NDOT Research Report

Report No. 339-12-803

Enhancing NDOT's Traffic Safety Programs

October 2013

Nevada Department of Transportation
1263 South Stewart Street
Carson City, NV 89712

Disclaimer

ENHANCING NDOT'S TRAFFIC SAFETY PROGRAMS

UTC Research Project

UNR Safety Engineering Support

October 2013

Prepared for the UTC-NDOT Research Program

Prepared by

Center for Advanced Transportation Education and Research (CATER), University of Nevada, Reno

Nevada Department of Transportation
Safety Capacity Building

UNR Safety Engineering Support
UTC Contract

Calibration of Highway Safety Manual's Safety Performance Functions
Summary of Before-and-After Study Approaches and Procedures
Safety Evaluation of I-580 and U.S. 395 ALT

Authors

Zong Tian, Ph.D., P.E.
Associate Professor and Director
Center for Advanced Transportation Education and Research (CATER)
University of Nevada, Reno, MS 258
Email: zongt@unr.edu

Laura Y Zhao
Research Assistant
Center for Advanced Transportation Education and Research
University of Nevada, Reno, MS 258
Email: yuezhao118@gmail.com

Saeedeh Farivar
Research Assistant
Center for Advanced Transportation Education and Research
University of Nevada, Reno, MS 258
Email: saeedeh.f1983@gmail.com

Chuck Reider, P.E.
Center for Advanced Transportation Education and Research
University of Nevada, Reno, MS 258
Email: charles.w.reider@gmail.com

ACKNOWLEDGEMENT

The authors would like to thank Mike Colety and Michael Mosley from the Kimley-Horn and Associates for their constructive comments and help during the study period. In addition, we would particularly like to thank Ken Mammen and Jaime Tuddao from NDOT Safety Division for providing directions and assisting with the data collection. We are also grateful to Kimberly Edwards, Chris Wright and Jennifer Hartley from NDOT Safety Division for providing the comprehensive crash data for each research task.

PREFACE

This is the final report for the University Transportation Center (UTC) research project titled **Enhancing NDOT's Traffic Safety Programs—Calibration of the Highway Safety Manual's Safety Performance Functions, Summary of Before-After Study Procedures and Methodologies, and Safety Evaluation of I-580 and U.S. 395 ALT.** This research is sponsored by the Nevada Department of Transportation (NDOT) and supported in part by the U.S. Department of Transportation (USDOT) Research and Innovative Technology Administration (RITA) funds. The research project was divided into three separate tasks. The first task, Calibration of the Highway Safety Manual's (HSM) Safety Performance Functions (SPFs) for the State of Nevada, was conducted in a one-year period from July 2012 to July 2013. The second task, Summary of Before-and-After Study Procedures and Methodologies, and the third task, Safety Evaluation of I-580 and U.S. 395 ALT, were conducted in a six-month period from January to June 2013.

The release of the first-edition Highway Safety Manual (HSM) includes comprehensive and well-established procedures for conducting various traffic safety analyses. Incorporating these procedures into NDOT's traffic safety program is essential for making cost-effective recommendations on safety improvement projects. Several critical issues have been identified within NDOT's current safety programs: (1) most HSM procedures are new to NDOT's safety engineers, thus timely staff training is necessary; (2) the HSM does not provide Nevada-specific Safety Performance Functions (SPF); (3) the HSM procedures are data-extensive, and identification of the data needs in Nevada is critical; (4) there are multiple procedures that can be applied to a before-and-after study; however, a clear recommendation on which procedure should be used is not defined in the HSM to meet NDOT's needs; (5) most procedures deal with large amount of data, and in most cases the analyses can only be carried out by using software tools.

Current practice on traffic safety mostly relies on empirical approaches. The safety analysis procedures in the HSM are developed based on advanced statistical methods. Such methods span over a wide range of applications. One example is the use of Empirical Bayes (EB) method for Network Screening and for Before-and-After Studies. Additionally, the EB method requires well-calibrated SPFs and CMFs. Both SPF and CMFs are used to predict the number of crashes by types based on traffic, geometry, and other site-related variables. The first edition HSM provides SPFs and CMFs for a selected set of facility types. These SPFs and CMFs are developed based on limited data sources, thus the results are likely to vary due to differences in driver population, weather conditions and other variables. The validity of a study largely depends on the accuracy of the SPFs and CMFs, which must be calibrated based on site-specific conditions. The effort on calibrating SPFs and CMFs has already been carried out in several states and this trend is expected to continue at the national level.

Calibration of the HSM models for rural two-lane highway segments has been done in several states, including Utah, Oregon and North Carolina. Several states have also calibrated SPFs for various intersection control types, including Illinois, Colorado, and Virginia.

To meet NDOT's needs, three tasks were undertaken concurrently by UNR CATER research team with the first task focusing on the calibration of the SPFs for rural two-lane two-way highway, the second task focusing on the developing before-and-after study procedures and methodologies for Nevada and the third task applying the HSM to evaluate and predict the safety of I-580 and U.S. 395 ALT segments between Carson City and Reno. This final report transmits our findings and conclusions pertaining to each task. Concerning the arrangement of the report, the executive summary presents how the research tasks were conducted throughout the one-year period and the major conclusions drawn by executing each research task. The objectives and scope of each task are presented in the corresponding chapter. Chapter 1 presents the calibration process and results of the HSM's SPFs for rural two-lane two-way undivided roadways. Chapter 2 focuses on the before-and-after study procedures and recommendations for Nevada's applications. Chapter 3 summarizes the Interstate 580 (I-580) and U.S. Route 395 Alternate (U.S. 395A) safety evaluation case study. In the end, all the findings and conclusions are summarized in Chapter 4.

ABSTRACT

This report summarizes three tasks of the UTC project. Firstly, the report presents the SPF calibration procedures and results for rural two-way two-lane roads. A calibration factor was found to be 1.21 for both total and Fatal and Injury crashes. In addition, the performance of the existing (un-calibrated) HSM SPF and the calibrated HSM SPF was compared using a variety of statistical GOF tests and also the CURE plots. The comparison results indicate that the calibrated HSM SPF exhibited a better fit to the local data than the un-calibrated HSM SPF. Nevertheless, the calibrated HSM SPF was not determined as a good-fitting model. Therefore, further research needs to be done to develop Nevada specific SPF to better represent the observed crash frequency. Secondly, the report summarizes the procedures and methodologies for conducting a before-and-after study. Four commonly used before-and-after study approaches are discussed including the Naïve approach, the Comparison Group approach, the Yoked Comparison as well as the Empirical Bayes method. This task provides a concise introduction of the concepts, merits and limitations of each approach to weigh into the decision process where data availability, resources, and other decisive factors are realities. This report recommends that NDOT develop related training materials and conduct training sessions for NDOT and local government traffic engineers in using the recommended methods. The third part of this report is a safety evaluation study on the new section of I-580 and the U.S. 395A. The analysis contains an overview of the roadway segments, and examines the horizontal alignment, vertical alignment and cross-section in more detail through a review of the detailed design plan/profile data. The EB method that weights the observed crash frequencies with the predicted crash frequencies using the base SPFs and CMFs is applied to calculate the expected crash frequencies of U.S. 395A and old U.S. 395 road segments. The freeway predictive method documented in the future Chapter 18 of HSM is applied to predict the safety of I-580 in 2014 as well. The evaluation results indicate that in 2014 53 crashes are predicted to occur along the new I-580 freeway section. In addition, 14 crashes are expected to occur along the U.S. 395A segments. The safety of the old U.S. 395/U.S. 395A will be improved significantly given the historical crash records involved approximate 69 crashes per year from 2007 to 2011 and a total of 72 crashes expected in 2014 without the building of the new freeway section. A benefit cost analysis was conducted for the I-580 Freeway Extension Project and the Benefit-Cost Ratio (BCR) of 1.76 was obtained. In the long run, the project will produce economic benefits in accident reductions, travel time savings, vehicle emission reductions, etc.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

The UTC research project *Enhancing NDOT's Traffic Safety Programs* was conducted in a one-year period from July 2012 to July 2013. The overall project was divided into three separate working tasks and several phases to achieve the research goals. With respect to the calibration of SPFs for Nevada, a major data collection, data processing and organization effort was involved. With respect to the before-and-after study procedures, a comprehensive literature review of existing methodologies and research findings based on national and local studies was documented. Furthermore, a synthesis of the practice was assembled and specific recommendations were developed for Nevada. Last but not least, a case study was conducted utilizing the predictive methods in the HSM and the Interactive Highway Safety Design Model (IHSDM) software to predict the safety on I-580/U.S. 395A pair compared to the scenario when only old U.S. 395 existed. The case study demonstrated the applications of major traffic safety management procedures documented in the American Association of State & Highway Transportation Officials (AASHTO) Highway Safety Manual (HSM), such as the safety performance functions (SPFs) and crash modification factors (CMFs). The findings will directly contribute to advancing NDOT's traffic safety programs.

Task 1: Calibration of SPFs for Nevada

The first task of the UTC project is to calibrate the SPFs in HSM for rural two-way two-lane roads in Nevada. Using five years of crash data and road characteristics data comprising of 541 roadway segments, the calibration factor was found to be 1.21 for Nevada rural two-way two-lane roads. This implies that the HSM model underestimates number of crashes for this type of facility in Nevada by 21%. Applying the calibration factor, the following equation gives the Nevada-calibrated SPF for rural two-way two-lane roads:

$$N_{spf} = 1.21 \times AADT \times L \times 365 \times 10^{-6} \times e^{-0.312}$$

To estimate the crash frequency for a particular crash severity, the crash severity levels are required to be multiplied by the above equation. In this study, the default values in the HSM were updated as depicted in Table 1. The local distribution values were derived using the crash data on the Nevada rural two-way two-lane roads.

Table 1 Nevada Crash Severity Levels versus HSM Default Values

Percentage of Total Roadway Segment Crashes		
Crash Severity Level	Locally Derived Percentages (%)*	HSM Provided Percentages (%)
Fatal Plus Injury	33.76	32.10
Property Damage Only	66.24	67.90
TOTAL	**100**	**100**

* *Source:* AASHTO, 2010

Statistical goodness-of-fit (GOF) tests were assessed to compare the performance of two HSM SPFs, the base SPF (uncalibrated) and the calibrated SPF. Three out of five statistical GOF parameters showed that the calibrated models provided a better fit to the local data than the uncalibrated model. In addition to the statistical tests, CURE (cumulative residuals) plots were utilized to perform this comparison. CURE plots were plotted against the model variables; AADT and the segment length to determine if the functional form for each variable in the model is appropriate. As a function of segment length, the cumulative residuals for the calibrated model fell mostly within its confidence limits, while for the uncalibrated model there was a significant deviation from the confidence limits. As a function of AADT, the calibrated model exhibited less deviation from its confidence limits especially when the AADT is more than 2000. For AADTs more than 2000, even the calibrated model showed a significant deviation from the confidence limit. In summary, even though the calibrated model exhibited a better fit to the data than the uncalibrated model, the calibrated HSM SPF still lacks the accuracy in predicting crash frequencies for the Nevada rural two-way two-lane roads. To better represent the observed crash frequency, further research needs to be done regarding the development of Nevada-specific SPF for this facility type by at least re-estimating the model parameters or considering other functional forms for the AADT.

Task 2: Summary of Before-and-After Study Approaches

Before-and-after studies are frequently used to evaluate the performance of a safety improvement plan or an operational change on a transportation facility. The Highway Safety Manual (HSM) recommends before-and-after studies as a standard approach to evaluate safety improvements and to develop Crash Modification Factors (CMFs). Unfortunately, there are cases where local studies have employed inferior analysis methodologies in before-and-after studies due to the lack of resources to provide sufficient data or the lack of proper understanding of the basic concept and different techniques in before-and-after studies. Therefore, it is the aim of this research task to provide better resources for Nevada practitioners to explicitly understand this type of study. This task mainly presents answers to the following two questions.

- What is the before-and-after study in road safety?
- How to conduct a valid before-and-after study in theory?

First and foremost, engineers need to have a profound understanding of the fundamental concepts within before-and-after studies. The effectiveness of a treatment, such as rumble strips, guard rail, turning lanes, pedestrian hybrid beacon, etc., should be considered as the changes in level of safety between the 'before' and 'after' periods purely due to the treatment. Therefore, the logical essence of a before-and-after study compares the level of safety of a specific site in the 'after' period without the treatment, to the level of safety with the treatment in the 'after' period. The estimation of the level of safety with the treatment in the 'after' period is usually obtained by counting the observed number of crashes. However, to predict the safety of the treated sites had the treatment not been applied is usually intricate. There are many ways to predict safety, and there are correspondingly many ways to conduct a before-and-after study. Nevertheless, a before-and-after study normally integrates four consecutive steps as shown in Figure 1, including the estimation of basic parameters (i.e., expected crashes in both 'Before' and 'After' periods as well as the measures of safety effectiveness) and their variances. The estimates are used to measure the size of the effectiveness, and their variances are to approximate the level of confidence of the estimation. The four-step procedure forms a standardized framework to conduct a before-and-after study.

Furthermore, in real-world applications, four approaches are commonly used to conduct before-and-after studies, including: (1) the Naïve approach; (2) the Comparison Group (C-G) approach; (3) the Yoked Comparison approach; and (4) the Empirical Bayes (EB) approach. The naïve before-and-after study simply compares the crash frequencies before and after the installation of the treatment assuming no interventions from the 'before' to the 'after' period. The comparison methods including the C-G approach and the Yoked comparison approach implicitly explain the potential factors that might influence the safety in the 'before' and 'after' periods by using a comparison site or group. The EB method predicts what would have happened at the treatment sites in the 'after' period without treatment based on a weighted combination of two factors: (1) the frequency of crashes on the treated sites in the 'before' periods; and (2) the crash frequency predictions from regression models developed with data from similar but untreated reference sites. By doing so, the EB method properly accounts for the regression-to-the-mean bias while normalizing for differences in traffic volume and other possible confounding factors between the 'before' and 'after' periods.

To conclude, each before-and-after study approach has its own strengths and limitations. Engineers need to follow the basic four-step process demonstrated in Figure 8 and select the suitable study approach based on data availability and the knowledge of the approach as recommended in the flow chart in Figure 25. This technical report documents key component related to before-and-after studies and provides the following recommendations for the Nevada Department of Transportation (NDOT) Safety Division to consider.

- The Naïve approach is not recommended in NDOT field applications.
- The C-G approach is recommended when over 5-years of crash data in the 'before' period is available for both treatment and comparison groups.
- The Yoked Comparison approach is recommended when the number of facilities is limited in the comparison group.

- When the number of facilities is limited in the comparison group, the Yoked Comparison approach is recommended to conduct one-to-one analysis.
- The Empirical Bayes approach is recommended as the standard approach for before-and-after studies when the Safety Performance Functions (SPFs) are available for specific study highway types.
- The related training materials need to be developed to conduct training sessions for NDOT and local government traffic engineers in applying the before-and-after approaches in field applications.

Task 3: Safety Evaluation of I-580 and U.S. 395 ALT

The third task is the safety evaluation case study of the Interstate 580 (I-580 hereafter) and U.S. Route 395 Alternate (U.S. 395A hereafter). The purpose of this task is to predict the safety benefit of the new section of I-580 and U.S. 395A by comparing the crashes per year estimated for old U.S. Route 395 (old U.S. 395 hereafter) considering no build of I-580 versus the predicted crashes for I-580 and U.S. 395A pair. A depiction of the study area is shown in Figure 1.

Figure 1 I-580 and U.S. 395A Case Study Area

Firstly, this study focused on the new opened section of I-580 from Washoe Valley to the extreme southern edge of Reno. The section starts at the diverging point at U.S. 395A and continues to the Mount Rose Highway (SR 431) interchange. The total analysis segment is approximately 9.5 miles long with three lanes in each direction. Utilizing the Interactive Highway Safety Design Model (IHSDM) software, the Freeway Predictive Method was applied to the I-580 segment to calculate the predicted total number of crashes for the study period.

Secondly, U.S. 395A is an alternate route of the old U.S. 395. This study focused on the section of U.S. 395A that parallels I-580 between the intersection of I-580/U.S. 395 and the junction of Geiger Grade (SR 341). The total analysis segment is about 7.84 miles long. Similarly, the IHSDM software was applied to predict the safety of U.S. 395A as well as the old U.S. 395 by including site specific crash data in the analysis using the Empirical Bayes Method.

The predicted number of crashes for I-580 and the expected number of crashes for U.S. 395A and old U.S. 395 are summarized in Table 2. A Benefit-Cost Analysis for the I-580 freeway extension project was also included in this study. The net present value of the benefits for this project with a discount rate of 7% is almost $946 million in 20 years and the present value of costs is about $536 million in 20 years giving a benefit cost ratio of 1.76 with a payback period of 12 years. In the long run, the project will produce benefits from accident reductions, travel time savings, vehicle emission reductions, etc.

Table 2 Predicted and Expected Number of Crashes

Predicted and Expected Number of Crashes in 2014	**Existing Condition**		**Comparison Scenario**
	I-580	U.S. 395A	Old U.S. 395
Total Number of Crashes	52.24	13.92	71.06
Fatal and Injury Crashes	15.59	7.92	32.68
Reduction in Total Crashes over Existing Conditions	–	–	4.90
Reduction in Fatal and Injury Crashes over Existing Conditions	–	–	9.17
Predicted and Expected Number of Crashes in 20 Years	**Existing Condition**		**Comparison Scenario**
	I-580	U.S. 395A	Old U.S. 395
Total Number of Crashes	2013.95	292.32	2316.89
Fatal and Injury Crashes	624.32	166.62	1065.76
Reduction in Total Crashes over Existing Conditions	–	–	10.62
Reduction in Fatal and Injury Crashes over Existing Conditions	–	–	274.82

CHAPTER 1 CALIBRATION OF SAFETY PERFORMANCE FUNCTIONS FOR NEVADA

INTRODUCTION

The Highway Safety Manual (HSM) (*1*) published in 2010 by the American Association of State Highway and Transportation Officials (AASHTO) provides Safety Performance Functions (SPFs) for different types of roadways and intersections. However, the crash predictive models in the HSM are developed using data from selected states which may not be directly applied to other states where different local factors exist. These factors include crash reporting threshold, crash reporting method, driver population, weather condition, and terrain type. To better account for the local factors, HSM recommends that its predictive models be calibrated using the respective data where the models will be applied. Efforts on SPF calibration have been made in several states including Alabama (*2*), Utah (*3*), North Carolina (*4*), and Oregon (*5*). Based on their results, the calibrated HSM SPFs showed better fit to the state data than the un-calibrated SPFs.

The main objectives of this task include: (1) to evaluate the performance of the base SPF provided in the HSM in predicting crash frequency for Nevada rural two-way two-lane roads, (2) to calibrate the HSM SPF, and (3) to compare the calibrated versus un-calibrated SPF based on Nevada data. The results will help NDOT in improving the accuracy of predicting crash frequencies for rural two-way two-lane roads

DESCRIPTION OF THE STUDY DATA

The main components necessary for calibration or development of SPFs are crash data and roadway characteristics. Crash data provides the number of crashes and crash severity, while the roadway data provides the information of traffic volumes and the route data such as functional classification, number of lanes, lane width. The required data were provided by NDOT. The following sections describe the details of available data and the data processing methodology.

NDOT's Data Availability

NDOT provided three datasets for the SPF calibration. The datasets included roadway related information, crashes, traffic volumes, and milepost location of intersections. Each dataset was provided separately, and then crash and AADT data were added to the obtained route data.

The original dataset contained information on 461 roadway segments with a total mileage of 683.1 along three major rural two-way two-lane roads in Nevada: US-93, US-95 and SR-318. Actually, these three roads were the only roads with their required data available in NDOT. Figure 2 shows where these roadways are located. A brief description of each dataset is provided below.

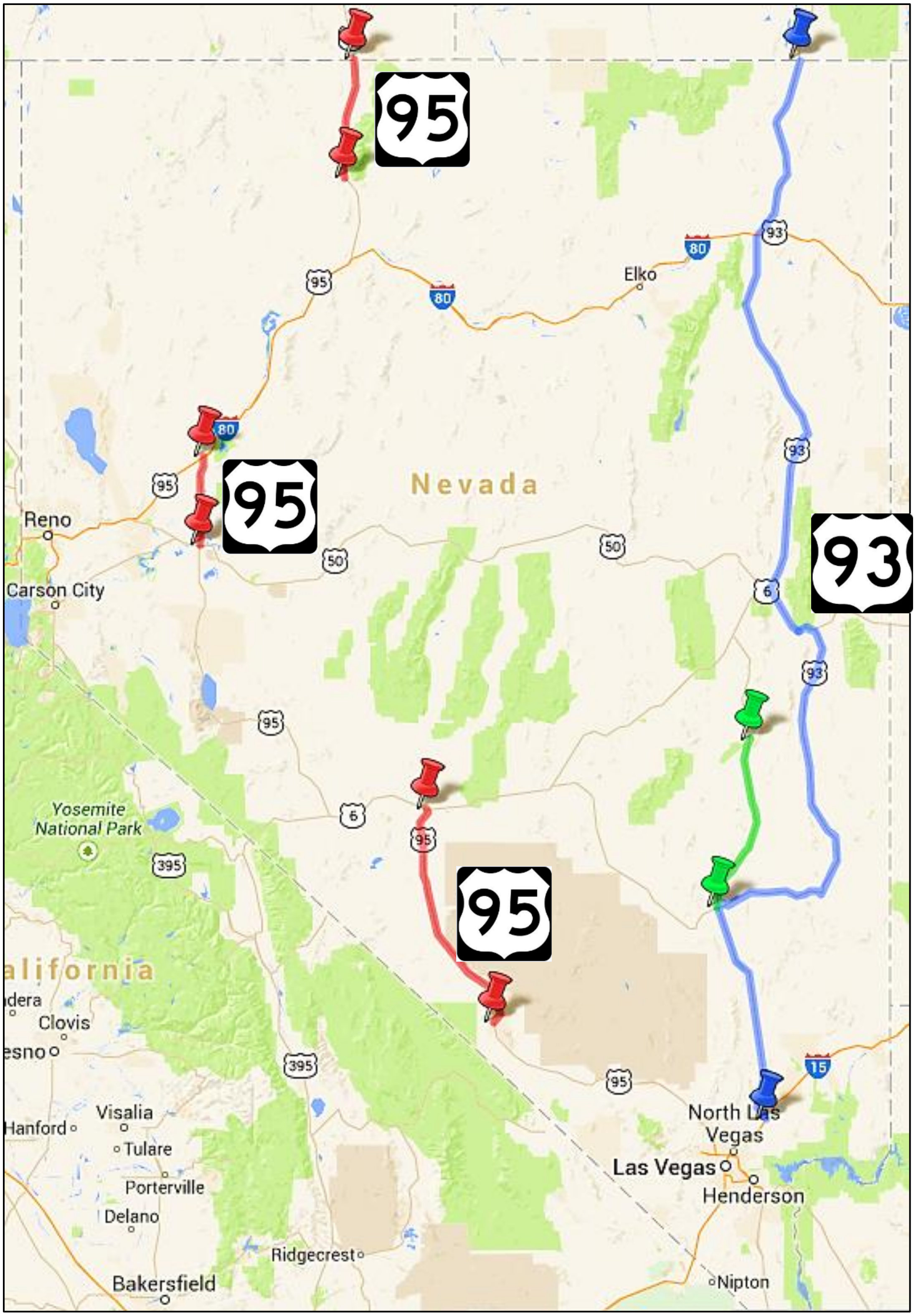


Figure 2 Map of the Study Routes

Route Data

The route data was obtained from the Highway Performance Monitoring System (HPMS), in Excel. The dataset included the following fields of route characteristics:

- Segment Type
- Beginning Mile Post
- Ending Mile Post
- Segment Length
- Automated Speed Enforcement
- Center Line/Shoulder Rumble Strip
- Lane Width
- Shoulder Width
- Shoulder Type
- Hazard Rating
- Horizontal Curve Radius
- Presence of Spiral Curve
- Super Elevation Variance
- Grade
- Driveway Density

Information of horizontal curves, superelevation, grade, and driveway density was not available for all the study segments except for the segments along US-95.

Crash Data

Similar to the route data, NDOT provided the 2006-2011 crash datasets for the study roadway segments. These files are also in Excel which include information about the severity, crash type, contributing factors, location, and time of each crash. The most relevant information for the SPF calibration from these files is:

- Crash Year
- Primary Street
- Adjusted Mile Marker
- Injury Type
- Crash Type
- Vehicle Sequence Events

The Adjusted Mile Marker indicates the location of the crash which is calculated from the reference point (mile marker). Crash Type and Vehicle Sequence Events were used to calculate the local crash distributions as described later in the report. Since the crash dataset did not include all the reported crashes in 2006, a five-year dataset (2007 to 2011) was used in the analysis.

Traffic Volumes

NDOT provided the average annual daily traffic (AADT) volume for the above provided routes. The AADT dataset is also an Excel file containing the five-year (2007-2011) traffic volumes for the study roadway segments. The data file includes the following fields:

- Route Name
- Beginning Mile Post
- Ending Mile Post
- From Street
- To Street
- AADTs for 2007 to 2011

The SPF for rural two-way two-lane roads in the HSM is valid for roads with AADT up to 17,000 vehicles per day. The original 5-year dataset contained 461 segments that meet this criterion.

Intersection Locations

The HSM segment-based predictive models predict only non-intersection crashes. Therefore, to calibrate the SPF for roadway segments, only the segment-related crash data is required and all the intersection related crashes should be eliminated from the dataset. Therefore, the information on the milepost location of all the intersections along the study roadways is required. NDOT provided this dataset in an Excel file. The given file neither has milepost locations for all the intersections nor the intersection control type. The missing information was collected using Google Maps by UNR CATER team.

Data Processing and Organizing

The datasets obtained from NDOT needed to be organized to produce a comprehensive dataset usable for SPF calibration. The following sections provide more details of the data processing steps which involved eliminating intersection related crashes, assigning AADTs and crashes to the route data, and finally merging all the route datasets.

Exclusion of Intersections

As mentioned earlier, it is important to determine whether a crash is related to a segment or influenced by an intersection area. Harwood and Bauer (6) defined intersection crashes as all the crashes occurred at intersections and within 250 feet of the intersection.

Although this criterion was commonly adopted in many studies, NDOT uses 200 feet of an intersection which was used in this study.

Prior to assigning the crashes to the route data, the route mileages located within 200 feet (or 0.04 mile) on either side of each intersection were excluded from the original route data. Figure 2 shows a sample of this technique. In this sample, the milepost location of an intersection is 40.50. Considering 200 feet (0.04 mile) on either side of the intersection, the segment from 40.46 to 40.54 would be excluded from the dataset and the segment covering this mileage is broken into two separate segments as indicated in Figure 3 (all the changes are highlighted in the figure). Since some segments were broken into two or more segments, the total numbers of segments were increased by 83 from 461 to 541.

Segment Type	Beginning Milepost	Ending Milepost	Segment Length	Speed Limit
2U	40.30	41.30	1.00	70
2U	41.30	41.40	0.10	70
2U	41.40	41.50	0.10	70
:	:	:	:	:
:	:	:	:	:

Segment Type	Beginning Milepost	Ending Milepost	Segment Length	Speed Limit
2U	40.30	40.46	0.16	70
2U	40.54	41.30	0.76	70
2U	41.30	41.40	0.10	70
2U	41.40	41.50	0.10	70
:	:	:	:	:

Figure 3 A Sample of Intersection Mileposts Exclusion

By excluding the intersection-related mileposts, the crashes that are assigned to the remaining mileposts would be only the segment related crashes.

Based on the FHWA recommendation (David Engstrom, Personal Communication, June, 4, 2013), only the intersections of "public roadways" with rural two-way two-lane roads which had some type of control on minor roads (at least one stop sign) were excluded from the dataset. NDOT defines a "public roadway" as a road that is maintained by the state, city, or county.

AADT and Crash Assignment

After excluding the intersection related mileages from the route data, all the datasets were merged together and then the AADTs of each study year were assigned based on the route name, beginning milepost, and ending milepost. The average AADT across all the study segments is 1,674 vehicles per day per year with the minimum and the maximum of 320 and 5,240 vehicles per day per year, respectively. The standard deviation is 753.87.

A macro code was developed to assign the crash data to the study routes by counting the number of crashes within each defined segment. To compute separate calibration factors for the total and the injury (fatal plus injury) crashes, PDO and injury crashes were counted and assigned separately to the route data. Then the total number of crashes occurred on a segment was calculated by a summation of PDO and injury crashes. From 2007 to 2011, there were a total of 1,546 crashes consisting of 1,024 PDO and 522 injury crashes.

CALIBRATION OF THE HSM SPFs

Based on the HSM calibration procedure, the calibration factor for each facility type is obtained by calculating the ratio of total number of observed crashes for a selected sample of sites to the total number of predicted crashes obtained from the predictive models in the HSM for the same sites. Calibration factor can be computed from the following equation:

$$C = \frac{\sum_{all\ sites} Observed\ crash\ frequency}{\sum_{all\ sites} Predicted\ crash\ frequency}$$

A calibration factor of more than 1.0 implies that the respective HSM model under-predicts the number of crashes.

Implementation of predictive models in the HSM requires the development of two main components: (1) a base SPF; and (2) crash modification factors (CMFs) for a particular facility type, in this study, rural two-way two-lane roads. First, the base SPF uses the roadway characteristics to estimate the crash frequency under the base conditions. Then the CMFs are applied if the actual roadway conditions are different from the base conditions. The following sections describe the base SPF and CMFs for the rural two-way two-lane roads crash prediction.

HSM recommends that the minimum sample size for each facility type to be 30 to 50 sites and that the entire group of sites should present at least 100 crashes per year in order for the calibration to be reliable. The sample sites should also be selected as random as possible. In order to minimize the site selection bias that is caused by random selection, in this study all the study segments were selected to calculate the calibration factor.

Predicted Crash Calculation

The predicted crash calculation is described in this section by using an example.

Base Safety Performance Function (SPF)

The HSM SPF for the base condition of rural two-way two-lane roads is shown in the following equation:

$$N_{spf} = AADT \times L \times 365 \times 10^{-6} \times e^{-0.312}$$

Where,

- N_{spf} is the predicted average crash frequency for a rural two-way two-lane segment with the base conditions;
- AADT is the AADT volume of the segment (veh/day); and
- L is the segment length (mile).

Crash Modification Factors (CMFs)

CMFs are used to adjust the estimated crash frequency for a specific change in geometric design and traffic control features and account for the effects of non-base conditions. One CMF is used for each design element (e.g., lane width) and it has a value of 1.0 when the design element is the same as the base condition. If a design element would result in a decrease in predicted crashes compared to the base condition, then the CMF should have a value smaller than 1.0. Table 3 summarizes the CMFs in chapter 10 of the HSM for rural two-way two-lane roads along with the base conditions.

Table 3 CMFs for Rural Two-Way Two-Lane Roadway Segments

Facility Type	CMF	CMF Description	Existing Condition
Rural Two-way Two-Lane Roadway Segments	CMF_{1r}	Lane Width	12-feet
	CMF_{2r}	Shoulder Width and Type	6-feet-Paved
	CMF_{3r}	Horizontal Curves: Length, Radius, and Presence or Absence of Spiral Transitions	None
	CMF_{4r}	Horizontal Curves: Superelevation	<0.01
	CMF_{5r}	Grades	<3% (Absolute Value)
	CMF_{6r}	Driveway Density	<=5 Driveways/mile
	CMF_{7r}	Centerline Rumble Strips	None
	CMF_{rumble}*	Shoulder Rumble Strips	None
	CMF_{8r}	Passing Lanes	NONE
	CMF_{9r}	Two-way Left-Turn Lanes	None
	CMF_{10r}	Roadside Hazard Rating	3
	CMF_{11r}	Lighting	None
	CMF_{12r}	Automated Speed Enforcement	None

** Not in the HSM, from FHWA CMF Clearinghouse: http://www.clearinghouse.org*

CMFs for each design element are provided in chapter 10 of the HSM. It should be noted that a direct implementation of the HSM predictive method would not include the application of the CMF for shoulder rumble strips because it was not one of the data elements included in SPF development for the rural two-way two-lane roads. Based on a consultation with FHWA, it was decided to apply the shoulder rumble strip CMF to predictive method calculations for the rural two-way two-lane roads.

In this study, the base condition (CMF equals to 1.0) was assumed for the segments with no information on the horizontal curves, superelevation, grade, and driveway density. It should be noted that the CMFs for lane width, shoulder width, and type apply only to crash types that are most likely affected such as single-vehicle run-off the road, multiple-vehicle head-on, opposite-direction sideswipe, and same-direction sideswipe crashes. Therefore, the CMFs for the lane width, shoulder width, and type are adjusted by considering the proportion of related crashes to the total crashes. The default crash type distribution in HSM is 0.574. In this study, it was updated based on the local data and was estimated as 0.5887. Table 4 shows the locally derived crash type distributions versus the HSM default values.

Table 4 HSM-Default Crash Distribution Versus Nevada Distribution (obtained based Jurisdiction data)

Percentage of Total Roadway Segment Crashes		
Crash Type	HSM Provided Values*	Locally Derived Vales
SINGLE-VEHICLE CRASHES		
Run-off the Road	52.10	52.86
MULTIPLE_VEHICLE CRASHES		
Head-on Collision	1.60	0.80
Sideswipe Collision	3.7	5.21
TOTAL	**57.4**	**58.87**

** Source: AASHTO, 2010*

To calculate particular calibration factors for total and injury crashes, the crash Severity levels are required that can be multiplied by the SPF and estimate the number of injury crashes. In this study, the default value in HSM is updated as depicted in Table 5.

Table 5 HSM-Default Crash Severity Levels Versus Nevada Levels (obtained based Jurisdiction data)

Percentage of Total Roadway Segment Crashes		
Crash Severity Level	HSM Provided Values*	Locally Derived Vales
Total Fatal Plus Injury	32.10	33.76
Property Damage Only	67.90	66.24
TOTAL	**100**	**100**

* *Source:* AASHTO, 2010

A Sample of Predicted Crash Calculation

Table 6 shows the part of the predicted crash calculation spreadsheet for the study segments. As can be seen in this table, the predicted numbers of crashes for the base condition are 0.216 and 0.019 for segments 65 and 66, respectively. After the CMFs are applied, the unadjusted[1] predicted number of crashes for segment 65 is 0.187 and for segment 66 is 0.016. After calculating the unadjusted predicted crash frequencies for all the study segments, the total predicted number of crashes can be determined as the sum of all the unadjusted predicted crash frequencies which is 256.58 crashes per year as shown in the table.

[1] Unadjusted means that the calibration factor has not been applied yet.

Table 6 A Sample of Unadjusted Predicted Crash Frequency Calculation Spreadsheet

Segment NO	Hwy Name	Beg Mp	End MP	L (mile)	Average AADT	N_{spf}	CMFs													$N_{predicted}$
							Lane Width	Shoulder Width and Type			Horizontal Curve	Superelevation	Grade	Driveway Density	Centerline Rumble Strip	Shoulder Rumble Strip	TWLTL	Roadside Hazard Rating	Automated Speed Enforcement	
							CMF_{1r}	CMF_{wra}	CMF_{tra}	CMF_{2r}	CMF_{3r}	CMF_{4r}	CMF_{5r}	CMF_{6r}	CMF_{7r}	CMF_{Rumble}	CMF_{9r}	CMF_{10r}	CMF_{12r}	
:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:
65	US93	39.52	39.98	0.46	1767	0.216	1	1.13	1	1.08	1	1	1	1	0.86	1	1	0.94	1	**0.187**
66	US93	40.06	40.10	0.04	1767	0.019	1	1.13	1	1.08	1	1	1	1	0.86	1	1	0.94	1	**0.016**
67	US93	40.10	40.31	0.21	1767	0.099	1	1.27	1	1.16	1	1	1	1	0.86	1	1	0.94	1	**0.092**
68	US93	40.39	42.05	1.66	1767	0.781	1	1.27	1	1.16	1	1	1	1	0.86	1	1	0.94	1	**0.726**
69	US93	42.13	43.60	1.47	1767	0.691	1	1.27	1	1.16	1	1	1	1	0.86	1	1	0.94	1	**0.643**
70	US93	43.60	44.16	0.56	1767	0.263	1	1.27	1	1.16	1	1	1	1	0.86	1	1	0.94	1	**0.245**
71	US93	44.24	44.40	0.16	1767	0.075	1	1.27	1	1.16	1	1	1	1	0.86	1	1	0.94	1	**0.070**
72	US93	44.40	44.80	0.40	1767	0.188	1	1.27	1	1.16	1	1	1	1	0.86	1	1	0.94	1	**0.175**
73	US93	44.80	45.10	0.30	1767	0.141	1	1.27	1	1.16	1	1	1	1	0.86	1	1	0.94	1	**0.131**
74	US93	45.10	45.40	0.30	1767	0.141	1	1.27	1	1.16	1	1	1	1	0.86	1	1	0.94	1	**0.131**
75	US93	45.40	45.60	0.20	1767	0.094	1	1.27	1	1.16	1	1	1	1	0.86	1	1	0.94	1	**0.087**
76	US93	45.60	45.90	0.30	1767	0.141	1	1.27	1	1.16	1	1	1	1	0.86	1	1	0.94	1	**0.131**
77	US93	45.90	46.40	0.50	1767	0.235	1	1.27	1	1.16	1	1	1	1	0.86	1	1	0.94	1	**0.219**
78	US93	46.40	49.61	3.21	1767	1.509	1	1.27	1	1.16	1	1	1	1	0.86	1	1	0.94	1	**1.404**
:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:	:
SUM						**264.97**														**256.58**

Calibration Results

By applying all the applicable CMFs, as shown in Table 5, the total predicted crash frequency for the five-year study period is 256.58 crashes per year. The reported crashes on the 541 roadway segments for 2007 to 2011 were 1,546, so the average observed crash frequency over the five-year study period would be 309.20 (1,546/5) crashes per year. The calibration factor for the rural two-way two-lane roads can be calculated using equation as follows:

$$C_{Total\ Crash} = \frac{309.20}{256.58} = 1.21$$

The calibration factor is 1.21 that is more than 1.0. This means HSM SPF underestimates the crash frequencies for rural two-way two-lane roads by 21%. By applying the calibration factor, the Nevada-calibrated HSM SPF for rural two-way two-lane roads would be as follows:

$$N_{spf} = 1.21 \times AADT \times L \times 365 \times 10^{-6} \times e^{-0.312}$$

Considering 33.67% as the percentage of FI (fatal plus injury) crashes, the predicted number of injury crashes during the study period would be 86.621. Using 104.40 as the observed number of injury crashes over the study period, the calibration factor for the injury crashes is obtained as follows:

$$C_{FI\ Crash} = \frac{104.400}{86.621} = 1.21$$

In this study, calibration factors were calculated for each of the five years (2007 to 2011) as well as the five-year average. The results of the calibration are shown in Table 7.

Table 7 Calibration Factors for the Nevada-Rural Two-Way Two-Lane Roads

Facility Type	**Severity**	**Calibration Factors**					
		2007	2008	2009	2010	2011	**Six-Year Average**
Rural Two-way Two-Lane Roads	Total	1.38	1.29	1.16	1.12	1.08	**1.21**
	FI	1.51	1.20	1.14	1.03	1.01	**1.21**

As can be seen in Table 7, the calibration factors do not vary significantly from year to year and all of the calibration factors are more than 1.0.

As mentioned earlier, the HSM predictive method would not include the application of the CMF for shoulder rumble strips. However, the shoulder rumble strip CMF was included in the calibration calculation. To see the impact of shoulder rumble strip CMF, the calibration factor was calculated without applying this CMF to the predictive method calculation. The results showed that the application of shoulder rumble strip CMF did not change the results and calibration factors for both total and injury crashes were found to be 1.20.

A statistical comparison between the HSM SPFs, both calibrated and uncalibrated, is provided in the following section.

COMPARE THE PERFORMANCE OF CALIBRATED VERSUS UNCALIBRATED HSM SPF

To evaluate the prediction capability of developed calibration factors, the calibrated HSM SPF was compared to the uncalibrated HSM SPF using a number of techniques including visual plots, statistical goodness of fit (GOF) tests, and CURE (Cumulative Residuals) plots.

CURE Plots and Goodness-Off-Fit Tests for Validation of SPFs

CURE plot is a graphical method introduced by Hauer and Bamfo (*7*) to compare the different forms of SPFs. In this method, cumulative residuals which defined as the difference between the observed and predicted crash frequency, are plotted for each SPF model against the model explanatory variables (in this study AADT and segment length). Residuals below zero indicate that the model overestimates the number of crashes, while the residuals above zero indicate the model underestimate the number of crashes. If a model fits the data along the entire variable values, the cumulative residuals will fluctuate around zero (the horizontal axis) with no significant drops. Additionally, for the good fitting model, the cumulative residuals lie between the limits of two standard deviations ($\pm 2\sigma$) from the mean, both above and below zero which represent a 95% confidence interval. Hauer (*8*) explained the reason behind these limits, so as the residuals are random variables, their sum is also a random variable. Therefore, the sum has a mean and a standard deviation. For an unbiased model, the mean of this sum is zero for all the variable values. The standard deviation at i which is the index of variable values, can be calculated from the following equation:

$$\pm\hat{\sigma}_i = \hat{\sigma}_{\Sigma i}\sqrt{1 - \frac{{\hat{\sigma}_{\Sigma i}}^2}{{\hat{\sigma}_{\Sigma n}}^2}}$$

Where,

- ${\hat{\sigma}_{\Sigma i}}^2$, is the sum of the squared residuals at i; and
- ${\hat{\sigma}_{\Sigma n}}^2$, is the sum of the squared residuals at n, the last index.

Figure 4 shows an example of a CURE plot with the solid line representing the cumulative residuals and the dashed lines showing positive and negative two standard deviations limits.

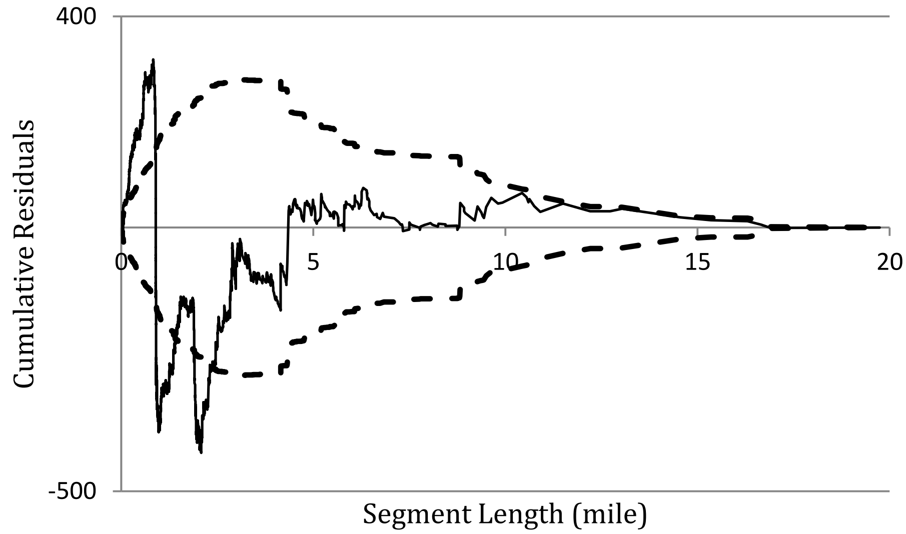


Figure 4 A Sample of CURE Plot for a Crash Prediction Model (*8*)

Washington et.al (*9*) recommended that several GOF tests be measured to assess how well the SPFs fit the data. They reported a series of statistical tests that can be used to validate models including the mean square error (MSE), the mean prediction bias (MPB), the mean absolute deviation (MAD), and the mean squared prediction error (MSPE). Three of these statistical tests were used to compare the performance of calibrated and uncalibrated SPFs in predicting Nevada crashes. These tests include the MPB, MAD, and MSPE that can be determined using the following equations.

$$MPB = \frac{\sum_{i=1}^{n} \hat{Y}_i - Y_i}{n}$$

$$MAD = \frac{\sum_{i=1}^{n} |\hat{Y}_i - Y_i|}{n}$$

$$MSPE = \frac{\sum_{i=1}^{n} (\hat{Y}_i - Y_i)^2}{n}$$

Where,

- $\hat{Y}_i$ is the predicted crash frequency at site i;
- Y_i is the observed crash frequency at site i; and

- n is the data sample size.

The smaller values of these tests are preferable to larger values and when their values are close to 0, it indicates a good fit or desirable SPF.

Another statistical test was measured in this study is called the Freeman Tukey R-Squared (R^2_{FT}) which was proposed by Fridstrom at.al (*10*). The equations required to calculate R^2_{FT} are given in the following equations.

$$R^2_{FT} = \frac{\sum_{i=1}^{n}(f_i - \bar{f})^2 - \sum_{i=1}^{n} \hat{e}_i^2}{\sum_{i=1}^{n}(f_i - \bar{f})^2}$$

$$f_i = \sqrt{Y_i} + \sqrt{Y_i + 1}$$

$$\bar{f} = \frac{\sum_{i=1}^{n}(\sqrt{Y_i} + \sqrt{Y_i + 1})}{n}$$

$$\hat{e}_i = \sqrt{Y_i} + \sqrt{Y_i + 1} - \sqrt{4 \times \hat{Y}_i + 1}$$

The larger value of R^2_{FT} are preferable and when its value is closer to 1.0, indicates a better fit.

The value of log likelihood (LL) was also considered as one of the GOF measures to be assessed in this study, because the HSM SPFs are developed based on the negative binomial model and these models parameters are estimated using the concept of likelihood maximization. The likelihood is the probability that the observed data will actually be realized under the given parameters estimates. The higher value of LL is an indicative of a better model. For each point in the dataset, the probability of having y_i crashes per year ($P(y_i)$) is calculated from the following equation showing the probability density function of a negative binomial model:

$$P(y_i) = \frac{\Gamma\left(y_i + \frac{1}{k}\right)}{y_i!\,\Gamma\left(\frac{1}{k}\right)} \left(\frac{k\mu_i}{1 + k\mu_i}\right)^{y_i} \left(\frac{1}{1 + k\mu_i}\right)^{\frac{1}{k}}$$

Where,

- y_i is the observed crash frequency at site i;
- μ_i is the predicted crash frequency at site i; and

- k is the overdispersion parameter.

The log likelihood (LL) value is computed as the natural log of the product of all probability values (for all the study sites).

$$LL = LN\left[\prod_{i=1}^{n} P(y_i)\right]$$

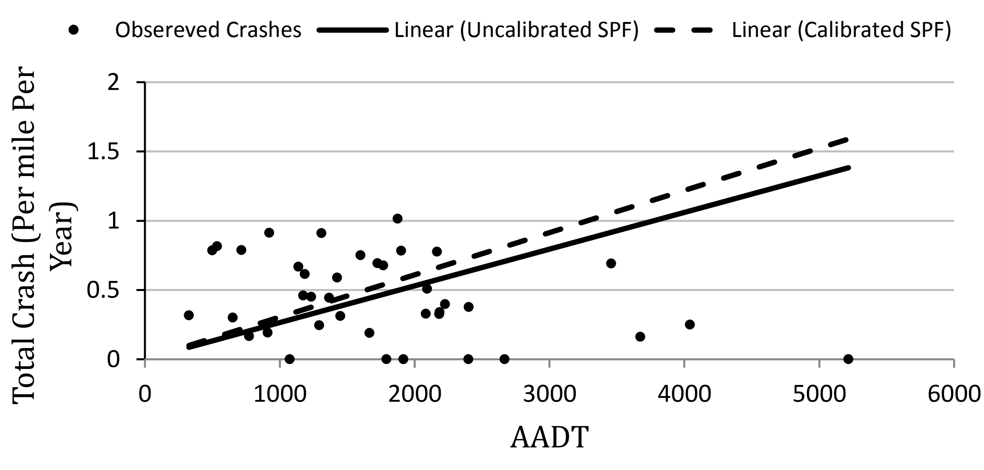
The overdispersion parameter was calculated from the equation below which is provided in HSM as a function of segment length (L):

$$k = \frac{0.236}{L}$$

Validation Results

Both calibrated and uncalibrated HSM SPFs were compared using the techniques were discussed in previous section.

Graphs showing the observed crashes (five-year averages), as well as uncalibrated and calibrated HSM SPF, are shown in Figure 5. It can be seen in both graphs that HSM SPF (the solid lines) consistently under-predict the total and FI crashes. These graphs only give a general idea of the SPFs' (uncalibrated and calibrated) fit to the observed data and it should be noted that a single point represents the average number of crashes (per mile per year) per number of sites which have the same AADTs.



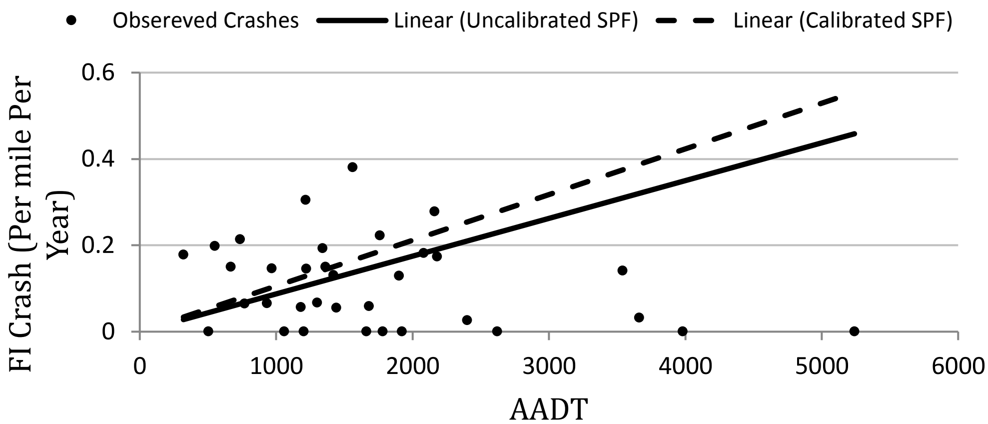


Figure 5 Fatal and FI Observed Crashes, and SPFs (Calibrated and Uncalibrated) for Rural Two-Way Tow-Lane Roads

Five statistical GOF tests including MPB, MAD, MSPE, Freeman Tukey R-Squared, and log likelihood were also used to compare the calibrated and uncalibrated HSM SPF for rural two-way two-lane roads. Table 8 summaries the results of this comparison. The highlighted values are related to the model which performed better.

Table 8 Statistical Comparison between Uncalibrated and Calibrated HSM SPFs

Severity	Uncalibrated HSM SPF					Calibrated HSM SPF				
	MSPE	MPB	MAD	R^2_{FT}	LL	MSPE	MPB	MAD	R^2_{FT}	LL
Total	0.567	-0.097	0.387	0.436	-442.104	0.633	0.002	0.424	0.445	-436.373
FI	0.087	-0.033	0.156	0.391	-232.430	0.090	0.001	0.165	0.415	-230.644

The calibrated HSM SPFs exhibited the lowest value for MPB, while the uncalibrated HSM SPFs exhibited the lowest values for MSPE and MAD. The calibrated SPFs exhibited the highest Freeman Tukey R-Squared and the maximum likelihood values. The results from theses GOF tests highlight the importance of performing several statistical tests, so that the result from one test does not reflect the results from the other tests. For example, based on MPB values the calibrated SPF performed better, while based on the MSPE values, the uncalibrated SPF exhibited a better fit. Therefore, in this study several GOF tests were measured as Washington et.al recommended. As can be seen in Table 8, the majority of measurements (3 out of 5) showed that the calibrated HSM SPF better fitted to the Nevada data.

CURE plots were also plotted for uncalibrated and calibrated HSM SPFs. Figure 6 shows the CURE plots as a function of AADT based on total and FI crashes. As can be seen in Figure 5, the cumulative residuals do not fluctuate around the horizontal axis for either the uncalibrated or the calibrated SPFs. The cumulative residuals for the uncalibrated SPF significantly deviate from the 95% confidence interval (shown as dashed lines) for all the AADTs, while the calibrated SPF exhibit less deviation from the 95% confidence interval especially for AADTs of more than 2000. For AADTs less than 2000, the calibrated SPFs still show significant deviation from the 95% confidence intervals. Based on the CURE plots, although the calibrated HSM SPF exhibited a better fit to the data than the uncalibrated HSM SPF, it still is not a representative of a good-fitting model. It might be required to modify the model parameters or consider a different functional form for the AADT to better represent the observed crash data in Nevada.

CURE plots as a function of segment length for total and FI crashes are also shown in Figure 7. As a function of segment length, the cumulative residuals for the calibrated HSM SPF fall mostly within the 95% confidence interval and fluctuate around the horizontal axis, while for the uncalibrated SPF there is a significant deviation from the 95% confidence interval. These results prove the existing of a linear relationship between the crash and the segment length for Nevada rural two-way two-lane roads.

Calibrated HSM SPF

Uncalibrated HSM SPF

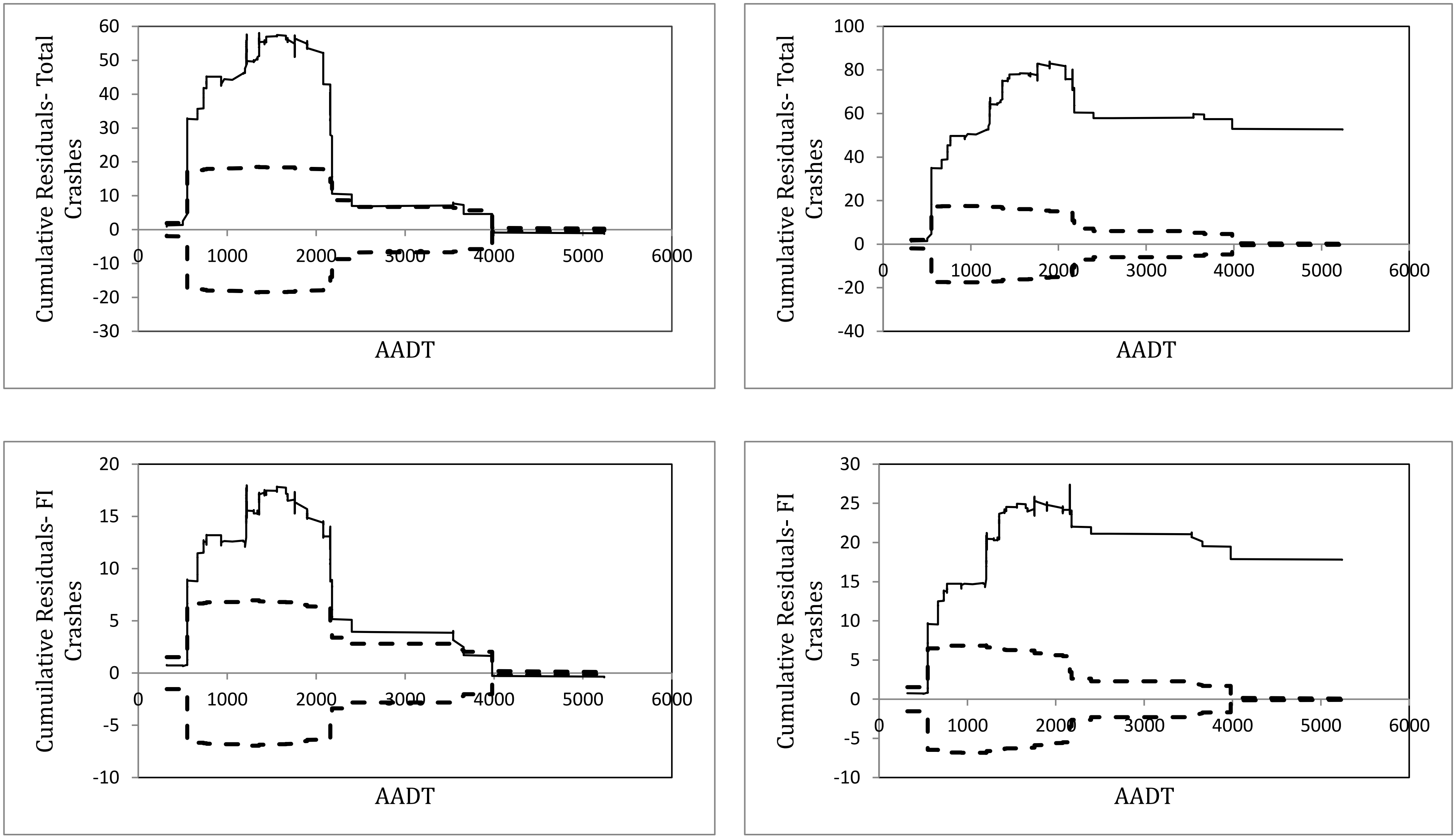


Figure 6 CURE Plots as a Function of AADT for Rural Two-Way Two-Lane Roads

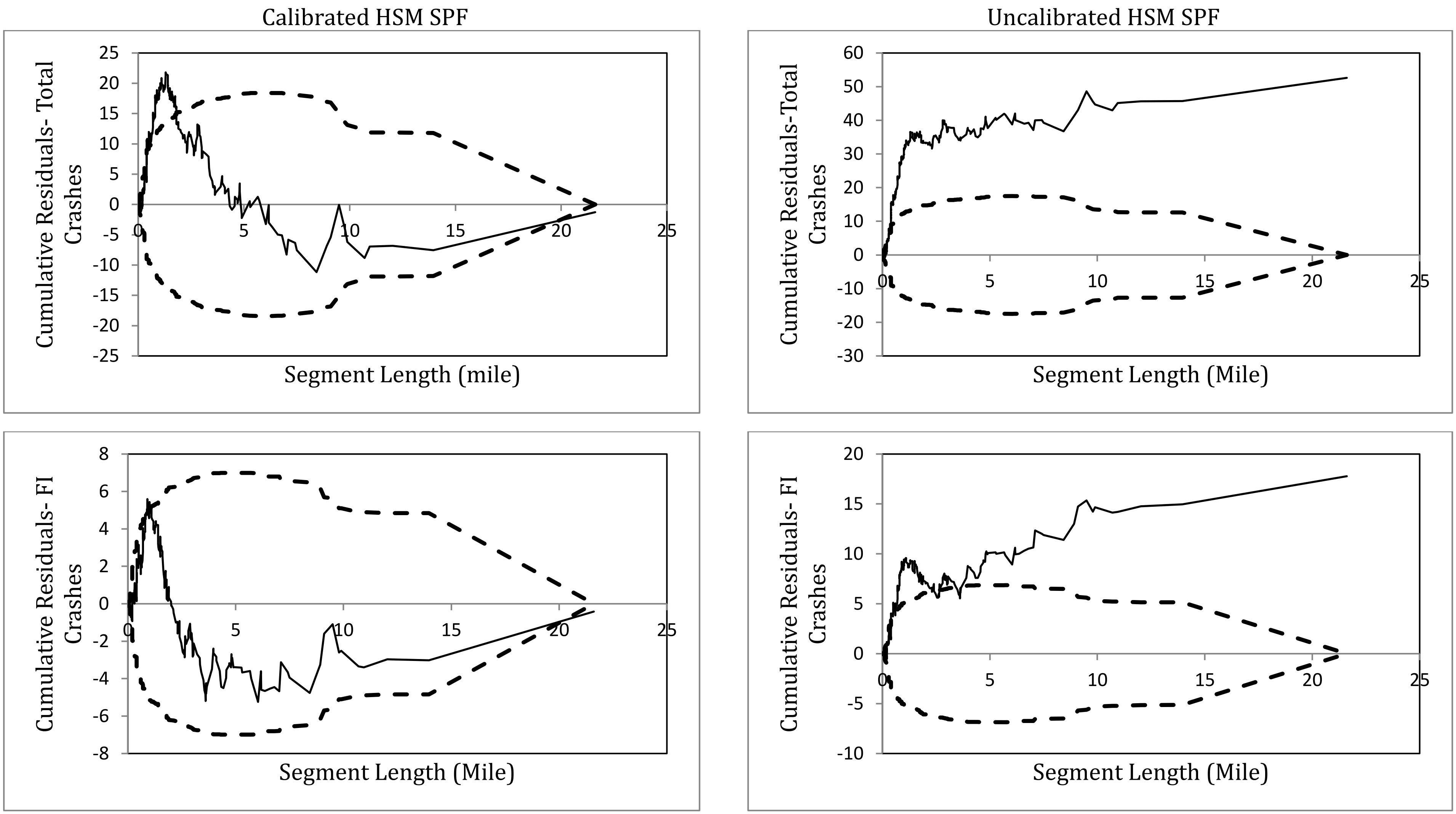


Figure 7 CURE Plots as a Function of Segment Length for Rural Two-Way Two-Lane Roads

CHAPTER 2 BEFORE-AND-AFTER STUDY

INTRODUCTION

Agencies are often required to evaluate the effectiveness of safety improvements and justify its implementation at other locations. The typical approach is to compare the crash data associated with the segment of a roadway before and after the implementation of specified treatment. This is the well-known Before-and-After Study. There are multiple procedures and methodologies that can be applied to such a study; however, a clear recommendation for NDOT is not provided in the Highway Safety Manual (HSM).

The primary objective of this task is to assist NDOT engineers and researchers to understand the before-and-after study concept, procedures, techniques, and data requirements. This understanding will enable them to effectively conduct such studies in the future. A secondary objective is to provide practitioners with a quick reference on the key elements and considerations of a valid before-and-after study.

This chapter documents the procedures and methodologies pertaining to the before-and-after evaluations. The first section outlines the background fundamentals and definitions required to understand the primary components of a before-and-after study. The concept section provides a detailed description of the basic four-step procedure for conducting a before-and-after study. An overview of the techniques adopted in conducting a before-and-after study was then presented. The four most commonly used approaches to perform a before-and-after study are explained in detail in terms of concept, data requirements, strengths and weaknesses. In the end, all the conclusions and recommendations are summarized. Along with the recommendations, implementation activities are identified for NDOT to utilize for conducting effective before-and-after studies. This will increase the likelihood that safety improvements will become more cost effective.

BACKGROUND INFORMATION

This chapter provides a propaedeutic of before-and-after studies including the definition of safety, the logical basis of a before-and-after study, the target crashes and potential causal factors that affect the safety performance of facilities.

Definition of Safety

Safety itself should be defined before attempting to discuss the level of safety of an entity, such as a road section, an intersection, etc. [*11*]. Safety can be seen as an attribute of the entity that is believed to be the same over time if all influencing parameters (such as environment, users, traffic volumes, etc.) remain unchanged [*11, 12*]. Consequently, the safety of an entity is defined as the number of crashes, or crash frequency, by kind and severity, expected to occur on the entity during a specified period of time. The expected value refers to a long-term average which would materialize if it were possible to keep constant traffic volumes, driver demography, environmental conditions, and all other relevant conditions of periods. In reality, the expected number of crashes is unknown; safety therefore can only be estimated. The precision of the estimate is usually expressed by its variance or standard deviation.

Logical Basis

The intuitive way to determine the effect of a treatment is to count the number of crashes in the 'before' period and compare it to the counting in the 'after' period. The 'before' and 'after' periods here are associated with the time point of the implementation of the treatment. The difference between the numbers of crashes is therefore attributed to the treatment assuming that nothing else has changed. This is the basic idea of a naïve before-and-after study which most agencies are executing. However, the safety of everything will change with time. Besides the treatment, other factors, such as traffic volume, traffic control device, environmental changes, etc. will have certain impact on safety. It is not rational to conclude that had no treatment been applied, safety in the 'after' period would have been the same as in the 'before' period. Therefore, to assess the effect of a treatment on some entity, it is necessary to compare what would have been the safety of the entity in the 'after' period had the treatment not been applied, to what the safety of the treated entity in the 'after' period was [*11*]. This statement is the logical basis of a before-and-after study. The safety of the treated entity in the 'after' period can be estimated by the observed number of crashes. The difficult task is to predict 'what would have been' the safety of the facility in the 'after' period had the treatment not been applied.

Target Crashes

In any before-and-after study, crashes can be grouped into two categories according to their relation to the treatment:

- **Target Crashes**: crashes that are expected to be materially affected by the treatment; and
- **Comparison Crashes**: crashes that are not expected to be affected by the safety treatment [*11*].

In real-world applications, there are challenges to differentiate and determine these two types of crashes. This division is not actually definitive in applications. Engineers and researchers need to have a firm understanding of the contributory factors in specific crash types in order to distinguish between the two types in a before-and-after study.

Potential Causal Factors

Two groups of factors will have an impact on the safety performance of facilities. The first group includes factors that can be recognized, measured, and explained, such as the exposure effect and treatment effect. The exposure effect is caused by the changes in traffic volumes and patterns on the roadway segment. Previous studies have shown that traffic volumes and crash frequencies have direct relationships. Common sense is that the crash frequency increases as traffic volume increases. In reality, this effect might be significant when the treatment applied changes the operation of the facility significantly. The other aspect of effect is from the treatment. The treatment effect is the target of a before-and-after study. It is therefore critical to isolate the treatment effect from other causal factors to determine the net improvement in terms of safety. The second group includes factors that cannot be recognized, measured or explained, such as the random and time trend effects. The type of effect refers to the regression-to-the-mean phenomenon which will be explained in detail in next section.

BASIC CONCEPT OF FOUR-STEP PROCEDURE FOR BEFORE-AND-AFTER STUDY

The key objective of a before-and-after study is to estimate the change of safety as a result of a treatment or countermeasure. The general idea to accomplish this goal is to estimate and compare the safety before and after the implementation of the treatment. According to the logical basis of the before-and-after study, it is necessary to predict what the expected number of crashes would have been in the 'after' period had the treatment not been implemented, and to compare this prediction with the expected number of crashes in the 'after' period with the treatment in place. Consequently, the many variants of before-and-after studies contain different ways to accomplish the following two tasks.

- Task 1: ***Predict*** what would have been the safety of treated entities in the 'after' period, had the treatment not been implemented; and
- Task 2: ***Estimate*** what the safety of treated entities in the 'after' period was.

A unified framework for conducting before-and-after studies was summarized by Ezra Hauer [*11*] and has been widely accepted and applied. The universal process consists of four basic steps and this chapter presents the concepts of this procedure.

General Notation

Before formulating the procedures, some notation and a few basic results are introduced first. The notation illustrated in Table 9 will be referred to throughout this report.

Generally the expected safety in the 'after' period is denoted as λ, and the expected safety in the 'after' period had the treatment not been installed is denoted as π. Using these safety indexes, the effectiveness of a treatment is estimated by two measures. Firstly, the difference between π and λ is defined as the change in safety due to the treatment (δ). Secondly, the index of effectiveness (θ) is calculated as the ratio of λ and π.

The expected number of target crashes refers to the average crash counts in a long run. Therefore, the parameters π, λ, δ, and θ are unknown parameters, but can be estimated from observed data. Estimated values are correspondingly designated by a caret above the symbol as $\hat{\pi}$, $\hat{\lambda}$, $\hat{\delta}$, and $\hat{\theta}$ hereafter.

Table 9 General Notation for Before-and-After Studies

Notation Marks	Description
π	The expected[2] number of target crashes of treated entities in the 'after' period had treatment not been installed. π is the value that has to be predicted.
λ	The expected number of target crashes of treated entities in the 'after' period with the implementation of the treatment. λ is usually estimated by the observed number of crashes in the 'after' period with the treatment in place.
$\boldsymbol{\delta = \pi - \lambda}$	Reduction of the expected number of target crashes due to the treatment.
$\boldsymbol{\theta = \lambda/\pi}$	Index of Effectiveness (also referred as the Odds Ratio): the ratio of what the safety was with the treatment to what it would have been without the treatment.

Intuitively, the expected safety in the 'after' period λ can be estimated by the observed number of target crashes. It might seem to be true that the observed number of target crashes in the 'before' period would be employed to predict the expected safety if the treatment was not applied, i.e., the value of π. However, this thought is naïve to some extent. The definition of π is the expected number of target crashes in the 'after' period had the treatment NOT been applied. This definition describes an eventuality that does not have a materialized circumstance to rely on. In reality, there are many factors that may potentially change from the 'before' to the 'after' period, such as traffic volumes, weather conditions, crash reporting thresholds, changes in vehicle codes, the probability of reporting, the driving populations, etc. All these changes on the other hand might influence the safety of the treated entity to different extents. Therefore, a sound approach is needed to extrapolate this variable. In fact, the many variants of before-and-after studies depend entirely on the approaches by which the estimate of π is obtained.

[2] The expected value is referred to as an average value that would be obtained in a very large number of trails, and is denoted as E[·].

Furthermore, before-and-after studies consider two measures of safety effectiveness. Firstly, the value of δ is defined here as the reduction of the expected number of target crashes in the 'after' period. To further explain, if δ is greater than 1, one can conclude the treatment is effective since there will be a reduction in the number of crashes. Secondly, the index of effectiveness θ is also widely used in real-world applications. To further explain, when the ratio θ is smaller than 1, one can conclude that the treatment is effective, but when θ is larger than 1, the treatment is considered as harmful to safety. Also, the value of $100 \times (1-\theta)$ defines the percentage of reduction in the expected crash frequency, which is well known as the Crash Modification Factor (CMF). For example, a value of $\theta = 0.75$ indicates an estimated 25% reduction in target crashes due to the treatment.

Using these indices, the following sections present the four-step procedures of before-and-after studies for a single entity (e.g., a highway segment, a signalized intersection, etc.) and composite entities (e.g., combinations of rural two-way two lane highways and freeway facilities). It should be noted that no matter what approaches are employed, the four-step procedure is the fundamental part in estimating the safety effect of a treatment.

Four-Step Procedure for a Single Entity

The estimates of π and λ are assumed to follow Poisson distribution and are statistically independent. The following four steps demonstrate the basic procedures of conducting a before-and-after study for a single entity.

Step 1 Estimate λ and predict π.

The first step is to estimate parameters λ and π. As mentioned previously, the estimate of λ is equal to the number of observed target crashes in the 'after' period. The predicted value of π can be obtained through several different approaches to be presented in the next chapter. Therefore,

$$E[\hat{\lambda}] = \lambda; E[\hat{\pi}] = \pi$$

Step 2 Estimate variances of λ and π, $V\hat{A}R[\hat{\lambda}]$ and $V\hat{A}R[\hat{\pi}]$.

The second step is to estimate the variance of $\hat{\lambda}$ and $\hat{\pi}$. The underlying assumption here is that the target crashes are Poisson distributed with the variance equal to the mean. The estimates of the variance also depend on the method chosen to estimate λ and predict π. Therefore,

$$V\hat{A}R[\hat{\lambda}] = VAR[\hat{\lambda}] = E[\hat{\lambda}] = \lambda$$

$$V\hat{A}R[\hat{\pi}] = VAR[\hat{\pi}] = E[\hat{\pi}] = \pi$$

Step 3 Estimate measures of safety effectiveness, δ and θ.

The third step is to estimate the measures of safety effectiveness. They are both closely related to the estimates of λ and π.

$$\hat{\delta} = \hat{\pi} - \hat{\lambda};\ E[\hat{\delta}] = E[\hat{\pi} - \hat{\lambda}] = E[\hat{\pi}] - E[\hat{\lambda}] = \pi - \lambda$$

The estimator $\hat{\delta}$ is an unbiased estimator of δ, however, the estimator $\hat{\theta}^* = \hat{\lambda}/\hat{\pi}$ is not an unbiased estimator for a small sample, given,

$$E[\hat{\theta}^*] = E[\hat{\lambda}/\hat{\pi}] \neq \lambda/\pi = \theta$$

An unbiased estimator of θ is given as follows (*11*):

$$\hat{\theta} \cong \frac{\left(\frac{\hat{\lambda}}{\hat{\pi}}\right)}{\left(1 + \frac{VAR[\hat{\pi}]}{\hat{\pi}^2}\right)}$$

Step 4 Estimate variances of δ and θ, $V\hat{A}R[\hat{\delta}]$ and $V\hat{A}R[\hat{\theta}]$.

The last step is to estimate the variance of δ and θ as follows.

$$V\hat{A}R[\hat{\delta}] = V\hat{A}R[\hat{\pi} - \hat{\lambda}] = V\hat{A}R[\hat{\pi}] + V\hat{A}R[\hat{\lambda}] = \hat{\pi} + \hat{\lambda}$$

$$VAR[\hat{\theta}] = \frac{\hat{\theta}^2 \left\{\frac{VAR[\hat{\lambda}]}{\hat{\lambda}^2} + \frac{VAR[\hat{\pi}]}{\hat{\pi}^2}\right\}}{\left\{1 + \frac{VAR[\hat{\pi}]}{\hat{\pi}^2}\right\}^2}$$

By using these statistical calculations, the magnitude of the measures of effectiveness, $\hat{\delta}$ and $\hat{\theta}$, and their variances are obtained. The results of a before-and-after study are therefore demonstrated and described by the two measures, $[\hat{\delta}, V\hat{A}R\{\hat{\delta}\}]$, and $[\hat{\theta}, V\hat{A}R\{\hat{\theta}\}]$. The estimates $\hat{\delta}$ and $\hat{\theta}$ are used to measure the effect size, and their variances can be used to approximate the 'level of confidence' of the results [*11*]. Table 10 summarizes the formulas required in each step for conducting a before-and-after study for a single entity. These equations are referred as a common platform hereafter in various before-and-after study approaches.

Table 10 Basic Formulas of the Four-Step Before-and-After Study

Steps	Goals	Formulas for Four-Step Before-and-After Study
Step 1	Estimate λ and predict π	$\hat{\lambda}$ = observed number of crashes (after) $\hat{\pi}$ = predicted number of crashes(after)
Step 2	Estimate $\mathrm{VAR}[\hat{\lambda}]$ and $\mathrm{VAR}[\hat{\pi}]$	$\mathrm{VAR}[\hat{\lambda}] = \hat{\lambda}$ $\mathrm{VAR}[\hat{\pi}] = \hat{\pi}$
Step 3	Estimate δ and θ	$\hat{\delta} = \hat{\pi} - \hat{\lambda}$ $\hat{\theta} \cong \dfrac{\left(\frac{\hat{\lambda}}{\hat{\pi}}\right)}{\left(1 + \frac{\mathrm{VAR}[\hat{\pi}]}{\hat{\pi}^2}\right)}$
Step 4	Estimate $\mathrm{VAR}[\hat{\delta}]$ and $\mathrm{VAR}[\hat{\theta}]$	$\mathrm{VAR}[\hat{\delta}] = \mathrm{VAR}[\hat{\lambda}] + \mathrm{VAR}[\hat{\pi}] = \hat{\lambda} + \hat{\pi}$ $\mathrm{VAR}[\hat{\theta}] = \dfrac{\hat{\theta}^2 \left\{\frac{\mathrm{VAR}[\hat{\lambda}]}{\hat{\lambda}^2} + \frac{\mathrm{VAR}[\hat{\pi}]}{\hat{\pi}^2}\right\}}{\left\{1 + \frac{\mathrm{VAR}[\hat{\pi}]}{\hat{\pi}^2}\right\}^2}$

Four-Step Procedure for a Composite Entity

Sometimes the same treatment is applied to several entities or locations. The corresponding safety effect is thought to be the combination of each individual entity or site. The similar and more general four-step procedure for a composite entity is illustrated below. Table 11 summarizes the general formulas required in each step.

Step 1 Estimate λ(j) and predict π(j) for each entity, j=1, ..., n.

The overall estimations of λ and π are $\hat{\lambda} = \sum \hat{\lambda}(j)$ and $\hat{\pi} = \sum \hat{\pi}(j)$.

Step 2 Estimate variances of λ(j) and π(j) for each entity, $VAR[\hat{\lambda}(j)]$ and $VAR[\hat{\pi}(j)]$.

The overall estimations of $\mathrm{VAR}[\hat{\lambda}]$ and $\mathrm{VAR}[\hat{\pi}]$ are $\mathrm{VAR}[\hat{\lambda}] = \sum \mathrm{VAR}[\hat{\lambda}(j)]$ and $\mathrm{VAR}[\hat{\pi}] = \sum \mathrm{VAR}[\hat{\pi}(j)]$.

Step 3 Estimate δ and θ.

Step 4. Estimate VAR{$\hat{\delta}$} and VAR{$\hat{\theta}$}.

Table 11 Formulas of the Four-Step Procedure for a Composite Entity

Steps	Goals	Formulas for Naïve Before-and-After Study
Step 1	Estimate λ and predict π	$\hat{\lambda} = \sum_{j=1}^{n} \hat{\lambda}(j)$ $\hat{\pi} = \sum_{j=1}^{n} \hat{\pi}(j)$
Step 2	Estimate $VAR[\hat{\lambda}]$ and $VAR[\hat{\pi}]$	$VAR[\hat{\lambda}] = \sum_{j=1}^{n} VAR[\hat{\lambda}(j)] = \sum_{j=1}^{n} \hat{\lambda}(j) = \hat{\lambda}$ $VAR[\hat{\pi}] = \sum_{j=1}^{n} VAR[\hat{\pi}(j)] = \sum_{j=1}^{n} \hat{\pi}(j) = \hat{\pi}$
Step 3	Estimate δ and θ	$\hat{\delta} = \hat{\pi} - \hat{\lambda}$ $\hat{\theta} \cong \frac{\left(\frac{\hat{\lambda}}{\hat{\pi}}\right)}{\left(1 + \frac{VAR[\hat{\pi}]}{\hat{\pi}^2}\right)}$
Step 4	Estimate $VAR[\hat{\delta}]$ and $VAR[\hat{\theta}]$	$VAR[\hat{\delta}] = VAR[\hat{\lambda}] + VAR[\hat{\pi}] = \hat{\lambda} + \hat{\pi}$ $VAR[\hat{\theta}] = \frac{\hat{\theta}^2 \left\{\frac{VAR[\hat{\lambda}]}{\hat{\lambda}^2} + \frac{VAR[\hat{\pi}]}{\hat{\pi}^2}\right\}}{\left\{1 + \frac{VAR[\hat{\pi}]}{\hat{\pi}^2}\right\}^2}$

Section Summary

This section presents the general concepts and expressions for conducting a before-and-after study. The entire process is structured into four basic steps illustrated in Figure 8. The results of a before-and-after study are described by measures of effectiveness, $[\hat{\delta}, \widehat{VAR}\{\hat{\delta}\}]$, and $[\hat{\theta}, \widehat{VAR}\{\hat{\theta}\}]$. In actual field applications, engineers can choose either $\hat{\delta}$ or $\hat{\theta}$ to estimate the safety effect of a treatment. In many previous literatures, researchers prefer θ, the index of effectiveness, as a measure of choice and it is often used to generate the CMFs.

This section also pointed out that to produce the prediction of π would be more complicated since it describes the eventuality that is not likely to occur. The different approaches to be introduced in next section will mainly focus on different before-and-after study approaches to predict values of π.

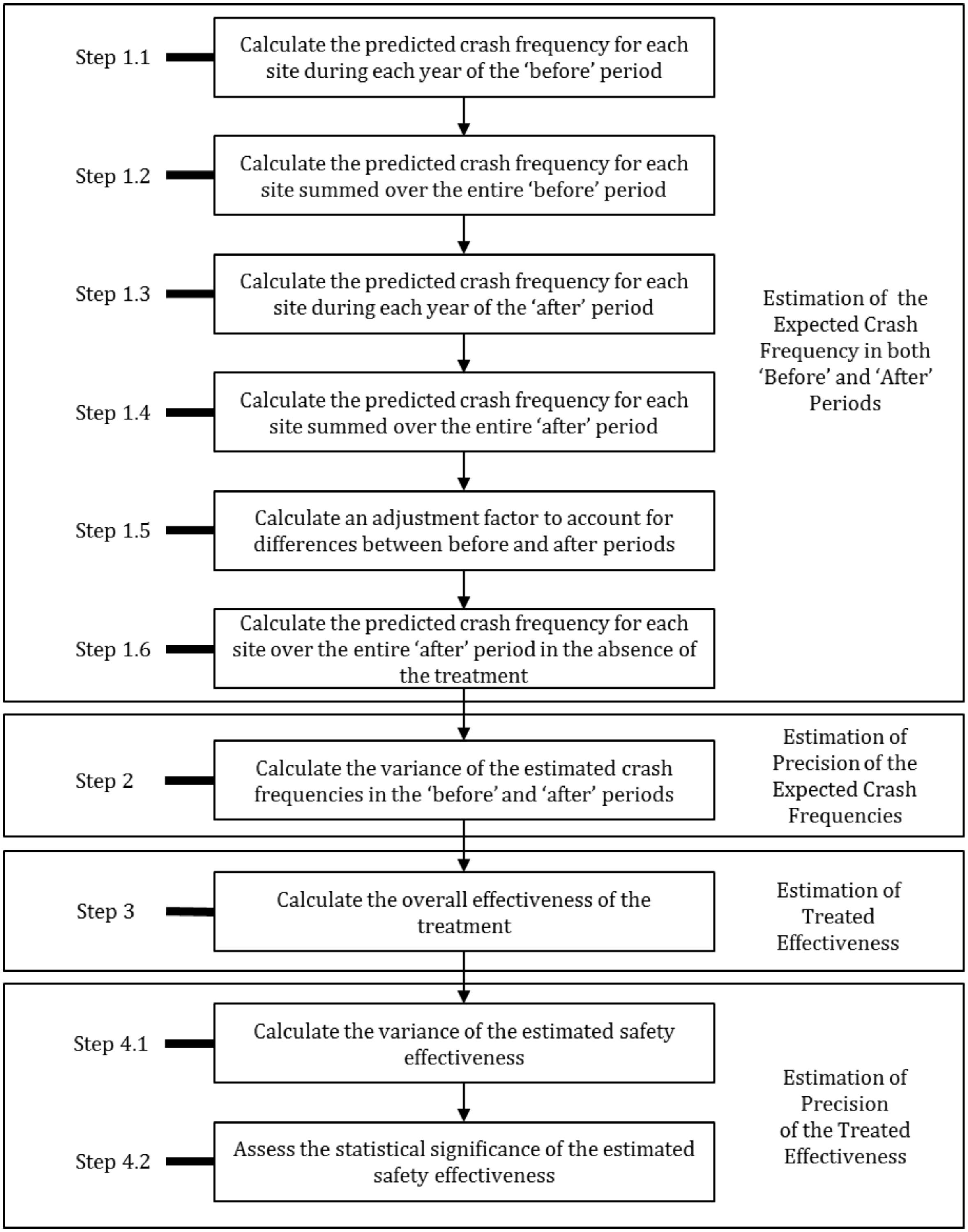


Figure 8 Overview of the Four-Step Procedure for Before-and-After Studies

BEFORE-AND-AFTER STUDY APPROACHES

As discussed previously, two essential tasks of a before-and-after study are the ***prediction*** of what would have been the safety in the 'after' period had the treatment not been installed, and the ***estimation*** of what safety in the 'after' period actually was. The estimation of safety in the 'after' period is usually obtained by counting the observed number of crashes. However, to predict the safety of the treated entities had the treatment not been applied is usually much more complicated. This is due to the fact that this is an eventuality that did not materialize and therefore could not have been observed. Previous studies have developed diverse ways to predict what would have occurred, ranging from assuming nothing would have changed from before to after circumstances, to considering and accounting for the changes in traffic volumes or other autonomous factors. Although there are many variants in predicting methods, assumptions, and applications, the general concept of the prediction in before-and-after evaluation approaches consist of two portions.

- A ***prediction foundation*** that estimates the expected crash frequency of target crashes in the 'before' period.
- A ***prediction method*** that predicts how the expected number of target crashes would have changed from the 'before' to 'after' periods as a result of changes in all other factors (e.g., traffic volumes, weather conditions and etc.) but not the addition of the treatment.

This section forms an overview of the four most commonly used approaches to perform a before-and-after study including the Naïve before-and-after study approach, before-and-after study with Yoked Comparison, before-and-after study with Comparison Group, and before-and-after study with Empirical Bayes (EB) approach. The approaches are presented in order of increasing complexity.

Starting Point: Naïve Before-and-After Study

The simplest technique for conducting a before-and-after study is known as the naïve approach. It is a natural starting point of a before-and-after study. In this approach, the observed crashes in the 'before' period are used to predict the expected number of crashes (π) if the safety treatment had not been implemented. Consequently, the change in number of crashes from the 'before' to the 'after' periods is considered as the treatment effect. An illustration of the naïve assumption is displayed in Figure 9 below. In this example, the duration of 'before' and 'after' data is 5-years, and k_i and l_i represent the observed number of target crashes in the 'before' and 'after' periods respectively. Hence, the estimate of π is considered the summation of all the observed number of crashes in the 'before' period.

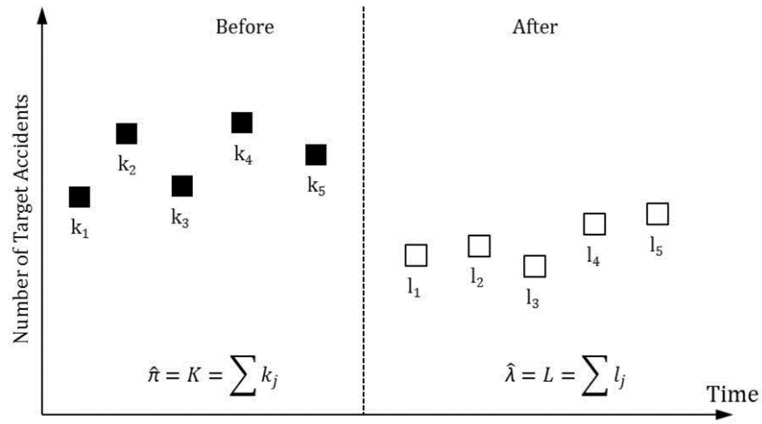


Figure 9 Illustration of a Naïve Before-and-After Study

This way of predicting is straightforward, but reflects an unrealistic belief that the passage of time from the 'before' to the 'after' period is not associated with the changes in safety. The following facts challenge the naïve assumption.

- In reality, traffic volumes and environmental factors will change autonomously over time. The change in safety from 'before' to 'after' period using the naive approach is actually the combinations of the effect of changes in all these factors in addition to the treatment.
- The count of Property Damage Only (PDO) crashes will be easily affected by the cost of repairs, which will also change gradually over time [*11*]. Similarly, the crash records may change suddenly because of the adjustments to the legally reporting limit.
- The treated entities would likely have been selected for treatment due to the unusual high number of crashes. It is hard to hope that this unusual high number is a good basis for predicting what would be expected in the future had the treatment not been applied. Therefore, if the analysis shows the treatment was effective, it is probably because of the regression-to-the-mean phenomenon, at least in part.

To conclude, the observed number of crashes in the 'before' period represents the safety in the 'after' period only if all safety related factors are constant in both the before and after periods, save for the treatment. The naïve approach actually estimates a mix of what is due to the treatment and what is caused by other factors and is influenced by other circumstances described above. It is not exactly known which part of the change is attributed to the treatment. The Naïve approach is the simplest approach for before-and-after studies. The strengths and weaknesses of the naïve before-and-after approach are summarized below.

  - The concept is a simple, straightforward approach and is easy to apply.
  - This method can provide very precise results with small standard deviations.
- **Weaknesses**
  - This approach is unable to separate the treatment effect from the other effects.
  - The effect of the passage of time on the safety of a facility is ignored.

Even though the underlying assumptions may be questionable, this approach serves as a starting point for the before-and-after analysis, and provides the results that might serve as a baseline. Table 12 summarizes the formulas for a Naïve before-and-after study.

Table 12 Formulas of the Four-Step Naïve Before-and-After Study

Steps	Goals	Formulas for Naïve Before-and-After Study
Step 1	Estimate λ and predict π	$\hat{\lambda} = \mathrm{L}$[3]; $\hat{\pi} = \mathrm{K}$[4]
Step 2	Estimate $\mathrm{VAR}[\hat{\lambda}]$ and $\mathrm{VAR}[\hat{\pi}]$	$\mathrm{VAR}[\hat{\lambda}] = \hat{\lambda} = \mathrm{L}$ $\mathrm{VAR}[\hat{\pi}] = \hat{\pi} = \mathrm{K}$
Step 3	Estimate δ and θ	$\hat{\delta} = \hat{\pi} - \hat{\lambda} = \mathrm{K} - \mathrm{L}$ $\hat{\theta} \cong \frac{\left(\frac{\hat{\lambda}}{\hat{\pi}}\right)}{\left(1 + \frac{\mathrm{VAR}[\hat{\pi}]}{\hat{\pi}^2}\right)} = \frac{\left(\frac{\mathrm{L}}{\mathrm{K}}\right)}{\left(1 + \frac{\mathrm{K}}{\mathrm{K}^2}\right)}$
Step 4	Estimate $\mathrm{VAR}[\hat{\delta}]$ and $\mathrm{VAR}[\hat{\theta}]$	$\mathrm{VAR}[\hat{\delta}] = \mathrm{VAR}[\hat{\lambda}] + \mathrm{VAR}[\hat{\pi}] = \hat{\lambda} + \hat{\pi} = \mathrm{K} + \mathrm{L}$ $\mathrm{VAR}\{\hat{\theta}\} = \frac{\hat{\theta}^2 \left\{\frac{\mathrm{VAR}[\hat{\lambda}]}{\hat{\lambda}^2} + \frac{\mathrm{VAR}[\hat{\pi}]}{\hat{\pi}^2}\right\}}{\left\{1 + \frac{\mathrm{VAR}[\hat{\pi}]}{\hat{\pi}^2}\right\}^2} = \frac{\hat{\theta}^2 \left\{\frac{\mathrm{L}}{\mathrm{L}^2} + \frac{\mathrm{K}}{\mathrm{K}^2}\right\}}{\left\{1 + \frac{\mathrm{K}}{\mathrm{K}^2}\right\}^2}$

[3] L: observed number of crashes in the 'after' period at treated entities.
[4] K: observed number of crashes in the 'before' period at treated entities.

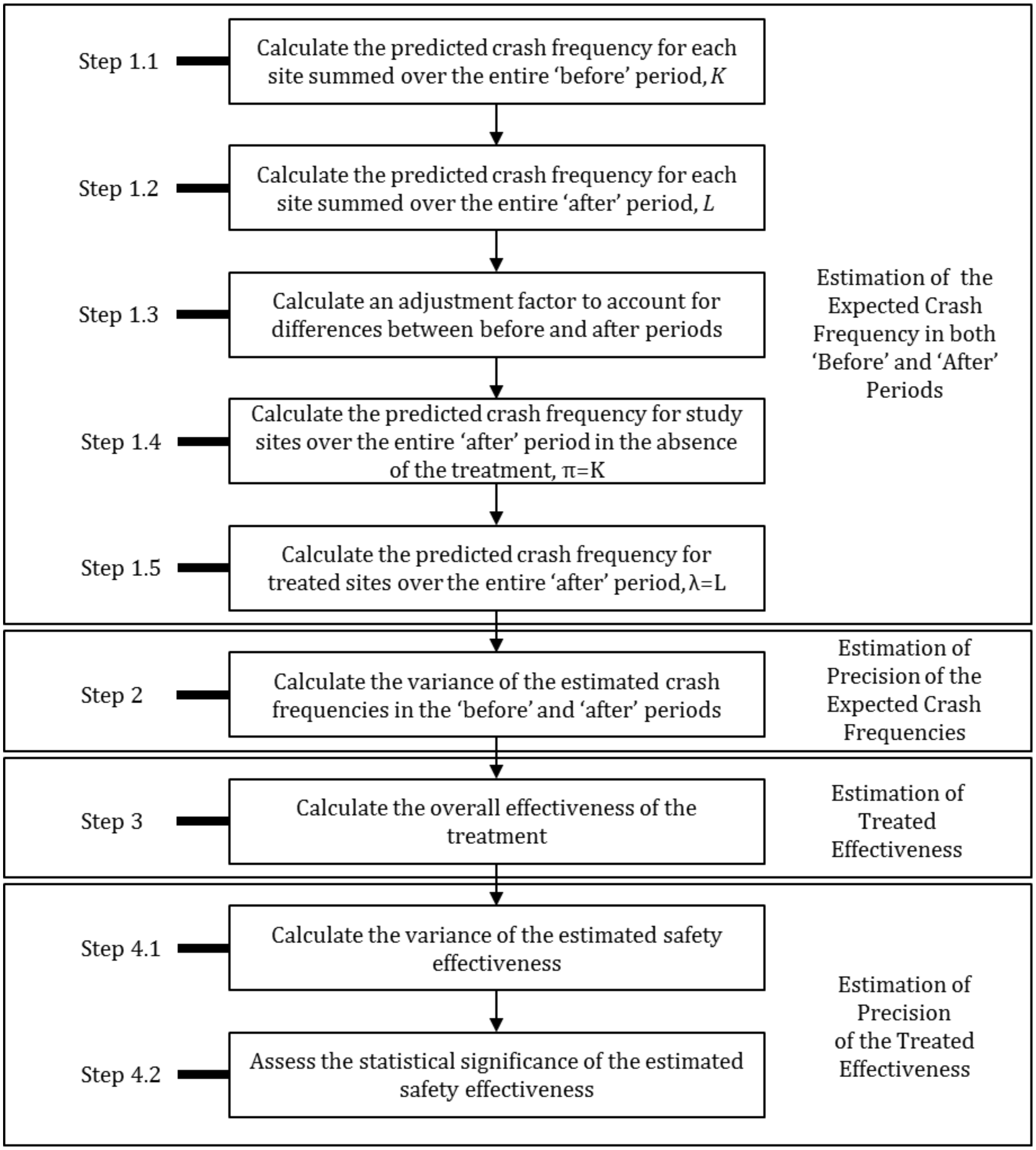


Figure 10 Four-Step Procedure for Naïve Before-and-After Studies

Before-and-After Study with Comparison Group

As time passes, many causal factors will have an impact on the safety from 'before' to 'after' periods besides the treatment. Some of the causal factors can be recognized, measured, and understood, such as traffic volume. However, some uncertain factors might not be recognized, measured, or understood. An idea of using the comparison group to implicitly explain those influences is introduced in this section. The approach is known as the Comparison Group (C-G) Method.

The idea of before-and-after studies with the C-G method is to identify a group of entities that are similar to the treated entities but remained untreated. The treated sites form the 'treatment group', and the untreated sites form the 'comparison group'. The basic assumptions behind the C-G method are,

- The safety changes from the 'before' to the 'after' period in the comparison group are indicative of how the safety on the treatment group would have changed.
- The sundry factors that impact safety will change from 'before' period to 'after' period in the same manner on both the comparison group and treatment group.
- The changes of various factors will influence the safety of the treatment group and the comparison group in the same way.

To provide a better demonstration of this approach, a set of hypothetical data of target crashes was created in Table 13 and Figure 11. Assuming that the study duration is 5-years in both the before and after periods. In Table 13, the letters, K, L, M and N denote the total number of target crashes that correspond to the before and after periods counts. The expected values of those counts are denoted by Greek letters, κ, λ, μ, and ν.

Table 13 Observed Number of Crashes and Expected Values in the C-G Method

Study Periods	Treated Sites	Comparison Sites
Observed crashes in the 'before' period	$K = \sum k_i$, i=1…5	$M = \sum m_i$, i=1…5
Observed crashes in the 'after' period	$L = \sum l_i$, i=1…5	$N = \sum n_i$, i=1…5
Expected crashes in the 'before' period	κ	μ
Expected crashes in the 'after' period	λ	ν

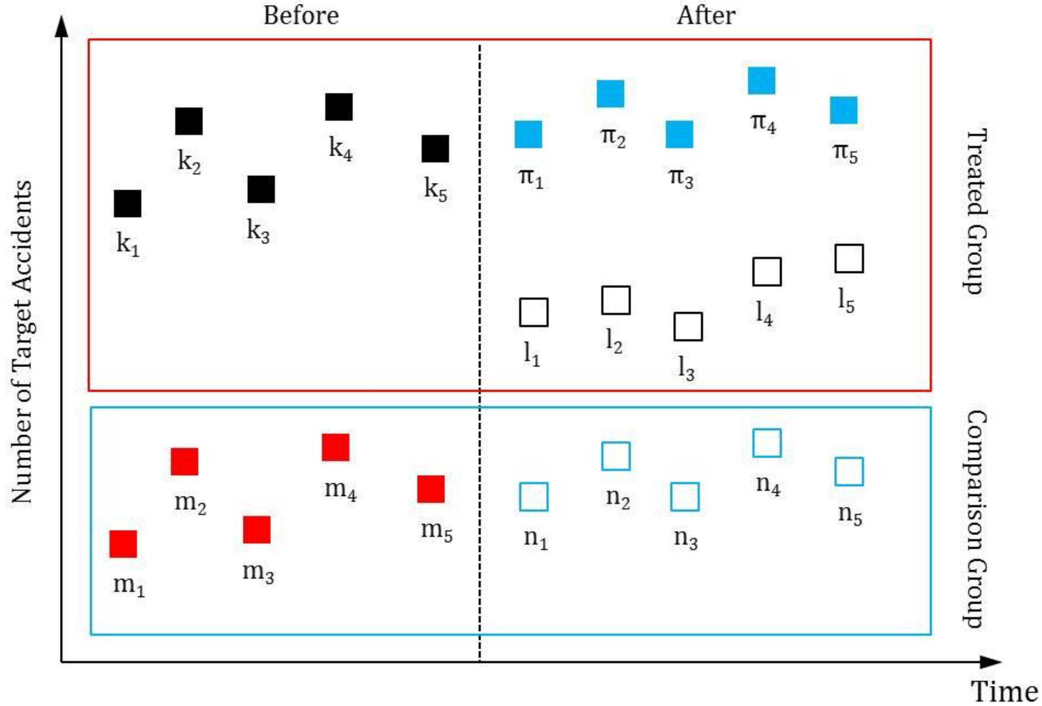


Figure 11 Illustration of Before-and-After Study with C-G Method

In order to predict π, the expected number of crashes in the 'after' period, had the treatment not been applied, two terms are defined to assist the explanation and inferences.

- $\mathbf{r_C = \nu/\mu}$ Comparison Ratio for the comparison group: the ratio of expected number of 'after' to the expected number of 'before' target crashes of the comparison group;
- $\mathbf{r_T = \pi/\kappa}$ Comparison Ratio for the treatment group: the ratio of expected number of 'after' to the expected number of 'before' target crashes of the treatment group.

According to the basic assumptions in the C-G method, it is rational to derive the relations between the treatment group and comparison groups that $r_C = r_T$. This inference means that, in the absence of treatment, the ratio of the expected number of crashes from before to after periods in the comparison group equals the ratio of the expected number of crashes from before to after period in the treatment group. Therefore, the following equations can be derived.

$$\pi = r_T\kappa \Rightarrow \pi = r_C\kappa \Rightarrow \pi = \frac{\nu}{\mu} \times \kappa$$

This derivation indicates that the predicted expected number of crashes in the 'after' period had the treatment not been applied equals the comparison ratio multiplied by the expected number of 'before' crashes on the treatment group. Since the values of r_C, μ and ν can be estimated from the number of crashes (M and N) in the comparison group, and κ can be estimated by the number of crashes (K), the value of π can easily be estimated. Table 14 below summarizes the formulas for a before-and-after study with the C-G Method.

Table 14 Formulas of the Four-Step Before-and-After Study with the C-G Method

Steps	Goals	Formulas for Before-and-After Study with the C-G method
Step 1	Estimate λ and predict π	$\hat{\lambda} = \mathrm{L}$ $\hat{\pi} = \hat{r_T} \cdot \kappa = \hat{r_C} \cdot \kappa = \frac{\left(\frac{N}{M}\right)}{\left(1+\frac{1}{M}\right)} \cdot K$
Step 2	Estimate $VAR[\hat{\lambda}]$ and $VAR[\hat{\pi}]$	$VAR[\hat{\lambda}] = \hat{\lambda} = \mathrm{L}$ $VAR[\hat{\pi}] \cong \hat{\pi}^2 \cdot \left[\frac{1}{K} + \frac{VAR[\hat{r_C}]}{\hat{r_C}^2}\right]$
Step 3	Estimate δ and θ	$\hat{\delta} = \hat{\pi} - \hat{\lambda}$ $\hat{\theta} \cong \frac{\left(\frac{\hat{\lambda}}{\hat{\pi}}\right)}{\left(1+\frac{VAR[\hat{\pi}]}{\hat{\pi}^2}\right)}$
Step 4	Estimate $VAR[\hat{\delta}]$ and $VAR[\hat{\theta}]$	$VAR[\hat{\delta}] = VAR[\hat{\lambda}] + VAR[\hat{\pi}] = \hat{\lambda} + \hat{\pi}$ $VAR\{\hat{\theta}\} = \frac{\hat{\theta}^2 \left\{\frac{VAR[\hat{\lambda}]}{\hat{\lambda}^2} + \frac{VAR[\hat{\pi}]}{\hat{\pi}^2}\right\}}{\left\{1+\frac{VAR[\hat{\pi}]}{\hat{\pi}^2}\right\}^2}$

The C-G method is recommended when over 5-years of crash data in the 'before' period is available and sufficient for both treatment group and comparison group. It can be indicated that the difference between the C-G before-and-after study and naïve before-and-after study lies in the estimation of π, the expected number of crashes in the after period had the treatment not been implemented. In the naïve approach, the estimates of π equal the observed number of crashes that occurred in the before period (K) assuming that nothing would change. By adding a correction factor ($\hat{r_T}$, or $\hat{r_C}$), the C-G method modifies the estimates of π as $\hat{\pi} = \hat{r_T}K$. The purpose is to account for the effect of changes in various uncontrolled causal factors using the comparison group. However, because of the additional source of variance, the statistical precision in the estimation of π using the C-G method is less than that achievable by the naïve method. Just as the underlying assumption in the Naïve method is not exactly true, so are the assumptions of the C-G method. The strengths and weaknesses of the before-and-after study with the Comparison Group are summarized below.

- **Strengths**
  - The concept is clear and easy to grasp.
  - It has better theoretical grounds than the naïve before-and-after study approach.
- **Weaknesses**
  - Application of this approach can be challenging because the development of a comparison group can be difficult.

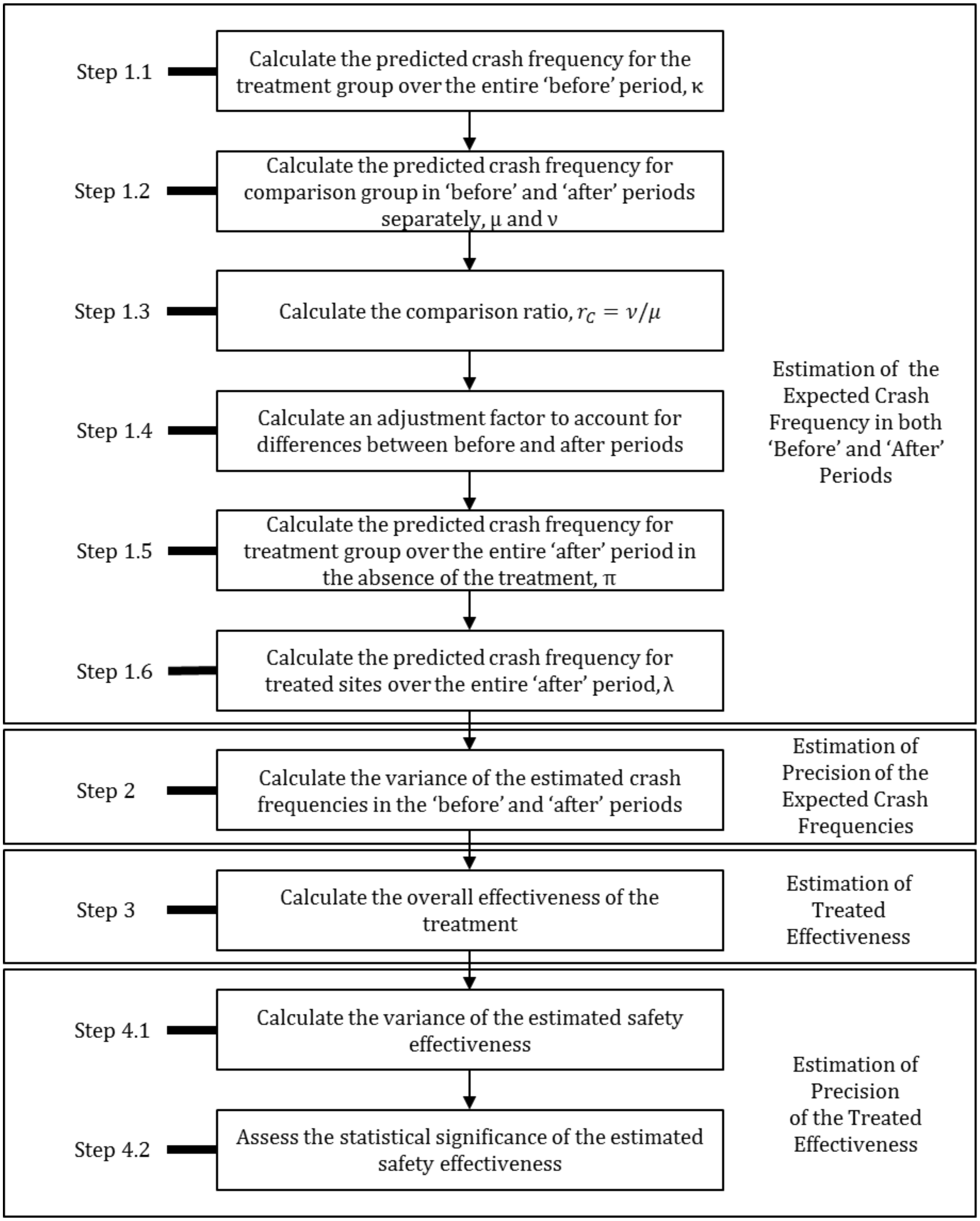


Figure 12 Four-Step Procedure for Before-and-After Studies with C-G Method

Before-and-After Study with Yoked Comparison

Another approach for the before-and-after evaluation is known as the Yoked Comparison. The rationale behind the before-and-after study with Yoked Comparison is the same as the C-G method. The formulas and four-step procedures are also the same as the C-G approach. However, this approach requires a *one-to-one* correspondence between each treatment site and comparison site [*13*].This approach is recommended when the number of facilities is limited in the comparison group. To be specific, if the treated facility is an intersection, the comparison site should be a similar intersection with respect to the intersection type (e.g., three-legged, or four-legged), area type (e.g., urban, or rural, or commercial business district, etc.), traffic control device (e.g., signalized, stop-controlled, etc.), intersection geometry, traffic volume, etc. The other important requirement to the comparison site is that there should be no geometric change or traffic control improvement during the before and after periods.

As discussed in the preceding section, the underlying assumption of the comparison group approach is that the unknown causal factors should affect the comparison group in the same manner that they influence the treatment group. Therefore, the change in the number of crashes from the 'before' period to the 'after' period had the treatment site been left unimproved, would have been in the same proportion as the matching comparison site. Under this assumption, the crash frequency at each treated site in the 'before' period is multiplied by the ratio of after-to-before crashes at the comparison site to predict the expected number of crashes in the 'after' period at the treated site without the improvement [*14*]. The inference and calculation procedures can follow the contents in previous section. The advantages and disadvantages of this approach are summarized below.

- **Strengths**
  - It has better theoretical grounds than the naïve before-and-after study approach.
- **Weaknesses**
  - This approach makes use of only one comparison site. It is therefore conceivable to have different estimates when other comparison sites are used.
  - The findings based on this approach are variable with relatively wide confidence limits [*15*].
  - This approach is unable to deal with cases where the comparison site has no history of crash occurrences [*14*].

Before-and-After Study with Empirical Bayes Approach

The aforementioned naïve approach assumed no interventions from the 'before' to the 'after' periods. This is often an unrealistic assumption because it is almost impossible to keep all conditions constant as in the laboratory. The comparison method made progress by considering the possible interventions from causal factors, observed and unobserved, in analyzing the treatment group versus comparison group. However, there are many challenges in randomly selecting assignment of entities to form large treatment and comparison groups. The comparison group is neither the only nor the best device for prediction.

Furthermore, none of the previous approaches have considered the possible regression-to-the-mean effect. In reality, some sites are selected for treatment because of the unusual high crash history. This leads to the acute problem well known as the 'selection bias' or the 'regression-to-the-mean' bias. The other issue with conventional approaches is regarding the time trend. The current habit of selecting the study years in the 'before' period is within minimum of three years. This cutoff somehow neglects the time trend and therefore comes to an arbitrary observation. It is quite difficult to discern the existence of a time trend with just a few years of crash counts. One plausible conclusion would be that there is no time trend because it cannot be seen in a few years of crash records. To discard crashes which are more than three years old amounts to a loss of potential useful information.

This section introduces the widely used Empirical Bayes (EB) method that accounts for the issues mentioned above. Firstly, the regression-to-the-mean bias is briefly described. Secondly, the detailed before-and-after study with EB method is presented. Last but not least, this section discusses two critical elements in the EB method, the Safety Performance Functions (SPFs) and the over-dispersion parameter.

Regression-To-the-Mean Phenomenon

Regression-To-the-Mean (hereafter RTM) is the phenomenon that when a variable is extreme on its first measurement, it will tend to be closer to the average on its second measurement, and paradoxically, if it is extreme on its second measurement, it will tend to have been closer to the average on its first [*16, 17*, *18*]. In road safety analysis, crashes are random and rare events that naturally fluctuate over time at any given site. The fluctuation is measured by variance and the variations are usually due to the typical randomness of crash occurrences. Because of this random fluctuation, an extreme high number of crashes chosen in one period is very likely to experience a lower number in the next period and vice versa. The specific concern in road safety is that entities are commonly selected for improvements or treatments because of their high crash records in only one year or a short period of time. The counts will naturally regress back toward the mean in subsequent years. Therefore, what happened in the 'before' period may not a good indicator of what might happen in the after period.

The following examples (Figure 13 and 14) illustrate how over a span of several years crash data fluctuates between several high and low points around an expected average crash frequency. In Figure 13, the short-term average crash frequencies might be significantly higher or lower than the long-term average crash frequency. It should also be clear that if an entity is treated because of abnormal high crash counts in the before period, this same count number cannot possibly be a good estimate of the expected number of crash counts in the before period. This difference can be seen in Figure 13 as the difference between the green bars and the blue bar. In other words, if the entity has been selected because it had an unusually high number of crashes, the expected average crash counts in the long run would tend to be overestimated.

There comes another question of which one should we trust, the average number of crashes in the long run, or the temporary extreme value. The answer should be clear with no doubt. Even if the temporary high crashes were observed, the best guess about the magnitude of the crash frequency in the next period should still be the expected average number. This is the essence and meaning of the expected value. When the first observed value of the crash happens to be a down-fluctuation or an up-fluctuation, a return towards the mean should be expected for the next observation.

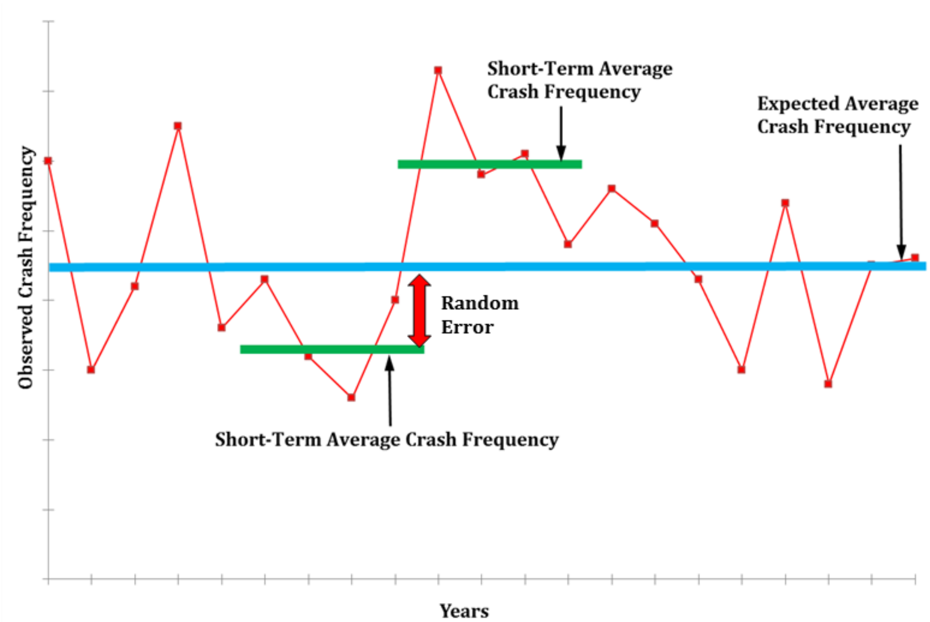


Figure 13 Variations in Crash Frequency

The natural fluctuation of crashes makes it difficult to identify whether the changes in observed crash frequency are due to changes in site conditions, treatment, or natural fluctuations. Figure 14 displays the history of crashes at an intersection which was identified as a high crash location in 2003 according to a relative high number of crashes in the before 3-year period.

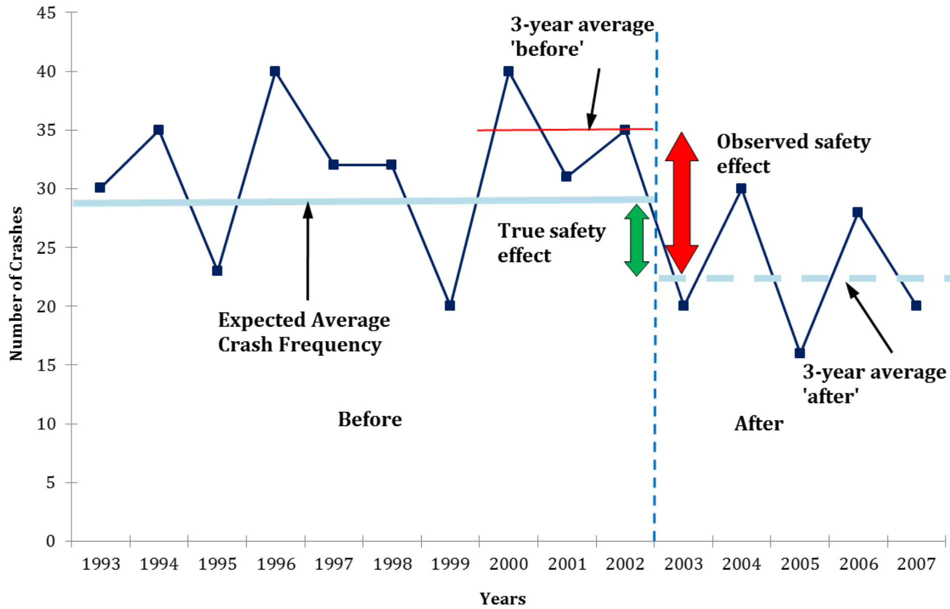


Figure 14 Example of RTM Bias in Before-and-After Study[5]

It can be seen that the crashes decreased from 2002 to 2003. Hence, even though a treatment was implemented in 2003, the differences between crash frequencies in 2002 and those in 2003 would not be totally attributed to the treatment to some unknown degree. Part of the reason should be the random error in the natural RTM phenomenon. In fact, if a decision is made to forgo safety improvements at this site, it would still be likely to show a reduction in crash frequencies in 2003 due to the natural variations. Clearly the RTM bias caused the perceived effectiveness of a treatment to be overestimated in this case. The RTM phenomenon must be considered when conducting a before-and-after study to avoid making wrong inferences.

Regardless of the problem identification method used to identify locations or road segments with potential for improvement, an appropriate time period needs to be defined for before-and-after analysis [*13*]. As discussed, crash experience can vary at a location from year to year, so it is important that more than one year of data is used for the analysis. Typically a minimum of three years of crash data is always used for safety related analysis. Multiple years of data are preferable to avoid the RTM bias. However, the use of multiple years of data can be misguided because the facility itself may have changed (e.g., adding a lane), the travel volume may have increased, or some other change has taken place that skews the analysis. In addition, it is sometimes difficult to obtain adequate multiple years

[5] Data Source: Highway Safety Improvement Program Manual. FHWA. 2010.

of data; therefore, it is often necessary to use a method for enhancing the estimate for sites with few years of data.

Overview of Empirical Bayes Approach

Let us recall the two critical tasks in a before-and-after study: (i) the prediction of what the expected number of crashes would have been in the 'after' period had the treatment not been implemented (π); and (ii) the estimation of the expected number of crashes in the 'after' period with the treatment in place. Both the naïve approach and comparison group method estimate π based on the observed number of crashes in the 'before' period (K). As explained in the RTM bias section, this conclusion is arbitrary to some extent.

Hauer in his book pointed out that the other important aspect of entity safety lies in the entity's traits, such as urban or rural, number of lanes, traffic control device, and geometry. Apparently a group of entities will share the same or similar traits, and they will have an average safety condition toward which the individual estimates will be seen to regress. Therefore, he defined the *reference group* of an entity as the group of entities that share the same set of traits as the entity in the safety of which one has an interest [*11*]. So far, the available clues toward the estimation of the entity safety in the 'after' period had the treatment not been installed include: (i) the observed crashes in the before period (K); and (ii) the average safety from the reference group. The second clue can be estimated using the Safety Performance Functions (SPFs). This perception brought up the idea of combining those available evidences to predict the safety without the treatment in the 'after' period. Therefore, the EB method is recommended to accomplish this goal.

The EB method is a statistical method that *combines* the observed crash frequency with the predicted crash frequency using the SPFs to calculate the expected crash frequency for a site of interest [*19*]. In the before-and-after study, this ultimate expected crash frequency that involves both historical crash information and the predicted crash frequency using SPFs is considered the expected safety that would have been in the 'after' period had the treatment not been implemented (i.e., π).

To be more specific, the EB method places an appropriate *weighted adjustment* to balance the observed crashes (K) and the predicted expected crashes using SPFs. It determines a relatively smoothed value for expected crashes and eliminates the randomness element of crashes. The EB estimate is illustrated in Figure 15, which shows how the observed crash frequency is combined with the predicted crash frequency using SPFs. It can be indicated that the EB estimate lies somewhere between the observed value and the predicted value from the SPF. Therefore, the EB method accounts for the RTM bias by pulling the crash count towards the mean as shown in Figure 15. The difference between the EB adjusted average crash frequency and the predicted average crash frequency from a SPF is referred to as the potential for safety improvement (PSI) [*19*]. The HSM also refers PSI as the excess expected average crash frequency.

The EB method is considered a suitable approach that fits the realities of observational before-and-after studies [*11*, *12*, *19*]. Many previous before-and-after studies have used EB method and proved that it provides a relatively more accurate estimation [*20*, *21*, *22*, *23*]. The recently released HSM recommends using EB method for all the steps in the road safety management process, especially for network screening and countermeasure selection and evaluation [*24*].

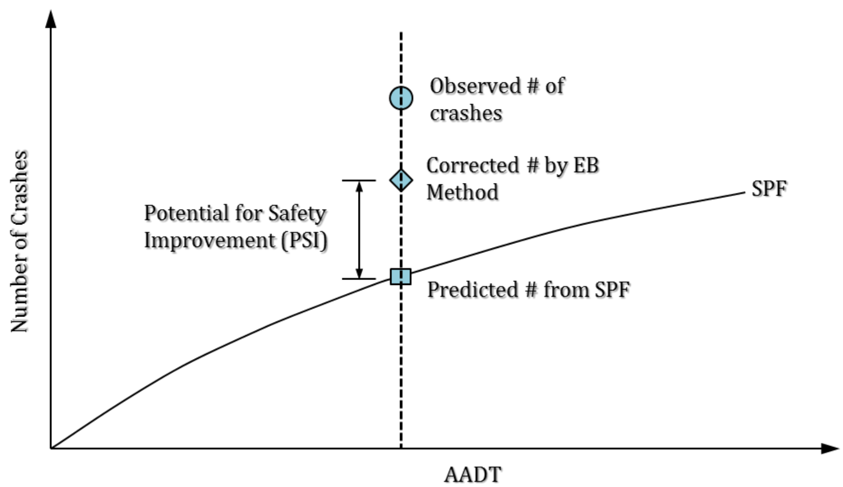


Figure 15 Illustration of the EB Estimate

Four-Step Procedure with Empirical Bayes Approach

The very first step in the before-and-after study procedure is to estimate the parameters λ and π. The ultimate goal is to obtain an unbiased estimate of π to account for the possible RTM bias and diverse impacts of other possible factors. The EB method was developed to fulfill this obligation. The essence of the before-and-after study with EB approach is to use statistical methodologies to join two separate pieces of information (observed number of crashes, and the predicted expected crashes using SPFs) to eventually estimate the safety of the entity with interest.

The EB method estimate π using the conditional expected value $E[\kappa|K]$[6]. The estimate of π consists of the expected value κ given observed crash counts K in the 'before' period. In other words, one can think of the observed crash frequency K as a sample, and the expected value κ as the prior knowledge about the expected safety performance of the reference group. Hence, the best estimation of π is the posterior given sample K based on Bayesian

[6] The detailed derivation process can be found in Hauer's book.

logic. The value of E[κ|K] locates somewhere between E[κ] (from the SPFs) and observed number of crashes K as demonstrated in Figure 15. Therefore,

$$\pi = E[\kappa|K] = w \cdot E[\kappa] + (1 - w) \cdot K$$

$$VAR[\kappa|K] = (1 - w) \cdot E[\kappa|K]$$

$$w = \frac{1}{1 + \frac{VAR[\kappa]}{E[\kappa]}} = \frac{1}{1 + \alpha \cdot E[\kappa]}$$

Where,

E[κ|K]: the expected safety for a site given observed crash frequency K;

VAR[κ|K]: the variance of expected safety for a site given observed crash frequency K;

E[κ]: the expected safety of reference group obtained from the SPF.

VAR[κ]: the variation around E[κ], the value is given by $VAR[\kappa] = E[\kappa]^2 \cdot \alpha$, in which α is the over-dispersion factor[7] of the Negative Binomial Regression Model used to generate SPF.

w: the weighted adjustment, assigned between 0 and 1.

α: the over-dispersion parameter obtained from SPF.

Two components, the expected safety of reference group E[κ] and the variation around E[κ] (i.e., V[κ]), play a pivotal role in obtaining the Bayesian estimator E[κ|K]. On one hand, the estimate of E[κ] determines part of the expected number of crashes on the entity in the 'after' period had the treatment not been applied. On the other hand, the weighted adjustment w consists of the average crash frequency of similar sites (i.e., E[κ]) and the variation around E[κ] (i.e., VAR[κ]). The weight w depends on the strength of the crash record (i.e., how many crashes are to be expected), and on the reliability of the SPFs (how different the safety may be of a specific site from the average which the SPF represents) [*25*]. If w is estimated to be close to 1, the estimate of E[κ|K] is close to the mean of its reference group E[κ]. On the contrary, if w is estimated to be close to 0, the estimate of E[κ|K] is mainly affected by the observed crash frequency K. Nonetheless, the estimate for E[κ|K] is always between K and E[κ].

[7] The detailed discussion of SPFs and over-dispersion parameter can be referred in Section 4.4.4.

Table 15 summarizes the modified procedures and formulas of the before-and-after study with the EB method. In conjunction with the EB estimate, the basic four-step process to estimate the change in safety (δ) and the index of effectiveness (θ) is modified.

Table 15 Modified Formulas of the Four-Step Before-and-After Study with EB Method

Steps	Goals	Formulas for Before-and-After Study with EB Method
Step 1	Estimate λ and predict π	$\hat{\lambda} = \mathrm{L}$ $\hat{\pi} = \mathrm{E}[\mathrm{k}\|\mathrm{K}] = \mathrm{w} \cdot \mathrm{E}[\kappa] + (1-\mathrm{w}) \cdot \mathrm{K}$
Step 2	Estimate $\mathrm{VAR}[\hat{\lambda}]$ and $\mathrm{VAR}[\hat{\pi}]$	$\mathrm{VAR}[\hat{\lambda}] = \mathrm{L}$ $\mathrm{VAR}[\hat{\pi}] = \mathrm{VAR}[\mathrm{k}\|\mathrm{K}] = (1-\mathrm{w}) \cdot \mathrm{E}[\mathrm{k}\|\mathrm{K}]$
Step 3	Estimate δ and θ	$\hat{\delta} = \hat{\pi} - \hat{\lambda}$ $\hat{\theta} \cong \dfrac{\left(\frac{\hat{\lambda}}{\hat{\pi}}\right)}{\left(1 + \frac{\mathrm{VAR}[\hat{\pi}]}{\hat{\pi}^2}\right)}$
Step 4	Estimate $\mathrm{VAR}[\hat{\delta}]$ and $\mathrm{VAR}[\hat{\theta}]$	$\mathrm{VAR}[\hat{\delta}] = \mathrm{VAR}[\hat{\lambda}] + \mathrm{VAR}[\hat{\pi}]$ $\mathrm{VAR}[\hat{\theta}] = \dfrac{\hat{\theta}^2 \left\{\frac{\mathrm{VAR}[\hat{\lambda}]}{\hat{\lambda}^2} + \frac{\mathrm{VAR}[\hat{\pi}]}{\hat{\pi}^2}\right\}}{\left\{1 + \frac{\mathrm{VAR}[\hat{\pi}]}{\hat{\pi}^2}\right\}^2}$

In short, the EB method increases the precision of estimation and corrects for the RTM bias. It is based on the recognition that crash counts are not the only clue to the safety of an entity. Another clue is in what is known about the safety of similar entities. A sensible estimate must be a mixture of the two clues. The strengths and weaknesses of the before-and-after study with the EB method are summarized below.

- **Strengths**
  - It has a solid theoretical framework that combines the information contained from observed crash data with the information contained in knowing the safety of similar entities.
  - It accounts for the RTM bias and provides a relatively more precise estimation.
  - It allows the estimation of the entire time series as required.
- **Weaknesses**
  - It completely relies on well-established SPFs.

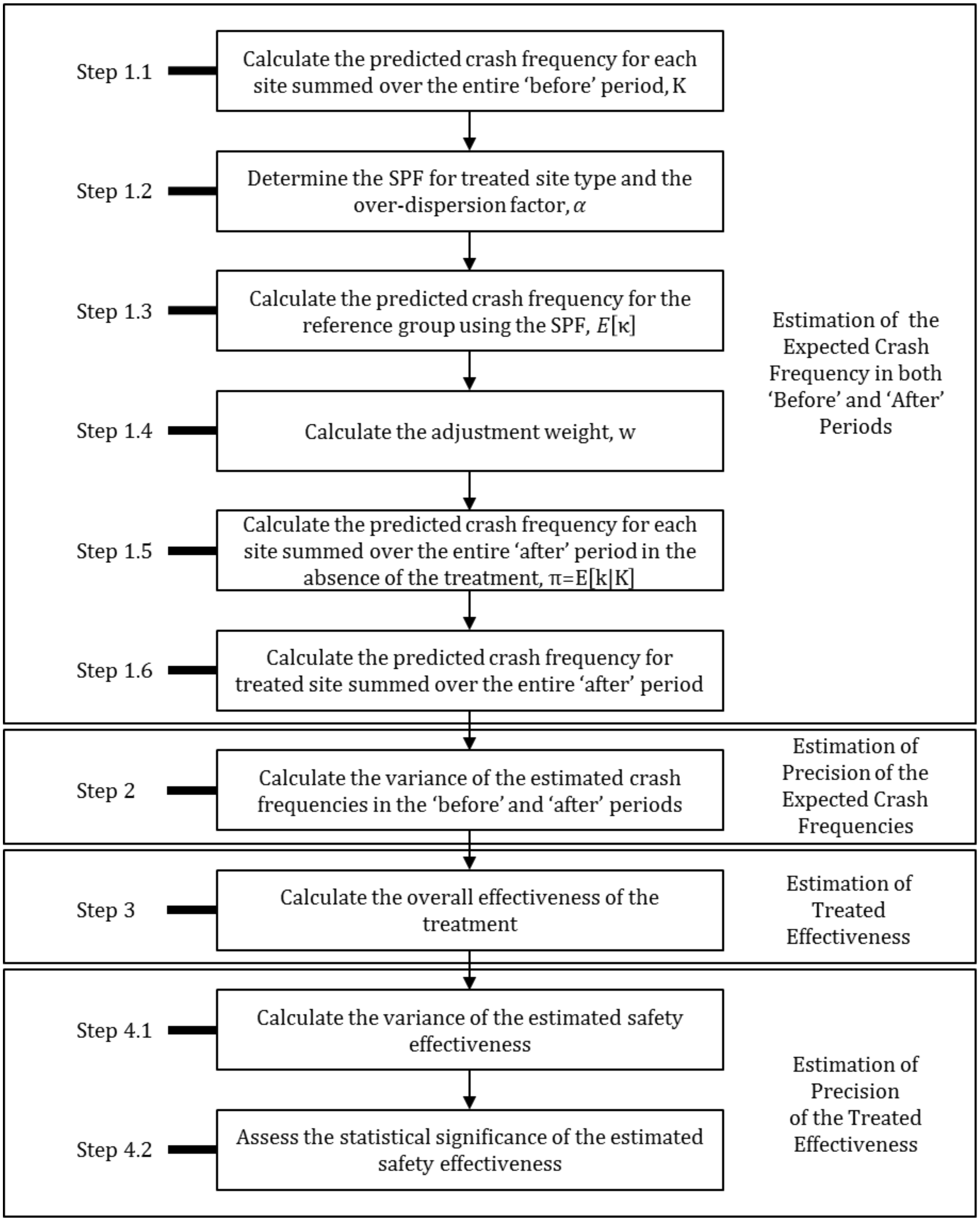


Figure 16 Four-Step Procedure for Before-and-After Studies with EB Method

Safety Performance Functions and Over-Dispersion Parameter

SPFs are statistical models used to estimate the average crash frequency for a specific site type (with specified base conditions), based on traffic volume and roadway segment length [*26*]. The EB method estimates the value of π using the conditional expected value E[κ|K]. The average crash frequency of the reference group (E[κ]) and the variation VAR[κ] around E[κ] are brought into the EB method through the SPFs. Two prerequisites play a critical role in generating the SPFs, the assumed probability distribution of crash frequency, and the over-dispersion parameter.

According to Hauer, crash occurrence is best modeled using a multivariate statistical model. Consequently, SPFs are generated using mathematical models (e.g., the Generalized Linear Models) to link the expected crash frequency on the roadway to measurable roadway traits such as AADT, length of roadway segment, roadway width, shoulder width, number of lanes, etc. As mathematical equations, SPFs also explain the relationships between crash frequency and explanatory variables such as traffic volumes on the facility [*27*]. To illustrate, the SPF for predicted average crash frequency for rural two-lane, two-way roadway segment is shown below that includes the information of AADT and segment length.

$$N_{spfrs} = AADT \times L \times 365 \times 10^{-6} \times e^{(-0.312)}$$

Where,

N_{spfrs}: predicted total crash frequency for roadway segment base conditions;

AADT: average annual daily traffic volume (vehicles per day); and

L: length of roadway segment (miles).

To estimate a roadway's SPF, it is necessary to assume an underlying probability distribution for the crash frequency. Traditionally, crash frequencies were often assumed to follow a Poisson distribution which assumes that the mean and variance observed for the crash frequency variable are equal. Previous studies have shown that the differences between the crash frequencies and model predictions based on the Poisson distribution is inconsistent, likely resulting from a violation of this equality assumption. In reality, crash data over a series of sites often exhibit a large variance and a small mean, and display over-dispersion with a variance-to-mean value greater than one [*28*, *29*]. Nowadays, researchers more commonly assume a Negative Binomial distribution, also known as the Poisson-Gamma distribution, to represent the distribution of crash frequencies and to generate SPFs.

The negative binomial distribution is considered to be able to handle over-dispersion better than other distributions, and has been widely used in many fields in addition to traffic safety [*30*]. The negative binomial distribution has two parameters, the mean and the dispersion parameter which is commonly considered to be fixed to measure over-dispersion. A typical negative binomial probability density function is outlined as follows,

$$Prob(Y_i = y_i) = \frac{\Gamma(y_i + \alpha)}{\Gamma(y_i + 1)\Gamma(\alpha)} \left(\frac{\mu_i}{\mu_i + \alpha}\right)^{y_i} \left(\frac{\phi}{\mu_i + \alpha}\right)^{\alpha}$$

Where,

$\mathrm{Y_i}$ = independently and identically (i. i. d.) NB distributed variable;

$\mathrm{y_i}$ = observed number of crashes at study site i (i = 1,2, … , n);

μ_i = expected number of crashes at study site i;

α = over − dispersion parameter of NB distribution;

Γ = Gamma function;

It can be indicated that the over-dispersion parameter α is an essential part of the model. Estimation of α is thus important given a sample of counts. The closer the over-dispersion parameter is to zero, the more statistically reliable the SPF. The widely used methods to estimate α include the Method of Moments Estimate (MME) [*31*], the Maximum Likelihood Estimate (MLE) method [*32*, *33*], and Maximum Quasi-Likelihood Estimate (MQLE) [*34*].

Section Summary

The essence of a before-and-after study lies in the estimation of the treatment effect on safety from the 'before' to the 'after' period. Two critical tasks need to be accomplished, including the prediction of what the expected number of crashes would have been in the 'after' period had the treatment not been implemented (π), and the estimation of the expected number of crashes in the 'after' period with the treatment in place. As noted in previous section, the expected safety of treated entities is often estimated using the observed number of crashes in the 'after' period. However, the prediction of π is intricate. This section presents four commonly used before-and-after study approaches, including the Naïve approach, Comparison Group approach, Yoked Comparison approach, and the Empirical Bayes approach. Overall they obey the procedures documented in previous section. Each of them however is equipped with its own concept and assumptions to predict π. This section describes the basic concept, data needs, required formulas, revised four-step procedures, and the shortcomings and limitations of each approach to provide a reference for this type of study.

CHAPTER 3 SAFETY EVALUATION OF I-580 AND U.S. 395 ALT

INTRODUCTION

The UNR CATER research team conducted a study on the new I-580 and U.S. Route 395 Alternate between south Reno and Carson City. The new section of I-580 between Washoe City and Mount Rose Highway (SR 341) opened on August 24, 2012. Afterwards, the old routing of U.S. 395 changed the name to U.S. Route 395 Alternate (i.e. U.S. 395A). A major benefit for the new I-580 freeway is anticipated to be traffic safety improvements. According to NDOT records, approximately, an annual average of 40,000 vehicles were using the old U.S. 395. From 1994 to 2012, there were a total of 1,610 crashes consisting of 33 fatal crashes and 521 injury crashes. It was estimated that approximately 75% of vehicles will take the new freeway, significantly reducing the number of crashes on U.S. 395A, making it much safer for both local residents and regional travelers.

The main goal of this study is to predict the safety on I-580/U.S. 395A pair compared to the scenario when only old U.S. 395 existed. The case study will demonstrate the applications of major traffic safety management procedures documented in the American Association of State & Highway Transportation Officials (AASHTO) Highway Safety Manual (HSM), such as the safety performance functions (SPFs) and crash modification factors (CMFs). All default SPFs' formats and parameters developed using national data were used in this study since Nevada specific SPFs are currently under development. It should be noted that this is a comparative study using national SPFs for both freeway facility and rural multi-lane highway. The results may not reflect actual number of crashes but will reliably indicate the safety benefits in relative terms.

Utilizing the predictive methods in the HSM and the Interactive Highway Safety Design Model (IHSDM) software, the predicted and expected crashes were estimated for a particular study year. The analysis results will assist NDOT in estimating the future safety along these roadways. Lessons learned through this case study can better guide the development of future NDOT traffic safety programs.

SAFETY EVALUATION OF I-580 FREEWAY SEGMENT

Freeway Segment Description

I-580 is the U.S. 395 freeway in the Reno-Carson City metropolitan area between Fairview Drive in Carson City and Interstate 80 (I-80) in Reno as shown in Figure 17. Unlike the old U.S. 395 which ran along the valley floor of Pleasant Valley, I-580 is routed in the mountains overlooking the valley. While traversing Pleasant Valley, the highway crosses the Galena Creek Bridge, the largest cathedral arch bridge in the world. Upon descent from these mountains the highway cuts through the center of one of the largest geothermal power plants in the United States, Ormat Industries' Nevada Power station, just before entering Reno.

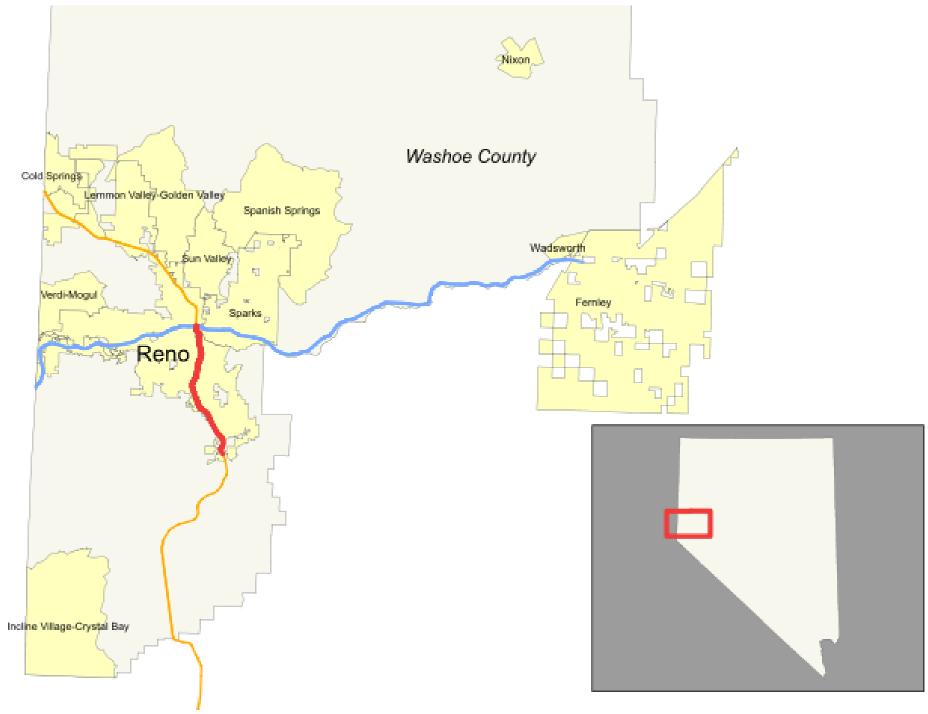


Figure 17 I-580 in Nevada

The new section of I-580 between Washoe City and Mt. Rose Junction opened on August 24, 2012. The total project length is approximately 9.5 miles (15.29 km). It is a six-lane freeway with nine bridges. It was predicted that 75% of the vehicles from the old U.S. 395 will use the new freeway. Ultimately, this new freeway cuts the travel time between Reno and Carson City by nearly five minutes.

In this analysis, the 9.5 miles stretch was divided into 33 segments to exclude interchanges and weaving sections. Detailed geometric design data and historical crash records are introduced in the data summary that follows.

I-580 Segment Data Summary

The section to be analyzed has a design speed of 75 mph. The horizontal alignment is generally curvilinear, including several compound curves. The study year is 2013. The overall existing conditions are provided in the following list which includes all the data required by the IHSDM Software.

- Analysis segment length: 9.5 miles (15.29 km)

- Project limits: Station "LS"289+70.500 to Station "P"145+81.114[8]
- Area type: Rural
- Functional classification: Freeway
- Design speed: 75mph
- Average Daily Traffic (ADT) (veh/day)[*35*]
    - In 2012: 26,200 vpd
    - In 2026: 65, 600 vpd (projected)
- Alignments
    - Horizontal: complete tangent and curve data[9]
    - Vertical: tangent data[10]
- Cross section[11]
    - Number of lanes: 6-lane (both southbound and northbound directions)
    - Lane width: 12 feet
    - Auxiliary lanes (speed change lanes): not considered
    - Shoulder width
        - Outside shoulder width: 12 feet
        - Inside shoulder width: varies from 4 to 20 feet
    - Median width: varies up to 70 feet
    - Ramps: no ramps along the analysis segment
- Roadside
    - Clear zone width: assumed to be 30 feet
- Other data
    - Median Barrier
        - Continuous median barrier from Station "SN"35+11.666 to Station "P"145+81.114
        - Median barrier offset from inside traveled way: 12 feet
    - Outside Barrier

---

[8] Stations are from the I-580 as built plans in metric units. Station "LS"289+70.500 starts at the intersecting point of SR 429 and I-580. Station "P"145+81.114 is located at the Mt. Rose Junction (SR 431).
[9] IHSDM requires complete horizontal alignment data. The detailed tangent and curve information is displayed in Table 4 and Figure 3.
[10] IHSDM and the Freeway model in the Highway Safety Manual (HSM) considers vertical alignment as level tangent only.
[11] Cross slope is used for system purposes only.

    - Continuous outside barrier from Station "SN"27+50.00 to Station "SN"29+57.00, Station "SN"31+76.000 to Station "P"57+80.000, Station "P"60+96.801 to Station "P1A" 172+80.000, and from Station "P1A" 178+52.60 to Station "P"145+81.114.
    - Outside barrier offset from outside traveled way: 12 feet
  - Shoulder rumble strips
    - Both sides of the shoulders from Station "LS"289+70.500 to Station "P"145+81.114

The analysis segment does not include any interchanges or on/off ramps. In addition, weaving sections are ignored in this analysis. The available data listed above is required by the IHSDM Crash Prediction Module. There have been 11 observed injury crashes and 49 property-damage-only crashes from August 2012 to May 2013. The detailed recorded crashes are summarized in Table 16 and 17. Since the segment has less than one year of crash data, which is not enough to use for the Empirical Bayes Method, the default base SPFs and CMFs for freeway models are applied to predict the safety.

Table 16 Crash Severity and Distribution for I-580 since August 2012

Severity Type	Number of Crashes
Fatal	0
Injury A	0
Injury B	1
Injury C	10
PDO	49
Total	60

Table 17 Crash Collision Type for I-580 since August 2012

Collision Type	Number of Crashes
Non-Collision	41
Angle	8
Rear End	10
Sideswipe	1
Total	60

Table 18 I-580 Freeway Segment and Station Information

No.	Type	Starting Station (m)	Starting Station (ft)	Starting Station in Design File (m)	Ending Station (m)	Ending Station (ft)	Ending Station in Design File (m)	Length (m)	Curve Radius (m)	Direction of Curve	Curve Side of Road
1	Tangent	28+970.500	46+282.720	"LS"289+70.500	29+890.500	49+301.093	"L"298+90.500	920.000	--	--	--
2	Tangent	29+890.500	49+301.093	"L"298+90.500	30+450.627	51+138.780	"LS"304+50.627 "SS"7+50.000 "SN"27+50.000	560.127	--	--	--
3	Tangent	30+450.627	51+138.780	"LS"304+50.627 "SS"7+50.000 "SN"27+50.000	31+212.293	53+637.684	"SN"35+11.666	761.666	--	--	--
4	Curve 1	31+212.293	53+637.684	"SN"35+11.666	32+177.552	56+804.544	"SN"44+76.925	965.259	1540.00	Right	Both Roadbeds
5	Tangent	32+177.552	56+804.544	"SN"44+76.925	32+267.265	57+098.878	"SN"45+66.638 "P"25+86.878	89.713	--	--	--
6	Tangent	32+267.265	57+098.878	"SN"45+66.638 "P"25+86.878	35+024.782	66+145.851	"P"53+44.395	2757.517	--	--	--
7	Curve 2	35+024.782	66+145.851	"P"53+44.395	35+777.188	68+614.374	"P"60+96.801	752.406	1220.00	Right	Both Roadbeds
8	Tangent	35+777.188	68+614.374	"P"60+96.801	36+516.909	71+041.281	"P1A"168+36.522	739.721	--	--	--
9	Tangent	36+516.909	71+041.281	"P1A"168+36.522	36+545.952	71+136.566	"P1"68+65.656	29.043	--	--	--
10	Curve3	36+545.952	71+136.566	"P1"68+65.656	37+013.892	72+671.802	"P1"73+33.505	467.940	620.00	Left	Both Roadbeds
11	Tangent	37+013.892	72+671.802	"P1"73+33.505	37+055.13	72+807.097	"P1A"173+74.742	41.238	--	--	--
12	Tangent	37+055.13[12]	72+807.097	"P1A"173+74.742	37+532.988[13]	74+374.873	"P1A"178+52.600	477.858	--	--	--
13	Tangent	37+532.988	74+374.873	"P1A"178+52.600	37+710.765	74+958.131	"P1A"180+30.377	177.777	--	--	--
14	Tangent	37+710.765	74+958.131	"P1A"180+30.377	37+731.352	75+025.674	"P1"80+71.806	20.587	--	--	--
15	Curve 4	37+731.352	75+025.674	"P1"80+71.806	37+991.315	75+878.571	"P1A"183+10.927 "P1"83+28.229 "P"83+01.665	259.963	1142.50	Right	Both Roadbeds
16	Curve 5	37+991.315	75+878.571	"P1A"183+10.927 "P1"83+28.229 "P"83+01.665	38+573.025	77+787.068	"P"88+83.381	581.710	1150.00	Left	Both Roadbeds

[12] Starting point of Galena Creek Bridge.
[13] Ending point of Galena Creek Bridge.

Table 18 I-580 Freeway Segment and Station Information (continued)

No.	Type	Starting Station (m)	Starting Station (ft)	Starting Station in Design File (m)	Ending Station (m)	Ending Station (ft)	Ending Station in Design File (m)	Length (m)	Curve Radius (m)	Direction of Curve	Curve Side of Road
17	Tangent	38+573.025	77+787.068	"P"88+83.381	38+880.398	78+795.510	"P"91+90.754	307.373	--	--	--
18	Tangent	38+880.398	78+795.510	"P"91+90.754	39+098.689	79+511.688	"P"94+09.045	218.291	--	--	--
19	Tangent	39+098.689	79+511.688	"P"94+09.045	39+327.488	80+262.341	"P"96+37.844	228.799	--	--	--
20	Curve 6	39+327.488	80+262.341	"P"96+37.844	39+545.496	80+977.590	"P"98+55.853	218.008	1500.00	Right	Both Roadbeds
21	Tangent	39+545.496	80+977.590	"P"98+55.853	39+989.643	82+434.765	"P"103+00.000 "P2"103+00.000 "P3"103+00.000	444.147	--	--	--
22	Tangent	39+989.643	82+434.765	"P"103+00.000 "P2"103+00.000 "P3"103+00.000	40+062.261	82+673.013	"P2"103+72.618	72.618	--	--	--
23	Tangent	40+062.261	82+673.013	"P2"103+72.618	40+115.879	82+848.925	"P3"104+26.236	53.618	--	--	--
24	Curve 7	40+115.879	82+848.925	"P3"104+26.236	40+326.035	83+538.413	"P2"106+36.392	210.156	850.00	Right	Both Roadbeds
25	Tangent	40+326.035	83+538.413	"P2"106+36.392	40+401.374	83+785.589	"P3"107+11.731	75.339	--	--	--
26	Tangent	40+401.374	83+785.589	"P3"107+11.731	40+652.382	84+609.106	"P2"109+62.739	251.008	--	--	--
27	Curve 8	40+652.382	84+609.106	"P2"109+62.739	41+292.922	86+710.615	"P3"116+03.279	640.54	850.00	Left	Both Roadbeds
28	Tangent	41+292.922	86+710.615	"P3"116+03.279	41+376.787	86+985.763	"P2"116+87.144 "P3"116+78.808 "P"117+00.000	83.865	--	--	--
29	Tangent	41+376.787	86+985.763	"P2"116+87.144 "P3"116+78.808 "P"117+00.000	41+775.085	88+292.515	"P"120+98.298	398.298	--	--	--
30	Tangent	41+775.085	88+292.515	"P"120+98.298	42+200.217	89+687.305	"P"125+23.430	425.132	--	--	--
31	Tangent	42+200.217	89+687.305	"P"125+23.430	42+841.186	91+790.221	"P"131+64.399	640.969	--	--	--
32	Tangent	42+841.186	91+790.221	"P"131+64.399	43+531.166	94+053.935	"P"138+54.379	689.980	--	--	--
33	Curve 9	43+531.166	94+053.935	"P"138+54.379	44+257.901	96+438.237	"P"145+81.114	726.735	1066.802	Right	Both Roadbeds

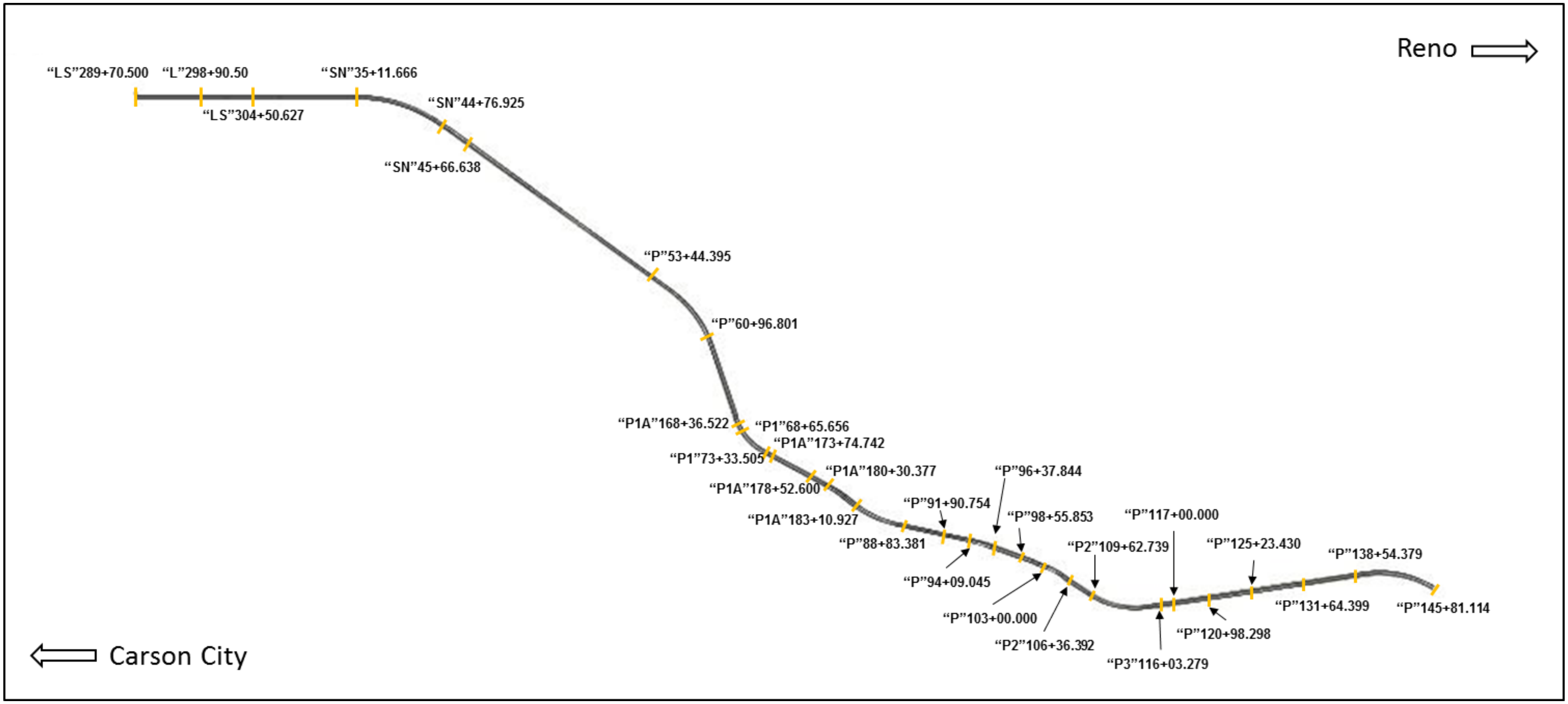


Figure 18 I-580 Freeway Segment and Stations

Predictive Method for Freeway Segment

This analysis is intended to determine the predicted safety of the new section of the I-580 for a particular year using the Predictive Methods in the Highway Safety Manual (HSM). The implementation of the predictive methods requires the development of three main components: (1) base Safety Performance Function (SPF); (2) Crash Modification Factors (CMFs); and (3) local Calibration Factor for a particular category of highway, in this study, freeway. Firstly, the base SPF uses roadway geometry and characteristics to estimate a base safety condition of the freeway segment. Secondly, appropriate CMFs are applied to create a site specific function that reflects the actual freeway condition more accurately. Thirdly, a calibration factor can be used to account for the regional variations.

The base SPFs and CMFs for the freeway crash prediction are presented below. The freeway predictive methods and models are documented in the future HSM Chapter 18 and are derived from the NCHRP Project 17-45: Enhanced Safety Prediction Methodology and Analysis Tool for Freeways and Interchanges. All default SPFs' formats and parameters are used. However, at the time of this study, NDOT had not developed a local calibration factor for freeway and therefore no calibration factor was applied for this analysis.

Base Safety Performance Functions (SPFs)

a. <u>Freeway Segment Multi-Vehicle SPF</u>

$$N_{spf,fs,n,mv,z} = L^* * \exp\ (\ a + b * \ln(c * AADT_{fs}))$$

$$L^* = L_{fs} - (0.5 * Sum_{i=1to2}\ L_{en,seg,i}) - (0.5 * Sum_{i=1to2}\ L_{ex,seg,i})$$

Where:

- $N_{spf,fs,n,mv,z}$ is the predicted average multiple-vehicle crash frequency of a freeway segment with base conditions, n lanes, and severity z (z is fi: fatal and injury, pdo: property damage only) (crashes/yr);
- L^* is the effective length of the freeway segment (mi);
- L_{fs} is the length of the freeway segment (mi);
- $L_{en,seg,i}$ is the length of the ramp entrance i adjacent to the subject freeway segment (mi);
- $L_{ex,seg,i}$ is the length of the ramp exit i adjacent to the subject freeway segment (mi);
- $AADT_{fs}$ is the AADT volume of the freeway segment (veh/day); and
- *a, b, c* are regression coefficients. The coefficients values are shown in Table 19.

Table 19 Coefficients of the Base SPF for Freeway Segments with Multi-Vehicle Crashes

Area Type	Model Class	Segment Type	Intercept (a)	Intercept (b)	AADT (c)	d	Over-dispersion (k)
Rural	Property Damage Only	Six-Lane Freeway	-7.141	1.936	0.001	1.000	18.800
Rural	Fatal and Injury	Six-Lane Freeway	-6.092	1.492	0.001	1.000	17.600

b. *Freeway Segment Single-Vehicle SPF*

$$N_{spf,fs,n,sv,z} = L^* * exp\ (\ a + b * ln(c * AADT_{fs}))$$

$$L^* = L_{fs} - (0.5 * Sum_{i=1to2}\ L_{en,seg,i}) - (0.5 * Sum_{i=1to2}\ L_{ex,seg,i})$$

Where:

- $N_{spf,fs,n,sv,z}$ is the predicted average single-vehicle crash frequency of a freeway segment with base conditions, n lanes, and severity z (z = fi: fatal and injury, pdo: property damage only) (crashes/yr); L^* is the effective length of the freeway segment (mi);
- L_{fs} is the length of the freeway segment (mi);
- $L_{en,seg,i}$ is the length of the ramp entrance i adjacent to the subject freeway segment (mi);
- $L_{ex,seg,i}$ is the length of the ramp exit i adjacent to the subject freeway segment (mi);
- $AADT_{fs}$ is the AADT volume of the freeway segment (veh/day); and
- *a, b, c* are regression coefficients.

Table 20 demonstrates the coefficients used for this type of crash.

Table 20 Coefficients of the Base SPF for Freeway Segments with Single-Vehicle Crashes

Area Type	Model Class	Segment Type	Intercept (a)	Intercept (b)	AADT (c)	d	Over-dispersion (k)
Rural	Fatal and Injury	Six-Lane Freeway	-2.055	0.646	0.001	1.000	30.100
Rural	Property Damage Only	Six-Lane Freeway	-2.274	0.876	0.001	1.000	20.700

Crash Modification Factors (CMFs)

Appropriate CMFs are used to modify the crash frequency for a specific change in geometry and adapt the base SPF to non-base conditions. One CMF should be used for each design element (e.g., lane width). CMF equals to 1.00 when the specific design element is the same as the base condition. For this study, Table 21 below summarizes the base conditions for the SPF along with the existing conditions for the application of CMFs.

Table 21 CMFs for I-580 Freeway Segments

CMFs[14]	Freeway Design Element	HSM Base Condition	Existing Condition
$CMF_{1,fs,ac,y,z}$	Horizontal Curve	Tangent Only	Curves and tangent
$CMF_{2,fs,ac,y,fi}$	Lane Width	12-feet	12 feet
$CMF_{3,fs,ac,y,z}$	Inside Shoulder Width	6-feet	4-20 feet
$CMF_{4,fs,ac,y,z}$	Median Width	60-feet	0-70 feet
$CMF_{5,fs,ac,y,z}$	Median Barrier	None	Present
$CMF_{6,fs,ac,y,z}$	High Volume	None	None
$CMF_{7,fs,ac,mv,z}$	Lane Change	None	None
$CMF_{8,fs,ac,sv,z}$	Outside Shoulder Width	10-feet	12 feet
$CMF_{9,fs,ac,sv,fi}$	Shoulder Rumble Strip	None	Present
$CMF_{10,fs,ac,sv,fi}$	Outside Clearance	30-feet	30 feet
$CMF_{11,fs,ac,sv,z}$	Outside Barrier	None	Present

Crash Prediction Results of I-580

By using the basic geometric data, SPFs and CMFs presented, the predicted number of crashes for I-580 in 2014 was obtained and is demonstrated in Table 22, Table 23 and Figure 19. The default HSM values of crash type distributions were used for this analysis since there have not been any local values prepared for Nevada at this time. A total of 52.24 crashes are predicted to occur in 2014 along the 9.5 mile stretch.

[14] $CMF_{\#,fs,ac,y,z}$: *fs*=freeway segment; ac=any cross section; y=crash type; and z=severity.

Table 22 Predicted Crash Rates and Frequencies Summary of I-580 in 2014

General Summary	
Year of Analysis	2014
Evaluated Length	9.5 miles (15.29 km)
Average Future Road AADT (vpd)	31,828
Predicted Crashes	
Total Crashes	52.24
Fatal and Injury Crashes	15.59
Property-Damage-Only Crashes	36.65
Percent of Total Predicted Crashes	
Percent Fatal and Injury Crashes (%)	30%
Percent Property-Damage-Only Crashes (%)	70%
Predicted Crash Rate	
Crash Rate (crashes/mi/yr)	5.4997
Fatal and Injury Crash Rate (crashes/mi/yr)	1.6410
Property-Damage-Only Crash Rate (crashes/mi/yr)	3.8587
Predicted Travel Crash Rate	
Travel Crash Rate (crashes/million veh-mi)	0.47
Travel Fatal and Injury Crash Rate (crashes/million veh-mi)	0.14
Travel Property-Damage-Only Crash Rate (crashes/million veh-mi)	0.33

Table 23 Predicted Crash Type Distributions on I-580 in 2014

Crash Type	Fatal and Injury	% Fatal and Injury	PDO	% PDO	Total Crashes	% Total Crashes
Collision with Animal	0.11	0.2	1.89	3.9	2.00	4.1
Collision with Fixed Object	6.34	13.0	18.14	37.2	24.48	50.2
Collision with Other Object	0.35	0.7	3.63	7.4	3.97	8.2
Other Single-vehicle Collision	4.11	8.4	4.70	9.6	8.82	18.1
Collision with Parked Vehicle	0.27	0.6	0.67	1.4	0.94	1.9
Single Vehicle Crashes	11.18	22.9	29.02	59.5	40.20	82.5
Right-Angle Collision	0.25	0.5	0.23	0.5	0.48	1.0
Head-on Collision	0.08	0.2	0.03	0.1	0.11	0.2
Other Multi-vehicle Collision	0.26	0.5	0.60	1.2	0.86	1.8
Rear-end Collision	2.78	5.7	0.39	0.8	3.16	6.5
Sideswipe, Same Direction Collision	1.04	2.1	2.90	5.9	3.94	8.1
Multiple Vehicle Crashes	4.41	9.0	4.14	8.5	8.55	17.5
Total Crashes	**15.59**	**32.0**	**33.16**	**68.0**	**48.75**	**100.0**

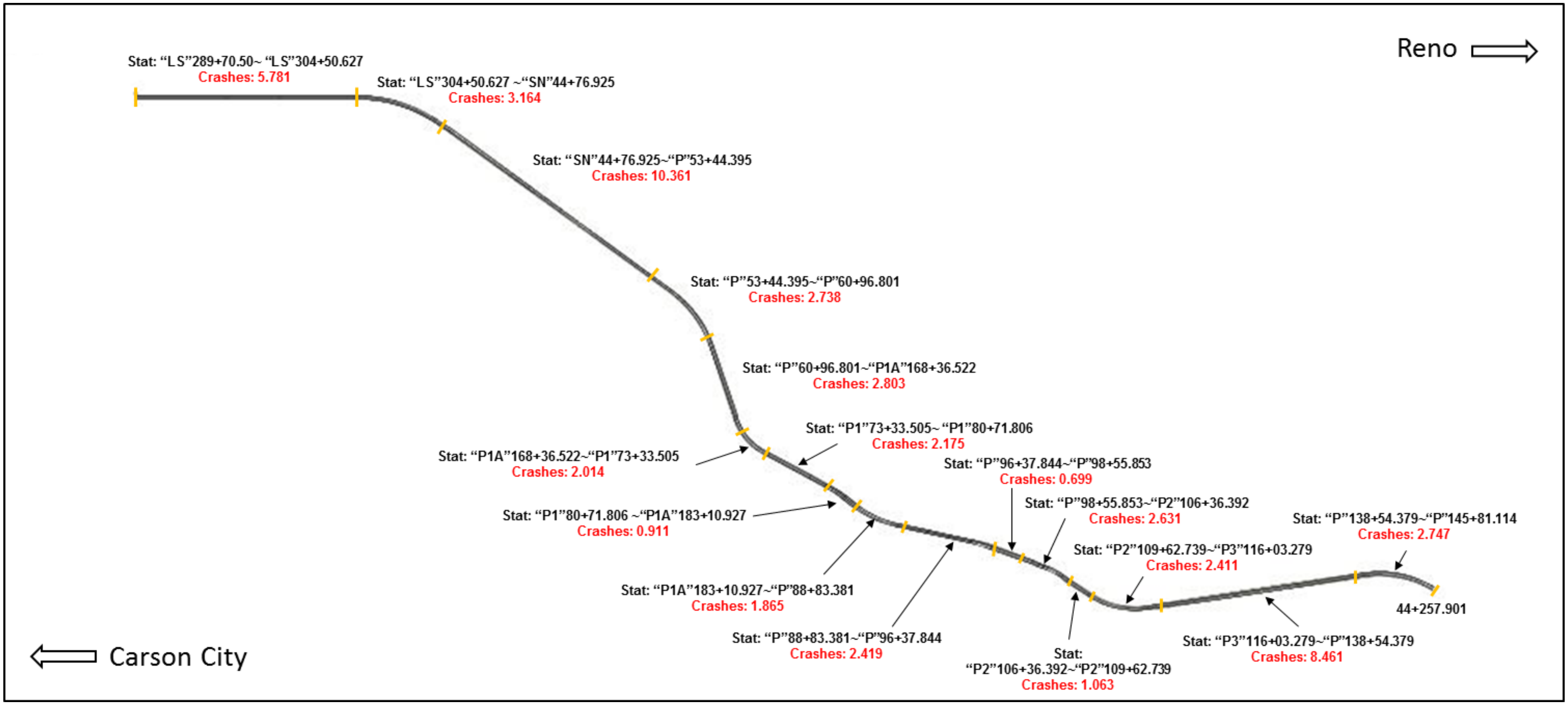


Figure 19 Predicted Crashes on I-580 by Segments

SAFETY EVALUATION OF U.S. 395 A

Highway Segment Description

The analyzed section of U.S. 395 is a four lane rural highway. This section, also known as Old U.S. 395, was renamed on May 18, 2012 to U.S. 395A. The analyzed segment is located between Mt. Rose Highway and Washoe City. Some portions of the highway segment are divided with a median barrier and some are undivided with a Two-way Left Turn Lane (TWLTL) in the middle. The study section is approximately 7.84 miles in length. After the opening of the new freeway I-580, the traffic volumes on the parallel section of U.S. 395A dropped about 80%-90%.

U.S. 395A Segment Data Summary

The U.S. 395A has a design speed of 70 mph. Similar to the freeway I-580, the horizontal alignment is curvilinear. The overall existing conditions present during 2012-2013 are provided in the following list and are also coded into the IHSDM Software. Table 24 summarizes the detailed segment information.

- Analysis segment length: 7.84 miles
- Project limits: Station "G1"53+71.20 to Station "OC"861+87.34[15]
- Area Type: Rural
- Functional classification: Arterial
- Design speed: 50 mph
- Average Daily Traffic (ADT) (veh/day)[16]
    - In 2012: 6,000 vpd
    - In 2013: 6,200 vpd
- Alignments
    - Horizontal: segment length[17]
    - Vertical: level tangent only[18]
- Cross Section

[15] Stations are the same as the design plans in English units. Station "G1"5+371.200 starts at the intersecting point of SR 429 and I-580. Station "OC"861+87.34 is located at junction with the Mount Rose Highway (SR 431) and Geiger Grade (SR 341).
[16] Analysis is based only on volumes since I-580 was opened.
[17] IHSDM requires horizontal segment lengths for the analysis whereas the detailed curvature data is not required. The detailed station information is displayed and discussed in Table 10.
[18] IHSDM and the rural multi-lane model in the HSM consider vertical alignment as level tangent only.

- Number of lanes:
    - 4-lane (both southbound and northbound directions) from Station "C"46+38.97 to Station "X"129+08.39, and Station "O"806+76.95 to Station "OC"861+87.34[19]
    - 4-lane on both directions and a TWLTL in the middle from Station "G1"53+71.20 to Station "G1"95+95.20, and from Station "X"129+08.39 to Station "O"806+76.95
- Lane width
    - Through lane width:11-12 feet
    - Left turning lane width: 10 feet
    - TWLTL width: 11 feet[20]
- Shoulder width
    - Outside shoulder width: 7 feet
    - Inside shoulder width: 5-6 feet from Station "G1"53+71.20 to Station "G1"95+95.20, and from Station "X"129+08.39 to Station "O"806+76.95
- Roadside Fore slope
    - Varies from 1:1.5 to 1:6
    - Width of Slope: 7 feet

- Intersections: intersections with historical crash data were involved along the study segment and the following information was gathered for each intersection in Table 25.
    - Number of legs
    - AADT's for major and minor roads
    - Type of traffic control
    - Approach leg type (i.e., major/minor)
    - Skew of the intersections
    - Number of left and right turn lanes from the major roadway
- Crash history data
    - Segment-related crashes
    - Intersection-related crashes

---

[19] There are median barrier rails from Station "C"46+38.97 to Station "X"129+08.39, and Station "O"806+76.95 to Station "OC"861+87.34. However, the current version of the HSM and IHSDM module do not have methods to analyze median barriers.

[20] The TWLTL data is not input into the IHSDM model for analysis because the current HSM and IHSDM module do not contain models to evaluate TWLTLs.

Table 24 U.S. 395A Highway Segment and Station Information

No.	Type[21]	Starting Station (ft)	Starting Station in Design File (ft)	Ending Station (ft)	Ending Station in Design File (ft)	Length (ft)	Left Lane Width (ft)	Right Lane Width (ft)	Left Shoulder width (ft)	Right Shoulder Width (ft)	TWLTL	Median Barrier	Left Side Slope	Right Side Slope
1	4U	5+371.200	“G1”53+71.20	6+088.680	“G1”60+88.68	717.48	12.00	12.00	7.00	7.00	YES	NO	1:6	1:6
2	4U	6+088.680	“G1”60+88.68	6+246.620	“G1”62+46.62	157.94	12.00	12.00	7.00	7.00	YES	NO	1:6	1:6
3	4U	6+246.620	“G1”62+46.62	6+759.960	“G1”67+59.96	513.34	12.00	12.00	7.00	7.00	YES	NO	1:6	1:6
4	4U	6+759.960	“G1”67+59.96	6+964.710	“G1”69+64.71	204.75	12.00	12.00	7.00	7.00	YES	NO	1:6	1:6
5	4U	6+964.710	“G1”69+64.71	8+651.700	“G1”86+51.70	1,686.99	12.00	12.00	7.00	7.00	YES	NO	1:6	1:6
6	4U	8+651.700	“G1”86+51.70	9+364.230	“G1”93+64.23	712.53	12.00	12.00	7.00	7.00	YES	NO	1:1.5	1:1.5
7	4U	9+364.230	“G1”93+64.23	9+595.200	“G1”95+95.20	230.97	12.00	12.00	7.00	7.00	YES	NO	1:6	1:6
8	4D	9+595.200	“G1”95+95.20	12+941.780	“C”46+38.97	3,346.58	11.00	11.00	7.00	7.00	NO	YES	1:6	1:6
9	4D	12+941.780	“C”46+38.97	13+721.920	“C”54+19.11	780.14	11.00	11.00	7.00	7.00	NO	YES	1:6	1:6
10	4D	13+721.920	“C”54+19.11	14+553.530	“C”62+50.72	831.61	11.00	11.00	7.00	7.00	NO	YES	1:6	1:1.5
11	4D	14+553.530	“C”62+50.72	15+124.490	“C”68+21.68	570.96	11.00	11.00	7.00	7.00	NO	YES	1:1.5	1:1.5
12	4D	15+124.490	“C”68+21.68	19+161.330	“X”108+58.52	4,036.84	11.00	11.00	7.00	7.00	NO	YES	1:1.5	1:1.5
13	4D	19+161.330	“X”108+58.52	21+107.660	“X”128+04.85	1,946.33	11.00	11.00	7.00	7.00	NO	YES	1:1.5	1:1.5
14	4D	21+107.660	“X”128+04.85	21+211.200	“X”129+08.39	103.54	11.00	11.00	7.00	7.00	NO	YES	1:1.5	1:1.5
15	4U	21+211.200	“X”129+08.39	21+733.920	“X”134+31.11	522.72	11.00	11.00	7.00	7.00	YES	NO	1:1.5	1:1.5
16	4U	21+733.920	“X”134+31.11	22+230.500	“X”139+27.69P.C. “O4”616+50.37P.C. “O4”616+40.34P.O.T	496.58	11.00	11.00	7.00	7.00	YES	NO	1:1.5	1:1.5
17	4U	22+230.500	“X”139+27.69P.C. “O4”616+50.37P.C. “O4”616+40.34P.O.T.	23+273.630	“O4”626+83.47	1,043.13	11.00	11.00	7.00	7.00	YES	NO	1:1.5	1:1.5
18	4U	23+273.630	“O4”626+83.47	23+728.200	“OA”631+38.04	454.57	11.00	11.00	7.00	7.00	YES	NO	1:1.5	1:1.5
19	4U	23+728.200	“OA”631+38.04	23+966.030	“OA”633+75.87	237.83	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
20	4U	23+966.030	“OA”633+75.87	25+473.760	“OA”648+83.60P.O.T	1,507.73	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
21	4U	25+473.760	“OA”648+83.60P.O.T	26+690.090	“O”660+99.93	1,216.33	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6

[21] 4U: 4-lane undivided; 4D: 4-lane divided, in this case divided by a median barrier.

Table 24 U.S. 395A Highway Segment and Station Information (continued)

No.	Type	Starting Station (ft)	Starting Station in Design File (ft)	Ending Station (ft)	Ending Station in Design File (ft)	Length (ft)	Left Lane Width (ft)	Right Lane Width (ft)	Left Shoulder width (ft)	Right Shoulder Width (ft)	TWLTL	Median Barrier	Left Side Slope	Right Side Slope
22	4U	26+690.090	"O"660+99.93	28+839.750	"O"682+49.59	2,149.66	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
23	4U	28+839.750	"O"682+49.59	30+959.200	"O"703+69.04P.O.T	2,119.45	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
24	4U	30+959.200	"O"703+69.04P.O.T	31+694.150	"O"711+03.99	734.95	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
25	4U	31+694.150	"O"711+03.99	32+053.860	"O5"714+63.70	359.71	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
26	4U	32+053.860	"O5"714+63.70	33+082.980	"O8"724+84.73	1,029.12	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
27	4U	33+082.980	"O8"724+84.73	34+413.100	"O8"738+14.85	1,330.12	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
28	4U	34+413.100	"O8"738+14.85	34+797.250	"O8"741+99.00	384.15	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
29	4U	34+797.250	"O8"741+99.00	37+276.530	"O"766+78.28	2,479.28	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
30	4U	37+276.530	"O"766+78.28	38+510.090	"O"779+11.84	1,233.56	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
31	4U	38+510.090	"O"779+11.84	39+650.400	"O"790+52.15	1,140.31	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
32	4U	39+650.400	"O"790+52.15	40+398.130	"O"797+99.88	747.73	11.00	11.00	7.00	7.00	YES	NO	1:6	1:6
33	4U	40+398.130	"O"797+99.88	40+418.130	"O"798+19.88	20.00	11.00	11.00	7.00	7.00	YES	NO	1:1.5	1:1.5
34	4U	40+418.130	"O"798+19.88	41+275.200	"O"806+76.95	857.07	11.00	11.00	7.00	7.00	YES	NO	1:1.5	1:1.5
35	4D	41+275.200	"O"806+76.95	41+510.110	"O3"809+11.86	234.91	11.00	11.00	7.00	7.00	NO	YES	1:6	1:6
36	4D	41+510.110	"O3"809+11.86	43+717.590	"O"831+19.34	2,207.48	11.00	11.00	7.00	7.00	NO	YES	1:6	1:6
37	4D	43+717.590	"O"831+19.34	46+783.590	"OC"861+87.34	3,068.00	11.00	11.00	7.00	7.00	NO	YES	1:6	1:6

Table 25 Intersections along Study Highway Segment

No.	Intersection[22]	Station	Major AADT	Minor AADT[23]	Legs	Traffic Control	Intersection Type	Major approaches w/Left Turn Lanes	Major approaches w/Right Turn Lanes	Skew 1	Skew 2
1	Viola Way and U.S. 395A	5+807	2,600	500	3	Stop-Controlled	Three-Legged w/STOP control	1	1	5.00	
2	Washoe Dr and U.S. 395A	8+011	2,600	500	3	Stop-Controlled	Three-Legged w/STOP control	0	1	0.63	
3	Eastlake Blvd and U.S. 395A	13+819	2,600	3,500	3	Stop-Controlled	Three-Legged w/STOP control	1	1	19.80	
4	Pagni Ln and U.S. 395A	22+890	6,200	600	3	Stop-Controlled	Three-Legged w/STOP control	2	1	12.52	
5	Rawhide Dr and U.S. 395A	23+000	6,200	500	3	Stop-Controlled	Three-Legged w/STOP control	1	1	5.11	
6	Laramie Dr and U.S. 395A	25+258	6,200	600	4	Stop-Controlled	Four-Legged w/STOP control	2	2	9.06	8.54
7	Ames Ln and U.S. 395A	26+842	6,200	500	3	Stop-Controlled	Three-Legged w/STOP control	1	1	0.73	
8	Pleasant Valley Dr and U.S. 395A	28+426	6,200	500	3	Stop-Controlled	Three-Legged w/STOP control	1	1	8.04	
9	Andrew Ln and U.S. 395A	33+706	6,200	600	3	Stop-Controlled	Three-Legged w/STOP control	1	1	11.95	
10	Rhodes Rd and U.S. 395A	38+458	6,200	300	4	Stop-Controlled	Four-Legged w/STOP control	2	2	3.33	3.05
11	Towne Dr and U.S. 395A	42+682	6,200	1,000	3	Stop-Controlled	Three-Legged w/STOP control	1	1	35.09	

[22] All intersections along the old U.S. 395 with crash history from 2007 to 2012 were demonstrated in this table. For the U.S. 395A segment, 3 intersections with observed crashes from August 2012 to May 2013 were involved as shown in Table 12.
[23] The AADT for the Eastlake Blvd was extracted from the NDOT TRINA site. Other minor street AADT was estimated by UNR CATER.

Table 26 Site-Specific Crash Data of U.S. 395A since August 2012

Year	Severity	Type	Station	Direction	Relation To Intersection	Intersection Name
2012	Injury	Single Vehicle	12+997.400	Southbound	Non-intersection-related	
2012	Injury	Single Vehicle	13+819.200	Northbound	Intersection-related	Eastlake Blvd and U.S. 395A
2012	Injury	Single Vehicle	18+355.400	Southbound	Non-intersection-related	
2012	Injury	Single Vehicle	34+016.000	Northbound	Intersection-related	Andrew Ln and U.S. 395A
2012	Property damage only	Single Vehicle	34+017.000	Northbound	Intersection-related	Andrew Ln and U.S. 395A
2012	Injury	Single Vehicle	32+700.400	Southbound	Non-intersection-related	
2012	Property damage only	Single Vehicle	38+308.000	Southbound	Intersection-related	Rhodes Rd and U.S. 395A
2012	Injury	Single Vehicle	41+064.400	Northbound	Non-intersection-related	
2012	Property damage only	Multi Vehicle	46+238.800	Northbound	Non-intersection-related	

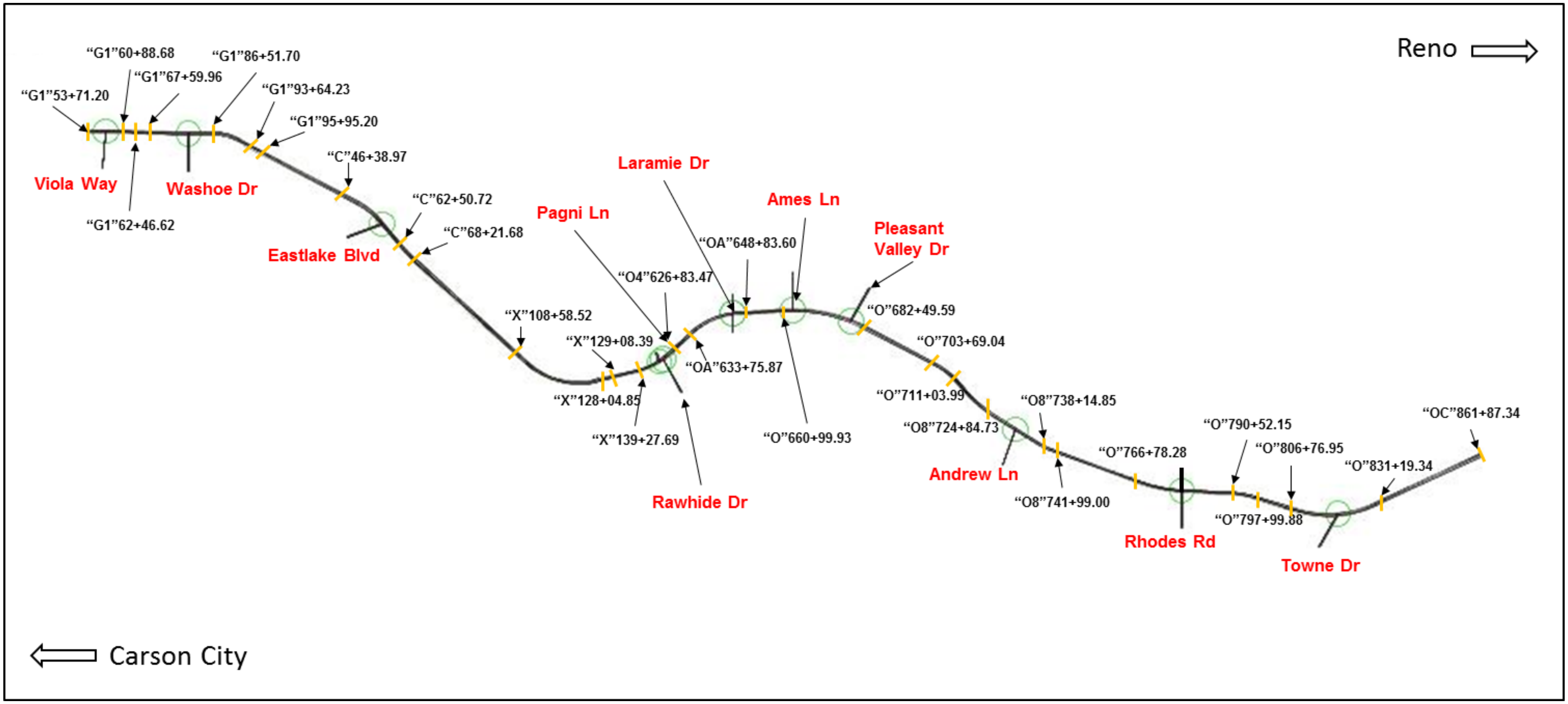


Figure 20 U.S. 395A Highway Segment and Stations

Predictive Method for Rural Multi-Lane Highway Segment

The Rural Multi-Lane crash prediction models and the corresponding CMFs documented in Chapter 11 of the HSM were used to predict the safety of the U.S. 395A segments. The base SPF is as follows.

$$N=exp\ (\ a + (b * Ln\ (AADT)) + Ln\ (L^{d}))$$

Where:

- *a, b, and d* are regression coefficients in Table 27;
- *AADT* is the segment annual average daily traffic;
- *L* is the segment length in miles.

Table 27 Coefficients of the Base SPF for Rural Multi-Lane Highways

Model Class	Segment Type	Intercept (a)	AADT (b)	d
Total	Four-Lane Undivided	-9.653	1.176	1.000
Total	Four-Lane Divided	-9.025	1.049	1.000
Fatal and Injury	Four-Lane Undivided	-9.410	1.094	1.000
Fatal and Injury	Four-Lane Divided	-8.837	0.958	1.000
Fatal and Injury excluding possible injury	Four-Lane Undivided	-8.577	0.938	1.000
Fatal and Injury excluding possible injury	Four-Lane Divided	-8.505	0.874	1.000
Total	Four-Lane Undivided	-9.653	1.176	1.000

Table 28 below summarizes the base conditions for the SPF along with the existing conditions for the application of CMFs. However, the HSM Chapter 11 and thus the IHSDM Crash Prediction Module does not contain methods to evaluate rural multilane highways with TWLTLs, median barrier or inside edge rumble strips.

Table 28 CMFs for U.S. 395A Highway Segments

CMF	Roadway Element	HSM Base Condition	Existing Condition
CMF_{1ru}/CMF_{1rd}	Lane Width Factors	12-feet	Varies (11-12 feet)
CMF_{2ru}	Shoulder Width Factors	6-feet	Varies (6-7 feet)
CMF_{2ru}	Shoulder Type Factors	Paved	Paved
CMF_{3ru}	Side Slope Factors	1:7	Varies (1:1.5-1:6)
CMF_{3rd}	Median Factors	30 feet	10-feet
CMF_{4ru}/CMF_{4rd}	Lighting Factors	None	None
CMF_{5ru}/CMF_{5rd}	Automated Speed Enforcement	None	None

Predictive Method for Intersections on Rural Multi-Lane Highway Segment

The Rural Multi-Lane crash prediction models for intersections and the corresponding CMFs documented in Chapter 11 of the HSM were used to predict the safety of intersections along the analyzed section of the U.S. 395A. The base SPF for intersections is as follows.

$$N=exp\ (a + (b * Ln\ (AADT_{maj})) + (c * Ln\ (AADT_{min}))$$

Where:

- *a, b, and c* are regression coefficients in Table 29;
- $AADT_{maj}$ is the major segment annual average daily traffic;
- $AADT_{min}$ is the minor segment annual average daily traffic.

Table 29 Intersection SPF Factors

Intersection Type	Model Class	Intercept (a)	AADT Major (b)	AADT Minor (c)	Over-dispersion (k)
Three-Legged w/STOP control	Total	-12.526	1.204	0.236	0.460
Three-Legged w/STOP control	Fatal and Injury	-12.664	1.107	0.272	0.569
Three-Legged w/STOP control	Fatal and Injury excluding possible injury	-11.989	1.013	0.228	0.566
Four-Legged w/STOP control	Total	-10.008	0.848	0.448	0.494
Four-Legged w/STOP control	Fatal and Injury	-11.554	0.888	0.525	0.742
Four-Legged w/STOP control	Fatal and Injury excluding possible injury	-10.734	0.828	0.412	0.655

The CMFs for intersection skew angle is in the form of the equation below.

$$CMF_{1i} = Skew\ A * skew / (Skew\ B + Skew\ C * skew) + 1.0$$

Where:

- *Skew A, Skew B and Skew C* are factors from Table 30;
- *skew* is the intersection skew angle (in degrees)[24].

[24] The intersection skew angle is the absolute value of the difference between 90 degrees and the actual intersection angle.

Table 30 Intersection CMFs

Intersection Type	Model Class	Skew A	Skew B	Skew C	Left TL One Approach	Left TL Both Approach	Right TL One Approach	Right TL Both Approach
Three-Legged w/STOP control	Total	0.016	0.98	0.016	0.56	--	0.8600	--
Three-Legged w/STOP control	Fatal and Injury	0.017	0.52	0.017	0.45	--	0.7700	--
Three-Legged w/STOP control	Fatal and Injury excluding possible injury	0.017	0.52	0.017	0.45	--	0.7700	--
Four-Legged w/STOP control	Fatal and Injury	0.048	0.72	0.048	0.65	0.42	0.77	0.59
Four-Legged w/STOP control	Fatal and Injury excluding possible injury	0.048	0.72	0.048	0.65	0.42	0.77	0.59
Four-Legged w/STOP control	Total	0.053	1.43	0.053	0.72	0.52	0.86	0.74

Crash Prediction Results of U.S. 395A

By using the IHSDM Software and the geometric parameters, base SPF and CMFs listed above, the Predictive Method was applied to predict the safety of the U.S. 395A in 2014. Table 31-34 and Figure 21 and 22 present the analysis results. A total of 13.92 crashes are expected to occur in 2014 along this 7.84 mile segment.

Table 31 Expected Crash Rates and Frequencies Summary of U.S. 395A in 2014

General Summary	
Year of Analysis	2014
Evaluated Length (mi)	7.8433
Average Future Road AADT (vpd)	4,778
Expected Crashes	
Total Crashes	13.92
Fatal and Injury Crashes	7.92
Fatal and Serious Injury Crashes	5.18
Property-Damage-Only Crashes	6.00
Percent of Total Expected Crashes	
Percent Fatal and Injury Crashes (%)	57%
Percent Fatal and Serious Injury Crashes (%)	37%
Percent Property-Damage-Only Crashes (%)	43%
Expected Crash Rate	
Crash Rate (crashes/mi/yr)	1.7747
Fatal and Injury Crash Rate (crashes/mi/yr)	1.0094
Fatal and Serious Injury Crash Rate (crashes/mi/yr)	0.6609
Property-Damage-Only Crash Rate (crashes/mi/yr)	0.7654
Expected Travel Crash Rate	
Travel Crash Rate (crashes/million veh-mi)	1.02
Travel Fatal and Injury Crash Rate (crashes/million veh-mi)	0.58
Travel Fatal and Serious Injury Crash Rate (crashes/million veh-mi)	0.38
Travel Property-Damage-Only Crash Rate (crashes/million veh-mi)	0.44

Table 32 Expected Crash Type Distribution of U.S. 395A

Crash Type	Fatal and Injury	% Fatal and Injury	PDO	% PDO	Total Crashes	% Total Crashes
Angle Collision	2.86	20.5	1.85	13.3	4.74	34.1
Head-on Collision	0.24	1.7	0.05	0.3	0.19	1.3
Other Collision	0.27	1.9	0.35	2.5	0.73	5.3
Rear-end Collision	2.32	16.7	1.49	10.7	3.52	25.3
Sideswipe	0.39	2.8	0.82	5.9	1.47	10.5
Single	1.84	13.2	1.44	10.3	3.27	23.5
Total Crashes	**7.92**	**56.9**	**5.99**	**43.0**	**13.92**	**100.0**

Table 33 Expected Crash Rates and Frequencies by Highway Segment of U.S. 395A

Intersection Name/Cross Road	Start Station	End Station	Length (mi)	Expected No. Crashes	Expected Crash Rate (crashes/mi/yr)	Travel Crash Rate (crashes/million veh-mi)	Expected No. Crashes/Year (crashes/million veh)	Expected Crash Rate (crashes/yr)
	5+371.200	5+682.200	0.0589	0.036	0.6091	0.64		
	5+682.200	5+807.200	0.0237	0.014	0.6091	0.64		
Viola and US395A	5+807.200			0.100			0.10	0.1005
	5+807.200	5+932.200	0.0237	0.014	0.6091	0.64		
	5+932.200	6+088.680	0.0296	0.018	0.6091	0.64		
	6+088.680	6+246.620	0.0299	0.018	0.6091	0.64		
	6+246.620	6+759.960	0.0972	0.060	0.6162	0.65		
	6+759.960	6+964.710	0.0388	0.025	0.6343	0.67		
	6+964.710	7+886.200	0.1745	0.112	0.6389	0.67		
	7+886.200	8+011.200	0.0237	0.015	0.6506	0.69		
Washoe Dr and US395A	8+011.200			0.164			0.16	0.1635
	8+011.200	8+651.700	0.1213	0.079	0.6517	0.69		
	8+651.700	9+364.230	0.1349	0.089	0.6573	0.69		
	9+364.230	9+595.200	0.0437	0.029	0.6636	0.70		
	9+595.200	12+941.780	0.6338	0.426	0.6719	0.71		
	12+941.780	13+694.200	0.1425	0.225	1.5768	1.66		
	13+694.200	13+721.920	0.0052	0.004	0.6813	0.72		
	13+721.920	13+819.200	0.0184	0.013	0.6813	0.72		
Eastlake Blvd and US395A	13+819.200			0.259			0.16	0.2589
	13+819.200	13+944.200	0.0237	0.016	0.6813	0.72		
	13+944.200	14+553.530	0.1154	0.079	0.6813	0.72		
	14+553.530	15+124.490	0.1081	0.074	0.6813	0.72		
	15+124.490	19+161.330	0.7646	0.648	0.8482	0.89		
	19+161.330	21+107.660	0.3686	0.246	0.6665	0.70		
	21+107.660	21+211.200	0.0196	0.013	0.6555	0.69		
	21+211.200	21+733.920	0.0990	0.065	0.6550	0.69		
	21+733.920	22+230.500	0.0940	0.140	1.4884	0.66		
	22+230.500	22+765.000	0.1012	0.150	1.4783	0.65		
	22+765.000	22+875.000	0.0208	0.030	1.4639	0.65		
	22+875.000	22+890.000	0.0028	0.004	1.4610	0.65		
Pagni Lane and US395A	22+890.000			0.476			0.20	0.4755

	22+890.000	23+000.000	0.0208	0.030	1.4606	0.64		

Table 33 Expected Crash Rates and Frequencies by Highway Segment of U.S. 395A (continued)

Intersection Name/Cross Road	Start Station	End Station	Length (mi)	Expected No. Crashes	Expected Crash Rate (crashes/mi/yr)	Travel Crash Rate (crashes/million veh-mi)	Expected No. Crashes/Year (crashes/million veh)	Expected Crash Rate (crashes/yr)
Rawhide Dr and US395A	23+000.000			0.264			0.11	0.2641
	23+000.000	23+015.000	0.0028	0.004	1.4576	0.64		
	23+015.000	23+125.000	0.0208	0.030	1.4570	0.64		
	23+125.000	23+273.630	0.0281	0.041	1.4511	0.64		
	23+273.630	23+728.200	0.0861	0.124	1.4430	0.64		
	23+728.200	23+966.030	0.0450	0.064	1.4181	0.63		
	23+966.030	25+133.000	0.2210	0.315	1.4265	0.63		
	25+133.000	25+258.000	0.0237	0.035	1.4625	0.65		
Laramie Dr and US395A	25+258.000			0.477			0.19	0.4767
	25+258.000	25+383.000	0.0237	0.035	1.4646	0.65		
	25+383.000	25+473.760	0.0172	0.025	1.4668	0.65		
	25+473.760	26+690.090	0.2304	0.338	1.4684	0.65		
	26+690.090	26+717.000	0.0051	0.008	1.4876	0.66		
	26+717.000	26+842.000	0.0237	0.035	1.4877	0.66		
Ames Ln and US395A	26+842.000			0.250			0.11	0.2499
	26+842.000	26+967.000	0.0237	0.035	1.4886	0.66		
	26+967.000	28+301.000	0.2527	0.376	1.4895	0.66		
	28+301.000	28+426.000	0.0237	0.036	1.4986	0.66		
Pleasant Valley Dr and US395A	28+426.000			0.272			0.12	0.2724
	28+426.000	28+551.000	0.0237	0.036	1.4995	0.66		
	28+551.000	28+839.750	0.0547	0.082	1.5003	0.66		
	28+839.750	30+959.200	0.4014	0.603	1.5023	0.66		
	30+959.200	31+694.150	0.1392	0.211	1.5167	0.67		
	31+694.150	32+053.860	0.0681	0.104	1.5217	0.67		
	32+053.860	33+082.980	0.1949	0.583	2.9889	1.32		
	33+082.980	33+581.000	0.0943	0.144	1.5311	0.68		
	33+581.000	33+706.000	0.0237	0.036	1.5344	0.68		
Andrew Ln and US395A	33+706.000			0.563			0.24	0.5629
	33+706.000	33+831.000	0.0237	0.036	1.5353	0.68		
	33+831.000	34+413.100	0.1102	0.169	1.5361	0.68		

	34+413.100	34+797.250	0.0728	0.112	1.5400	0.68		

Table 33 Expected Crash Rates and Frequencies by Highway Segment of U.S. 395A (continued)

Intersection Name/Cross Road	Start Station	End Station	Length (mi)	Expected No. Crashes	Expected Crash Rate (crashes/mi/yr)	Travel Crash Rate (crashes/million veh-mi)	Expected No. Crashes/Year (crashes/million veh)	Expected Crash Rate (crashes/yr)
	34+797.250	37+276.530	0.4696	0.724	1.5426	0.68		
	37+276.530	38+333.000	0.2001	0.309	1.5429	0.68		
	38+333.000	38+458.000	0.0237	0.036	1.5429	0.68		
Rhodes Rd and US395A	38+458.000			0.505			0.21	0.5049
	38+458.000	38+510.090	0.0099	0.015	1.5429	0.68		
	38+510.090	38+583.000	0.0138	0.021	1.5429	0.68		
	38+583.000	39+650.400	0.2022	0.312	1.5429	0.68		
	39+650.400	40+398.130	0.1416	0.218	1.5429	0.68		
	40+398.130	40+418.130	0.0038	0.006	1.5429	0.68		
	40+418.130	41+275.200	0.1623	0.539	3.3233	1.47		
	41+275.200	41+510.110	0.0445	0.066	1.4914	0.66		
	41+510.110	42+557.000	0.1983	0.281	1.4181	0.63		
	42+557.000	42+682.000	0.0237	0.034	1.4181	0.63		
Towne Dr and US395A	42+682.000			0.372			0.15	0.3717
	42+682.000	42+807.000	0.0237	0.034	1.4181	0.63		
	42+807.000	43+717.590	0.1725	0.245	1.4181	0.63		
	43+717.590	46+783.590	0.5807	1.089	1.8755	0.83		

Table 34 Expected Crash Rates and Frequencies by Horizontal Design Element of U.S. 395A

Title	Start Station	End Station	Length (mi)	Expected No. Crashes	Expected Crash Rate (crashes/mi/yr)	Travel Crash Rate (crashes/million veh-mi)
Tangent	5+371.200	6+088.680	0.1359	0.083	0.6091	0.64
Curve 1	6+088.680	6+246.620	0.0299	0.018	0.6091	0.64
Tangent	6+246.620	6+759.960	0.0972	0.060	0.6162	0.65
Curve 2	6+759.960	6+964.710	0.0388	0.025	0.6343	0.67
Tangent	6+964.710	8+651.700	0.3195	0.206	0.6446	0.68
Curve 3	8+651.700	9+364.230	0.1349	0.089	0.6573	0.69
Tangent	9+364.230	12+941.780	0.6776	0.455	0.6714	0.71
Curve 4	12+941.780	13+721.920	0.1478	0.228	1.5450	1.63
Tangent	13+721.920	14+553.530	0.1575	0.107	0.6813	0.72
Curve 5	14+553.530	15+124.490	0.1081	0.074	0.6813	0.72
Tangent	15+124.490	19+161.330	0.7646	0.648	0.8482	0.89
Curve 6	19+161.330	21+107.660	0.3686	0.246	0.6665	0.70
Tangent	21+107.660	22+230.500	0.2127	0.218	1.0236	0.68
Curve 7	22+230.500	23+273.630	0.1976	0.290	1.4682	0.65
Tangent	23+273.630	23+966.030	0.1311	0.188	1.4344	0.63
Curve 8	23+966.030	25+473.760	0.2856	0.410	1.4351	0.63
Tangent	25+473.760	26+690.090	0.2304	0.338	1.4684	0.65
Curve 9	26+690.090	28+839.750	0.4071	0.607	1.4919	0.66
Tangent	28+839.750	30+959.200	0.4014	0.603	1.5023	0.66
Curve 10	30+959.200	31+694.150	0.1392	0.211	1.5167	0.67
Tangent	31+694.150	32+053.860	0.0681	0.104	1.5217	0.67
Curve 11	32+053.860	33+082.980	0.1949	0.583	2.9889	1.32
Tangent	33+082.980	34+413.100	0.2519	0.386	1.5340	0.68
Curve 12	34+413.100	34+797.250	0.0728	0.112	1.5400	0.68
Tangent	34+797.250	37+276.530	0.4696	0.724	1.5426	0.68
Curve 13	37+276.530	38+510.090	0.2336	0.360	1.5429	0.68
Tangent	38+510.090	39+650.400	0.2160	0.333	1.5429	0.68
Curve 14	39+650.400	40+418.130	0.1454	0.224	1.5429	0.68
Tangent	40+418.130	41+510.110	0.2068	0.606	2.9292	1.29
Curve 15	41+510.110	43+717.590	0.4181	0.593	1.4181	0.63
Tangent	43+717.590	46+783.590	0.5807	1.089	1.8755	0.83

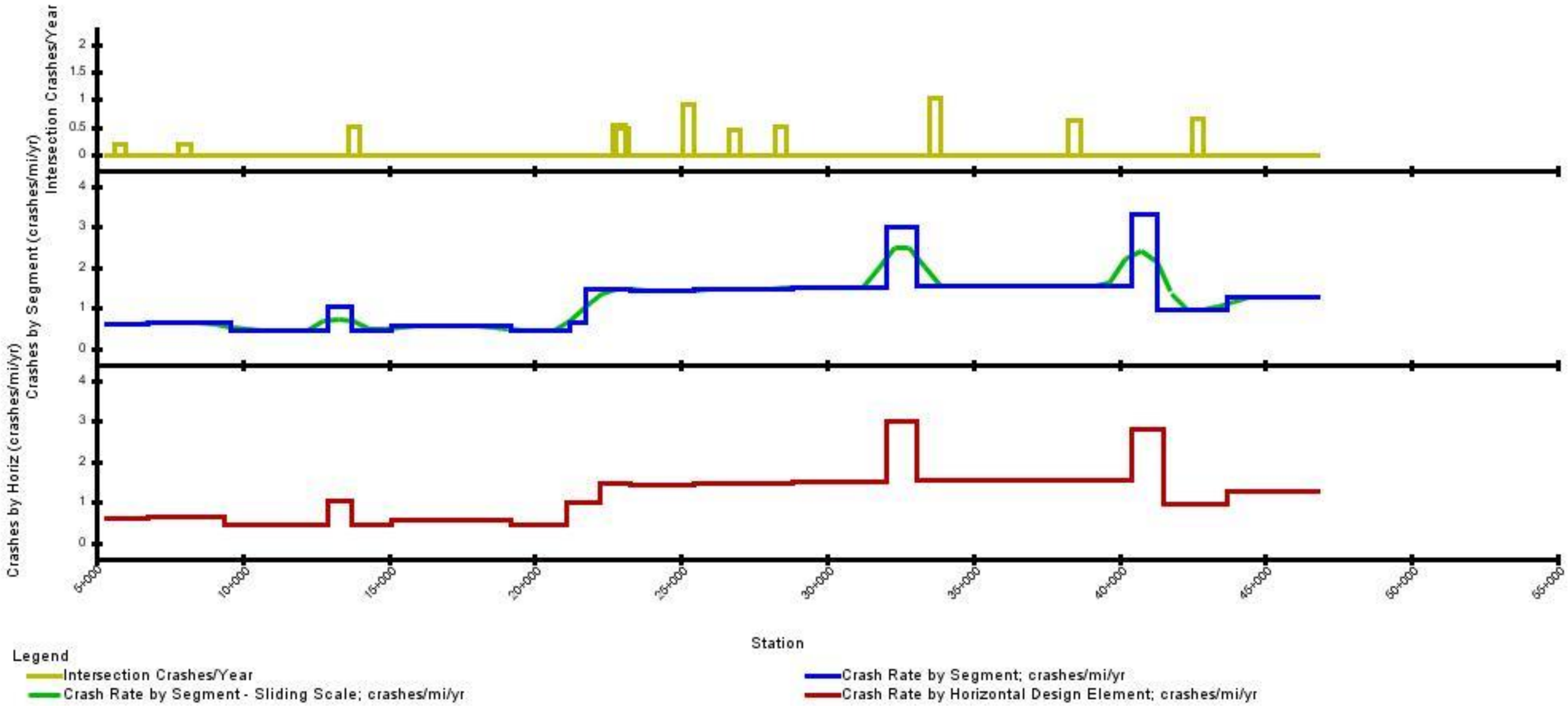


Figure 21 Crash Prediction Results of U.S. 395A

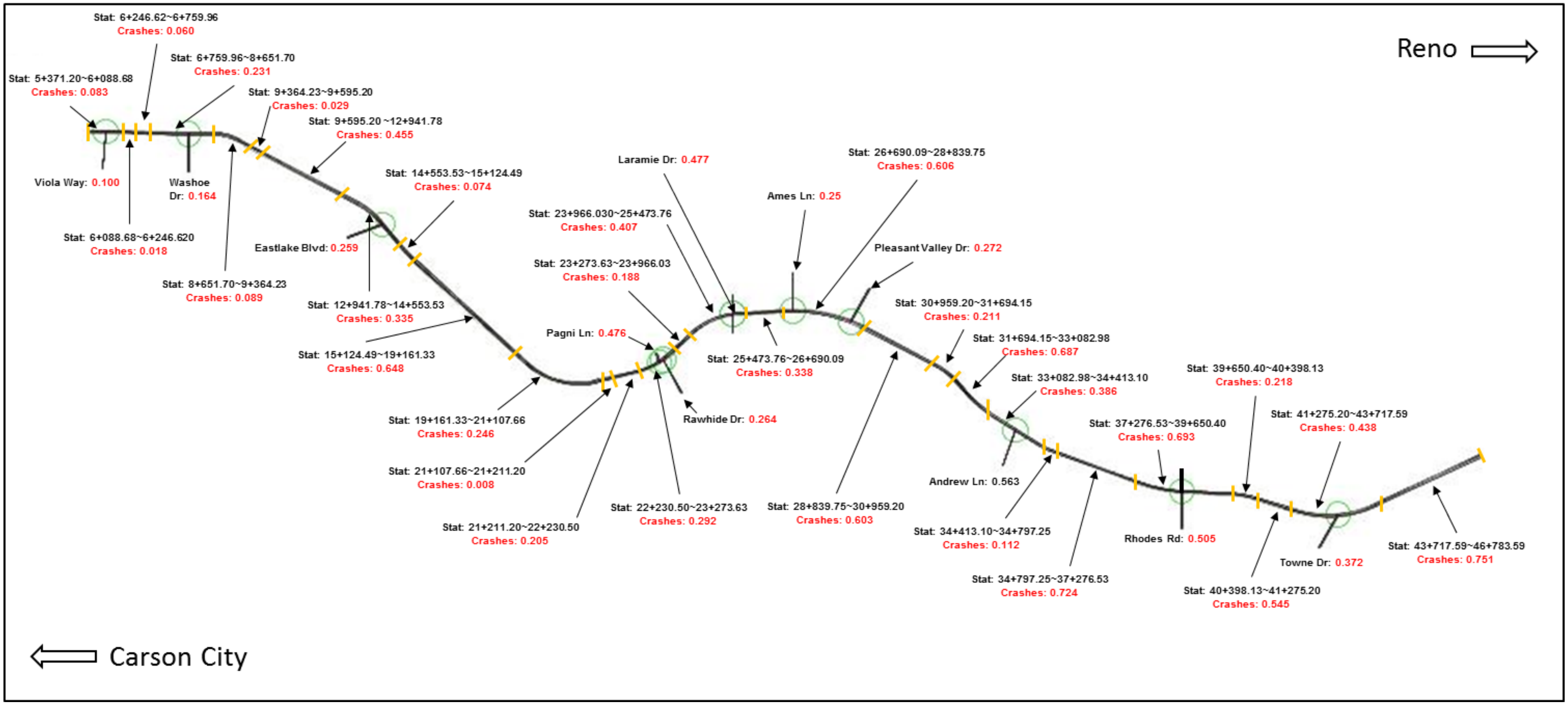


Figure 22 Expected Crash Frequencies on U.S. 395A by Highway Segments and Intersections

SAFETY EVALUATION OF OLD U.S. 395

Old U.S. 395 Segment Data Summary

The old U.S. 395 is the same as the U.S. 395A in terms of roadway geometry and characteristics except for the AADT data as shown in Table 33. The historical crash data was also used for the analysis within the project limits for the five-year period of 2007 to 2011.

Table 35 AADT of Old U.S. 395

Starting Station	Ending Station	Year[25]	AADT (vpd)
5+371.200	21+733.920	2013	31,500
		2014	32,000
21+733.920	46+783.590	2013	33,600
		2014	33,200

Table 36 Crash Severity and Distribution for Old U.S. 395

Severity Type	Number of Crashes
Fatal	2
Injury A	4
Injury B	38
Injury C	48
PDO	253
Total	**345**

Table 37 Crash Collision Type for Old U.S. 395

Collision Type	Number of Crashes
Non-Collision	185
Angle	52
Rear End	74
Head On	3
Sideswipe	28
Other (not reported, unknown, etc.)	3
Total	**345**

Crash Prediction Results of Old U.S. 395

Utilizing the IHSDM Software, the Rural Multi-Lane crash prediction models were applied to calculate the expected total number of crashes for the old U.S. 395 using the Empirical Bayes Method. The subsequent tables summarize the total number of crashes, as well as

[25] AADT data for the year 2013 and 2014 was predicted by UNR CATER.

the severity and manner of collision. A total of 71.06 crashes are predicted to occur in 2014 on the old U.S. 395 if the new freeway segment was not open.

Table 38 Expected Crash Rates and Frequencies Summary of Old U.S. 395

General Summary	
Year of Analysis	2014
Evaluated Length (mi)	7.8433
Average Future Road AADT (vpd)	36,050
Expected Crashes	
Total Crashes	71.06
Fatal and Injury Crashes	32.68
Fatal and Serious Injury Crashes	18.48
Property-Damage-Only Crashes	38.38
Percent of Total Expected Crashes	
Percent Fatal and Injury Crashes (%)	46%
Percent Fatal and Serious Injury Crashes (%)	26%
Percent Property-Damage-Only Crashes (%)	54%
Expected Crash Rate	
Crash Rate (crashes/mi/yr)	9.0601
Fatal and Injury Crash Rate (crashes/mi/yr)	2.8440
Fatal and Serious Injury Crash Rate (crashes/mi/yr)	1.4062
Property-Damage-Only Crash Rate (crashes/mi/yr)	6.2161
Expected Travel Crash Rate	
Travel Crash Rate (crashes/million veh-mi)	0.69
Travel Fatal and Injury Crash Rate (crashes/million veh-mi)	0.32
Travel Fatal and Serious Injury Crash Rate (crashes/million veh-mi)	0.18
Travel Property-Damage-Only Crash Rate (crashes/million veh-mi)	0.37

Table 39 Expected Crash Type Distribution of Old U.S. 395

Crash Type	Fatal and Injury	% Fatal and Injury	PDO	% PDO	Total Crashes	% Total Crashes
Angle Collision	11.81	16.61	12.53	17.63	24.33	34.25
Head-on Collision	1.03	1.44	0.19	0.27	1.21	1.71
Other Collision	1.17	1.65	2.30	3.24	3.47	4.89
Rear-end Collision	9.48	13.34	9.18	12.92	18.66	26.26
Sideswipe	1.63	2.29	5.01	7.05	6.63	9.34
Single	7.56	10.64	9.17	12.91	16.73	23.55
Total Crashes	**32.68**	**45.66**	**38.38**	**54.34**	**71.06**	**100.0**

Table 40 Expected Crash Rates and Frequencies by Highway Segment of Old U.S. 395

Intersection Name/Cross Road	Start Station	End Station	Length (mi)	Expected No. Crashes	Expected Crash Rate (crashes/mi/yr)	Travel Crash Rate (crashes/million veh-mi)	Expected No. Crashes/Year (crashes/million veh)	Expected Crash Rate (crashes/yr)
	5+371.200	5+682.200	0.0589	0.935	15.8762	1.23		
	5+682.200	5+807.200	0.0237	0.022	0.9394	0.07		
Viola and Old US395	5+807.200			0.835			0.06	0.8350
	5+807.200	5+932.200	0.0237	0.022	0.9394	0.07		
	5+932.200	6+088.680	0.0296	0.732	24.6886	1.91		
	6+088.680	6+246.620	0.0299	0.028	0.9394	0.07		
	6+246.620	6+759.960	0.0972	0.091	0.9402	0.07		
	6+759.960	6+964.710	0.0388	0.566	14.5936	1.13		
	6+964.710	7+886.200	0.1745	1.400	8.0237	0.62		
	7+886.200	8+011.200	0.0237	0.022	0.9437	0.07		
Washoe Dr and Old US395	8+011.200			0.886			0.07	0.8863
	8+011.200	8+651.700	0.1213	0.645	5.3159	0.41		
	8+651.700	9+364.230	0.1349	0.127	0.9444	0.07		
	9+364.230	9+595.200	0.0437	0.041	0.9450	0.07		
	9+595.200	12+941.780	0.6338	4.674	7.3739	0.57		
	12+941.780	13+694.200	0.1425	1.376	9.6560	0.75		
	13+694.200	13+721.920	0.0052	0.005	0.9466	0.07		
	13+721.920	13+819.200	0.0184	0.017	0.9466	0.07		
Eastlake Blvd and Old US395	13+819.200			4.173			0.31	4.1728
	13+819.200	13+944.200	0.0237	0.200	8.4359	0.65		
	13+944.200	14+553.530	0.1154	1.350	11.7012	0.90		
	14+553.530	15+124.490	0.1081	0.457	4.2258	0.33		
	15+124.490	19+161.330	0.7646	6.752	8.8313	0.68		
	19+161.330	21+107.660	0.3686	1.588	4.3074	0.33		
	21+107.660	21+211.200	0.0196	0.018	0.9442	0.07		
	21+211.200	21+733.920	0.0990	0.447	4.5168	0.35		
	21+733.920	22+230.500	0.0940	0.439	4.6700	0.35		
	22+230.500	22+765.000	0.1012	0.446	4.4028	0.33		
	22+765.000	22+875.000	0.0208	0.020	0.9359	0.07		
	22+875.000	22+890.000	0.0028	0.003	0.9358	0.07		
Pagni Lane and Old	22+890.000			1.413			0.11	1.4132

Table 40 Expected Crash Rates and Frequencies by Highway Segment of Old U.S. 395 (continued)

Intersection Name/Cross Road	Start Station	End Station	Length (mi)	Expected No. Crashes	Expected Crash Rate (crashes/mi/yr)	Travel Crash Rate (crashes/million veh-mi)	Expected No. Crashes/Year (crashes/million veh)	Expected Crash Rate (crashes/yr)
US395								
	22+890.000	23+000.000	0.0208	0.195	9.3490	0.70		
Rawhide Dr and Old US395	23+000.000			0.671			0.05	0.6713
	23+000.000	23+015.000	0.0028	0.003	0.9356	0.07		
	23+015.000	23+125.000	0.0208	0.545	26.1703	1.97		
	23+125.000	23+273.630	0.0281	0.026	0.9353	0.07		
	23+273.630	23+728.200	0.0861	0.606	7.0366	0.53		
	23+728.200	23+966.030	0.0450	0.217	4.8155	0.36		
	23+966.030	25+133.000	0.2210	0.731	3.3086	0.25		
	25+133.000	25+258.000	0.0237	0.022	0.9359	0.07		
Laramie Dr and Old US395	25+258.000			1.600			0.12	1.6001
	25+258.000	25+383.000	0.0237	0.022	0.9360	0.07		
	25+383.000	25+473.760	0.0172	0.016	0.9361	0.07		
	25+473.760	26+690.090	0.2304	1.092	4.7421	0.36		
	26+690.090	26+717.000	0.0051	0.005	0.9371	0.07		
	26+717.000	26+842.000	0.0237	0.022	0.9371	0.07		
Ames Ln and Old US395	26+842.000			1.144			0.09	1.1437
	26+842.000	26+967.000	0.0237	0.198	8.3519	0.63		
	26+967.000	28+301.000	0.2527	1.641	6.4957	0.49		
	28+301.000	28+426.000	0.0237	0.022	0.9377	0.07		
Pleasant Valley Dr and Old US395	28+426.000			1.159			0.09	1.1594
	28+426.000	28+551.000	0.0237	0.022	0.9377	0.07		
	28+551.000	28+839.750	0.0547	0.403	7.3614	0.55		
	28+839.750	30+959.200	0.4014	2.309	5.7516	0.43		
	30+959.200	31+694.150	0.1392	0.834	5.9903	0.45		
	31+694.150	32+053.860	0.0681	0.592	8.6819	0.65		
	32+053.860	33+082.980	0.1949	1.238	6.3525	0.48		
	33+082.980	33+581.000	0.0943	1.496	15.8601	1.19		
	33+581.000	33+706.000	0.0237	0.374	15.8037	1.19		
Andrew Ln and Old	33+706.000			1.826			0.14	1.8256

Table 40 Expected Crash Rates and Frequencies by Highway Segment of Old U.S. 395 (continued)

Intersection Name/Cross Road	Start Station	End Station	Length (mi)	Expected No. Crashes	Expected Crash Rate (crashes/mi/yr)	Travel Crash Rate (crashes/million veh-mi)	Expected No. Crashes/Year (crashes/million veh)	Expected Crash Rate (crashes/yr)
US395								
	33+706.000	33+831.000	0.0237	0.374	15.8043	1.19		
	33+831.000	34+413.100	0.1102	0.456	4.1317	0.31		
	34+413.100	34+797.250	0.0728	0.772	10.6158	0.80		
	34+797.250	37+276.530	0.4696	0.969	2.0643	0.16		
	37+276.530	38+333.000	0.2001	0.540	2.6992	0.20		
	38+333.000	38+458.000	0.0237	0.022	0.9398	0.07		
Rhodes Rd and Old US395	38+458.000			0.761			0.06	0.7612
	38+458.000	38+510.090	0.0099	0.009	0.9398	0.07		
	38+510.090	38+583.000	0.0138	0.013	0.9398	0.07		
	38+583.000	39+650.400	0.2022	1.950	9.6470	0.72		
	39+650.400	40+398.130	0.1416	0.661	4.6687	0.35		
	40+398.130	40+418.130	0.0038	0.180	47.4102	3.56		
	40+418.130	41+275.200	0.1623	0.505	3.1086	0.23		
	41+275.200	41+510.110	0.0445	0.568	12.7756	0.96		
	41+510.110	42+557.000	0.1983	1.059	5.3430	0.40		
	42+557.000	42+682.000	0.0237	0.372	15.7054	1.18		
Towne Dr and Old US395	42+682.000			1.040			0.08	1.0402
	42+682.000	42+807.000	0.0237	0.022	0.9335	0.07		
	42+807.000	43+717.590	0.1725	1.385	8.0308	0.60		
	43+717.590	46+783.590	0.5807	10.334	17.7964	1.34		

Table 41 Expected Crash Rates and Frequencies by Horizontal Design Element of Old U.S. 395

Title	Start Station	End Station	Length (mi)	Expected No. Crashes	Expected Crash Rate (crashes/mi/yr)	Travel Crash Rate (crashes/million veh-mi)
Tangent	5+371.200	6+088.680	0.1359	1.711	12.5935	0.97
Curve 1	6+088.680	6+246.620	0.0299	0.028	0.9394	0.07
Tangent	6+246.620	6+759.960	0.0972	0.091	0.9402	0.07
Curve 2	6+759.960	6+964.710	0.0388	0.566	14.5936	1.13
Tangent	6+964.710	8+651.700	0.3195	2.068	6.4710	0.50
Curve 3	8+651.700	9+364.230	0.1349	0.127	0.9444	0.07
Tangent	9+364.230	12+941.780	0.6776	4.715	6.9588	0.54
Curve 4	12+941.780	13+721.920	0.1478	1.381	9.3465	0.72
Tangent	13+721.920	14+553.530	0.1575	1.568	9.9524	0.77
Curve 5	14+553.530	15+124.490	0.1081	0.457	4.2258	0.33
Tangent	15+124.490	19+161.330	0.7646	6.752	8.8313	0.68
Curve 6	19+161.330	21+107.660	0.3686	1.588	4.3074	0.33
Tangent	21+107.660	22+230.500	0.2127	0.905	4.2551	0.32
Curve 7	22+230.500	23+273.630	0.1976	1.237	6.2605	0.47
Tangent	23+273.630	23+966.030	0.1311	0.823	6.2737	0.47
Curve 8	23+966.030	25+473.760	0.2856	0.792	2.7723	0.21
Tangent	25+473.760	26+690.090	0.2304	1.092	4.7421	0.36
Curve 9	26+690.090	28+839.750	0.4071	2.313	5.6808	0.43
Tangent	28+839.750	30+959.200	0.4014	2.309	5.7516	0.43
Curve 10	30+959.200	31+694.150	0.1392	0.834	5.9903	0.45
Tangent	31+694.150	32+053.860	0.0681	0.592	8.6819	0.65
Curve 11	32+053.860	33+082.980	0.1949	1.238	6.3525	0.48
Tangent	33+082.980	34+413.100	0.2519	2.700	10.7169	0.80
Curve 12	34+413.100	34+797.250	0.0728	0.772	10.6158	0.80
Tangent	34+797.250	37+276.530	0.4696	0.969	2.0643	0.16
Curve 13	37+276.530	38+510.090	0.2336	0.572	2.4466	0.18
Tangent	38+510.090	39+650.400	0.2160	1.963	9.0902	0.68
Curve 14	39+650.400	40+418.130	0.1454	0.841	5.7821	0.44
Tangent	40+418.130	41+510.110	0.2068	1.073	5.1882	0.39

Curve 15	41+510.110	43+717.590	0.4181	2.838	6.7888	0.51
Tangent	43+717.590	46+783.590	0.5807	10.334	17.7964	1.34

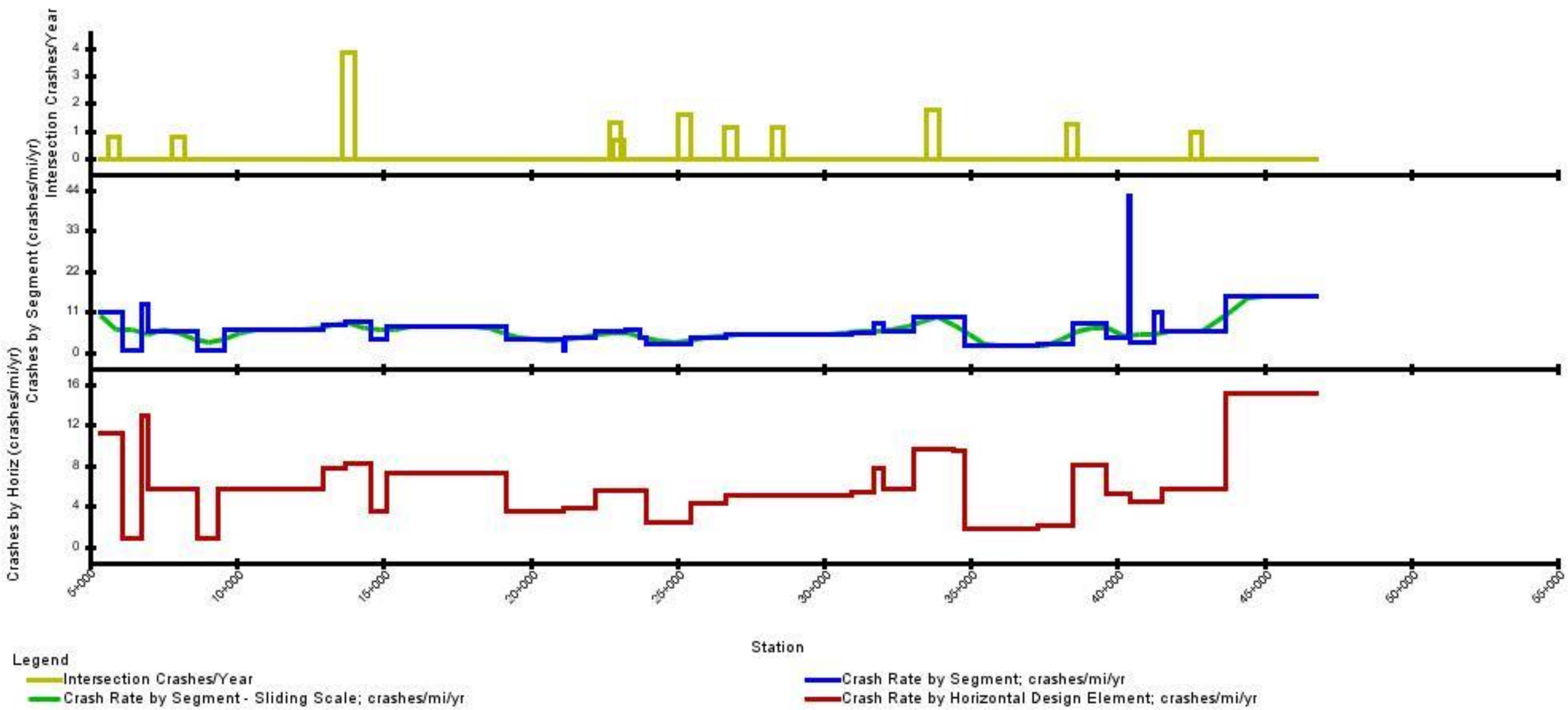


Figure 23 Crash Prediction Results of Old U.S. 395

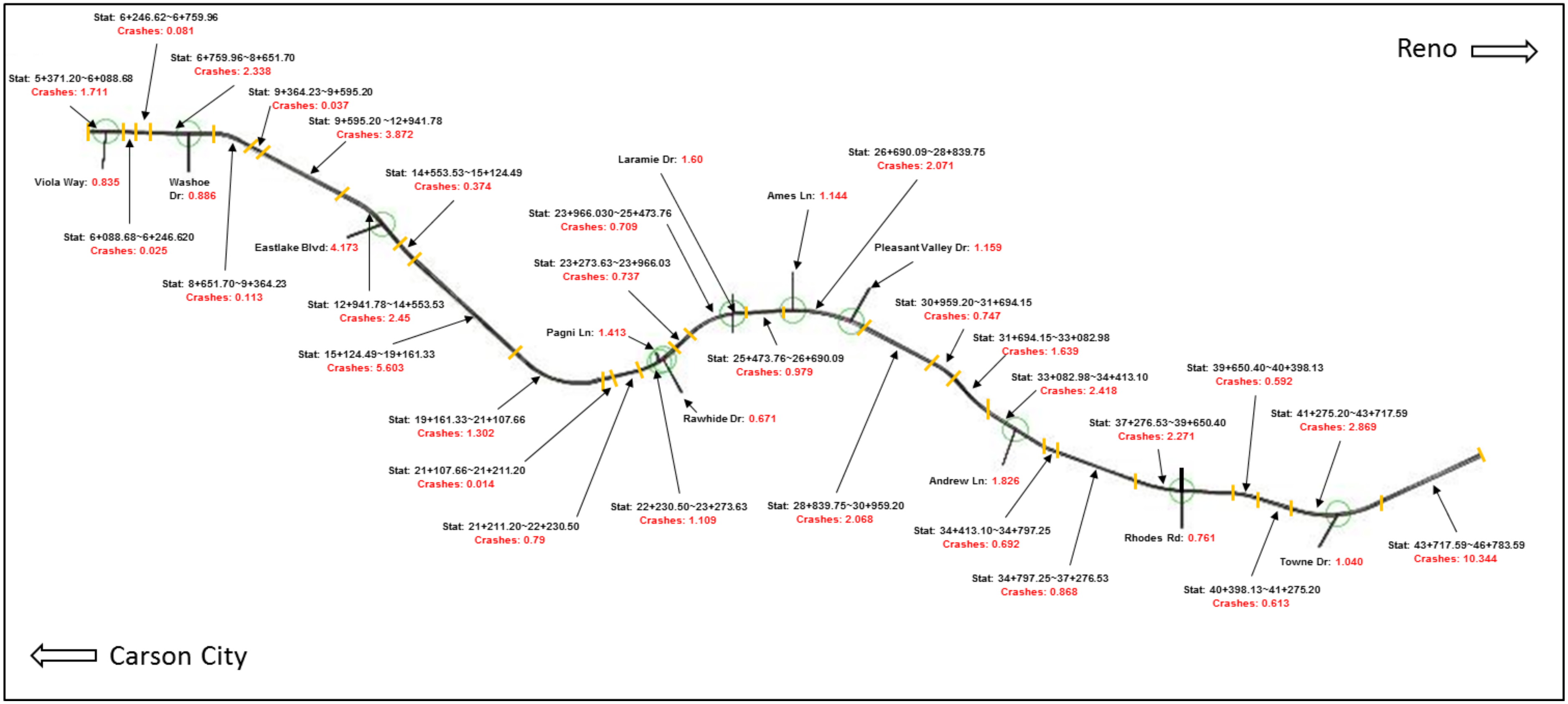


Figure 24 Expected Crash Frequencies on Old U.S. 395 by Highway Segments and Intersections

BENEFIT-COST ANALYSIS

A benefit cost analysis was conducted by UNR CATER for the I-580 Freeway Extension project between Reno/Sparks and Carson City in May 2013. The whole project included the construction of a 9.5-mile new six-lane freeway segment as discussed in Chapter 2. The primary project benefits, as outlined by NDOT, are listed below:

- Completion of I-580 from Reno to Carson City
- Connect Carson City, the state's capital, to the Interstate System
- Decrease congestion
- Shorter commute times
- Improve safety
- Accommodate projected local traffic
- Meet stakeholder/public transportation expectations
- Reduce idling and vehicle emissions
- Beautify the corridor

This freeway extension results in 27 miles of uninterrupted controlled access facility that meets interstate standards. This segment also provides only all-weather route connection between Carson City and Reno, Sparks and I-80. It was predicted that the new I-580 section will effectively alleviate congestion and explosive growth of over 61,700 vehicles per day in North Carson on I-580/U.S. 395 [*36*]. Furthermore, the extension was projected to reduce the over 200 accidents and 16 fatalities that occurred in a 20 year span within similar limits.

This chapter transmits our findings related to the benefit-cost analysis of the I-580 Freeway Extension project. Supporting this analysis, NDOT staff has undertaken engineering studies of the proposed operations under no-build and build conditions using existing and horizon-year traffic volumes [*35*]. In addition, the California Life Cycle Benefit-Cost Analysis Model (CAL-B/C v5.0) was used to conduct this analysis with the pertinent Nevada parameters. This software package was selected since it allows a planning-level analysis of the proposed safety and operational improvements. Additional supporting data used in this analysis was provided by NDOT and includes project information, traffic volumes, freeway speed, etc. CAL-B/C uses the net present value (NPV) method to compare the project costs including initial construction costs along with ongoing operational and maintenance expenses with the monetized user benefits. In the long run, the project will produce quantitative economic benefits accrue within one or more of the following categories:

- Accident reductions
- Travel time savings

- Vehicle operation cost savings
- Vehicle emission reductions

The ratio of user benefits to construction costs when expressed in terms of present value yields the Benefit-Cost Ratio (BCR). In this analysis, a 7% discount rate was applied. The length of the analysis period is 20-year, from year 2014 to year 2034. Overall benefits and costs for the Build Alternative versus the No-Build Alternative are summarized below. The detailed benefit-cost analysis report of this project can be found in Appendix A.

- The net present value of the benefits, assuming a discount rate of 7%, will be about $946.02 million and the net present value of the implementation costs will be about $535.66 million. The Benefit-Cost Ratio (B/C) is 1.76.
- The payback period is 12 years at a discount rate of 7% after the project opens.

CHAPTER 4 CONCLUSIONS AND RECOMMENDATIONS

Task 1: Calibration of Safety Performance Functions for Nevada

This study was conducted to calibrate the existing SPF in HSM for rural two-two-lane roads to represent the conditions in Nevada. The calibration factor was found to be 1.21 for both total and FI crashes indicating that the HSM underestimates the number of crashes by 21%.

The performance of the existing (uncalibrated) HSM SPF and the calibrated HSM SPF was compared using a variety of statistical goodness-of-fit tests and also the CURE plots. Based on the statistical goodness-of-fit tests and also the CURE plots, the calibrated HSM SPF exhibited a better fit to the local data than the uncalibrated HSM SPF. However, the calibrated HSM SPF was not determined as a good-fitting model. To better represent the observed crash frequency, further research needs to be done regarding the development of Nevada-specific SPF for this facility type by considering other functional forms for the variables or adding other explanatory variables.

Task 2: Before-and-After Study Procedures and Methodologies

Before-and-after studies are widely used to evaluate the performance of safety improvement plans. Properly performed before-and-after studies can be used to quantify and assess the safety improvements of a particular treatment. The fundamental reference is the HSM Chapter 9 Safety Effectiveness Evaluation. However, there are no clear recommendations in the HSM on which procedure or methodology should be applied to meet NDOT's needs. This technical research report extracts the essence of the before-and-after evaluations to provide NDOT engineers a clear reference to understand the basic concept, procedures, approaches, and data requirements for conducting a valid before-and-after study.

First of all, the core of a before-and-after study is to compare the estimated safety of the treated group of entities in the 'after' period with the predicted safety of the same group of entities if no treatment had been applied in the 'after' period. The challenge inherent is the fact that the crashes are random and rare events that fluctuate over time. Other factors that affect the safety of a facility, such as traffic volume, environmental conditions, etc. will change over time as well. Consequently, specific evaluation techniques are required to account for those changes in order to estimate the true effects of safety improvements.

Secondly, a before-and-after study normally involves four consecutive steps, including the estimation of basic parameters and their variances, and the estimation of safety effectiveness measurements and their variances. The four-step procedure forms a standardized framework to undertake a before-and-after study no matter what approach is applied.

Thirdly, a before-and-after study can be accomplished using different approaches. In this technical report, four commonly used before-and-after study approaches are presented and compared. The naïve before-and-after study is the simplest technique for this kind of study. It assumes that nothing changes from the 'before' to the 'after' period. This is an unreasonable assumption in terms of statistical validity and tends to overestimate the safety performance of treatments. The application of this approach in real-world projects is not recommended.

The Comparison Group approach takes the potential impact factors besides the treatment into consideration by contrasting the treatment group with the comparison group. This method is recommended when over 5-years of crash data in the 'before' period is available and sufficient for both treatment group and comparison group.

In addition, the rationale behind the before-and-after study with Yoked Comparison is the same as the Comparison Group approach, but there is a strict requirement for a one-to-one matching between the treated site and comparison site. Hence, this approach is recommended when the number of facilities is limited in the comparison group.

Among the four reviewed approaches, the Empirical Bayes method is the most statistically rigorous method. The EB method combines the historical crash data of the treatment group with the predicted safety of the reference group. Therefore, it increases the precision of estimation and corrects for the RTM bias. Real-world engineering studies tend to focus on high crash locations, or hot spots. These type of studies are vulnerable to the RTM bias which might be the most important cause of erroneous conclusions in highway-related evaluations. In the before-and-after study, the ability to obtain an unbiased estimate and prediction is desired to account for the possible RTM bias. Therefore, the HSM recommends the EB method for all the steps in the road safety management process, especially for network screening and countermeasure selection and evaluation. However, employing the EB method requires significantly more data, such as geometry data, traffic volume data, etc. Bedsides, the accuracy of the EB estimates totally relies on well-established SPFs. Nowadays, researchers tend to use more advanced approaches, such as the Full Bayes approach to account for data deficiency. Nonetheless, the Full Bayes is much more complicated for field applications. The lack of required data and accurate SPFs will jeopardize the application of the Bayes method in field applications. Therefore, this technical report recommends that NDOT use the EB method to undertake before-and-after studies if the corresponding SPFs are available for specific highway types.

To conclude, the many variants of before-and-after studies contain different ways to obtain the estimated safety and predicted safety in the 'after' period. This technical report provides a concise introduction of the concepts, merits and limitations of each before-and-after study approach to weigh into the decision process where data availability, resources, and other decisive factors are realities. To conduct a before-and-after study, engineers can refer the process flow chart in Figure 10 for Naïve approach, Figure 12 for C-G and Yoked Comparison approaches, and Figure 16 for the E-B method. Finally, Table 40 summarizes the data requirements, strengths, weaknesses, and implementation recommendations associated with each approach documented in this report. Based on the Table 8, Figure 25 provides a flow chart for selecting suitable before-and-after approaches. The final decision to choose one approach versus another will ultimately depends on data availability, the knowledge of the approach, and perhaps the agency preferences. This report also recommends that NDOT develop related training materials and conduct training sessions for NDOT and local government traffic engineers in using the recommended methods.

Table 42 Summary of Before-and-After Study Approaches

Approaches	Data Requirements	Strengths	Weaknesses	Recommendations
Naïve Before-and-After Study	• Crash Frequency in the 'before' period; • Crash Frequency in the 'after' period.	• Concept is straightforward; • Natural starting point of before-and-after study; • Simple to carry out; • Few data requirements.	• Assumptions are questionable; • Does not address RTM bias; • Ignores the exposure and trend effects over time.	• Not recommended in real-world applications.
Before-and-After Study with Comparison Group (C-G)	• A comparison group that is in conformity with the treatment group in the 'before' period; • Crash Frequency in the 'before' period for the treatment and comparison groups; • Crash Frequency in the 'after' period for the treatment and comparison groups.	• Concept is straightforward; • Treatment group and comparison group need to be similar.	• Not easy to form a comparison group; • Requires a conformity check between treatment and comparison groups; • Does not address RTM bias.	• Recommend when over 5-years of crash data in the 'before' period is available for both treatment and comparison groups.
Before-and-After study with Yoked Comparison	• A comparison group that has a one-to-one similarity with the treatment group; • Crash Frequency in the 'before' period for the treatment and comparison site; • Crash Frequency in the 'after' period for the treatment and comparison site.	• Concept is straightforward; • Simple to carry out; • Fewer data requirements; • Suitable when the sites are limited in the comparison group.	• Requires a one-to-one match between treated and comparison sites; • Results are not stable; • Does not address RTM bias.	• Recommend when over 5-years of crash data in the 'before' period is available for both treatment and comparison sites; • Recommend when number of facilities is limited in the comparison group.
Before-and-After Study with Empirical Bayes (EB) Method	• Safety performance functions that suit the facility and the type of target accidents; • Crash Frequency in the 'before' period; • Crash Frequency in the 'after' period.	• Concrete theoretical background; • Recommended approach by HSM; • Effectively address the RTM bias.	• Require more data; • Requires well-established SPFs and CMFs; • SPFs do not exist for all facilities; • Both SPFs and CMFs need calibrations.	• Recommend when SPFs are available for study highway type.

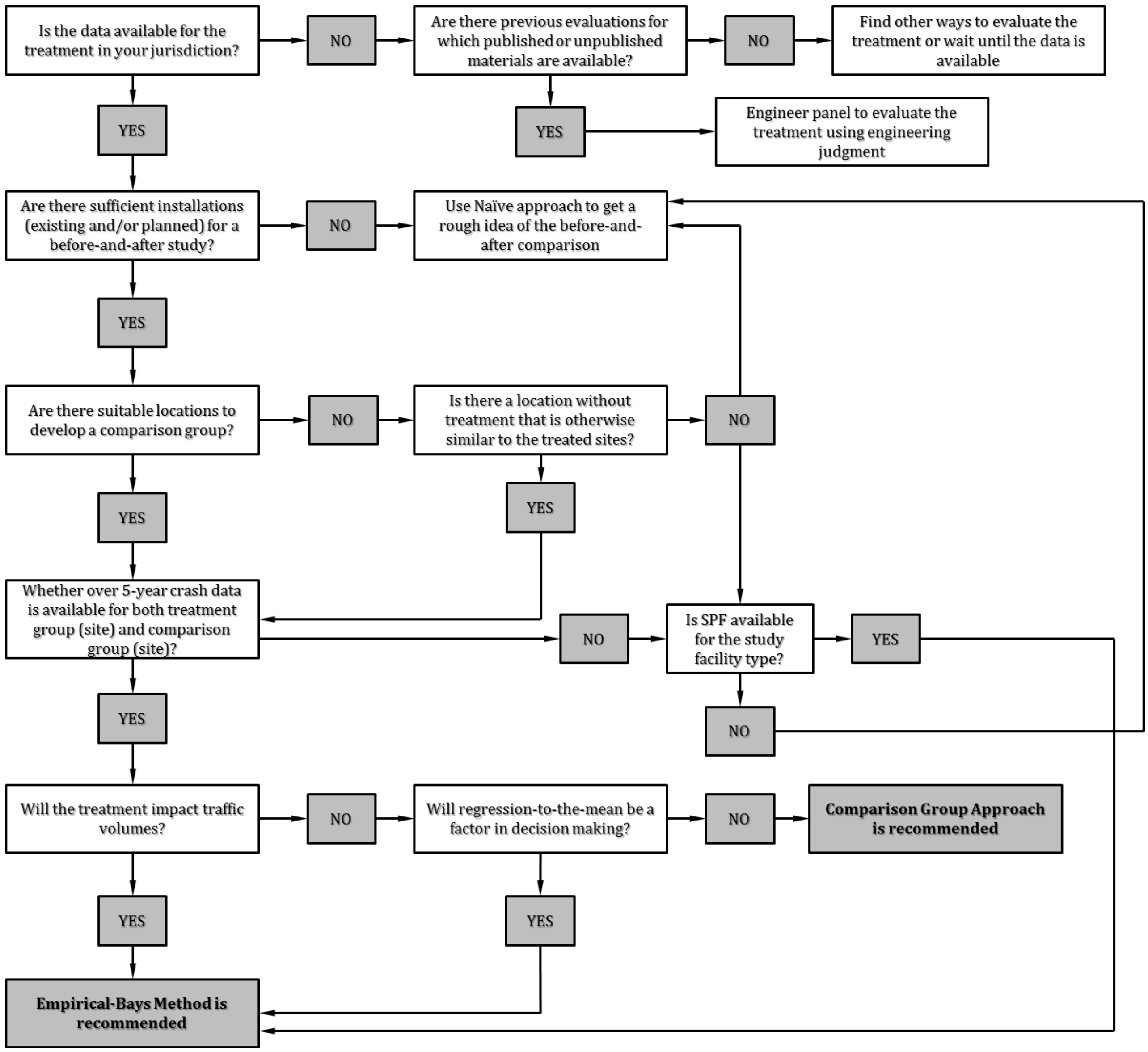


Figure 25 Flow Chart for Selecting Before-and-After Approaches

Task 3: Safety Evaluation of I-580 and U.S. 395 Alt

This task introduces the new section of I-580 and the U.S. 395A—the parallel road segments that serve as a case study for the current UTC contract task. The analysis contains an overview of the roadway segments, and examines the horizontal alignment, vertical alignment and cross-section in more detail through a review of the detailed design plan/profile data. The EB method that weights the observed crash frequencies with the predicted crash frequencies using the base SPFs and CMFs is applied to calculate the expected crash frequencies of U.S. 395A and old U.S. 395 road segments for a one-year period, 2014. The freeway predictive method documented in the future Chapter 18 of HSM is applied to predict the safety of I-580 in 2014 as well.

The evaluation results indicate that in 2014 53 crashes are predicted to occur along the new I-580 freeway section. In addition, 14 crashes are expected to occur along the U.S. 395A segments. The safety of the old U.S. 395/U.S. 395A will be improved significantly given the historical crash records involved approximate 69 crashes per year from 2007 to 2011 and a total of 72 crashes expected in 2014 without the building of the new freeway section. The U.S. 395A is improved for crossing traffic with the new lower AADT.

A benefit cost analysis was conducted for the I-580 Freeway Extension Project and the Benefit-Cost Ratio (BCR) of 1.76 was obtained. In the long run, the project will produce economic benefits in accident reductions, travel time savings, vehicle emission reductions, etc. The opening of I-580 freeway between Mt. Rose Highway and Washoe Valley is more of a benefit than a cost, according to this analysis.

APPENDIX A BENEFIT-COST ANALYSIS REPORT OF I-580 FREEWAY EXTENSION

Project Data

The benefit-cost analysis procedure involves identifying the project type and then providing necessary and available information that permits the Cal-B/C analysis tool to differentiate between the Build and No-Build conditions extending out 20 years after the project opens. The required information includes traffic volumes, lane configuration, speed limit, vehicle occupancies, crash reports, construction costs, operation & maintenance costs, rehabilitation costs, etc. The previous sections present the project specific data used in the Cal-B/C model and the potential benefits of the proposed development.

Study Parameters

Cost analysis parameters obtained from NDOT for the year 2011 were inflated by 7% in order to conduct the analysis using 2013 dollars. Analysis parameters for this analysis include the following four groups.

1) Travel Time Parameters

Consistent with the U.S. Department of Transportation guidance for the valuation of travel time in economic analysis, it is assumed that the local personal travel is to be valued at 50% of the local median wage while business travel by truck/bus drivers was 100% of the mean wage for these occupations plus fringe benefits. A 50% fringe was used because it was an average of several labor groups. Vehicle occupancy values of different counties in Nevada were calculated based on household surveys, census data, and travel demand outputs. Since the I-580 freeway extension project is located in Washoe County, a vehicle occupancy of 1.28 was applied in the analysis. The travel cost and vehicle occupancy parameters provided by NDOT are summarized in Table 43.

Table 43 Nevada Travel Cost ($/hour) and Vehicle Occupancy (2013$)

Location	Local Personal Travel	Business Travel	Vehicle Occupancy
Clark County	$11.14	$34.63	1.45
Carson City/Douglas County	$10.57	$34.06	1.43
Washoe County	$11.35	$34.84	1.28

2) Accident Cost Parameters

The rates of crash occurrences resulting in fatalities, injuries, and property damage only (PDO) were obtained from Chapter 3. These parameters are used to relate incident frequency and/or severity in economic terms. The total cost of accident types is contained in Table 44. These costs were derived from a report from NDOT-Safety Information Management System.

Table 44 Nevada Accident Cost Assumptions (2013$)

Accident Type	Cost
Fatality	$3,658,390
Injury	$98,656
Property Damage Only (PDO)	$4,873

3) Operating Cost Parameters

The cost of operating a vehicle is segregated into fuel and non-fuel costs. Fuel costs are used to estimate impacts due to changes in vehicle miles traveled (VMT) and travel speeds. While non-fuel costs include expenses such as maintenance, repairs, insurance, depreciation, etc. Table 45 shows the operating cost parameters for Nevada.

Table 45 Nevada Operating Cost Parameters (2013$)

Parameter	Mid-Grade Fuel	Diesel Fuel
Fuel Cost per Gallon	$3.87	$4.01
Non-fuel Cost per Mile	Car ($/mile)	Truck ($/mile)
Tires	$0.0103	$0.0246
Depreciation	$0.2659	$0.3431
Maintenance	$0.0475	$0.1102
Insurance	$0.0690	$0.0685
License, Registration, Taxes	$0.0425	$0.0225
Finance Charge	$0.0587	$0.1712

4) Vehicle Emission Parameters

The rate of motor vehicle emissions and associated health costs was based on data from California and are summarized in Table 46.

Table 46 Nevada Vehicle Emission Health Cost Assumptions ($/Ton) (2013$)

Emission Type		Cost
Carbon monoxide	CO	$136
Fine Participates	PM_{10}	$452,610
Nitrogen oxides	NO_X	$55,212
Hydrocarbons	HC	$7,929

Estimated Project Costs

Project costs considered in this analysis include the construction related expenditures followed by annual estimates for ongoing operations, maintenance, and periodic rehabilitation costs. The construction costs, including construction administration, were provided by NDOT. Future cost impacts related to operations, maintenance, and rehabilitation were estimated as those costs likely to be incurred above the expense to maintain already constructed facilities. The detailed estimates of the annualized operations, maintenance, and rehabilitation expenses were assumed to be no more than 10% of the total costs. The estimated project costs including 20-year post construction operations and maintenance costs in constant dollars are presented in Table 47.

Table 47 Estimates of Project Costs

I-580 Freeway Extension	Project Costs in Constant Dollars	Project Costs in Present Value
Right-of-way Cost[26]	$50,021,603	$50,021,603
Construction Costs[15]	$433,194,086	$433,194,086
Design Cost[15]	$6,322,902	$6,322,902
Engineering Cost[15]	$42,596,028	$42,596,028
20-Year O&M and Rehabilitation Cost[27]	$5,200,000	$3,533,485
TOTAL	$537,334,619	$535,668,104

Estimated Project Benefits

The opening of I-580 freeway between Mt. Rose Highway and Washoe Valley is more of a benefit than a cost. The analyzed benefit resulted in terms of travel time, vehicle operating cost savings, and emission cost savings in net present values which are summarized in Table 48. In using the California Life Cycle Benefit-Cost Analysis Model (Cal B/C v5.0) for the freeway extension project it was determined that these improvements provide a benefit cost ratio of 1.76.

[26] Direct project costs from NDOT.
[27] Estimation from similar project (CATER).

Table 48 Summary of Project Benefits (2013$)

Itemized Benefits	Average Annual	Total Over 20 Years
Travel Time Savings	$45,469,034	$909,380,674
Vehicle Operating Cost Savings	$1,341,014	$26,820,281
Accident Cost Savings	$384,594	$7,691,889
Emission Cost Savings	$106,493	$2,129,864
Total Benefits	$47,301,135	$946,022,708
Person-Hours of Time Saved	5,521,729	110,434,587
Additional CO2 Emission (tons)	10,156	203,129
Additional CO2Emissions Saved	$157,494	$3,149,878

Conclusions of Present Values of Overall User Benefits and Project Costs

As can be seen from Table 49 the net present value of the benefits for this project with a discount rate of 7% is almost $946 million and the present value of costs is about $536 million giving a benefit cost ratio of 1.76 with a payback period of 12 years.

Table 49 Present Values of Overall User Benefits and Project Costs (2013$)

Year	PRESENT VALUE OF USER BENEFITS				Present Value of Total User Benefits	Present Value of Total Project Costs	NET PRESENT VALUE
	Travel Time Savings	Vehicle Op. Cost Savings	Accident Reductions	Vehicle Emission Reductions			
Construction Period							
1					$0	$532,134,619	($532,134,619)
Project Open							
1	$23,294,093	($2,457,472)	$345,734	($346,056)	$20,836,299	$250,000	$20,586,299
2	$25,192,701	($2,140,612)	$355,684	($321,688)	$23,086,085	$240,385	$22,845,700
3	$27,135,075	($1,901,043)	$364,358	($304,730)	$25,293,660	$231,139	$25,062,521
4	$29,124,899	($1,653,569)	$371,838	($283,977)	$27,559,191	$222,249	$27,336,942
5	$31,166,507	($1,380,739)	$378,204	($260,835)	$29,903,137	$213,701	$29,689,436
6	$33,264,965	($1,214,614)	$383,531	($247,742)	$32,186,140	$205,482	$31,980,658
7	$35,426,161	($843,266)	$387,888	($211,933)	$34,758,849	$197,579	$34,561,271
8	$37,656,920	($369,475)	$391,342	($55,314)	$37,623,473	$189,979	$37,433,493
9	$39,965,136	$112,573	$393,957	($8,636)	$40,463,031	$182,673	$40,280,358
10	$42,359,943	$612,555	$395,792	$40,475	$43,408,764	$175,647	$43,233,118
11	$44,851,914	$905,040	$396,903	$70,901	$46,224,758	$168,891	$46,055,867
12	$47,453,316	$1,491,755	$397,343	$133,577	$49,475,991	$162,395	$49,313,595
13	$50,178,413	$2,061,815	$397,163	$198,135	$52,835,525	$156,149	$52,679,376
14	$53,043,853	$2,780,521	$396,408	$280,798	$56,501,580	$150,144	$56,351,436
15	$56,069,150	$3,503,757	$395,123	$364,819	$60,332,849	$144,369	$60,188,480
16	$59,277,291	$3,910,328	$393,352	$417,195	$63,998,166	$138,816	$63,859,349
17	$62,695,510	$4,736,403	$391,132	$520,362	$68,343,407	$133,477	$68,209,930
18	$66,356,289	$5,576,118	$388,500	$625,081	$72,945,988	$128,343	$72,817,645
19	$70,298,654	$6,367,654	$385,493	$731,262	$77,783,063	$123,407	$77,659,656
20	$74,569,885	$6,722,555	$382,142	$788,172	$82,462,754	$118,661	$82,344,093
Total	**$909,380,674**	**$26,820,281**	**$7,691,889**	**$2,129,864**	**$946,022,708**	**$535,668,104**	**$410,354,604**
Project Benefit-Cost Ratio					**$946,022,708/$535,668,104=1.766**		

REFERENCES

[1].American Association of State Highway and Transportation Officials (AASHTO). *Highway Safety Manual.* Washington, D.C: American Association of State Highway and Transportation Officials. 2010.

[2].Gaurav, M., and Lou, Y. *Safety Performance Function Calibration and Development for the State of Alabama: Two-Lane Two-Way Rural Roads and Four-Lane Divided Highways*. 92th Transportation Research Board Annual Meeting, Washington D.C. Paper No. 13-4221. 2013.

[3].Saito, M., Brimley, B.K., Schultz, G.G. *Transportation Safety Data and Analysis Volume 2: Calibration of the Highway Safety Manual and Development of New Safety Performance Functions*. Utah Department of Transportation, Research Division, Report No. UT-10.12b. 2011.

[4].Srinivasan, R., Carter, D. *Development of Safety Performance Function for North Carolina*. North Carolina Department of Transportation, FHWA/NC/2010-09. 2011.

[5].Xie, F. *Calibrating the Highway Safety Manual Predictive Methods for Oregon Rural State Highways*" A thesis submitted to Oregon State University. 2011.

[6].Harwood, D.W., and K.M., Bauer. *Statistical Models of At-Grade Intersection Accidents-Addendum*. Federal Highway Administration, FHWA-RD-99-094. 2000.

[7].Hauer, E., and Bamfo, J. *Two Tools for Finding What Functional Form Links the Dependent Variable to the Explanatory Variables*. Published in Proceedings of ICTCT 97 Conference. 1997.

[8].Hauer, E. *Safety Performance Functions: A Workshop*. 2013.

[9].Washington, S., Persaud, B., Lyon, C., and Oh, J. *Validation of Accident Models for Intersections*. FHWA Report, Publication No. FHWA-RD-03-037. 2005.

[10]. Fridstrom, L. Ifver, J., Ingebrigtsen, R., Kulmala, R., and Thomsen L.K. *Measuring the contribution of randomness, exposure, weather, and daylight to the variation in road accident counts*. Accident Analysis and Prevention, Vol. 27. pp. 1-20. 1995.

[11]. Hauer, E. *Observational Before-After Studies in Road Safety*. Pergamon Press, Elsevier Science Ltd., Oxford, England.1997.

[12]. Institute of Transportation Engineers. Transportation Safety Council. *ITE TSC Before-and-After Study*. ISBN-10: 1-933452-46-3. 2009.

[13]. Griffin, L.I. and R.J. Flowers. *A Discussion of Six Procedures for Evaluating Highway Safety Projects*. Washington, D.C., USA: U.S. Department of Transportation (U.S. DOT). Federal Highway Administration (FHWA). 1997.

[14]. Griffith, M. *Safety Evaluation of Continuous Rolled-In Rumble Strips Installed on Freeways*. Transportation Research Record, No. 1665:28-34. 1999.

[15]. Harwood, D.W., Bauer, K.M., and I.B., Potts. *Safety Effectiveness of Intersection Left- and Right-Turn Lanes*. Report No. FHWA-RD-02-089. Washington, D.C., USA: U.S. Department of Transportation (U.S. DOT). Federal Highway Administration (FHWA). 2002.

[16]. Bland, J.M. and D.G., Altman. *Statistic Notes: Regression towards the mean*. British Medical Journal 308 (6942): 1499. PMC 2540330. 1994.

[17]. Bland, J.M. and D.G. Altman. *Statistic Notes: Regression towards the mean*. British Medical Journal 308 (6942): 1499. 1994.

[18]. Michael R. C. and H.F. Robert. *Introductory Biostatistics for the Health Sciences*. ISBN 978-0-471-41137-6. 2003.

[19]. Highway Safety Improvement Program Manual. Washington, D.C., USA: U.S. Department of Transportation (U.S. DOT). Federal Highway Administration (FHWA). http://safety.fhwa.dot.gov/hsip/resources/fhwasa09029/sec2.cfm. 2009.

[20]. Persaud, B.N., R.A., Retting, P.E. Garder and D. Lord. *Safety Effect of Roundabout Conversions in the United States: Empirical Bayes Observational Before-After Study*. Transportation Research Record, No. 1751:1-8. 2001.

[21]. Lord, D. *The Prediction of Accidents on Digital Networks: Characteristics and Issues Related to the Application of Accident Prediction Models*. Toronto, Ontario, Canada: University of Toronto. 2000.

[22]. Bonneson, J.A. and P.T. McCoy. *Estimation of Safety at Two-Way Stop-Controlled Intersections on Rural Highways.* Transportation Research Record, No. 1401: 83-89. 1993.

[23]. Lyon, C., K. Eccles, and R. Amjadi. *Evaluated the Safety Effectiveness of TWLTLs based on the Before-and-After Analysis using the EB Approach*. FHWA-HRT-08-042. Washington, D.C., USA: U.S. Department of Transportation (U.S. DOT). Federal Highway Administration (FHWA). 2008.

[24]. Highway Safety Manual. American Association of State Highway and Transportation Officials (AASHTO). 2010.

[25]. Carlin, B.P. and T.A. Louis. *Bayes and empirical Bayes methods for data analysis, 2nd edition*. New York, Chapman & Hall. 2000.

[26]. Piper, D. *Highway Safety Manual Case Study 4: Development of Safety Performance Functions for Network Screening in Illinois*. Washington, D.C., USA: U.S. Department of Transportation (U.S. DOT). Federal Highway Administration (FHWA). 2010.

[27]. Hauer, E. *Estimating Safety by the Empirical Bayes Method*. Transportation Research Record, Vol. 1784, No. 02-2181, pp. 126-131. 2002.

[28]. Bliss, C.I., and R.A. Fisher. *Fitting the Negative Binomial Distribution to Biological Data and Not on the Efficient Fitting of the Negative Binomial*. Biometrics 9, pp. 176-200.

[29]. Hauer, E. *Overdispersion in Modelling Accidents on Road Sections and in Empirical Bayes Estimation*. Accident Analysis and Prevention. 33(6):799-808. 2001.

[30]. Zhang, Y.L., Z.R., Ye and D. Lord. *Estimating the Dispersion Parameter of the Negative Binomial Distribution for Analyzing Crash Data Using a Bootstrapped Maximum Likelihood Method.* 2006.

[31]. Anscombe, F.J. *The Statistical Analysis of Insect Counts Based on the Negative Binomial Distributions*. Biometrics 5, pp. 165-173. 1949.

[32]. Fisher, R.A. *The Negative Binomial Distribution*. Annals of Eugenics 11, pp. 182-187. 1941.

[33]. Lawless, J.F. *Negative Binomial and Mixed Poisson Regression*. The Canadian Journal of Statistics 15, pp. 209-225, 1987.

[34]. Clark, S.J., and J.N. Perry. *Estimation of the Negative Binomial Parameter K by Maximum Quasi-Likelihood*. Biometrics 45, pp. 309-316. 1989.

[35]. *I-580 Design File: Contract 3292*. Nevada Department of Transportation. Project No. EB-NH-580-1(025)000. August, 2006.

[36]. *Quarterly Report for Major Projects: I-580 Freeway Extension*. Nevada Department of Transportation. December, 2012.

Nevada Department of Transportation
Rudy Malfabon, P.E. Director
Ken Chambers, Research Division Chief
(775) 888-7220
kchambers@dot.nv.gov
1263 South Stewart Street
Carson City, Nevada 89712