200	0.510
Urban 4-leg stop control MV		0.820	0.250		0.930	0.280		0.770	0.230
Urban 3-leg signalized MV		1.110	0.260		1.020	0.170		1.140	0.300
Urban 4-leg signalized MV		1.070	0.230		1.180	0.220		1.020	0.240
Urban 3-leg stop control SV		0.160	0.510					0.250	0.550
Urban 4-leg stop control SV		0.330	0.120					0.360	0.250

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Table 8 (continued). SPF Coefficients from the *HSM* by facility type, crash severity, and crash type.

Facility Type	Total			Fatal and Injury			Property Damage Only		
	AADT (a)	$AADT_{maj}$ (b)	$AADT_{min}$ (c)	AADT (a)	$AADT_{maj}$ (b)	$AADT_{min}$ (c)	AADT (a)	$AADT_{maj}$ (b)	$AADT_{min}$ (c)
Urban 3-leg signalized SV		0.420	0.400		0.270	0.510		0.450	0.330
Urban 4-leg signalized SV		0.680	0.270		0.430	0.290		0.780	0.250
Freeway segment MV				1.492			0.936		
Freeway segment SV				0.646			0.876		
Entrance speed-change lane				1.173			1.215		
Exit speed-change lane				0.903			0.932		
Ramp/collector-distributor MV				0.524			1.256		
Ramp/collector-distributor SV				0.718			0.689		
Parclo type A2/B2 signalized terminal					0.325	0.212		0.592	0.516
Diamond 3-leg exit and Parclo Type A4 signalized terminal					0.379	0.394		0.797	0.384
Diamond 3-leg entrance and Parclo type B4 signalized terminal					0.265	0.905		0.741	0.845
Diamond 4-leg signalized terminal					1.191	0.131		0.879	0.545
Parclo type A2/B2 stop control terminal					0.260	0.947		0.773	0.878
Diamond 3-leg exit and Parclo Type A4 stop control terminal					0.582	0.899		0.595	0.937
Diamond 3-leg entrance and Parclo type B4 stop control terminal					0.709	0.730		0.885	0.350
Diamond 4-leg stop control terminal					1.008	0.177		0.845	0.476

Note: MV = multiple-vehicle; SV = single-vehicle; TWLTL = two-way left-turn lane

For urban arterial and freeway facilities, SPFs are differentiated by crash type (multiple-vehicle versus single-vehicle) and crash severity (fatal and injury versus property damage only). For the most reliable estimate of design year crash frequency, the observed crash frequency may be broken down by applicable crash type and severity with the equations in Figure 8 and Figure 9 being applied at the appropriate level. From there, the individual crash types and severities are aggregated to account for the combined effect of change in traffic volume on the crash frequency of interest (e.g., total crash frequency).

The resulting estimated baseline average annual crash frequency in the design year under no-build conditions serves as the baseline against which the alternatives are measured.

For example, assume a practitioner wants to estimate the design year baseline average annual crash frequency for an urban three-leg signalized intersection without access to a locally calibrated national SPF, and the observed crash frequency based on the most recent five years of data include the following:

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- 2.4 multiple-vehicle fatal and injury (MVFI) crashes per year.
- 4.4 multiple-vehicle property damage only (MVPDO) crashes per year.
- 0.1 single-vehicle fatal and injury crashes (SVFI) per year.
- 0.2 single-vehicle property damage only (SVPDO) crashes per year.
- 7.1 total observed crashes per year (2.4 + 4.4 + 0.1 + 0.2).

The intersection has study period AADTs of 40,000 and 8,000 vehicles per day for the major and minor road, respectively. The analyst estimated design year AADTs of 47,000 and 9,500 vehicles per day. Applying the equation in Figure 9, the baseline design year crash frequencies for each crash and severity type are estimated in Figure 10 through Figure 13.

$$N_{baseline,design,MVFI} = 2.4 \times \frac{47{,}000^{1.02} \times 9{,}500^{0.17}}{40{,}000^{1.02} \times 8{,}000^{0.17}} = 2.91\ MVFI\ crashes\ per\ year$$

Figure 10. Equation. Estimated average annual MVFI baseline crash frequency for example.

$$N_{baseline,design,MVPDO} = 4.4 \times \frac{47{,}000^{1.14} \times 9{,}500^{0.30}}{40{,}000^{1.14} \times 8{,}000^{0.30}} = 5.57\ MVPDO\ crashes\ per\ year$$

Figure 11. Equation. Estimated average annual MVPDO baseline crash frequency for example.

$$N_{baseline,design,SVFI} = 0.1 \times \frac{47{,}000^{0.27} \times 9{,}500^{0.51}}{40{,}000^{0.27} \times 8{,}000^{0.51}} = 0.11\ SVFI\ crashes\ per\ year$$

Figure 12. Equation. Estimated average annual SVFI baseline crash frequency for example.

$$N_{baseline,design,SVPDO} = 0.2 \times \frac{47{,}000^{0.45} \times 9{,}500^{0.33}}{40{,}000^{0.45} \times 8{,}000^{0.33}} = 0.23\ SVPDO\ crashes\ per\ year$$

Figure 13. Equation. Estimated average annual SVPDO baseline crash frequency for example.

Based on the increase in major road and minor road traffic volume, the study period observed crash frequency of 7.1 crashes per year is estimated to increase to the following: for the design year under no-build conditions.

- 2.91 MVFI + 5.57 MVPDO + 0.11 SVFI + 0.23 SVPDO = 8.82 crashes/year for the design year under no-build conditions.

This calculation used the SPF coefficients for an urban three-leg signalized intersection from the HSM to account for the non-linear relationship between AADT and crash frequency. Had this example used a crash rate assumption (the SPF coefficients are all equal to 1.0), the resulting calculation would be:

- 3.35 MVFI + 6.14 MVPDO + 0.14 SVFI + 0.28 SVPDO = 9.91 crashes/year for the design year under no-build conditions.

This example highlights that assuming a linear crash rate when the SPF coefficient for AADT is less than 1.0 will tend to overestimate the change in average annual crash frequency given a change in traffic volume. If the SPF coefficient for

AADT is greater than 1.0, an assumed linear crash rate will underestimate the change in average annual crash frequency given a change in traffic volume.

Step 2: Alternative-Specific Baseline Safety Performance

In most cases, practitioners can use the estimated average annual baseline crash frequency for the design year under no-build conditions as the baseline for alternatives directly. However, there are some cases that may warrant an alternative-specific estimated average annual baseline crash frequency for application in Steps 3 and 4. The following provide examples where an alternative-specific estimated average annual baseline crash frequency may be needed:

- **If an alternative design is expected to have an impact on the forecast traffic volume**. In other words, if the alternative-specific projected traffic volume differs from the design year under no-build conditions, then it is appropriate to develop an alternative-specific baseline. However, while rare, if the change in traffic volume is already accounted for in a CMF the practitioner selects for application or can be accounted for when using the predictive method, then an alternative-specific baseline is not necessary.
- **If the estimated average annual crash frequency in the design year under no-build conditions is not applicable as the baseline for assessing the change in safety performance for the alternative**. For example, if an existing condition is a three-leg intersection with stop control on the minor road approach, and the proposed alternative is a Continuous Green-T intersection, there is not a predictive method or CMF practitioners can use to estimate the change in safety performance. In this case, an intermediate alternative-specific baseline is required for application of the CMF for converting a standard three-leg signalized intersection into a Continuous Green-T intersection.

Estimated Average Annual Baseline Crash Frequency Considerations when AADT Differs from No-Build

When the projected AADT for the design year for an alternative differs from the projected AADT for the design year under no-build conditions, the practitioner should determine whether the methodology used in Step 3 will account for the change in traffic volume. For example, if the practitioner plans to use the predictive method to assess the relative safety effects of geometric changes from the no-build condition to the alternative condition, changes in traffic volume can be incorporated by using the applicable SPFs for each condition. If the practitioner plans to use a CMF to assess the relative safety effects, then the practitioner should determine if the CMF includes the effects of anticipated changes in traffic volume due to the change in geometric condition.

For example, if a practitioner is evaluating a no-build condition of an urban four-lane undivided roadway and the alternative includes a road diet with one through-lane in each direction and a center two-way left-turn lane, a reduction in traffic volume may be anticipated for the alternative condition. A CMF based on a naïve before-after analysis does not consider what the traffic volume in the after period would have been had no geometric changes been implemented. In this case, the CMF includes the effects of the geometric changes as well as the effects of the reduction in traffic volume.

While uncommon, a CMF based on a more rigorous approach, such as an EB before-after methodology, may account for the change in traffic volume when estimating the CMF value. This is the case if the expected crash frequency in the after period was obtained using a projection of the forecast AADT for the no-build condition rather than the observed

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AADT after the project was implemented. If the CMF included the observed AADT in the after period, then the CMF does not include the effects of the change in traffic volume, potentially discounting the overall effects of the treatment. Analysts should seek to understand the applicability of CMFs employed and potential biases in their application, including whether they account for changes in traffic volume due to geometric changes. Additionally, analysts should consider the influence area of the alternative and include a broader analysis area if the alternative will have broad impacts. In this example, if traffic volumes are reduced on the road diet corridor, the analyst should determine if the reduction in traffic volume will have an impact at other locations or on other modes within the corridor (for example will more people walk and bike, potentially impacting non-motorized safety, specifically). The safety analysis should include the study corridor and other locations where volumes are expected to shift to gain a better understanding of the net safety tradeoffs.

When an alternative-specific baseline condition is needed, based on changes in traffic volume, the equations provided in Figure 7 and Figure 8 can still be used for the alternative-specific estimated average annual baseline crash frequency ($N_{baseline,design,alternative}$).

When using Method 1 or Method 2 to estimate average annual baseline crash frequency, the practitioner can directly use the predicted crash frequency for the design year using the alternative-specific AADT($N_{pr,design,alternative}$) for $N_{pr,design}$. When using Method 3, the practitioner can substitute the alternative-specific AADT for the design year ($AADT_{alternative}$) for $AADT_{design}$, where $AADT_{alternative}$.

If no adjustment to the estimated average annual baseline crash frequency in the design year for the no-build condition is needed, then $N_{baseline,design,nobuild}$ is directly used as $N_{baseline,design,alternative}$. The resulting alternative-specific estimated average annual baseline crash frequency in the design year serves as the baseline for calculating the estimated average crash frequency for the alternative after accounting for the safety effects of project improvements.

Estimated Average Annual Baseline Crash Frequency Considerations when Alternative-Specific Baseline Differs from No-Build Baseline

There are many cases where the no-build geometric and operational conditions are not applicable as the baseline for comparative analysis for a project alternative. Most commonly, this occurs when the estimate of relative safety effectiveness of a project alternative requires a baseline that differs from the no-build condition. This is primarily the case when a CMF does not exist, or cannot be directly estimated, for comparing the no-build baseline condition to the alternative-specific condition. When this is the case, the practitioner can estimate a new alternative-specific baseline condition for comparative analysis.

The practitioner should identify the applicable predictive method(s) or applicable CMF(s) for the project alternative and determine the baseline geometric conditions. For example, if the project alternative for an existing three-leg intersection with stop-control on the minor road approach includes a Continuous Green-T intersection, and the preferred CMF only includes conversion from a standard signalized three-leg intersection to a Continuous Green-T intersection, the standard signalized three-leg intersection will serve as the alternative-specific baseline. In this case, the practitioner can use a preferred CMF or predictive method (pseudo-CMF) to evaluate the relative change in safety from the no-build geometric and operational conditions (in this case a three-leg intersection with stop-control on the minor road approach) to the new baseline (three-leg signalized intersection). The resulting alternative-specific estimated average annual baseline crash frequency in the design year ($N_{baseline,design,alternative}$) serves as the baseline for calculating the estimated average annual crash frequency for the alternative after accounting for the safety effects of project improvements (in this case conversion to a Continuous Green-T intersection).

Step 3: Safety Effectiveness of Project Alternatives

Practitioners have two primary methods for estimating the safety effectiveness of project alternatives. The first method is to apply a preferred CMF, or CMFs for multiple treatments, for changes from the no-build condition to the project alternative condition. The second method is to develop a pseudo-CMF by applying the *HSM* predictive method for the no-build and alternative geometric and operational conditions. Each approach includes advantages and disadvantages, including potential biases that should be considered. Practitioners are encouraged to use available resources and information to identify the applicability and limitations of each approach. This memorandum describes several of the more important applications and limitations of each. If no preferred CMF or pseudo-CMF is available for evaluating project alternatives, practitioners could estimate a CMF from safety surrogate measures or choose an alternative approach.

Applying Preferred CMFs for Alternatives

CMFs are the primary method for assessing the relative safety effects of a proposed countermeasure or geometric change. The CMF is a multiplicative factor applied to the baseline crash frequency to estimate the crash frequency given the proposed alternative conditions. The *HSM* Part D contains CMFs for countermeasures and geometric changes on roadway segments, intersections, interchanges, special facilities, and road networks. Additionally, the FHWA CMF Clearinghouse serves as a repository for CMFs from research studies. State agencies have developed their own CMFs as well as identified preferred CMFs from the CMF Clearinghouse for use in alternatives analysis or general safety analysis. Practitioners can apply CMFs for total crash frequency or for specific crash or severity types.

Unlike the AFs estimated as part of the *HSM* predictive method, CMFs for countermeasures or geometric changes are typically estimated in isolation using datasets developed for the specific countermeasure. Therefore, practitioners should use caution when assessing the safety effects of multiple changes when multiple CMFs are required (i.e., the effects of combined changes are not assessed together in a single CMF). NCHRP Report 991: Guidelines for the Development and Application of Crash Modification Factors developed guidance and a procedure for estimating the combined safety effect of two treatments.[19] Agencies generally have policies on combining CMFs, typically acknowledging that no more than two or three CMFs should be considered in combination. When combining CMFs for multiple countermeasures, practitioners should evaluate whether the CMFs apply to the same crash type and severity as well as consider several factors, including the following:

- Countermeasure applicability.
- Countermeasure overlap.
- CMF magnitude.
- CMF value.
- CMF combination method.

It is imperative to understand the context of the CMF to properly apply it for project alternatives analysis. For instance, the CMF may be applicable to a specific facility type (e.g., rural, two-lane highways), crash type (e.g., run-off-road crashes), or crash severity (e.g., fatal and injury crashes). Moreover, CMFs may apply to specific conditions within a facility type (e.g., rural two-lane highways with AADT less than 2,000 vehicles per day). FHWA's CMF Clearinghouse is the primary resource for identifying and contextualizing CMFs.

While CMFs are relatively simple to apply, they alone may not represent the full complexity of a suite of improvements included in a proposed alternative. As such, analysts may choose to apply supplemental approaches to assess the change in safety performance.

Applying the HSM Predictive Method for Alternatives

The *HSM* predictive method has direct applications for estimating the safety effectiveness of geometric changes from the no-build condition to the project alternative condition. According to NCHRP Report 1029, before the prediction models were included in Part C of the *HSM*, the models predicting intersection crashes were calibrated with HSIS data from California and the models predicting segment crashes were calibrated with data from Washington.[5] It was suggested that end users could utilize these recalibrated models to directly compare the expected safety performance of different facility types. A number of calibration efforts conducted by State agencies have shown that the relative safety performance across facility types differs from jurisdiction to jurisdiction. Developing jurisdiction-specific SPFs or calibrating national SPFs provides more reliable estimates of relative safety performance across facility types. It is not advisable, or statistically reliable, to use a jurisdiction-specific or calibrated national SPF for one alternative and an uncalibrated national SPF for another alternative. Instead, practitioners should consider using jurisdiction-specific SPFs or calibrated national SPFs for both alternatives or uncalibrated national SPFs for both alternatives. When applicable, practitioners should prioritize jurisdiction-specific or calibrated national SPFs and AFs for comparison between a no-build condition and an alternative condition.

When using jurisdiction-specific or calibrated national SPFs, practitioners can use pseudo-CMFs, as shown in Figure 14.

$$CMF_{pseudo} = \frac{N_{Alternative}}{N_{NoBuild}}$$

Figure 14. Equation. Calculation of the pseudo-CMF using predicted crash frequency for the alternative and no-build conditions.

Where $N_{Alternative}$ is the predicted crash frequency for the alternative condition and $N_{NoBuild}$ is the predicted crash frequency for the no-build condition, using an absolute comparison. If uncalibrated national SPFs are used, practitioners can use a relative comparison.

Relative Comparison

If applying uncalibrated national SPFs, practitioners can develop relative comparisons assuming the SPFs come from the same set of calibration data (in this case, the *HSM* single-state calibration). Practitioners can use the ratio of predicted crash frequency in the alternative condition to predicted crash frequency in the no-build condition to create a pseudo-CMF. The relative comparison approach requires future no-build and alternative-specific traffic volumes as well as the required input for the SPFs. These inputs may vary as national SPFs contained in the *HSM* have different geometric inputs by facility type.

This approach can be useful as it does not require calibrated or jurisdiction-specific SPFs. As a reminder, by not using jurisdiction-specific SPFs or locally calibrated national SPFs, the practitioner is working under the assumption that the relative relationship calculated by the SPFs is the same for the study area as it was for the *HSM* single-State calibration, which should be documented accordingly.

Absolute Comparison

Practitioners with access to jurisdiction-specific SPFs or locally calibrated national SPFs can perform an absolute comparison of safety performance. As with the relative comparison, agencies collect and enter all relevant data for the SPF(s) to produce predicted crashes under the no-build and alternative condition. If the baseline no-build average annual crash frequency is calculated using predicted crash frequency, then the change in safety performance is simply reflected by the predicted crash frequency under the alternative condition. If baseline no build average annual crash frequency is calculated using observed or expected crash frequency, then the ratio of alternative predicted crash frequency to no-build predicted crash frequency can be used as a pseudo-CMF to calculate the expected change in safety performance.

The absolute comparison approach, calculated using jurisdiction-specific SPFs or locally-calibrated national SPFs, is the preferred approach for estimating the change in safety effectiveness using the predictive method. Jurisdiction-specific or locally calibrated SPFs increase the reliability in the relative safety performance.

Pseudo-CMF Calculation

If a project alternative results in no change in traffic volume or facility type, *HSM* SPF AFs can be compared directly for geometric changes of interest. For example, the AF for the alternative condition divided by the AF for the no-build condition directly provides the relative safety effectiveness of the geometric change. Figure 15 provides the generalized approach for estimating a pseudo-CMF for multiple geometric changes within a given facility type using the *HSM* predictive method. Note this approach focuses on SPF AFs and further guidance is provided below in regard to estimating the effect of multiple countermeasures using CMFs.

$$CMF_{PM1} = \frac{AF_{i,A} \times \ldots \times AF_{n,A}}{AF_{i,NB} \times \ldots \times AF_{n,NB}}$$

Figure 15. Equation. Pseudo-CMF for multiple geometric changes within a given facility type.

Where:

CMF_{PM1} = pseudo-CMF based on the predictive method within a given facility type for geometric changes.

$AF_{i,A}$ = SPF AF for geometric elements i to n in the alternative condition.

$AF_{i,NB}$ = SPF AF for geometric elements i to n in the no-build condition.

For this analytical approach, the practitioner only needs to include geometric elements changed between the no-build and alternative conditions.

If a project alternative results in a change in traffic volume, the relative effects of the change in traffic volume can be incorporated into the CMF. Figure 16 provides the generalized approach for estimating a pseudo-CMF for multiple geometric changes as well as a change in traffic volume within a given facility type using the predictive method.

$$CMF_{PM2} = \frac{AADT_{A}^{\beta} \times AF_{i,A} \times \ldots \times AF_{n,A}}{AADT_{NB}^{\beta} \times AF_{i,NB} \times \ldots \times AF_{n,NB}}$$

Figure 16. Equation. Pseudo-CMF for multiple geometric changes and traffic volume changes within a given facility type.

Where:

CMF_{PM2} = pseudo-CMF based on the predictive method within a given facility type for geometric and traffic volume changes.

$AADT_{A}^{\beta}$ = traffic volume for the alternative condition to the power of beta.

$AADT_{NB}^{\beta}$ = traffic volume for the no-build condition to the power of beta.

Beta is the coefficient assigned to the effects of traffic volume for the given SPF in the applicable predictive method. Table 8 provides the values of beta for each facility type included in the *HSM*. For facility types with different SPFs by crash severity and number of vehicles, pseudo-CMFs can be calculated for each SPF application.

If a project alternative results in a change in facility type, the equation provided in Figure 14 can be used directly. The SPF and associated AFs are applied for the no-build condition to arrive at predicted crash frequency. Similarly, the applicable SPF and associated AFs are applied for the alternative condition to arrive at predicted crash frequency.

The primary advantages to the pseudo-CMF approach include the following:

- This approach allows practitioners to estimate the expected change in safety performance for an alternative when a specific CMF does not exist.
- This approach provides additional flexibility beyond typical CMF applications. CMFs typically indicate the change in safety performance at an aggregate level (e.g., widen from two-lanes to four-lanes). The pseudo-CMF method allows the practitioner to consider additional safety effects of geometric changes, such as widening from two-lanes to four-lanes along with a wider median and inside shoulders versus a narrower median and inside shoulders.

The primary limitation of this approach is the underlying assumption that the predictive method for different facility types can be directly compared, whether the SPFs are calibrated to local conditions or if the SPFs are uncalibrated national SPFs considering an HSM single-state calibration.

Applying Surrogate Safety Measures

If the practitioner cannot obtain applicable CMFs, it may be possible to estimate a CMF from surrogate measures or other means as practical. For example, traffic conflicts have been shown to serve as a safety surrogate and have shown promise when compared to estimated CMFs for different treatment scenarios.[20,21]

Step 4: Project Alternative Estimated Average Annual Crash Frequency

After establishing the CMF in Step 3, the practitioner can calculate the project alternative estimated average annual crash frequency using Figure 17.

$$N_{estimated,design,alternative} = N_{baseline,design,alternative} \times CMF_{alternative}$$

Figure 17. Equation. Calculating estimated average annual crash frequency for the alternative.

Where:

$N_{estimated,design,alternative}$ = the estimated average annual crash frequency in the design year for the alternative.

$CMF_{alternative}$ = the CMF applied for the countermeasure or geometric changes in the alternative.

Step 5: Change in Estimated Average Annual Crash Frequency

The final step includes calculating the change in estimated average annual crash frequency from the baseline estimated average annual crash frequency in the design year under no-build conditions to the estimated average annual crash frequency in the design year for the alternative using Figure 18.

$$N_{change,design,alternative} = N_{baseline,design} - N_{estimated,design,alterantive}$$

Figure 18. Equation. Calculation of expected change in estimated average annual crash frequency for the alternative.

Where:

$N_{change,design,alternative}$ = the expected change in estimated average annual crash frequency in the design year for the alternative.

It is important to note that the baseline estimated average annual crash frequency in the design year under no-build conditions from Step 1 is used for this calculation regardless of whether an adjusted value is obtained for alternative-specific analysis in Step 2.

Case Studies Revisited

This section revisits the WisDOT and *HSM User Guide* Examples provided in the Additional Considerations section, using the recommended five-step process to evaluate project alternatives. The underlying data are provided in the Additional Considerations section, specifically under each example. These case study updates identify how the analysis would be conducted using the recommended approach and provide a comparison to the results in the Additional Considerations section.

Wisconsin DOT Example: Four-Leg Roundabout

Overview

The WisDOT example included a comparison of predicted crash frequency for the no-build condition (rural, 4-leg, stop control) to the predicted crash frequency for the alternative condition (rural, 4-leg, single lane roundabout) based on the *HSM* recommendation. This extension of the case study shows how this evaluation can be conducted using the recommended project alternative analysis approach. The methodology section applies the five-step process.

Methodology

Step 1: Future No-Build Safety Performance

Since WisDOT had reliable crash history data and a calibrated SPF for the no-build condition, Method 1 is applicable for estimating the future no-build baseline average annual crash frequency, based on expected crash frequency. Using a 10-year horizon period, WisDOT estimated the expected crash frequency for the baseline no-build condition. The estimated baseline average annual crash frequency, based on expected crash frequency consists of the following:

- 3.03 fatal and injury crashes/year + 2.17 property damage only crashes/year = 5.20 total crashes/year.

Over ten years, the following is calculated:

- 3.03 × 10 = 30.3 fatal and injury crashes + 2.17 × 10 = 21.7 property damage only crashes = 52.0 total crashes.

Step 2: Alternative-Specific Baseline Safety Performance

There is no expected adjustments needed to refine the baseline estimated average crash frequency for the roundabout alternative since no difference in traffic volume is anticipated between the alternatives and since an alternative-specific baseline is not needed. Therefore, the result of Step 2 remains the same as Step 1.

Step 3: Safety Effectiveness of Project Alternatives

In this case, WisDOT has a calibrated SPF for each alternative; therefore, the calibrated SPFs can both be used to develop a pseudo-CMF using the equation provided as Figure 14. Based on the 10-year horizon analysis, WisDOT predicted the following:

- 0.64 fatal and injury crashes/year + 1.43 property damage only crashes/year = 2.07 total crashes/year for the no-build condition.
- 0.11 fatal and injury crashes/year + 1.54 property damage only crashes/year = 1.66 total crashes/year for the roundabout condition.

The resulting pseudo-CMFs are calculated as the following:

- Fatal and injury crashes: 0.11 ÷ 0.64 = 0.17.

- Property damage only crashes: 1.54 ÷ 1.43 = 1.08.

To support economic evaluation, the pseudo-CMF for fatal and injury crashes can be broken down further by specific injury severity level.

Step 4: Project Alternative Estimated Average Annual Crash Frequency

The estimated average annual crash frequency for the roundabout alternative can be calculated by severity using the equation provided in Figure 18. Estimated average annual fatal and injury crash frequency is calculated as the following:

- Average annual fatal and injury crashes/year = 3.03 fatal and injury crashes/year × 0.17 = 0.52 average annual fatal and injury crashes/year.
- Average annual property damage only crashes/year = 2.17 property damage only crashes/year × 1.08 = 2.34 average annual property damage only crashes.

Over the 10-year horizon period, this results in the following for the roundabout alternative:

- Fatal and injury crashes = 0.52 average annual crashes/year × 10 years = 5.20 estimated fatal and injury crashes.
- Property damage only crashes = 2.34 average annual crashes/year × 10 years = 23.40 estimated average annual property damage only crashes.

Step 5: Change in Estimated Average Annual Crash Frequency

The estimated change in average annual crash frequency can be calculated by severity using the equation provided in Figure 18. The no-build condition results in an expected 30.3 fatal and injury crashes over ten years while the roundabout condition results in an expected 5.2 fatal and injury crashes over ten years. The roundabout alternative results in the following reduction in expected fatal and injury crashes:

- 30.3 fatal and injury crashes for the no-build condition – 5.20 fatal and injury crashes for the roundabout alternative = 25.1 reduction in fatal and injury crashes over ten years.

The roundabout alternative results in the following increase in expected property damage only crashes:

- 21.7 property damage only crashes for the no-build condition – 23.4 property damage only crashes for the roundabout alternative = 1.7 (increase) in property damage only crashes over ten years.

Discussion

The initial case study example presented for this project alternatives analysis focused on a comparison of predicted crash frequency for the rural four-leg stop-control intersection no-build to a rural four-leg roundabout intersection alternative. The predicted crash frequency-based analysis suggested the roundabout alternative would result in the following:

- A reduction of 5.3 fatal and injury crashes over ten years.
- An increase of 1.1 property damage only crashes over ten years.

The alternative analysis conducted under the approach recommended in this memo suggested the roundabout alternative would result in the following:

- A reduction of 25.1 fatal and injury crashes over ten years.
- An increase of 1.7 property damage only crashes over ten years.

The recommended project alternatives analysis approach included in this memorandum provides practitioners with the opportunity to incorporate historic crash data where this information would have been discarded previously. In this specific example, the intersection had a long-term crash history that suggested a higher observed crash frequency than would be predicted for similar sites, when accounting for regression to the mean. Accounting for observed crash history in the no-build baseline allows the practitioners to better capture the potential benefits of the alternative, in this case installing a roundabout. Furthermore, the observed crash history suggested a higher-than-average proportion of crashes that were fatal and injury, further increasing the potential benefits of this project. The results of the analysis can be carried forward into economic analysis, which would be more likely to show that this project is economically justified.

Highway Safety Manual User Guide Example: EB Method Application for Alternatives within a Facility Type

Overview

The *HSM User Guide* case study provided an example project alternatives analysis for rural two-lane highway segments where two alternatives were considered to improve safety performance. For the three study segments, Alternative 1 consisted of shoulder widening from 1 foot to 6 feet in each direction and Alternative 2 consistent of the same shoulder widening, plus installing roadway lighting, improving roadside hazard rating, and implementing automated speed enforcement. The case study example included a comparison of expected crash frequency from the no-build to the alternative conditions. For this case study, the observed crash history was included to calculate expected crash frequency for each alternative in addition to the no-build. This extension of the case study shows how this evaluation can be conducted using the recommended project alternative analysis approach. The methodology section applies the five-step process.

Methodology

Step 1: Future No-Build Safety Performance

Since this example had reliable crash history data and a locally calibrated *HSM* rural-two lane highway SPF for the no-build condition, Method 1 is applicable for estimating the future no-build average annual crash frequency. Table 6 provided the predicted, observed, and expected crash frequency for each segment considering the existing conditions. The following provide the expected average annual crash frequencies for segments in the no-build condition:

- Segment 1: 9.99 expected crashes/year.
- Segment 2: 34.32 expected crashes/year.
- Segment 3: 10.54 expected crashes/year.

These values serve as the estimated average annual baseline crash frequency for the no-build alternative since there is not an anticipated change in traffic volume in the future.

Step 2: Alternative-Specific Baseline Safety Performance

There are no expected adjustments needed to refine the estimated average annual baseline crash frequency for either alternative since no change in traffic volume is anticipated and since an alternative-specific baseline is not needed. Therefore, the result of Step 2 remains the same as Step 1.

Step 3: Safety Effectiveness of Project Alternatives

In this case, the proposed enhancements do not result in a change in facility type for either alternative. Therefore, the equation shown as Figure 15 can be used to evaluate the pseudo-CMF for the changes proposed in each alternative:

1. Shoulder widening from 1 foot to 6 feet in each direction.
2. Shoulder widening from 1 foot to 6 feet in each direction, installing roadway lighting, improving roadside hazard rating, and implementing automated speed enforcement.

For Alternative 1, the pseudo-CMF is calculated based on the AF for shoulder width.

- The AF for the baseline condition (1-foot shoulder) is 1.23 based on the AADT and assuming the *HSM* default of 57.4 percent of crashes is applicable.
- The AF for the Alternative 1 condition is 1.00.
- Therefore, the pseudo-CMF for Alternative 1 is calculated as 1.00 ÷ 1.23 = 0.813.

Table 9 provides the SPF AFs for the geometric changes from the no-build condition to Alternative 2.

- The pseudo-CMF for Alternative 2 is calculated as (1.00 x 0.92 x 0.93 x 1.00) ÷ (1.23 x 1.00 x 1.00 x 1.14) = 0.610.

Table 9. Adjustment factors for no-build, Alternative 1, and Alternative 2.

Geometric Feature	No-Build	Alternative 2
Shoulder width	1.23	1.00
Roadway lighting	1.00	0.92
Automated speed enforcement	1.00	0.93
Roadside hazard rating	1.14	1.00

The pseudo-CMFs calculated for Alternative 1 and Alternative 2 are applicable for all three segments.

Step 4: Project Alternative Estimated Average Annual Crash Frequency

Table 10 provides the expected average annual crash frequency for each alternative, including the no-build condition, for each segment based on the results of Steps 1 and 3. The alternative-specific estimated average annual crash frequency is based on the no-build expected crash frequency multiplied by the alternative-specific pseudo-CMF.

Table 10. Alternative expected average annual crash frequency.

Segment	No-Build	Alternative 1	Alternative 2
1	9.99	8.12	6.09
2	34.32	27.90	20.94
3	10.54	8.57	6.43
Total	54.85	44.59	33.46

Step 5: Change in Estimated Average Annual Crash Frequency

The estimated change in average annual crash frequency can be directly calculated for each alternative from the no-build condition.

- The change in average annual crash frequency for Alternative 1 is 54.85 – 44.59 =10.26 crashes/year reduced.
- The change in average annual crash frequency for Alternative 2 is 54.85 – 33.46 = 21.39 crashes/year reduced.

The change in crash frequency can be further broken down by severity level for project-level economic evaluation.

Discussion

The case study provided in the *HSM User Guide* quantified the change in safety based on expected crash frequency for each alternative. The expected crash frequency for each alternative was the following:

- No-build condition: 54.85 crashes/year.
- Alternative 1: 52.91 crashes/year.
- Alternative 2: 49.75 crashes/year.

The change in estimated average annual crash frequency was calculated as the following:

- Alternative 1: 54.85 – 52.91 = 1.94 crashes/year reduced.
- Alternative 2: 54.85 – 49.75 = 5.10 crashes/year reduced.

Comparing expected crash frequency relies on the assumption that the historic crash data remain valid for the alternative conditions and assumes that the portion of expected crash frequency relying on historic crash data will remain unchanged. This results in undervaluing the effects of the geometric changes, particularly when applying the predictive method. Had the analyst first developed the baseline estimated average annual crash frequency and then directly applied a pseudo-CMF for the treatment, the historic crash data would not be captured in this manner. This method is consistent with the application of preferred CMFs, which are not subject to the influence of historic crash data. Applying the historic crash data to the new condition biases the estimate by assuming the treatment would have no effect on the historic crash data. Therefore, the recommended approach in this memo provides a consistent application approach for applying preferred CMFs and applying the predictive method through pseudo-CMFs that should not undervalue these effects.

Conclusions and Recommendations

The primary objectives of this technical memorandum include:

1. Exploring a combined predictive method that accounts for a project alternatives analysis with and without an EB adjustment.
2. Investigating the appropriate traffic volumes to use in alternatives analysis.
3. Evaluating the need for calibrating prediction models.

A review of the *HSM*, literature, and current State practices uncovered several inconsistencies in practitioner application of the *HSM* predictive method for alternatives analysis, identified gaps in knowledge surrounding the primary objectives of this memorandum, and identified a process already conceived to overcome many of these issues.

A key takeaway of the review was that the first edition of the *HSM* focuses on straightforward applications of the predictive method and the EB method; however, the *HSM* Part C provides no examples and little context on how to handle project alternative analysis. Developers can improve the *HSM* by providing more context and examples on evaluating project alternatives as well as providing specific guidance on the process for alternative analysis. Examples in the *HSM* and *HSM User Guide* are overly simplistic and skip over key steps practitioners struggle with. For example, the *HSM* suggests practitioners use projected traffic volumes but commonly assumes traffic volumes do not change for future periods or between alternatives.

The project team provided a practical framework for conducting alternatives analysis based on the process outlined in the MassDOT *Safety Alternatives Analysis Guide*. This framework focuses on establishing the estimated baseline average annual crash frequency for the future no-build condition and assessing the expected relative change in safety performance due to a project alternative. Step 1 identifies three common methods practitioners use for estimating the baseline average crash frequency, provided in order of reliability according to the *HSM*:

1. Expected crash frequency calculated using the EB method.
2. Predicted crash frequency calculated using a jurisdiction-specific or locally calibrated national SPF.
3. Observed crash frequency.

In this manner, the framework encourages practitioners to establish the estimated baseline average annual crash frequency using the most reliable method for which data are available. This allows practitioners to include historic crash data for a specific project location regardless of the analysis methodology (preferred CMF application or pseudo-CMF development using the HSM predictive method) when assessing the safety effectiveness of proposed alternative improvements and regardless of whether the improvements result in a change in facility type.

More research is needed to verify the extent to which local calibration of national SPFs overcomes potential biases when developing pseudo-CMFs based on the predictive method. Additionally, more research is needed to assess how well the preferred CMF method and pseudo-CMF method perform during alternatives analysis compared to actual project results after implementation. Each method has advantages and limitations, and future research would help to address when each method should be prioritized for project alternatives analysis.

Additionally, the recommended framework presented in this memorandum is sensitive to expected changes in traffic volumes and acknowledges potential biases when incorrectly applying preferred CMFs or pseudo-CMFs based on the *HSM* predictive method without accounting for the change in traffic volume or accounting for the change in traffic volume if it is already included in the CMF being applied. While not addressed here directly, practitioners should consider the overall study area when assessing alternative safety performance if a change in traffic volume is expected. An increase or decrease in traffic volume at the project location may be associated with a similar decrease or increase at nearby locations due to traffic diversion. Alternatively, changes in traffic volume may come from induced demand (for example when widening a roadway to include more travel lanes) or come from a mode shift (for example a road diet adding bicycle lanes may decrease vehicular traffic but increase bicycle traffic). Ignoring these tradeoffs will create a bias in the assessment of the safety effects of a project alternative.

The proposed framework addresses the need for developing jurisdiction-specific SPFs or calibrating national SPFs to local conditions. When establishing the baseline average annual crash frequency for the future no-build condition in Step 1, a jurisdiction-specific SPF or locally calibrated SPF is recommended. When estimating the safety performance of project alternatives in Step 3, practitioners can use jurisdiction-specific SPFs, locally calibrated SPFs, or uncalibrated SPFs. To increase reliability, practitioners should prioritize using jurisdiction-specific SPFs or locally calibrated SPFs. However, there are a few key components users should consider, including the following:

1. Practitioners should be consistent when developing pseudo-CMFs from locally calibrated or uncalibrated national SPFs for estimating the relative safety performance for the alternative condition relative to the baseline no-build condition when the project alternative results in a change in facility type. Practitioners should not use a jurisdiction-specific or locally calibrated SPF for one condition and an uncalibrated SPF for the second condition. This will introduce a bias in the estimate of the relative safety effectiveness.
2. Local calibration factors do not matter when a facility type does not change. Within a facility type, the estimated effects of changes to site geometrics can be directly assessed by using the ratio of the adjustment factors from the alternative condition to the baseline condition.
3. If using uncalibrated SPFs across a change in facility type, the practitioner is relying on the *HSM* single-State calibration to assess the relative safety performance. Inherently, this assumption means the jurisdiction would have the same calibration factor for both facility types had the jurisdiction calibrated the SPFs. When possible, practitioners should use jurisdiction-specific or locally calibrated SPFs to increase the reliability of the relative change in safety performance. If the uncalibrated *HSM* SPFs are used, practitioners should document this limitation.

Finally, the proposed framework in this memorandum approaches project alternatives analysis differently than the *HSM User Guide* when the facility type remains unchanged relative to the no-build condition. The *HSM User Guide* recommends applying the site-specific crash history as part of the EB adjustment for both the no-build and alternative conditions. That approach assumes the observed crash frequency continues to hold true for the alternative condition, introducing bias to the estimate of expected crash frequency for the alternative condition The framework proposed in this memorandum uses the EB method solely to establish the baseline measure for the no-build condition and then adapts the predictive method to develop a pseudo-CMF for application with the baseline measure. By doing so, this methodology allows the practitioner to incorporate site-specific crash history, when available, rather than relying solely on predicted crash frequency for projects resulting in a change in facility type. Furthermore, this methodology includes the full safety benefit of the proposed change rather than only including the safety benefit for the predicted crash frequency portion of the EB method. Both of these concepts are imperative to provide agencies with a more reliable representation of the baseline no-build conditions and estimated safety benefit for project alternatives for a wider range of project applications.

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Appendix: Literature and State Practices Review

Introduction

Part C of the *Highway Safety Manual* (*HSM*) provides a predictive method that can be used to estimate crashes by severity and crash type based on different road conditions such as facility type, site type, and geometric conditions. The predictive method allows users to estimate crashes using safety performance functions (SPFs), which are regression models used to predict crashes for base conditions and are a function of a few variables shown to have a significant relationship with crashes, such as average annual daily traffic (AADT). SPFs can be used to estimate crashes for existing conditions, modifications to an existing roadway, or a proposed new roadway. Advantages of the predictive method include (AASHTO, 2010b):

- Addresses regression-to-the-mean bias which can occur when using short-term average crash frequency in the Empirical Bayes (EB) method.
- Reduces reliance on observed crash data.
- Uses negative binomial distribution which accounts for crash data variability.
- Provides a method to estimate crashes for sites or facilities that are unconstructed or have been recently constructed.

To help states and jurisdictions with implementation of the *HSM* predictive method (e.g., for alternatives analysis), there is a need to explore the following topics, which this literature review will discuss:

- Explore a combined predictive method that accounts for an alternatives analysis with and without an EB adjustment.
- Investigate the appropriate traffic volume to use in alternatives analysis.
- Evaluate the need for calibrating prediction models.

Crash frequency can vary by state and jurisdiction due to specific conditions in the different areas. For example, some states receive more snowfall than others which can result in more snow-related crashes. When using the predictive method, the *HSM* recommends calibrating SPFs to local conditions using calibration factors for more reliable crash estimates (AASHTO, 2010). Calibration factors can be developed for an entire state or by jurisdiction using a procedure outlined in the *HSM* which uses state or jurisdiction specific crash data. Many States have developed calibration factors to use with the SPFs in the *HSM*, and others have developed jurisdiction-specific SPFs. The Federal Highway Administration (FHWA) CMF Clearinghouse provides a summary of State calibration factors and SPFs. The *HSM* also notes that if uncalibrated models are used, only the percent differences in prediction should be used for comparison.

The EB method can be used to combine the predicted and observed crashes through weighting to calculate the expected crashes. The EB method, if properly used, can produce more reliable crash estimates (Persaud and Lyon, 2007) because both the predicted and observed crashes are included in the analysis. However, the EB method can only be used if both observed and predicted crashes are available. Additionally, the *HSM* notes that the EB method cannot be used for projects where a substantial proportion of the alignment is new because past crash experience is not necessarily indicative of future crash experience with proposed substantial geometric changes (AASHTO, 2010b). Other changes that would result in not being able to use the EB method could include alterations such as changes in basic facility types where historical crash history would not be applicable.

In the predictive method, the EB method occurs after applying calibration factors to the predicted crashes from SPFs. However, the *HSM* does not mention if the SPFs used for predicting crashes in the EB method need to be calibrated. For example, some jurisdictions may want to use the EB method but do not have jurisdiction-specific calibration factors. The NCHRP 17-50 *HSM User Guide* (Kolody et al., 2014); however, indicates users can make relative assessments within the same facility and control type. The User Guide expressly states that the output from an *HSM* SPF cannot be used to describe an actual prediction (Kolody et al., 2014).

While the *HSM* outlines and describes how to use the predictive method, implementation of the predictive method has been difficult for some jurisdictions. Some challenges identified by these jurisdictions include:

- Having an insufficient sample size for developing jurisdiction-specific SPFs or calibration factors.
- Inability to use the EB method if observed crashes are not available and inability to compare expected crashes for all alternatives in an alternatives analysis.
- Lack of clarity if uncalibrated predictive models can be used to compare alternatives when the alternatives include different facility types and if uncalibrated predictive models can be used in the EB method.
- Deciding if current or forecasted AADT should be used and if different AADT values should be used for different alternatives if road changes substantially increase capacity.

The following sections explore a combined predicted method for an alternatives analysis, investigate appropriate traffic volume to use in an alternatives analysis, and evaluate the need for calibrating crash prediction models.

Predictive Method for Alternatives Analysis with and without Using the EB Method

It is generally desirable to apply the EB method during the alternatives analysis process to address potential site selection bias; however, practitioners may encounter scenarios where the EB method cannot or should not be used for one or more alternatives. An example scenario where the EB method could be used includes an alternative analysis for minor improvements, which do not result in the change of a facility type. A second scenario where the EB method can be used in alternative analysis is when the analyst evaluates alternatives to the no-build scenario using crash modification factors (CMFs).

It is common to analyze alternative scenarios that result in a change in facility type, such as the introduction of a leg to an intersection or change in traffic control. When a proposed change results in a change in facility type, the *HSM* recommends the EB method should not be used because the crash history needed for the analysis is not relevant. The Wisconsin Department of Transportation (DOT) also clarifies that the EB method cannot be used when the number of through lanes changes on a long segment and it cannot be used when evaluating other major geometric improvements (Wisconsin DOT, 2022).

As noted previously, when conducting analysis of multiple alternatives, one or more alternatives may result in a change in facility type. In this case, the EB method may not be applicable for all alternatives. According to the *HSM* (AASHTO, 2010b), "If alternative improvements are being evaluated for a given project and the EB method is being considered, then the EB method will need to be consistently applied to all alternatives being evaluated." The *HSM* goes on to say that the EB method should not be used for any alternatives if it cannot be used for all alternatives. In cases where the EB method cannot be used to compare alternatives, the *HSM* recommends using the predicted method without the EB Method to compare alternatives. Comparing alternatives with and without the EB method could introduce bias between the alternatives (AASHTO, 2010b; Bonneson et al., 2021).

The important distinction in this case is that this *HSM* reference is distinctly associated with using the EB method in combination or application of the Part C predictive method. The *HSM* does not distinguish or provide guidance when one alternative uses the predictive method (and by extension the EB method) and other alternatives are evaluated using CMFs. Further clarification is needed for guidance on when the EB method should be used in relation to the predictive method and/or through use of crash modification factors when evaluating alternatives that result in a change in facility type.

Practice and Examples

In the Massachusetts Department of Transportation (MassDOT) *Safety Alternatives Analysis Guide* (MassDOT, 2021), MassDOT provides guidance for determining when expected, predicted, or observed crash frequency should be used to establish the no-build baseline crash frequency for alternative analysis, shown in Figure 19. MassDOT recommends using expected crash frequency when a Massachusetts calibrated SPF is available and observed crash data are available and reliable. If observed crashes are not available and predicted crashes cannot be calculated, MassDOT (2021) mentions that other options should be discussed with the MassDOT Safety Group. MassDOT specifies using CMFs from their State-Preferred CMF list to evaluate alternatives against the baseline. If no CMF is available, MassDOT's guidance suggests working with MassDOT to determine a value for use. While not specifically stated, the predictive method is one approach used to determine a value if no direct CMF is available.

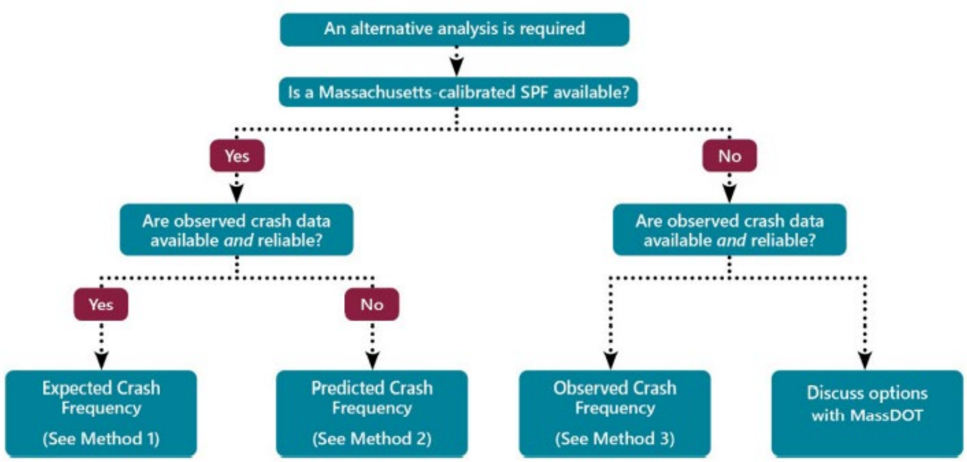

Figure 19. Flow chart for determining if expected or predicted crash frequency should be used in an alternatives analysis (Figure 3; MassDOT, 2021).

While the *HSM* does not mention if predicted crashes for the EB analysis need to be calibrated, Oregon Department of Transportation (DOT) noted in their *Analysis Procedures Manual* (Oregon DOT, 2022) that "the EB method requires a calibrated prediction model." Oregon DOT (2022) continued to describe when the EB method can and cannot be used, shown in Table 11. When the expected crashes cannot be calculated for all alternatives, Oregon DOT (2022) notes to use predicted crashes. Oregon DOT further suggests when an Oregon-specific calibration factor cannot be used, the analyst should document relative comparisons such as net difference in predicted crashes or percent change in predicted crashes. Further, Oregon DOT requires uncalibrated predicted crash frequency to be identified as such and reported separately from calibrated predicted crashes or expected crashes.

Table 11. When EB method can and cannot be used (table from Oregon DOT (2022)).

EB Method Can be Used	**EB Method Cannot be Used**
• Existing Conditions • Future conditions with traffic volume changes only • Future conditions with minor geometric changes (i.e., wider shoulders) or addition of turn or passing lanes.	• Uncalibrated predictive models, such as ISATe • Where no crash data from any time period are available • Future conditions with entirely new roadways where none existed before • Future conditions modifying existing roadways that include major alignment changes, the addition of through lanes, or a change in traffic control devices

The *HSM User Guide* (Kolody, et al., 2014) includes an example alternatives analysis for a rural highway that compares a no build scenario with three alternatives. One of the alternatives included changing the existing two-lane roadway to a four-lane divided facility. The example states that the EB method could not be used due to the substantial change in the roadway (two-lane to four-lane conversion), so all alternatives were compared using predicted average crash frequency. Kolody et al. (2014) also noted that "if the conversion from two-lane to four-lane divided was done in a short section of the corridor [less than 2 miles] to allow for more passing opportunities, then the expected number of crashes could have been calculated."

Persaud et al. (2001 and 2003) incorporated the EB method into observational before-after studies and analyzed alternatives with and without EB method. Persaud et al. (2001) compared the reduction in expected number of crashes due to a roundabout conversion and calculated the treatment's index of effectiveness. The study used the ratio of predicted crashes during the after period if a roundabout conversion did not occur and the predicted crashes in the before period, and then multiplied the ratio by the expected number of crashes during the before period to calculate the number of crashes that would have occurred in the after period if the conversion did not occur. Persaud et al. (2001) also calculated the index of effectiveness, which compared the number of crashes occurring after the roundabout conversion to the expected number of crashes if the conversion did not take place.

Similarly, Persaud et al. (2003) used the EB method as a part of a before-and-after study to calculate the safety impacts of converting a stop-controlled intersection to a signalized intersection, which typically should not use the EB method due to the substantial change at the intersection. The study proposed the following approach for analysis:

1. Estimate the number of crashes that would be expected if the intersection was stop-controlled. Prediction models for stop-controlled intersections would be used here in the EB method along with expected traffic volumes and recent crash counts.
2. Estimate the number of crashes that would be predicted if the intersection was signalized. Prediction models for signalized intersections would be used here along with expected traffic volumes.

3. Estimate the expected change in safety as the difference between estimates 1 and 2.

Persaud et al. (2003) also calibrated the prediction models and recommended recalibrating the models for specific jurisdiction and each year of analysis.

Rodegerdts et al. (2010) used a similar method as Persaud et al. (2003) in an example in their report to calculate the change in crashes due to conversion from two-way stop-controlled intersection to a roundabout. They first calculated the number of expected crashes using the EB method if the intersection was not converted to a roundabout. The study then calculated the predicted number of crashes for the roundabout using the roundabout crash prediction models developed as a part of the study. The change in crashes due to the intersection conversion was calculated using the expected crashes from the stop-controlled intersection and the predicted crashes for the roundabout. They also illustrated an alternative method for cases where a roundabout crash prediction model may not be available. In this, a CMF for conversion from stop controlled to roundabout operation is applied to the number of expected crashes using the EB method if the intersection was not converted to a roundabout.

Appropriate Traffic Volume in Alternatives Analysis

As noted in the previous section, a common application of the predictive method is for users to assess build and no-build alternatives in predictive safety analysis. Typically, alternatives analysis considers a horizon year analysis (typically 20 years or the service life of the changes proposed). The *HSM* recommends forecasting AADT for future analysis periods using planning and forecasting models or assuming AADT will remain constant during the analysis period. As displayed in Figure 20, SPFs universally demonstrate that predicted crash frequency increases as traffic volume increases. Therefore, any scenario expected to increase traffic volume over another scenario should be expected to predict a higher crash frequency, given all other features are equal.

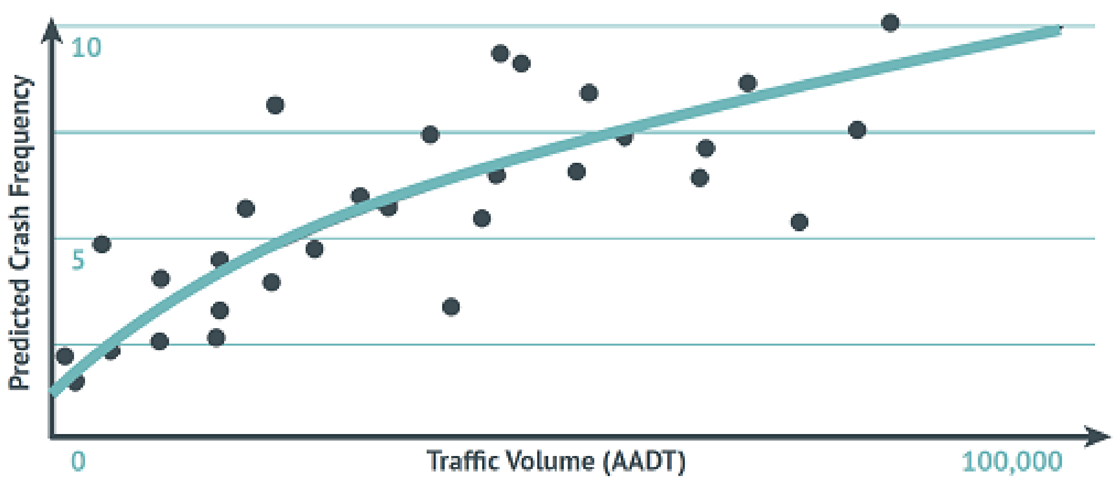

Figure 20. Example graph of an SPF (Figure 4-3; Carter and Srinivasan, 2017).

Practitioners have struggled to determine if alternatives should be analyzed assuming the same traffic volume for each alternative, or if alternative-specific build volumes should be used. This is especially true when traffic forecasts can be unreliable or when considering if an alternative should be penalized for providing greater capacity for increased demand.

The Florida Department of Transportation (FDOT*) Safety Analysis Guidebook for Project Development and Environment Studies* (FDOT, 2019) makes an important distinction when comparing alternatives that result in a change in traffic volume. For example, a no-build analysis of an existing two-lane highway indicated a 2042 volume of 17,800 vehicles per day and a build analysis of a proposed four-lane highway indicated a 2042 volume 20,000 vehicles per day. The FDOT guide notes that it is important to consider the forecasted difference in exposure in the design year conditions as this would result in no-build crashes being under-represented by 2,200 daily vehicles that would be elsewhere on the vehicle network. Therefore, the guide notes, the crash reduction benefit of the build condition may be greater than actually shown.

Practice and Examples

As previously discussed, the *HSM User Guide* (Kolody, et al., 2014) includes an example alternatives analysis on a rural highway. One of the alternatives includes a substantial change to the roadway, which entails changing the existing rural two-lane road to a four-lane divided road. When predicting crashes for all alternatives, the example uses the same future year AADT for all alternatives, even though there were substantial changes in one of the alternatives (two-lane to four-lane conversion). As a result of using the same AADT for the alternatives analysis, the four-lane divided road experienced fewer crashes than the no build alternative and alternatives with less significant changes.

An Introduction to the Highway Safety Manual (AASHTO, 2010a) also includes an alternatives analysis example; however, AADT is treated differently than in the *HSM User Guide* example. The example in AASHTO (2010a) includes a no-build option and two alternatives. Alternative 1 is a mix of a three- and five- lane road and Alternative 2 is a five-lane road. To calculate the predicted crash frequency for each alternative, the example uses the same AADT for the no-build option and Alternative 1 but used a higher forecasted AADT for Alternative 2, which results in a higher predicted crash frequency for Alternative 2, as shown in Table 12. Forecasting higher traffic volume for a facility will typically show higher predicted crash frequency unless the increase in AADT is offset by a reduction due to the alternative alignment or treatment.

Table 12. Forecasted crash frequency for alternatives analysis example (Table 6; AASHTO, 2010a).

	2035 Forecast Crash Frequency (Crashes/Year)								
	No-Build			Alternative 1 (Mix 3- and 5-Lane)			Alternative 2 (5-Lane)		
Intersection/ Segment[1]	Facility	AADT[2]	Crashes/ Year	Facility	AADT[2]	Crashes/ Year	Facility	AADT[2]	Crashes/ Year
Int: Main & Oak	Stop	35,730/ 3,650	3.26	Roundabout	35,730/ 3,650	1.67	Signal	39,080/ 5,280	6.93
Seg: Oak to 3rd St.	3-Lane	34,580	8.30	3-Lane	34,580	5.74	5-Lane	38,150	9.32
Int: Main & 3rd	Signal	33,910/ 25,790	6.63	Roundabout	33,910/ 25,790	3.43	Roundabout	36,900/ 29,400	3.86
Seg: 3rd to 5th	5-Lane	33,270	5.05	5-Lane	33,270	1.51	5-Lane	37,310	1.74
Int: Main & 5th	Signal	33,200/ 5,940	6.40	Roundabout	33,200/ 5,940	3.32	Roundabout	37,860/ 7,230	3.99
Total Prediction	29.6 crashes/year			15.7 crashes/year			25.8 crashes/year		
Change Relative to No-Build				47% Decrease			13% Decrease[3]		

FHWA (2019) developed the *Incorporating Data-Driven Safety Analysis in Traffic Impact Analysis: A How-To Guide* to support agencies making trade-off decisions for quantifying safety impacts with and without development. This guide provides a practical demonstration for analysts evaluating alternatives with and without projected traffic growth based on site development and corresponding proposed geometric and traffic control changes. From a traffic impact perspective, it is common to expect an increase in crash frequency due to increased traffic demand. This guide provides examples of mitigation measures and their economic evaluation when assessing the impacts of proposed development. Traffic volume is assessed accordingly for each alternative with the expectation based on development and infrastructure.

Need for Calibrating Prediction Models

The *HSM* (AASHTO, 2010b) recommends calibrating crash prediction models for local conditions using a minimum of 30 to 50 sites with at least 100 crashes per year for the calibration. Many agencies have taken to calibrating some or all planning-level and project design-level SPFs. Agencies have also developed jurisdiction specific planning-level and project design-level SPFs. The Crash Modification Factor Clearinghouse provides a general overview for agencies who have calibrated and developed SPFs to supplement and/or replace those in the *HSM*.

Persaud and Lyon (2007) indicate that for CMF development, SPFs need to be calibrated for each of the before and after periods and desirably for each year of these periods. This should also extend to predictive models for similar purposes.

Many agencies, however, have not had the resources, or ability to calibrate and/or develop jurisdiction-specific SPFs. In these cases, they have used relative comparisons within and across facility types using uncalibrated versions of the *HSM* predictive models. However, it is unclear whether the relative comparisons can be made, particularly across facility types, even considering a single-state calibration for the *HSM* – as the relative difference across facility types may differ from State to State or even among local jurisdictions within a State.

Farid et al. (2016) examined the transferability of SPFs from one or multiple jurisdictions to another using Rural Multilane highways as a case study. The authors found that some States, individually, provided SPFs that were more transferable, but that SPFs developed from pooled States (such as the *HSM* approach) were more transferable than those from individuals. The authors developed a transfer index to examine single-State, two-State, and three-State SPFs. Additionally, the authors proposed a Modified Empirical Bayes measure providing segment-specific calibration factors for transferring SPFs to local jurisdictions. Their proposed measure outperformed the *HSM* calibration factor based on the datasets used in their study.

Similarly, Yuan et al. (2021) studied the transferability of freeway SPFs between multiple states, including California, Florida, and Virginia. The results show the SPFs could be transferred between Florida and Virginia but no models were transferable with the California models. Looking at the transferability of SPFs for rural divided multilane highway segments in multiple states, Farid et al. (2018) found that SPFs for Ohio, Illinois, Minnesota, and California are transferable between each other. However, SPFs from the other states (Florida, Washington, and North Carolina) are not transferable.

While not specifically related to calibrating prediction models, Gross et al. (2018, 2021) studied CMFs and developed crash prediction models for access management strategies. When prediction models were not able to be developed for specific jurisdictions or areas, the study used prediction models for areas similar to the jurisdiction of interest. Gross et al. (2018) direct analysts to review summary data for agencies used to develop models and select the land use and region which would be most comparable to that being analyzed. Matarage and Dissanayake (2020) developed calibration functions and calibration factors for freeway facilities in Kansas. The authors found that calibration functions were superior to calibration factors and replaced the need for developing jurisdiction-specific SPFs.

Explicitly stated, the research on calibration factors and calibration functions indicate the need for calibration for accurate predictions within a state or local jurisdiction. Calibration functions serve as a surrogate for new SPFs, and generally provide improvement over calibration factors. Farid et al. also found improvement from their Modified Empirical Bayes measure, all of which provide an indication for the need to use calibrated models when conducting safety assessments where any form of numeric crash frequency is needed for economic evaluation. Caution should be used when considering relative assessments when using uncalibrated models where the relative assessment provides a numeric value for change in crash frequency. However, when the EB method is used, if crash sample sizes are large, less weight is put on predicted crash frequency. For larger crash sample sizes, a calibrated SPF may be less necessary due to the low weight provided on predicted crash frequency. As the SPF has less reliability (i.e., higher overdispersion parameter) and lower predicted crash frequency, the need for calibration is much greater.

Studies have found that the sample size requirements in the HSM may be inadequate (Alluri et al., 2014; Dissanayake and Karmacharya, 2020; Shirazi et al., 2016; Rajabi et al., 2021). Alluri et al. (2014) determined that "the minimum sample size of 30 to 50 sites, as recommended by the *HSM*, is insufficient to achieve the desired accuracy" and noted that sample size is variable and depends on many factors. Dissanayake and Karmacharya (2020) compiled a table of site sample sizes used for calibration in various studies of urban intersections (Table 13). Most sample sizes for site shown in Table 13 were above the 30 to 50 site recommendation in the *HSM*.

Table 13. Site sample sizes used in various intersection studies for calibration (Table 2.2.; Dissanayake and Karmacharya, 2020).

State	Site Sample Size			
	3-leg Unsignalized Intersection with Stop Control on Minor Approach	3-leg Signalized Intersection	4-leg Unsignalized Intersection with Stop Control on Minor Approach	4-leg Signalized Intersection
Maryland	152	167	90	244
Missouri	70	35	70	35
Massachusetts	86	48	59	52
Ohio	50	50	125	50
Florida	-	45	-	121
Oregon	73	49	48	57

Dissanayake and Karmacharya (2020) also summarized the number of observed crashes used to developed calibration factors for various states, shown in Table 14. Crash sample sizes ranged from 52 to 2,426 crashes.

Table 14. Number of observed crashes used to develop calibration factors by State (Dissanayake and Karmacharya, 2020).

State	Number of Crashes			
	3-leg Unsignalized Intersection with Stop Control on Minor Approach	**3-leg Signalized Intersection**	**4-leg Unsignalized Intersection with Stop Control on Minor Approach**	**4-leg Signalized Intersection**
Maryland	103	789	173	1763
Missouri	52	531	179	1347
Massachusetts	310	767	339	2426
Florida	-	80 to 123	-	690 to 815
Oregon	103	321	105	-

Bahar and Hauer (2014) offered a series of equations to determine an appropriate sample size for calibration based on a desired standard deviation. Table 15 displays suggested sample sizes needed for different calibration factors (C) and standard deviations (σ). For example, a calibration factor of 0.9 developed using 90 crashes would have a standard deviation of 0.1. However, if the number of crashes used to develop the calibration factor increases, the standard deviation decreases.

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Table 15. Suggested number of crashes (sample size) for different standard deviations (Table B.1; Bahar and Hauer, 2014).

σ	C										
	0.5	0.6	0.7	0.8	0.9	1	1.1	1.2	1.3	1.4	1.5
0.01	5000	6000	7000	8000	9000	10000	11000	12000	13000	14000	15000
0.02	1250	1500	1750	2000	2250	2500	2750	3000	3250	3500	3750
0.03	556	667	778	889	1000	1111	1222	1333	1444	1556	1667
0.04	313	375	438	500	563	625	688	750	813	875	938
0.05	200	240	280	320	360	400	440	480	520	560	600
0.06	139	167	194	222	250	278	306	333	361	389	417
0.07	102	122	143	163	184	204	224	245	265	286	306
0.08	78	94	109	125	141	156	172	188	203	219	234
0.09	62	74	86	99	111	123	136	148	160	173	185
0.1	50	60	70	80	90	100	110	120	130	140	150
0.11	41	50	58	66	74	83	91	99	107	116	124
0.12	35	42	49	56	63	69	76	83	90	97	104
0.13	30	36	41	47	53	59	65	71	77	83	89
0.14	26	31	36	41	46	51	56	61	66	71	77
0.15	22	27	31	36	40	44	49	53	58	62	67
0.16	20	23	27	31	35	39	43	47	51	55	59
0.17	17	21	24	28	31	35	38	42	45	48	52
0.18	15	19	22	25	28	31	34	37	40	43	46
0.19	14	17	19	22	25	28	30	33	36	39	42
0.20	13	15	18	20	23	25	28	30	33	35	38

Studies have also used the coefficient of variation of crashes to determine sample size requirements (Shirazi et al, 2016; Rajabi et al., 2021). Another method for determining a needed sample size is using Figure 21 which considers coefficient of variation of the observed crashes [CV(N_o)] and coefficient of variation of the calibration factor [CV(CF)] (Rajabi et al., 2021).

$$Sample\ size = \frac{CV(N_o)^2}{CV(CF)^2}$$

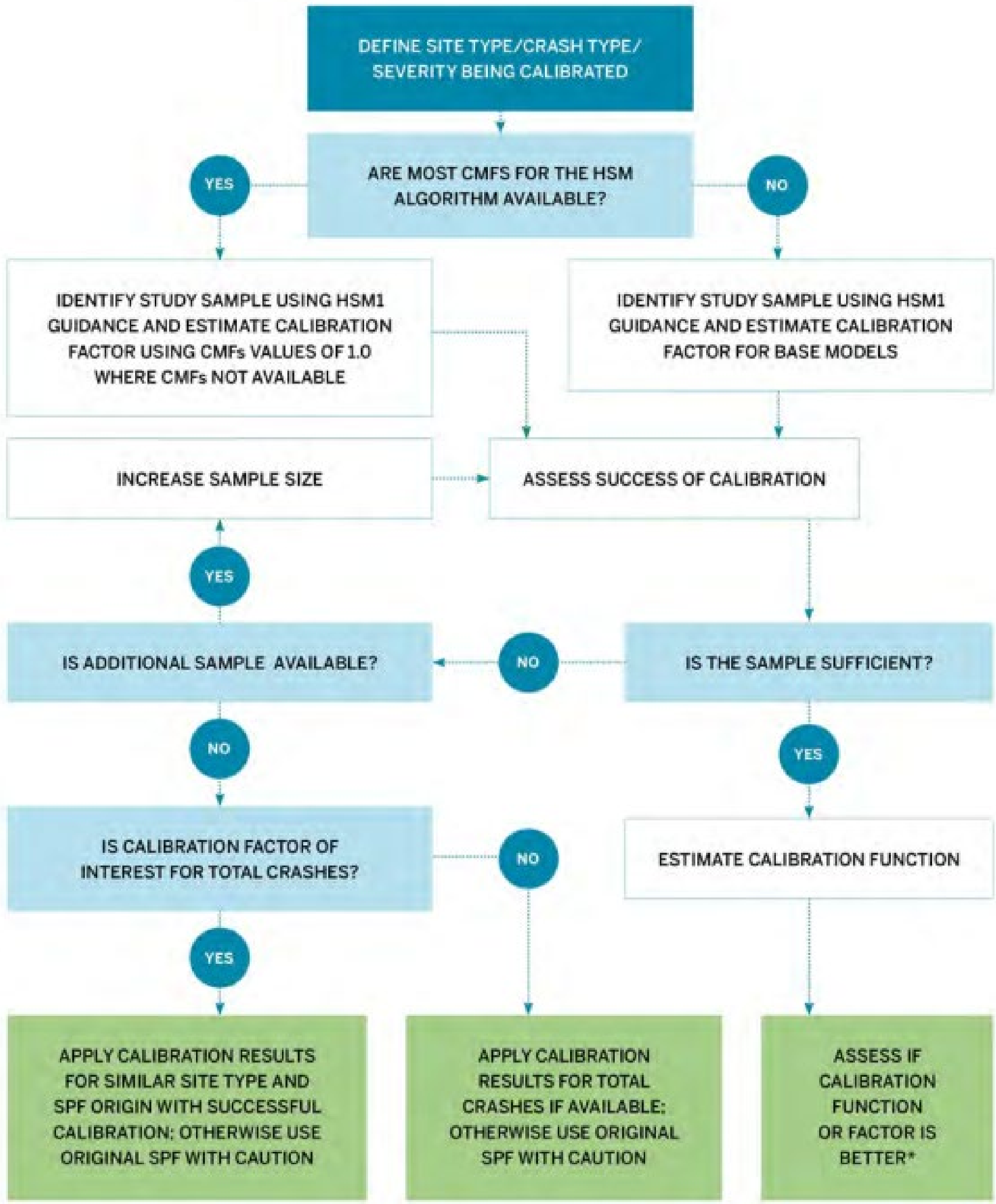
Figure 21. Sample size equation.

Shirazi et al. (2016) developed guidelines to determine sample size for multiple different confidence levels. Users of the guidelines must first calculate the coefficient of variation for the crash data using the crash mean and standard deviation. The coefficient of variation can then be used in Table 16 to identify the sample size for the needed confidence level.

Table 16. Sample size guidelines (Table 4; Shirazi et al., 2016).

CV	Confidence Level		
	90%	**80%**	**70%**
3.0	1500	1100	700
2.8	1400	1000	650
2.6	1300	900	600
2.4	1200	800	550
2.2	1000	650	450
2.0	900	550	400
1.8	750	450	300
1.6	600	350	250
1.4	450	300	200
1.2	300	200	150
1.0	200	125	75
0.8	100	75	50
≤0.6	50	30	30

Ivan et al. (2021), in suggesting refinements to the *HSM* calibration procedure, suggest that the required sample size needs to be determined iteratively based on goodness-of-fit measures for which there are thresholds for a successful calibration as outlined in the user guide for FHWA's CALIBRATOR software tool used for estimating and assessing calibration factors. Ivan et al. also offer a way to develop calibration factors if an adequate sample size is not available. Just as Alluri et al. (2014) discussed, Ivan et al. (2021) agrees that sample size for calibration is highly variable and adds that there has not been consistent guidance regarding an appropriate sample since in research since 2010. "If a successful calibration cannot be achieved with the entire sample available for total crashes, then the calibration results for a similar site type (from which a successful calibration was achieved) may be assumed to apply (Ivan et al., 2021). Similarly, if calibration is not successful for a specific crash type or severity, then it can be assumed that calibration for total crashes can be used (Ivan et al., 2021). Ivan et al. (2021) developed a flow chart for a suggested calibration process, shown in Figure 22. The process includes what to do if an adequate sample size is not available. If an adequate sample is not available and additional data cannot be obtained, the process recommends applying calibration results for similar site types or total crashes.



*If calibration is being done for base models and appropriate skills are available or could be acquired, it is recommended to try to directly estimate a model with the final calibration dataset and adopt the model if successfully estimated and performs better than the calibration factor and calibration function.

Figure 22. Calibration flow chart (Figure 6-1; Ivan et al., 2021).

Conclusions

Researchers and practitioners have proposed and utilized several approaches to apply the *HSM* and jurisdiction-specific predictive methods and SPFs for project alternatives analysis. The research reviewed herein provides guidance and best practices for using the EB method, identifying appropriate traffic volumes, and evaluating the need for locally-calibrated national SPFs for use in project alternatives analysis.

Predictive Method for Project Alternatives Analysis with and without Using the EB Method: Based on guidance included in the *HSM* Part C, if the EB method can be applied for all alternatives in a project alternatives analysis, then the EB method should be used for the safety impacts comparison. However, if the EB method is not applicable for one or more project alternatives, only the predicted crashes should be compared for all project alternatives. The EB method should only be used if it is applicable for all alternatives.

Researchers have proposed using the EB method for a baseline estimate of future crash frequency for no-build conditions. For the estimate of safety effectiveness, the researchers have used predicted crash frequency for the alternative condition. For project alternatives analysis, this approach will result in bias, accounting for site-specific crash history under the no-build condition and ignoring site-specific crash history for the project alternative condition. There may be underlying reasons why crash frequency or severity may be higher or lower at a given location, beyond site selection bias, and considering average crash frequency from an SPF will introduce bias when site-specific conditions are considered in one alternative (in this base the future no-build condition).

Agencies have demonstrated that using the EB method to establish baseline average annual crash frequency under no-build conditions and then applying a CMF for the alternative(s) of interest is also valid. This approach allows the analyst to account for site-specific conditions through consideration of observed crash frequency and assess the change in safety performance through an assessment of relative safety performance.

Appropriate Traffic Volume in Alternatives Analysis: If a project alternative introduces a higher traffic demand, a higher crash frequency is likely to be predicted, unless geometric or operational changes are being considered that overcome the increase in demand. Based on the nature of SPFs, alternatives that increase traffic volume will likely yield a higher predicted crash frequency in the comparison. While this may seem unfair in some circumstances, engineers should use all available information when making trade-off decisions, including identifying the potential area of impact for inclusion in the project alternatives analysis. Considering the actual expected traffic volumes will provide the analyst with an opportunity to evaluate mitigation measures to improve or offset increases in traffic volume or differences in traffic volume among alternatives.

Evaluate the need for calibrating prediction models: The *HSM* notes that crash prediction models should be calibrated to local conditions to account for jurisdiction-specific differences. However, some jurisdictions may not have sufficient sample sizes or resources to develop their own calibration factors. Further research is needed to examine the extent of this issue and at what point in the EB method calibration may not be necessary. Additionally, research is needed to examine the effects of using even relative safety performance as a measure when comparing alternatives that use different SPFs or predictive methods. The *HSM* indicates a relative comparison can be made when the facility type does not change, but many agencies have been using this assessment when comparing alternatives that change facility type.

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