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Evaluation of Safety Strategies at Signalized Intersections

Appendices to Final Report

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APPENDIX A:

Safety Effects of Installing Dynamic Signal Warning Flashers at Signalized Intersections

INTRODUCTION

Intersections account for a small portion of the total highway system; however, in 2008, approximately 2.31 million intersection-related crashes occurred. Intersection crashes accounted for 40 percent of all reported crashes and 22 percent (7,421) of all fatal crashes (NHTSA, 2008). The disproportionately high percentage of intersection crashes is not surprising because intersections present more points of conflict than non-intersection locations. Crashes at signalized intersections represent about 51 percent (1.18 million) of all intersection-related crashes, of which 2,511 involved a fatality in 2008 (NHTSA, 2008).

The National Cooperative Highway Research Program (NCHRP) 500 Series Report, Volume 12, identifies safety issues related to signalized intersections and potential countermeasures to address the safety issues (Antonucci et al., 2004). Specifically, the report identifies driver awareness of both intersections and traffic control devices as a critical factor affecting safety. These factors generally lead to rear-end and angle crashes. Objective 17.2 D identifies strategies to improve driver awareness of intersections and signal control. A specific strategy listed in this section is "improve signing and delineation", which includes dynamic signal warning flashers (DSWF).

This Appendix is a summary of the investigation of the safety effects of DSWF at signalized intersections. DSWF provide drivers with advance notice of the phase change. Specifically, the DSWF is linked to the signal and flashers are actuated when the signal is about to change from green to yellow. The flashers are located in advance of the intersection and are actuated at a time when the driver would not be able to clear the intersection before the onset of the red phase. Enhancing awareness of signalized intersections and the impending red phase is expected to reduce conflicts related to sight distance issues or perception-reaction time.

It is important to note that there are many variations of the dynamic signal warning flasher, including roadside and overhead signs. Figure A-1 and Figure A-2 show examples of the types of DSWF found in Virginia. Figure A-3, Figure A-4, and Figure A-5 show examples of DSWF applications in North Carolina. Regardless of the type of advance sign to be installed, agencies should be consistent in the type and placement of the signs.

The visibility and, therefore, placement and maintenance are critical to the effectiveness of advance warning signs. The *Manual on Uniform Traffic Control Devices* (MUTCD) provides guidelines for installation or upgrade of advance warning signs (FHWA, 2008). As identified in NCHRP 500 Series Report, Volume 12, the following are keys to success for this strategy (Antonucci et al., 2004):

- Visibility and clarity of the signal should be improved without creating additional confusion for drivers.
- Additional signing to warn drivers should not clutter the intersection and should not present confusing or conflicting messages to drivers.
- Care should be taken to ensure that new or relocated signs do not present additional sight distance, roadside, or driver distraction hazards.

Figure A-1. Example of Roadside DSWF in Virginia.

Figure A-2. Example of Overhead DSWF in Virginia.

Figure A-3. Example of a Roadside DSWF in North Carolina with a LED display.

Figure A-4. Example of a Roadside "Be Prepare to Stop" DSWF in North Carolina.

Figure A-5. Example of a Roadside "Signal Ahead" DSWF in North Carolina.

Literature Review

There have been a number of evaluations of DSWF over the last three decades employing a variety of measures and designs. Some were crash-based studies while others examined speed or conflict data (e.g., instances of red light running). Among the crash-based studies, the following designs have been used: naïve before-after, before-after with crash rates, before-after with control group, and cross sectional regression models. The results of these various studies have been diverse with many suggesting positive safety effects and others suggesting negligible or negative effects.

Baker et al., (1980) analyzed crash data for intersections with "Prepare to Stop When Flashing" signs as shown in Figure A-1. Their findings suggested statistically significant reductions in total, rear-end, property-damage-only, and truck-related crashes. Using a before-after with comparison group design, another study examined the effects of installing "Red Signal Ahead" signs at signalized intersections (Styles, 1982). The study found a statistically significant reduction in right angle crashes at intersections with limited sight distance due to geometric conditions. The study found no conclusive effects on total, rear-end, or truck-related crashes but discovered some indications of an increase in rear-end crashes. Klugman et al., (1992) analyzed red-light-running rates and speed distributions at intersections with and without DSWF. They observed that red-light-running rates were consistently higher at intersections without DSWF. Upon analyzing the speed distributions, they found that speeds during the green and red phases were the same but speeds during the yellow phase were higher at intersections with DSWF. Gibby et al., (1992) collected 10 years of crash data from 40 intersections and found that intersections with DSWF had lower rates of total, right-turn, left-turn, and rear-end crashes compared to intersections without DSWF. A study of 25 metropolitan intersections in Australia where DSWF was installed utilized a naïve before-after comparison as well as a multivariate regression analysis (Radalj, 2003). The naïve before-after analysis indicated a reduction in severe, right angle, right-turn, and side-swipe crashes but an increase in rear-end crashes. The results of the regression analysis were similar.

More recently, the North Carolina Department of Transportation (NCDOT) conducted an evaluation of the safety effects of installing DSWF at signalized intersections (Simpson, 2010). The evaluation utilized two study designs: naive before-after comparison and a before-after comparison with a linear traffic volume adjustment. A total of 15 signalized intersections were identified in North Carolina where one of the three types of DSWF shown in Figures A-3, A-4, and A-5 had been installed. The evaluation examined the change in the number and severity of total crashes and two target crash types: 1) frontal impact crashes where the mainline vehicle ran the red light, and 2) rear-end crashes on the mainline approaching the signal. The results of the naïve before-after comparison indicated a reduction in total crashes, injury crashes, and target angle crashes and an increase in rear-end crashes. However, only the changes in the injury and target angle crashes were statistically significant at the five percent significance level. The results based on the traffic volume-adjusted before-after comparison indicated a reduction in total crashes, injury crashes, target angle crashes, and target rear-end crashes; only the reduction in target rear-end crashes was statistically insignificant at the five percent level. The NCDOT evaluation also included a further analysis of crashes by level of injury severity. For total crashes, target frontal crashes, and target rear-end crashes, the declines in high severity injury crashes (i.e. level K or A) were greater than the declines in low severity crashes (i.e. level B or C). Based on their evaluation, NCDOT concluded that there is no overwhelming evidence to suggest DSWF reduce total crashes but they do appear to reduce high severity and target frontal crashes (Simpson, 2010).

Other studies suggested that the safety effects of DSWF were negligible or negative. Sayed et al., (1999) developed regression models using 106 intersections and three years of data to estimate the effects of DSWF. Their regression analysis indicated that DSWF reduced total and injury crashes, but the reductions were not significant based on a five percent significance level. McCoy and Pesti (2003)

analyzed red-light-running, abrupt stopping, and accelerating during the yellow phase. They reported no statistically significant benefits. Pant and Xie (1995) examined speed and conflict data and concluded that installations of DSWF at intersections with tangent approaches should be discouraged since they appeared to encourage higher vehicle speeds.

OBJECTIVE

The objective was to estimate the general safety effectiveness of installing dynamic (i.e., signal-actuated) signal warning flashers (DSWF) at signalized intersections, as measured by crash frequency. Target crash types included:

- All crash types (all severities).
- Rear-end crashes (all severities).
- Right-angle (side impact) crashes (all severities).
- Fatal and injury crashes (all crash types).
- Truck-related crashes (all severities).

Meeting these objectives placed some special requirements on the data collection and analysis tasks, including the need to:

- Select a large enough sample size to detect, with statistical significance, what may be small changes in safety for some crash types.
- Carefully select comparison or reference sites.
- Properly account for traffic volume changes.
- Pool data from multiple jurisdictions to improve reliability of the results and facilitate broader applicability of the research products.

Roadway, traffic volume, and crash data were acquired for sites in Nevada, Virginia, and North Carolina to facilitate the analysis. The States identified treatment and reference sites. The States also provided information related to the installation of the strategy (i.e., location and date).

METHODOLOGY

The treatment of interest is the installation of DSWF at signalized intersections. The study team worked with the Nevada, Virginia, and North Carolina Departments of Transportation to identify signalized intersections where DSWF had been installed in the recent past. Ideally, the installation of DSWF would be at some point after the signal was installed. However, the study team discovered during the data collection process that many of the DSWF in Nevada and Virginia were installed at the same time as the signal.

The desired method for this evaluation is the state-of-the-art empirical Bayes (EB) methodology for observational before-after studies. For the treatment sites in Nevada and Virginia, employing the EB procedure would have only produced an estimate of the combined effect of installing a signal and DSWF simultaneously at stop-controlled intersections. As the intent is to estimate the effect of DSWF alone, it was necessary to investigate the use of alternative methods for developing crash modification factors (CMFs) for the treatment in those two States. The data from North Carolina revealed that many of the DSWF were installed well after the intersections became signalized. However, the data from North Carolina were provided near the end of this study; after the analysis had been completed for Nevada and Virginia. Therefore, the EB method was employed to investigate the safety effects of DSWF in North Carolina, but the results are considered supplemental as the sample size is relatively limited.

Cross-Sectional Analysis

If a before-after method is not possible to evaluate the safety effects of DSWF, an alternative evaluation method is a cross-sectional regression analysis. Negative binomial regression is a common method for developing relationships between crashes and roadway characteristics (e.g., traffic volume, area type, etc). The negative binomial regression model was applied in this evaluation framework to estimate the safety effects of DSWF. Specifically, an indicator variable was included in the models to represent the presence of DSWF. Once the models were estimated, the coefficient for the DSWF term was exponentiated to estimate the CMF. The general functional form of the model assumed for this analysis is shown in Equation 1.

$$Crashes\ /\ year = \exp\left(\alpha + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_n X_n\right) \tag{1}$$

Where,
a and b_1 - b_n = parameters estimated in the model calibration process.
X_1 - X_n = covariates included in the model.

Average annual daily traffic (AADT) and DSWF were included in every cross-sectional model developed for this evaluation. Additional variables were considered based on available data and included in the models if the following conditions were met:

a) The variable significantly improved the model.
b) The effect of the variable was intuitive (e.g., crashes increase as number of approaches increases).

The following additional variables were considered in the model development.

- Area Type (urban/rural indicator).
- Number of Intersection Approaches (3-legged/4-legged indicator).
- Speed Limit (< 55 mph/55+ mph indicator).

These variables were entered the model form as adjustments to the base value of α in Equation 1. The base value of α was estimated for a particular baseline condition (e.g., urban, three-legged intersections with speed limit less than 55 mph). When the condition of the intersection was anything other than the baseline, an adjustment was applied to the base value of α. The parameter values (b's) indicate the magnitude and direction of the adjustment to the base α value.

Common concerns related to cross-sectional analyses include:

- Misspecification of model functional form.
- Confounding effects.
- Interaction effects.
- Inconsistency among results from different studies of the same treatment.

The following techniques were used to address the common concerns associated with cross-sectional studies.

- Several functional forms are explored to test the sensitivity of the coefficients.
- Several covariates are considered for inclusion in the model.
- Interaction effects are tested and included as necessary.
- Models are developed for each State individually and compared for consistency.

Before-After Analysis with Comparison Group

As a validation, the before-after with comparison group method was used to check the reasonableness of the cross-sectional study results for Virginia and Nevada. In Virginia, there were several reference sites identified for which the signal was installed during the study period. The framework of the before-after with comparison group analysis was as follows:

1. Estimate the safety effect of installing a signal only (using the reference sites).
2. Estimate the safety effect of installing a signal and DSWF simultaneously (using the treatment sites).
3. Estimate the individual safety effect of DSWF by comparing the results from 1 and 2.

It was not possible to complete this type of before-after evaluation in Nevada because the reference sites only included intersections that were signalized over the entire study period. Hence, this method could only be used as a validation.

In this method, the comparison group is essentially used to control for factors (other than the treatment itself) that may cause a change in safety when a treatment is implemented (Persaud, 2001). While a comparison group can be used to control for regression-to-the-mean (RTM), this can be difficult because it would be necessary to match treatment and comparison sites on crash frequency (Hauer, 1997). This is a cumbersome process and often better handled by employing the empirical Bayes method. The drawback of the empirical Bayes method is that it requires a sufficient sample of reference sites to calibrate or recalibrate a safety performance function.

To check for evidence of RTM in this study, the crash frequency was aggregated over all treatment sites and plotted for each year before treatment (e.g., 1 year before treatment, 2 years before treatment, 3 years before treatment, etc.). Due to the fact that the treatment (i.e., DSWF) was not installed during the same

year for each site, the treatment sites have various before periods. In the first year before treatment, the total crashes represent eight sites while in the eighth year before treatment, the total crashes represent just one site. To account for this issue, sites with less than four years of data before treatment were excluded from the RTM evaluation and only year one through year four are presented for the remaining sites. As such, the same sites are represented in each year before treatment.

The potential for RTM bias is explored in Figure A-6. The dashed line represents the average crashes per year before treatment and the solid line represents the actual crashes over the four sites in each year before treatment. The solid line indicates that the crashes in year one and year two before treatment were slightly elevated, but overall there is not strong evidence of regression to the mean, especially with the many years of before data available for this study. Therefore, the before-after with comparison group method is pursued without adjustment for RTM.

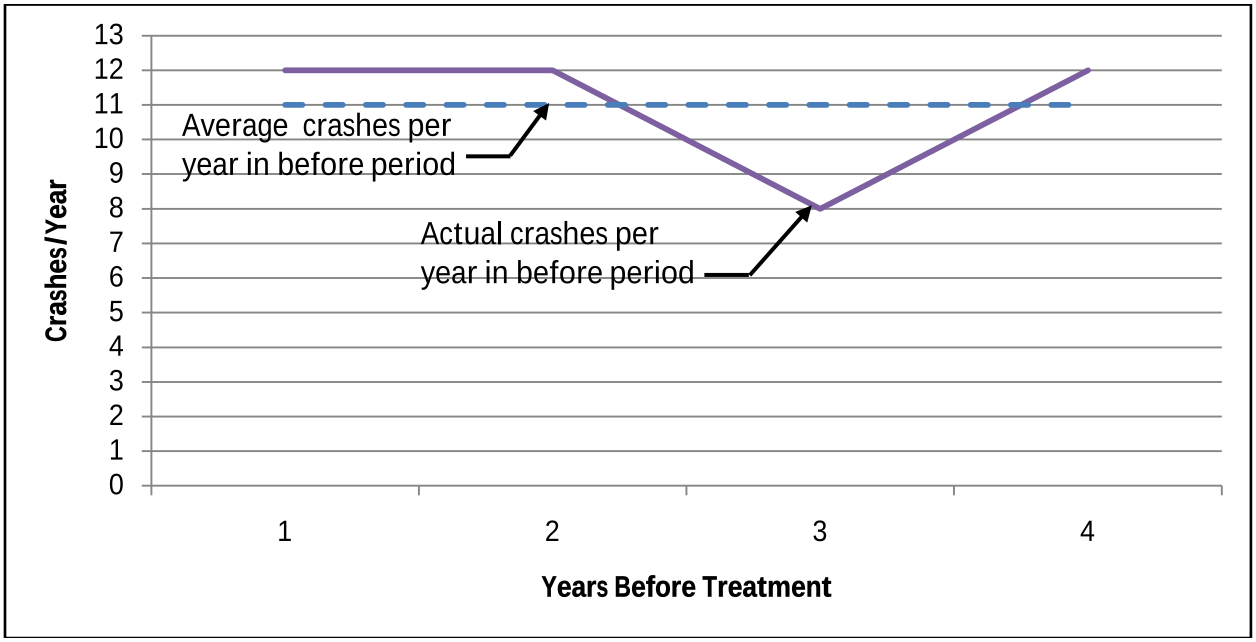

Figure A-6. Exploration of RTM Bias in Virginia.

The procedure outlined by Hauer was followed for this before-after with comparison group study (Hauer, 1997). In a simplified format, the data are structured according to Table A-1.

Table A-1. Simplified Data Structure for Before-After Analysis.

Period	Treatment	Comparison
Before	K	M
After	L	N

Where,
K = observed crash count for treatment group in the before period.
L = observed crash count for treatment group in the after period.
M = observed crash count for comparison group in the before period.
N = observed crash count for comparison group in the after period.

The procedure requires the estimation of the following parameters using Equations 2 through 8.

$$\lambda = L \qquad (2)$$

$$Var(\lambda) = L \qquad (3)$$

$$r_T = r_C = (\frac{N}{M})/(1 + \frac{1}{M}) \qquad (4)$$

$$\frac{Var(r_T)}{r_T^2} = \frac{1}{M} + \frac{1}{N} + Var(\omega) \qquad (5)$$

$$Var(\omega) = s^2(o) - \left(\frac{1}{K} + \frac{1}{L} + \frac{1}{M} + \frac{1}{N}\right) \text{if} > 0\text{, and 0 otherwise} \qquad (6)$$

$$\pi = r_T K \qquad (7)$$

$$Var(\pi) = \pi^2(\frac{1}{K} + \frac{Var(r_T)}{r_T^2}) \qquad (8)$$

Where,
λ = expected crashes at the treatment sites in the after period.
π = expected crashes at the treatment sites in the after period had no treatment been implemented.
Var(λ) = variance of expected crashes in after period.
Var(π) = variance of expected crashes in after period had no treatment been implemented.
r_T = ratio of expected crash counts for the treatment group.
r_C = ratio of expected crash counts for the comparison group.
Var(r_T) = variance of ratio of expected crash counts.
Var(ω) = variance of the underlying mean of the of the odds ratios.
s^2(o) = variance of the sample odds ratios.

The sample odds ratio for a specific time series is calculated from Equation 9 (Hauer, 1997).

$$o = (AD)/(BC)/(1 + \frac{1}{B} + \frac{1}{C}) \qquad (9)$$

Where,
o = estimate of sample odds ratio
A = observed crash count for treatment group in year 1.
B = observed crash count for treatment group in year 2.
C = observed crash count for comparison group in year 1.
D = observed crash count for comparison group in year 2.

While it is common to use a comparison group to account for the effect of changes in all factors, including traffic volume, it is more appropriate to use the comparison group to account for only those factors that cannot be accounted for explicitly (Hauer, 1997). In this study, the traffic volumes are known for both the treatment and comparison groups in both the before and after periods. Hence, the effect of changes in traffic volume can be accounted for explicitly.

The procedure outlined by Hauer was employed to account for changes in traffic volume from the before to the after period at both the comparison and treatment sites (Hauer, 1997). First, the observed crashes in the before period at each comparison site must be adjusted prior to calculating M, the sum of observed crashes for the comparison group in the before period. The adjustment removes the change in safety due to the change in traffic volume so that the comparison group can be used to account for only those factors

that cannot be accounted for explicitly (i.e., factors other than traffic volume). The adjustment to the comparison sites was accomplished using Equation 10. This adjustment makes use of a safety performance function (SPF) to estimate the change in crashes due to the change in traffic volume.

$$CAdj_i = \frac{AC_i}{BC_i} \tag{10}$$

Where,
$CAdj_i$ = adjustment factor for observed crashes in the before period at comparison site (i).
AC_i = expected crashes (from SPF) at comparison site (i) in the after period.
BC_i = expected crashes (from SPF) at comparison site (i) in the before period.

For each comparison site, the adjusted crash counts in the before period were calculated by multiplying the observed crash counts in the before period by the respective adjustment factor $CAdj_i$. The adjusted before period crash counts were then summed to estimate M_{adj}, which replaces M in Equations 6 and 7.

The next step was to adjust for changes in traffic volume from the before to the after period at the treatment sites. This was accomplished by applying the adjustment factor in Equation 11 to estimate π, the expected crashes in the after period had no treatment been applied. Again, Equation 11 makes use of an SPF to predict crashes based on traffic volume. Equation 7 is now replaced by Equation 12.

$$r_{tf} = \frac{AT}{BT} \tag{11}$$

$$\pi = r_T K r_{tf} \tag{12}$$

Where,
r_{tf} = adjustment factor for the change in traffic volume at the treatment sites.
AT = expected crashes (from SPF) at the treatment sites in the after period.
BT = expected crashes (from SPF) at the treatment sites in the before period.
All other variables defined previously.

An adjustment was also necessary because the duration of the before and after periods are different. This does not affect the estimate of π because the durations of the before and after periods are similar for the treatment and comparison sites. This does, however, affect the estimate of the Var(π). To account for this, an adjustment was made to Equation 8. Specifically, the adjustment was calculated using Equation 13 and Equation 8 wsa replaced by Equation 14. The computation of Var(π) in Equation 14 should also include an adjustment for $Var(r_{tf})$. However, the calculation of $Var(r_{tf})$ requires the coefficient of variation, which was not available in this case.

$$r_d = \frac{Years_A}{Years_B} \tag{13}$$

$$Var(\pi) = \pi^2 (\frac{1}{K} + \frac{Var(r_T)}{r_T^2}) r_d^2 \tag{14}$$

Where,
r_d = adjustment factor for the different durations of the before and after period.
$Years_A$ = duration of the after period (years).
$Years_B$ = duration of the before period (years).
All other variables defined previously.

Finally, the index of effectiveness (θ) and the variance of θ were estimated using Equations 15 and 16, respectively. The index of effectiveness is equivalent to the crash modification factor.

$$\theta = \frac{\pi / \lambda}{1 + \left(\frac{Var(\pi)}{\pi^2} \right)} \tag{15}$$

$$Var(\theta) = \sqrt{\frac{\theta^2 \left(\frac{Var(\pi)}{\pi^2} + \frac{Var(\lambda)}{\lambda^2} \right)}{\left(1 + \frac{Var(\lambda)}{\lambda^2} \right)^2}} \tag{16}$$

Where,
θ = index of effectiveness.

The percent change in crashes was calculated using Equation 17. Thus a value of $\theta = 0.8$ with a standard deviation of 0.05 indicates a 20 percent reduction in crashes with a standard deviation of five percent.

$$Percent\ Change\ in\ Crashes = 100 * (1 - \theta) \tag{17}$$

Empirical Bayes Before-After Analysis

As an additional validation, the EB before-after methodology was applied, following the procedure outlined in Hauer (1997). It was only possible to employ this method to the sites in North Carolina because the DSWF was implemented after the installation of the signal and there was a suitable reference group available.

The EB procedure estimates the effect of a treatment by comparing the crash frequency after implementation to the crash frequency that would have been observed had the treatment not been implemented. This approach addresses important problems found in observational before-after analyses. The advantages of the EB approach include the following:

- Properly accounts for regression-to-the-mean.
- Overcomes the difficulties of using crash rates in normalizing for volume differences between the before and after periods.
- Reduces the level of uncertainty in the estimates of safety effect.
- Provides a foundation for developing guidelines for estimating the likely safety consequences of contemplated installations.
- Properly accounts for differences in crash experience and reporting practice in amalgamating data and results from diverse jurisdictions.
- Avoids the difficulties of conventional treatment-comparison experimental designs caused by possible spillover and/or migration effects to natural comparison groups.

In an EB evaluation, the change in safety for a given crash type at a treated intersection is given by Equation 18.

$$B\text{-}A \qquad (18)$$

where *B* is the expected number of crashes that would have occurred in the "after" period without the treatment and *A* is the number of reported crashes in the after period.

Due to changes in safety that may result from changes in traffic volume, regression-to-the-mean, and trends in crash reporting and other factors, the count of crashes before treatment by itself is not a good estimate of *B,* a reality that has now gained common acceptance. Instead, *B* is estimated from an EB procedure in which a safety performance function (SPF) is used to first estimate the number of crashes that would be expected in each year of the before period at locations with traffic volumes and other characteristics similar to the treated site being analyzed. The sum of these annual SPF estimates (*P*) is then combined with the count of observed crashes (*x*) in the before period at the treatment site to obtain an estimate of the expected number of crashes (*m*) before the treatment. The estimate of *m* is given by Equation 19.

$$m = w(P) + (1\text{-}w)(x) \qquad (19)$$

The weight *w* is estimated from Equation 20.

$$w = 1/(1 + kP) \qquad (20)$$

where *k* is the over-dispersion parameter of the negative binomial distribution that is assumed for the crash counts used in estimating the SPF. The value of *k* is estimated from the SPF calibration process with the use of a maximum likelihood procedure.

A factor is then applied to *m* to account for the length of the after period and differences in traffic volumes between the before and after periods. This factor is the sum of the annual SPF predictions for the after period divided by *P*, the sum of these predictions for the before period. The result, after applying this factor, is an estimate of *B*. The procedure also produces an estimate of the variance of *B*, the expected number of crashes that would have occurred in the after period without the treatment.

The estimate of *B* is then summed over all sites in a treatment group of interest (to obtain B_{sum}) and compared with the count of crashes during the after period in that group (A_{sum}). The variance of *B* is also summed over all sections in the group of interest.

The index of safety effectiveness (q) is estimated by Equation 21.

$$\theta = \frac{A_{sum}/B_{sum}}{1 + \left(Var(B_{sum})/B_{sum}^2\right)} \qquad (21)$$

The index of effectiveness is equivalent to the crash modification factor. The standard deviation of q is given by Equation 22.

$$StDev(\theta) = \sqrt{\frac{\theta^2\left(\frac{A_{sum}}{A_{sum}^2} + \frac{B_{sum}}{B_{sum}^2}\right)}{\left(1 + \frac{Var(B_{sum})}{B_{sum}^2}\right)^2}} \tag{22}$$

The percent change in crashes is 100(1- q); thus a value of q = 0.7 with a standard deviation of 0.12 indicates a 30 percent reduction in crashes with a standard deviation of 12 percent.

DATA COLLECTION

This section provides a summary of the databases acquired for Nevada, Virginia, and North Carolina. These include data for signalized intersections with and without dynamic signal warning flashers. In Nevada, it was determined that the DSWF was installed at locations where signalized intersections are located along a high-speed rural road. In many cases, these are locations where a rural road is entering a rural village. In Virginia, the DSWF were installed for a number of reasons including:

- Vertical grade issues.
- Heavy truck volumes.
- Sight distance issues.

In North Carolina, the DSWF were installed on the high speed approaches to signalized intersections to provide drivers in the decision zone with more information and to inform them when a traffic signal is about turn red (Simpson, 2010). The main intent of the DSWF installations was to reduce light running and rear-end crashes (Simpson, 2010).

Reference sites were identified in both Nevada and Virginia from the same regions as the sites with DSWF. The reference sites were matched as closely as possible to the sites with DSWF. The matching variables included area type, speed limit, number of through lanes, and traffic volume. The same attempt was made when identifying reference sites in North Carolina for the EB analysis with a slightly lesser degree of success. As described in the *Empirical Bayes Before-After Analysis* section, the reference sites differed from the treatment sites in some respects.

Table A-2 describes how the crash types analyzed were defined for Nevada, Virginia, and North Carolina. Crash definitions are not identical between the States due to differences in crash reporting and available variables. The study team was unable to obtain crash information on truck involvement for the reference sites in North Carolina. Therefore, the EB before-after comparison with the North Carolina data lacks an analysis of this crash type.

Table A-2. Definitions of intersection-related, rear-end, right-angle, injury, and truck-related crashes used in the analyses for each jurisdiction.

Nevada

Intersection Related – All crashes at or within 500 feet of the intersection.
Rear-end – Defined as all rear-end crashes of any severity.
Angle – Defined as right-angle crashes of any severity.
Injury-related – Defined as those crashes resulting in an injury.
Truck-related – Defined as those crashes involving at least one semi, tractor truck (gasoline or diesel), flatbed, platform, dump, concrete mixer, or tanker.

Table A-2 (continued). Definitions of intersection-related, rear-end, right-angle, injury, and truck-related crashes used in the analyses for each jurisdiction.

Virginia

Intersection Related – All crashes at or within 250 feet of the intersection.
Rear-end – Defined as rear-end crashes of any severity.

Angle – Defined as right-angle crashes of any severity.
Injury-related – Defined as those crashes resulting in an injury.
Truck-related – Defined as those crashes involving at least one of the following vehicle types: straight truck (2 or more axles), flatbed, dump truck, wrecker, tractor truck, tractor trailer, tractor twin-trailer, tractor triple-trailer, or truck tractor (bobtail - no trailer).

North Carolina
Intersection Related – All crashes at or within 250 feet of the intersection.
Rear-end – All intersection-related rear-end crashes of any severity.
Angle – All intersection-related angle crashes of any severity.
Injury-related – All intersection-related crashes resulting in at least one injury and/or fatality.

Cross-Sectional Analysis

The data were structured differently and included different subsets of sites for the two analyses (cross-sectional and before-after). The datasets used in Nevada and Virginia for the cross-sectional analyses are described below.

Nevada Data

Crash data were obtained from the Nevada DOT for years 1994 through 2008. In some cases, all years of data were used in the analysis. In other cases, only a subset of the most recent data was used. The number of years included for each site is dependent on the time period for which other historical data could be identified. For example, if it was determined that no major improvements were made to a site from 2003 to 2008 then only those years were included in the analysis.

The Nevada data were categorized into two groups for the cross-sectional analysis. Tables A-3 and A-4 summarize the characteristics of the following groups.

1. Signalized intersections with DSWF installed.
2. Signalized intersections with no DSWF installed.

Table A-3. Signalized intersections in Nevada with DSWF installed.

Variable	Mean	Minimum	Maximum
Years	7.9	2.0	15.0
Crashes/site-year	14	0	99
Rear-end crashes/site-year	9	0	68
Right-angle crashes/site-year	2	0	21
Injury crashes/site-year	5	0	34
Heavy vehicle crashes/site-year	1	0	7
Major road AADT	36329	9765	99000
Minor road AADT	7263	1300	20100
Number of Sites = 15			

Table A-4. Signalized intersections in Nevada with no DSWF installed.

Variable	Mean	Minimum	Maximum
Years	7.9	6.0	10.0
Crashes/site-year	11	1	72
Rear-end crashes/site-year	6	0	59
Right-angle crashes/site-year	3	0	18
Injury crashes/site-year	3	0	19
Heavy vehicle crashes/site-year	1	0	5
Major road AADT	22970	9700	48500
Minor road AADT	11810	520	57000
Number of Sites = 18			

Virginia Data

Crash data were obtained from the Virginia DOT for years 1998 through 2008. In some cases, all years of data were used in the analysis. In other cases, only a subset of the data was used. The number of years included for each site was dependent on the time period for which other historical data could be identified. For example, if it was determined that no major improvements were made to a site from 2003 to 2008 then only those years were included in the analysis.

The Virginia data were categorized into two groups for the cross-sectional analysis. It should be noted that in some cases, the DSWF was installed prior to the study period while in other cases the DSWF was installed during the study period. Tables A-5 and A-6 summarize the characteristics of the following groups.

1. Signalized intersections with DSWF installed.
2. Signalized intersections without DSWF installed.

Table A-5. Signalized intersections in Virginia with DSWF installed.

Variable	Mean	Minimum	Maximum
Years	7.8	2.0	11.0
Crashes/site-year	2.6	0.0	12.0
Rear-end crashes/site-year	1.5	0.0	9.0
Right-angle crashes/site-year	0.7	0.0	5.0
Injury crashes/site-year	1.1	0.0	5.0
Heavy vehicle crashes/site-year	0.3	0.0	2.0
Major road AADT	18729	7500	33000
Minor road AADT	2408	40	5000
Number of Sites = 15			

Table A-6. Signalized intersections in Virginia without DSWF installed.

Variable	Mean	Minimum	Maximum
Years	8.8	1.0	11.0
Crashes/site-year	2.7	0.0	14.0
Rear-end crashes/site-year	1.5	0.0	9.0
Right-angle crashes/site-year	0.7	0.0	6.0

Injury crashes/site-year	1.1	0.0	8.0
Heavy vehicle crashes/site-year	0.2	0.0	3.0
Major road AADT	16947	2100	40000
Minor road AADT	4290	25	19000
Number of Sites = 35			

Empirical Bayes Before-After Analysis

The treatment and reference datasets from North Carolina used in the EB analysis are described below.

Treatment Sites

Initially, the study team worked with the North Carolina Departments of Transportation (NCDOT) to identify signalized intersections where DSWF had been installed in the recent past. NCDOT had conducted two basic evaluations of DSWF: a naive before-after comparison and a before-after comparison with linear traffic volume adjustment. For those evaluations, NCDOT identified 19 signalized intersections where DSWF had recently been installed. The study team attempted to utilize these sites as the basis for a rigorous EB evaluation of DSWF.

The study team requested and received all the raw roadway characteristics, traffic volume, and crash data that NCDOT had compiled for the treatment sites. Then the study team began analyzing these treatment sites to determine which could be included in the evaluation. Of the 19 sites, one intersection was immediately eliminated from the study because the DSWF for that site were removed shortly after installation. Thus, there was no viable after period for the site. Two of the remaining 18 sites were missing a crucial piece of information: the dates that the traffic signals were installed at the intersections. If the DSWF was installed at the same time or nearly the same time as the traffic signal, it would be difficult to separate the effects of the signalization from the effects of the DSWF installation. Consequently, these two sites were also eliminated. Of the remaining 16 sites, one site had less than a year of before data and was removed. Of the remaining 15 treatment sites, 14 were four-legged and one was three-legged. The three-legged signal was located at a commercial entrance. Therefore, the completeness and accuracy of the traffic volume and crash data were in question for this site. The team decided to eliminate this site because, whether or not accurate data were available for this site, one site could not serve as the basis for a statistically sound and meaningful analysis of thee-legged intersections. In this way, 14 treatment sites were ultimately selected in North Carolina.

Subsequently, the study team assessed the raw roadway characteristics, traffic volume, and crash data provided by NCDOT. The team obtained aerial imagery for each treatment site to verify the roadway characteristics data and collect any required data that was missing. Based on the aerial imagery, minor corrections were made to the raw data as necessary. The aerial imagery also provided area type information which was missing in the raw data. The aerial imagery revealed that most of the sites were rural. To confirm the area type designations, the functional classification of the mainline of each treatment site was queried using 2008 road data from the Highway Safety Information System (HSIS). Based on the mainline functional classification, three of the sites were determined to be urban while 11 were found to be rural.

The raw traffic volume data from NCDOT was then assessed, and the team found that the AADT values were missing for numerous years in the before and after periods of each treatment site. Since the EB analysis would require AADT values for every year in the before and after study periods, the team needed to obtain as many of the missing values as possible. To do so, the team located the treatment sites on

yearly traffic volume maps, which were publicly available on the NCDOT website. These traffic volume maps provided AADT values for most, but not all, of the missing years. In cases where the missing data fell between two years with known AADT values, the team used linear interpolation to fill-in the AADT values for the missing years. In the remaining cases, the nearest known AADT value was assumed. Table A-7 provides a summary of the traffic volume levels for the treatment sites during the entire study period (i.e. including the before and after period).

Table A-7. Traffic Volume Data for Treatment Sites.

Traffic Volume Category	**Minimum**	**Maximum**	**Mean**
Total Entering Volume (AADT)	8,600	30,750	18,362
Major Road Volume (AADT)	4,050	27,500	13,714
Minor Road Volume (AADT)	1,000	24,000	3,119

The team then assessed the raw crash data. Although this dataset was complete, there was another type of complication. The limits of the treatment intersections, as defined by the NCDOT, were problematic for an EB analysis. For each treatment site, NCDOT included the following crashes:

1) For approaches without DSWF, NCDOT included crashes that occurred up to 150 feet from the center of the intersection.
2) For approaches with DSWF, NCDOT included crashes that occurred up to 500 feet in advance of the DSWF.

As described earlier, the EB method requires the use of reference sites (i.e. untreated sites similar in nature to the treatment sites). In order conduct a proper EB analysis, the limits of the reference and treatment sites had to be consistent. The way NCDOT had defined the limits of the treatment sites made this outcome impossible. Since there currently are no standards in North Carolina for the placement of DSWF, the distances between the DSWF and intersection differed from site to site. Consequently, there was no simple way to match the intersection limits of the reference sites to those of the treatment sites. To eliminate this inconsistency, the study team redefined the limits of an intersection to be 250 feet from the center of an intersection for every approach. The study team then acquired crash data for the treatment sites from the NCDOT Traffic Engineering Accident Analysis System (TEAAS) using the radius of 250 feet.

The before and after time periods for the treatment group varied from site to site because the installation dates for the DSWF differed. The earliest installation occurred during September 1994 while the most recent installation occurred in June 2009. In order to eliminate the effects of any construction-related activities, three months of crash data surrounding the installation dates of the DSWF were removed. Since an intersection had to be signalized prior to the installation of the DSWF, the date that traffic signals were installed ideally would define the start of the before period for a treatment site. However, TEAAS data were only available from 1/1/1990 to 12/31/2009. Consequently, for three of the intersections which were signalized before 1990, the before periods had to be adjusted to start on 1/1/1990. The team then discovered that another adjustment to the before periods was necessary when obtaining reference data. Obtaining data for 1990, 1991, and 1992 from suitable reference sites proved difficult. Since these three years represented a small fraction of the before data, the team decided to remove them.

Table A-8 presents the before and after periods for the treatment sites after all adjustments. The team noted that three of the treatment sites had less than one year of after data. Therefore, the results associated with those particular sites were taken with caution. As described in the *Before-After Empirical Bayes Analysis* section, CMFs were calculated with and without these three sites to examine their impact on the overall results.

Table A-8. Before and After Periods for Treatment Sites.

	Before Period			After Period		
	Begin	**End**	**Duration (yrs)**	**Begin**	**End**	**Duration (yrs)**
Site 1	11/1/2001	3/31/2008	6.42	7/1/2008	12/31/2009	1.50
Site 2	1/1/1993	7/31/1994	1.58	11/1/1994	12/31/2009	15.18
Site 3	1/1/1993	6/30/2007	14.50	10/1/2007	12/31/2009	2.25
Site 4	11/1/2001	6/30/2007	5.66	10/1/2007	12/31/2009	2.25
Site 5	1/1/1993	11/30/2008	15.92	3/1/2009	12/31/2009	0.84
Site 6	1/1/1993	5/31/2002	9.42	9/1/2002	12/31/2009	7.34
Site 7	12/1/1994	12/31/2006	12.09	4/1/2007	12/31/2009	2.75
Site 8	6/1/1995	4/30/2009	13.92	8/1/2009	12/31/2009	0.42
Site 9	4/1/1999	5/31/2009	10.17	9/1/2009	12/31/2009	0.33
Site 10	1/1/2001	1/31/2008	7.08	5/1/2008	12/31/2009	1.67
Site 11	1/1/1993	11/30/2007	14.92	3/1/2008	12/31/2009	1.84
Site 12	11/1/1997	12/31/2007	10.17	4/1/2008	12/31/2009	1.75
Site 13	12/1/1994	8/31/2006	11.76	12/1/2006	12/31/2009	3.08
Site 14	9/1/2004	7/31/2007	2.91	10/1/2007	12/31/2009	2.25

Table A-9 provides crash information for the treatment sites.

Table A-9. Crash Data for Treatment Sites during the Before and After Periods.

Variable	**Mean**	**Minimum**	**Maximum**
Before Period Duration (years)	9.75	1.58	15.92
After Period Duration (years)	3.10	0.33	15.18
Crashes/site-year before	7.32	0	27
Crashes/site-year after	5.89	0	30
Rear-end crashes/site-year before	2.62	0	21
Rear-end crashes/site-year after	2.83	0	14
Angle crashes/site-year before	1.94	0	18
Angle crashes/site-year after	1.20	0	5
Injury crashes/site-year before	4.06	0	19
Injury crashes/site-year after	2.90	0	16
Number of Sites = 14			

Reference Sites

The study team attempted to identify reference sites that matched the characteristics of the treatment sites. The team ultimately identified 63 four-legged signalized intersections in North Carolina to serve as the reference group. The reference group selected for this study was similar to the treatment group in many respects. Both groups contained divided and undivided roadways with two or more through lanes and a mixture of lane configurations (e.g., some with exclusive right-turn lanes, some without). The traffic volumes for the reference sites encompassed and exceeded the range of traffic volumes for the treatment sites. Table A-10 provides a summary of the traffic volume data for the reference sites.

Table A-10. Traffic Volume Data for Reference Sites.

Traffic Volume Category	Minimum	Maximum	Mean
Total Entering Volume (AADT)	2,700	49,000	17,285
Major Road Volume (AADT)	2,600	40,000	12,793
Minor Road Volume (AADT)	50	23,000	4,491

Table A-11 gives crash information for these sites.

Table A-11. Crash Data for North Carolina Reference Sites.

Variable	Mean	Minimum	Maximum
Years of Data	15.97	15	16
Crashes/site-year	5.91	0	57
Rear-end crashes/site-year	1.11	0	10
Angle crashes/site-year	1.25	0	19
Injury crashes/site-year	2.20	0	16
Number of Sites = 63			

Despite the best efforts of the team, the reference sites differed from the treatment sites in some respects. Whereas many of the treatment sites had fully-protected left-turn phasing, the reference group was comprised entirely of intersections with permissive or protected-permissive left-turn phasing. This discrepancy could produce SPFs that overestimate the predicted crash frequency at the treatment sites. Previous studies on converting urban intersections from permissive or protected-permissive phasing to fully-protected phasing have estimated a reduction in total crashes ranging from one percent to 42 percent (Harkey et al. 2008; Davis and Aul, 2007). In order to account for these potentially large discrepancies, the team performed the EB analysis for total and angle crashes under two scenarios. In one scenario, the treatment sites with fully-protected left-turn phasing were included in the EB analysis. In the other scenario, the treatment sites with fully-protected left-turn phasing were excluded.

There was another issue with the reference group that limited the after period. Crash data were only available from 1/1/1993 to 8/31/2009. As described earlier, the before periods of the treatment sites were adjusted to begin on 1/1/1993 to help address this issue. The same approach could not be taken with respect to the after periods. While data from 1990, 1991, and 1992 represented a small portion of the before data, data from 2009 represented a significant portion of the after data. Therefore, simply eliminating 2009 data from the study was not a viable option. Another option was to retain the 2009 data from the treatment group, but exclude the 2009 data from the reference group. As such, the SPFs were generated from the reference group based on data from 1993 to 2008. SPF calibration factors were calculated for 2009 based on the partial year of reference data (i.e., 1/1/2009 to 8/31/2009). Although this overall approach was not ideal, the study team concluded it was the best option given the data availability constraints.

ANALYSIS BASED ON CROSS-SECTIONAL MODELS

Cross-sectional models were developed individually for Nevada and Virginia using the negative binomial regression techniques described in the Methodology section. The regression analysis is based on signalized intersections only. For sites that were stop-controlled, which had a signal installed during the study period, only the years for which the site was signalized were included in the analysis. Sites included both treatment sites (with DSWF) and reference sites (without DSWF). Models were developed for both states for total, rear-end, angle, and injury crashes. The data from the two States were also aggregated and combined models were developed for the same crash types. Heavy vehicle crashes were also analyzed, but only in the combined model due to sample size restrictions in the individual state models. The analysis is presented below, first for the two states separately, and then for the two states combined.

Nevada Analysis

Preliminary models were developed using various forms of AADT, including:

1. Separate terms for major and minor AADT.
2. Separate terms for the natural log of major and minor AADT.
3. Total entering AADT.
4. Natural log of total entering AADT.

For the Nevada model, total entering AADT was the most appropriate form for the AADT term. This decision was based on an evaluation of parameter estimates and other goodness of fit measures (i.e., log-likelihood and pseudo R-square). Other comparisons were made using the observed versus predicted values, including total predicted versus total observed crashes and the sum of squared residuals. The minor road AADT was not available for all sites. In these cases, an indicator was used to identify the lack of minor road data. The following variables were considered in the model development.

- Major road AADT (ln(major AADT) was also considered).
- Minor road AADT (ln(minor AADT) was also considered).
- Total AADT (ln(total AADT) was also considered).
- Missing Minor (indicator for sites where minor road AADT was unavailable).
- DSWF (1/0 indicator for dynamic signal warning flasher).
- Approaches (1/0 indicator, 1=four legged).
- Area Type (1/0 indicator, 1=rural).
- Speed Limit (1/0 indicator, 1=55mph or greater).
- Interaction (DSWF x AADT term).

Nevada Model for Total Crashes

There were 261 site-years included in the Nevada analysis, representing 3224 total crashes. The model for total crashes is presented in Table A-12. It is necessary to make an adjustment for the repeated observations (i.e., same site over multiple years). To accomplish this, the model used a clustered robust standard error, identifying individual sites as the cluster variable. In this case, the interaction term is the product of total AADT and the indicator for DSWF. The interaction term was insignificant, particularly after adjusting for the repeated measures. As such, the interaction was removed from the model and the final model is presented at the bottom of Table A-12. Based on the final model, the CMF for DSWF is exp(-0.1911058) = 0.826.

Table A-12. Nevada Model for Total Crashes.

```
WITH INTERACTION

Negative binomial regression                      Number of obs   =        261
Dispersion           = mean                       Wald chi2(6)    =     253.74
Log pseudolikelihood = -813.33403                 Prob > chi2     =     0.0000
------------------------------------------------------------------------------
             |               Robust
total_cras~s |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
   totalaadt |   .0000305   .0000131     2.32   0.020     4.77e-06    .0000562
missingminor |  -.1731692   .1971763    -0.88   0.380    -.5596277    .2132892
        dswf |    -.64564   .5318658    -1.21   0.225    -1.688078    .3967978
     fourleg |   .0851802   .2637229     0.32   0.747    -.4317072    .6020675
       rural |  -.3028919   .3353676    -0.90   0.366    -.9602003    .3544165
interaction1 |    .000012   .0000137     0.87   0.382    -.0000149    .0000388
       _cons |   1.307209   .7863715     1.66   0.096     -.234051    2.848469
-------------+----------------------------------------------------------------
       alpha |   .3365676   .0718544                      .2214868    .5114424
```

```
WITHOUT INTERACTION

Negative binomial regression                      Number of obs   =        261
Dispersion           = mean                       Wald chi2(5)    =     151.01
Log pseudolikelihood = -814.4338                  Prob > chi2     =     0.0000
------------------------------------------------------------------------------
             |               Robust
total_cras~s |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
   totalaadt |   .0000403   3.73e-06    10.80   0.000      .000033    .0000476
missingminor |   -.118618   .1805897    -0.66   0.511    -.4725672    .2353313
        dswf |  -.1911058   .1712693    -1.12   0.264    -.5267875     .144576
     fourleg |    .274548   .1378904     1.99   0.046     .0042878    .5448082
       rural |  -.1041887   .2341075    -0.45   0.656    -.5630311    .3546536
       _cons |   .7233852   .2909374     2.49   0.013     .1531584    1.293612
-------------+----------------------------------------------------------------
       alpha |    .340734    .074763                      .2216394    .5238222
```

Nevada Models by Crash Type and Severity

The functional form of the model for total crashes is applied to individual crash types and severities. Specifically, separate models were developed for rear-end, angle, and injury crashes. These models were based on the dataset with signals only and do not include interaction terms. The models are presented in Table A-13. At the bottom of the table, the CMF is presented for each crash type, including the related sample size.

Table A-13. Nevada Model for Individual Crash Types.

	Rear-end	Angle	Injury
Covariate	**Coefficient**	**Coefficient**	**Coefficient**

	(Robust SE)	(Robust SE)	(Robust SE)
Total AADT	4.46E-05 (3.62E-06)	3.40E-05 (5.51E-06)	4.03E-05 (4.12E-06)
Missing Minor	-0.1510 (0.2305)	0.0104 (0.2266)	-0.0918 (0.2033)
DSWF	-0.1030 (0.2348)	-0.5986 (0.1911)	-0.1584 (0.1878)
Approaches (1 = 4-legged)	0.1730 (0.1820)	0.4833 (0.2547)	0.1715 (0.1471)
Area Type (1 = rural)	-0.3269 (0.2797)	0.1250 (0.3335)	-0.3200 (0.2899)
Constant	0.1176 (0.3801)	-0.7102 (0.3670)	-0.3905 (0.2778)
Alpha	0.4910 (0.1377)	0.3992 (0.1148)	0.3660 (0.1026)
Log pseudo likelihood	-702.1	-491.0	-549.0
Wald chi2	256.92	40.47	131.90
Prob > chi2	0.0000	0.0000	0.0000
CMF	0.902	0.550	0.854
Sample Size (crashes)	1973	632	966

Virginia Analysis

Preliminary models were developed using various forms of AADT, including:

1. Separate terms for major and minor AADT.
2. Separate terms for the natural log of major and minor AADT.
3. Total entering AADT.
4. Natural log of total entering AADT.

For the Virginia model, the natural log of major and minor AADT was the most appropriate form for the AADT terms. This decision was based on an evaluation of parameter estimates and other goodness of fit measures (i.e., log-likelihood and pseudo R-square). Other comparisons were made using the observed versus predicted values, including total predicted versus total observed crashes and the sum of squared residuals. The following variables were considered in the model development. All sites are located in rural areas; hence, area type is not included in the model because there is no variation among sites.

- Major road AADT (ln(major AADT) was also considered).
- Minor road AADT (ln(minor AADT) was also considered).
- Total AADT (ln(total AADT) was also considered).
- Missing Minor (indicator for sites where minor road AADT was unavailable).
- DSWF (1/0 indicator for dynamic signal warning flasher).
- Approaches (1/0 indicator, 1=four legged).
- Speed Limit (1/0 indicator, 1=55mph or greater).
- Interaction1 (DSWF x major AADT term).
- Interaction2 (DSWF x minor AADT term).

Virginia Model for Total Crashes

There were 452 site-years included in the Virginia analysis, representing 1201 total crashes. The model for total crashes is presented in Table A-14. It was necessary to make an adjustment for the repeated observations (i.e., same site over multiple years). To accomplish this, the model used a clustered robust standard error, identifying individual sites as the cluster variable. The first interaction term (interaction1) is the product of ln(major AADT) and the indicator for DSWF (1 indicates flashers are present). The second indicator (indicator2) is the product of ln(minor AADT) and the indicator for DSWF. The first interaction term was marginally significant and the second interaction term was highly significant. This implies that a single CMF does not adequately describe the effect of DSWF, and the relationship is better defined by a crash modification function (CMFunction). Based on the model presented in Table A-14, the CMFunction is:

$$\text{CMFunction(DSWF)} = \exp(-4.442812 + 0.5036722 * \text{lnMajAADT} - 0.1032569 * \text{lnMinAADT})$$

The CMFunction was applied to the Virginia dataset to illustrate the relative effects of DSWF at sites with similar characteristics. The CMFuntion was applied to the dataset assuming two conditions. The first condition assumes that all sites had DSWF in all years. The second condition assumes that all sites had only signals in all years (no DSWF). A CMF was estimated by dividing the total predicted crashes over all site years from condition 1 (with) by the total predicted crashes from condition 2 (without). The total predicted crashes were summed over three groups to check for differences. The results are relatively consistent over all three groups.

- Summing over all sites, the resulting CMF is 0.843.
- Summing over all treatment sites, the resulting CMF is 0.828.
- Summing over all sites without DSWF, the resulting CMF is 0.849.

Table A-14. Virginia Model for Total Crashes.

```
Negative binomial regression                     Number of obs   =        452
Dispersion          = mean                       Wald chi2(7)    =      48.27
Log pseudolikelihood = -910.44677                Prob > chi2     =     0.0000
------------------------------------------------------------------------------
             |               Robust
totalcrashes |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
   lnmajaadt |    .535047   .1657234     3.23   0.001     .2102352    .8598588
   lnminaadt |   .0679468   .0341378     1.99   0.047      .001038    .1348557
        dswf |  -4.442812   3.106847    -1.43   0.153    -10.53212    1.646497
 speed55plus |   .2026891   .1698074     1.19   0.233    -.1301273    .5355056
     fourleg |  -.0886357   .1847021    -0.48   0.631    -.4506451    .2733737
interaction1 |   .5036722   .3130829     1.61   0.108    -.1099591    1.117303
interaction2 |  -.1032569    .037564    -2.75   0.006    -.1768809   -.0296329
       _cons |  -4.652425   1.640712    -2.84   0.005    -7.868162   -1.436688
-------------+----------------------------------------------------------------
    /lnalpha |  -1.127158   .2433592                     -1.604133   -.6501828
-------------+----------------------------------------------------------------
       alpha |   .3239526   .0788368                      .2010637    .5219504
```

The interaction effect is further explored in Figures A-7 to A-10. With the inclusion of interaction terms, the effect of DSWF is not a constant. The interaction terms allow the treatment effect to vary across sites. In this case, the interaction terms allow the treatment effect to vary by major and minor road AADT.

Figure A-7 shows the predicted values for signalized intersections with and without DSWF, assuming a range of major road AADT values and holding all other variables constant (i.e., minor road AADT, speed limit, and number of approaches). The range of AADT is based on the range of the data for signalized intersections with DSWF. The predicted number of crashes increases with increasing major road AADT for both signals with and without DSWF; however, the expected crashes increases at a greater rate for signals with DSWF. This indicates that the DSWF may be more effective at lower AADTs in Virginia (the effectiveness decreases as AADT increases). Figure A-8 further illustrates the point by showing the change in the resulting CMF as a function of major road AADT. The CMF increases as major road AADT increases.

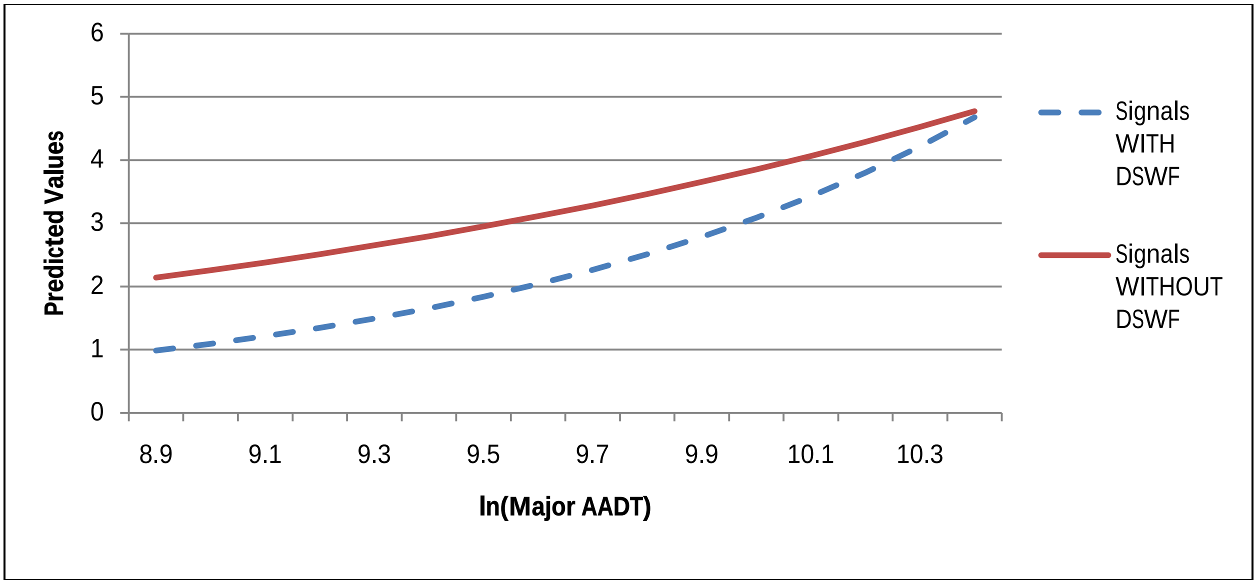

Figure A-7. Interaction Effect of Major Road AADT.

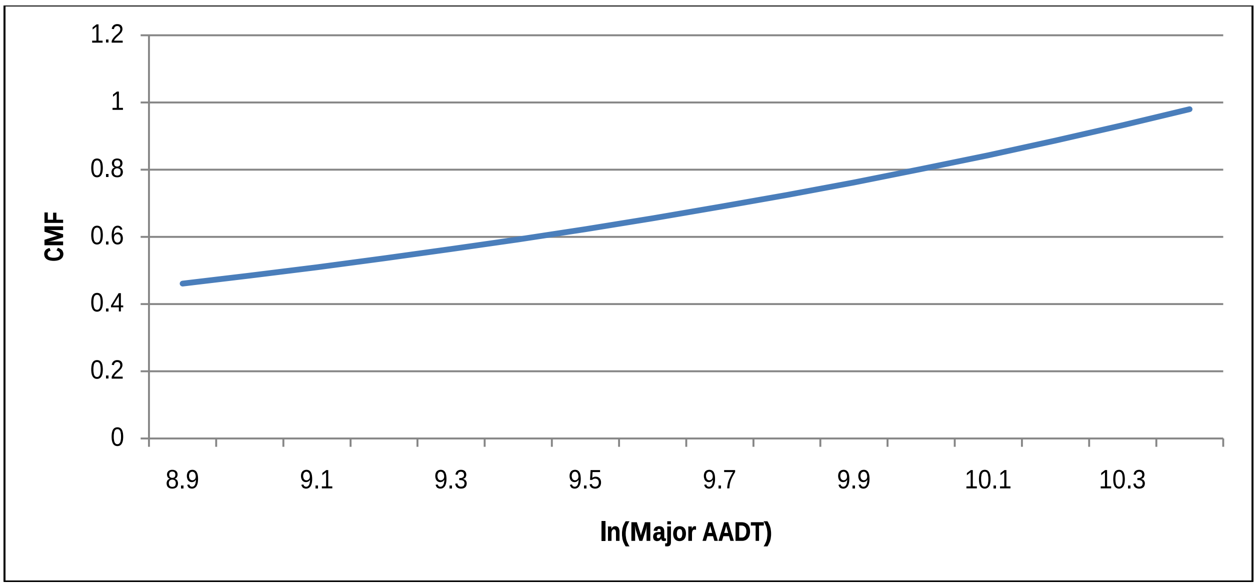

Figure A-8. Change in CMF from Interaction of Major Road AADT.

Figure A-9 shows the predicted values for signalized intersections with and without DSWF, assuming a range of minor road AADT and holding all other variables constant (i.e., major road AADT, speed limit, and number of approaches). The predicted number of crashes increases with increasing minor road AADT for signals without DSWF, but decreases for signals with DSWF. Signals without DSWF are expected to have more crashes than signals with DSWF and the difference in expected crashes increases as minor road AADT increases. Figure A-10 further illustrates the point by showing the change in the resulting CMF as a function of minor road AADT. The CMF is consistently less than 1.0 and decreases as minor road AADT increases.

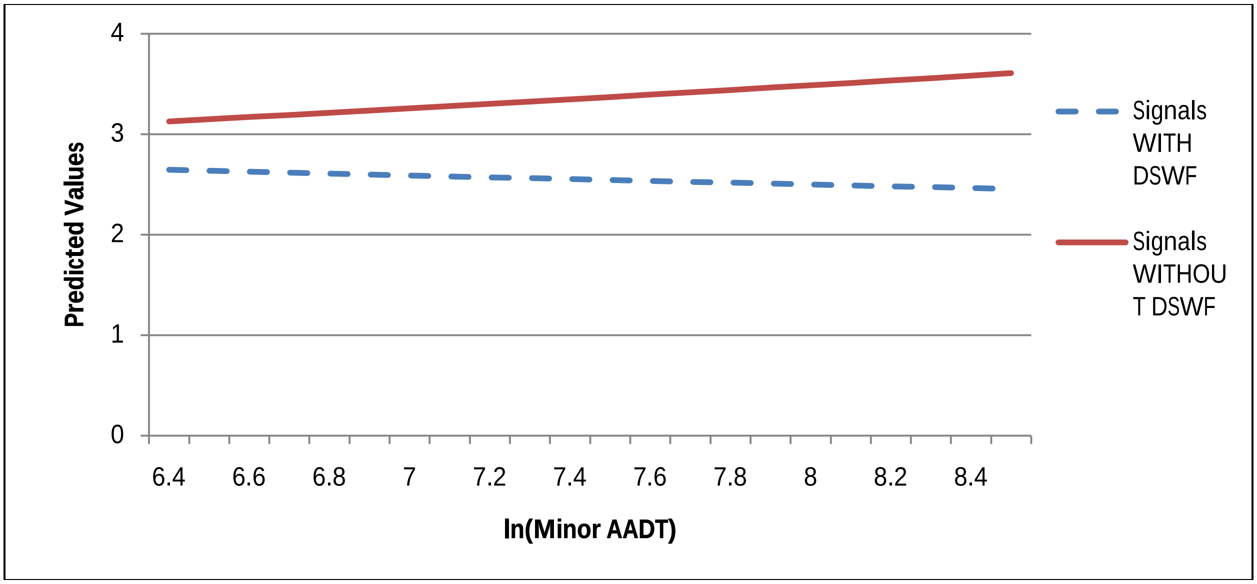

Figure A-9. Interaction Effect of Minor Road AADT.

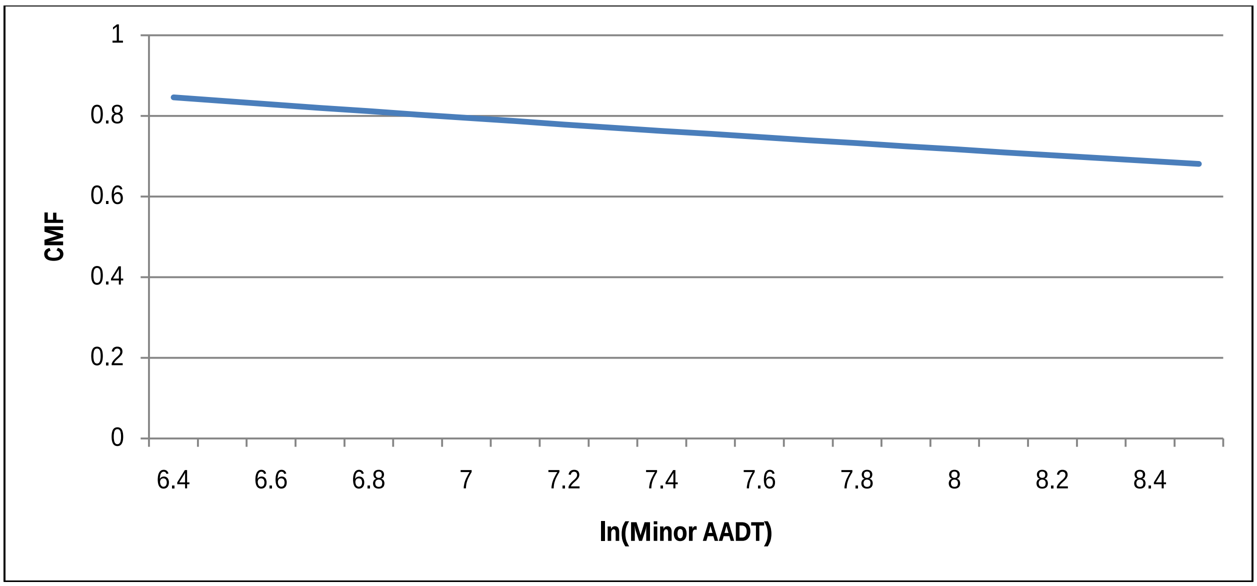

Figure A-10. Change in CMF from Interaction of Minor Road AADT.

The key point of the analysis with interaction is that a single value may not accurately represent the safety impact (i.e., CMF) of a treatment. Rather, it is conceivable that the CMF could vary over the range of values for specific variables (e.g., AADT). If there is evidence of a significant interaction, the relationship is better defined by a crash modification function (CMFunction) rather than a crash modification factor (CMF).

Virginia Models by Crash Type and Severity

The functional form of the model for total crashes was applied to individual crash types and severities. Specifically, separate models were developed for rear-end, angle, and injury crashes. These models were based on the dataset with signals only. The models are presented in Table A-15. Each crash type is then discussed, including the related sample size and CMFunction.

Table A-15. Virginia Model for Individual Crash Types.

Covariate	Rear-end Coefficient (Robust SE)	Angle Coefficient (Robust SE)	Injury Coefficient (Robust SE)
LN(Major AADT)	0.7232	0.3589	0.2318
	(0.2020)	(0.2647)	(0.1866)
LN(Minor AADT)	0.0596	0.0830	0.0864
	(0.0442)	(0.0354)	(0.0398)
DSWF	-8.2374	-1.3029	-5.9263
	(3.5115)	(6.2896)	(3.2238)
Speed (1 = 55+ mph)	0.0878	0.2883	0.2941
	(0.2150)	(0.2170)	(0.1732)
Approaches (1 = 4-legged)	-0.3333	0.3246	-0.0003
	(0.2244)	(0.2594)	(0.2149)
Interaction 1	0.8584	0.2123	0.6250
	(0.3560)	(0.6361)	(0.3255)
Interaction 2	-0.0782	-0.1246	-0.0487
	(0.0475)	(0.0523)	(0.0461)
Constant	-6.8272	-4.6206	-2.8300
	(2.0162)	(2.5209)	(1.8342)
Alpha	0.4291	0.6220	0.4098
	(0.1280)	(0.1688)	(0.1316)
Log pseudo likelihood	-717.8	-517.4	-625.8
Wald chi2	62.97	16.26	20.07
Prob > chi2	0.0000	0.0229	0.0054

Rear-end Crashes

The model of rear-end crashes is based on a total of 678 crashes. The CMFunction for rear-end crashes is:

$$CMF = \exp(-8.237414 + 0.8583682*\ln(majAADT) - 0.0782374*\ln(minAADT))$$

The model was applied to the dataset assuming two conditions. The first condition assumes that all sites had DSWF in all years. The second condition assumes that all sites have only signals in all years (no DSWF). A CMF was estimated by dividing the total predicted crashes over all site years from condition 1 (with) by the total predicted crashes from condition 2 (without). The total predicted crashes were summed over three groups to check for differences.

- Summing over all sites, the resulting CMF is 0.745.
- Summing over all treatment sites, the resulting CMF is 0.770.
- Summing over all sites without DSWF, the resulting CMF is 0.734.

Angle Crashes

The model of angle crashes is based on a total of 329 crashes. The CMFunction for angle crashes is:

$$CMF = \exp(-1.302905 + 0.2122964*\ln(majAADT) - 0.1245866*\ln(minAADT))$$

The model was applied to the dataset assuming two conditions. The first condition assumes that all sites had DSWF in all years. The second condition assumes that all sites have only signals in all years (no DSWF). A CMF was estimated by dividing the total predicted crashes over all site years from condition 1 (with) by the total predicted crashes from condition 2 (without). The total predicted crashes were summed over three groups to check for differences.

- Summing over all sites, the resulting CMF is 0.979.
- Summing over all treatment sites, the resulting CMF is 0.911.
- Summing over all sites without DSWF, the resulting CMF is 1.005.

Injury Crashes

The model of injury crashes is based on a total of 484 crashes. The CMFunction for injury crashes is:

$$CMF = \exp(-5.926328 + 0.6250468*\ln(majAADT) - 0.0486699*\ln(minAADT))$$

The model was applied to the dataset assuming two conditions. The first condition assumes that all sites had DSWF in all years. The second condition assumes that all sites have only signals in all years (no DSWF). A CMF was estimated by dividing the total predicted crashes over all site years from condition 1 (with) by the total predicted crashes from condition 2 (without). The total predicted crashes were summed over three groups to check for differences.

- Summing over all sites, the resulting CMF is 0.861.
- Summing over all treatment sites, the resulting CMF is 0.878.
- Summing over all sites without DSWF, the resulting CMF is 0.855.

Summary of Individual State Analyses

Table A-16 provides a summary of the CMFs from the Virginia and Nevada cross-sectional analyses. The results are relatively consistent among the two states, particularly for the analysis of total, rear-end, and injury crashes (i.e., those groups with the largest sample sizes). Based on the relative consistency in the results, a combined model is explored, aggregating the data from the two states.

Table A-16. Summary of Regression Analysis by State.

Crash Type	Nevada CMF	Virginia CMF	Virginia Dataset
Total	0.826	0.843	Average over all sites
(NV sample size = 3224)		0.828	Average over sites with DSWF
(VA sample = 1201)		0.849	Average over sites without DSWF
Rear-end	0.902	0.745	Average over all sites
(NV sample size = 1973)		0.770	Average over sites with DSWF
(VA sample = 678)		0.734	Average over sites without DSWF

Angle	0.550	0.979	Average over all sites
(NV sample size = 632)		0.911	Average over sites with DSWF
(VA sample = 329)		1.005	Average over sites without DSWF
Injury	0.854	0.861	Average over all sites
(NV sample size = 966)		0.878	Average over sites with DSWF
(VA sample = 484)		0.855	Average over sites without DSWF

Combined State Analysis

The results presented in this section are for the combined regression model, including data for Virginia and Nevada. The models employ an indicator variable to identify the state (1 = Virginia). Similar to the individual state analyses, preliminary models were developed using various forms of AADT. The following variables were considered in the development of the combined model.

- Major road AADT (ln(major AADT) was also considered).
- Minor road AADT (ln(minor AADT) was also considered).
- Total AADT (ln(total AADT) was also considered).
- Missing Minor (indicator for sites where minor road AADT was unavailable).
- DSWF (1/0 indicator for dynamic signal warning flasher).
- State (1/0 indicator for state, 1=Virginia).
- Approaches (1/0 indicator, 1=four legged).
- Speed Limit (1/0 indicator, 1=55mph or greater).
- Interaction (DSWF x AADT term).

Table A-17 summarizes the results of four models based on the combined data for total crashes. The four models include the same covariates, but assume different forms for AADT. Interaction terms were explored as well, but were not included in the final model because they were highly insignificant. Table A-18 shows the specific form of AADT for each of the four models. The coefficients for Model 2 through Model 4 are relatively consistent. Based on the goodness of fit statistics shown at the bottom of Table A-17, Model 3 is selected as the final model for the combined analysis. Model 3 employs the total AADT as the term to represent exposure and also includes an indicator to identify those intersections where minor road AADT is unavailable. The remainder of this section presents results based on the functional form in Model 3.

Table A-17. Comparison of Functional Forms for Combined Analysis.

Covariate	Model 1		Model 2		Model 3		Model 4	
	Coeff	P-val	Coeff	P-val	Coeff	P-val	Coeff	P-val
AADT 1	0.812	0.000	3.84e-5	0.000	4.05e-5	0.000	1.184	0.000
AADT 2	0.041	0.000	5.03e-5	0.000	-0.058	0.417	0.099	0.189
DSWF	-0.226	0.013	-0.118	0.165	-0.206	0.007	-0.171	0.029
State(Virginia=1)	-0.697	0.000	-0.655	0.000	-0.663	0.000	-0.702	0.000
Legs (Four=1)	-0.190	0.020	-0.025	0.752	-0.072	0.334	-0.108	0.163
Area (Rural=1)	-0.683	0.000	-0.107	0.363	-0.168	0.155	-0.158	0.209
Speed (55+=1)	-0.021	0.785	0.058	0.404	0.081	0.245	0.006	0.931
Constant	-5.655	0.000	0.880	0.000	0.994	0.000	-9.823	0.000
Log-Likelihood	-1789		-1728		-1731		-1746	
Pseudo R2	0.13		0.16		0.16		0.15	
Total Observed	4425		4425		4425		4425	
Total Predicted	4336		4531		4489		4226	
SSR	51,116		39,300		34,855		39,211	
Observations	713		713		713		713	
CMF	0.798		0.889		0.814		0.843	

Table A-18. Functional Form of AADT.

Model #	AADT 1	AADT 2
Model 1	LN(Major AADT)	LN(Minor AADT)
Model 2	Major AADT	Minor AADT
Model 3	Total AADT	Missing Minor AADT
Model 4	LN(Total AADT)	Missing Minor AADT

Combined Model for Total Crashes

There were 713 site-years included in the combined analysis, representing 4425 total crashes. The combined model for total crashes is presented in Table A-19. It is necessary to make an adjustment for the repeated observations (i.e., same site over multiple years). To accomplish this, the model uses a clustered robust standard error, identifying individual sites as the cluster variable. The CMF for total crashes is 0.814 with an adjusted standard error of 0.124.

The standard error is calculated based on the procedure outlined in the first edition of the Highway Safety Manual. A method correction factor (MCF) is applied to the standard error, which is based on the strength of the study design. Relatively strong study designs are assigned a MCF that is close to 1.0. As the strength of the study decreases, the MCF increases. A MCF value of 2 was used to adjust the results from this analysis. This value is related to regression-based cross-sectional studies that use a conventional functional form and account for several important confounding factors.

Table A-19. Combined Model for Total Crashes.

```
Negative binomial regression                      Number of obs   =       713
Dispersion           = mean                       Wald chi2(7)    =    266.28
Log pseudolikelihood = -1730.7929                 Prob > chi2     =    0.0000

                                (Std. Err. adjusted for 83 clusters in siteid)
```

totalcrashes	Coef.	Robust Std. Err.	z	P>|z|	[95% Conf.	Interval]
totaadt	.0000405	3.92e-06	10.32	0.000	.0000328	.0000482
missingminor	-.0581259	.1434646	-0.41	0.685	-.3393114	.2230595
dswf	-.2057469	.1412018	-1.46	0.145	-.4824974	.0710036
stateva	-.6626921	.1673489	-3.96	0.000	-.99069	-.3346942
fourleg	-.0722631	.1449603	-0.50	0.618	-.3563801	.211854
rural	-.1681806	.2237763	-0.75	0.452	-.6067741	.2704129
speed55plus	.0809165	.1299138	0.62	0.533	-.1737098	.3355428
_cons	.9944558	.2877819	3.46	0.001	.4304138	1.558498
/lnalpha	-1.056702	.159524			-1.369363	-.7440407
alpha	.3476003	.0554506			.2542688	.4751899

Combined Models by Crash Type and Severity

The functional form of the combined model for total crashes is applied to individual crash types and severities. Specifically, separate models were developed for rear-end, angle, injury, and heavy vehicle crashes. These models were based on the combined dataset for Nevada and Virginia. The models are presented in Table A-20. At the bottom of the table, the CMF is presented for each crash type, including the adjusted standard error and related sample size.

Table A-20. Combined Model for Individual Crash Types.

	Rear-end	Angle	Injury	Heavy Vehicle
Covariate	**Coefficient (Robust SE)**	**Coefficient (Robust SE)**	**Coefficient (Robust SE)**	**Coefficient (Robust SE)**
Total AADT	-1.80E-02	-1.15E-01	-1.07E-01	-3.33E-01
	(1.78E-01)	(1.81E-01)	(1.67E-01)	(2.99E-01)
Missing Minor	-0.0180	-0.1148	-0.1067	-0.3333
	(0.1776)	(0.1808)	(0.1672)	(0.2991)
DSWF	-0.2326	-0.2946	-0.1984	-0.0450
	(0.1839)	(0.2143)	(0.1651)	(0.2403)
State (1 = Virginia)	-0.4972	-0.6643	-0.1296	-0.0024
	(0.1677)	(0.2945)	(0.2207)	(0.3215)
Approaches (1 = 4-legged)	-0.2833	0.2820	-0.0687	0.0910
	(0.1865)	(0.1947)	(0.1613)	(0.2149)
Area Type (1 = Rural)	-0.3928	0.0713	-0.4599	-0.1845
	(0.2705)	(0.3126)	(0.2707)	(0.3788)
Speed (1 = 55+ mph)	0.0646	-0.0060	0.2163	0.3403
	(0.1530)	(0.1965)	(0.1520)	(0.2469)
Constant	0.4406	-0.4453	-0.1651	-2.1864
	(0.3952)	(0.3029)	(0.3237)	(0.4637)
Alpha	0.4872	0.4996	0.3981	0.7827
	(0.1031)	(0.1030)	(0.0772)	(0.2546)
Log pseudo likelihood	-1423.2	-1020.2	-1178.7	-524.7
Wald chi2	268.14	81.90	150.28	113.69
Prob > chi2	0.0000	0.0000	0.0000	0.0000
CMF	0.792	0.745	0.820	0.956
Adjusted Standard Error	0.157	0.171	0.165	0.354
Sample Size (crashes)	2651	961	1450	267

MODEL VALIDATION

Several additional modeling techniques were employed to validate the results of the negative binomial regression analysis. The additional techniques included variations of the cross-sectional analysis (e.g., different functional forms and subsets of data) and a before-after with comparison group analysis. The results of the additional analyses are presented below.

Cross-Sectional Analysis

Variations of the cross-sectional analysis were used to verify the reasonableness and test the sensitivity of the results. The results from the combined model illustrated the sensitivity to model functional form. Various forms of AADT were used in the modeling framework and the CMFs from each model were relatively consistent. In this section, two additional variations of the cross-sectional analysis are presented, including:

1. Separate models for the subsets of signalized intersections with and without DSWF.
2. Models based on all sites, including both signalized and stop-controlled intersections.

Variation 1

This analysis includes only signalized intersections and two separate models are developed. The first model is for signalized intersections WITHOUT DSWF. The second model is for signalized intersections WITH DSWF. The analysis was conducted for Nevada and Virginia separately.

<u>Nevada Analysis (Variation 1)</u>

There were 143 site-years included in the model of signals without DSWF, representing 1529 total crashes. There were 118 site-years included in the model of signals with DSWF, representing 1695 total crashes. The model includes similar variables to the original Nevada model, minus the indicator for DSWF. Again, a clustered robust standard error is used to account for repeated observations.

Table A-21 presents the models for total crashes at signalized intersections with and without DSWF. Taking the “with divided by without” approach for developing CMFs, the two regression equations are applied to the dataset of signalized intersections. Specifically, the ratio of expected crashes with and without DSWF is computed for each site and the results are averaged over various groups to determine the overall CMF.

- Averaging over all signalized intersections, the CMF is 0.94.
- Averaging over all signalized intersections with DSWF, the CMF is 1.15.
- Averaging over all signalized intersections without DSWF, the CMF is 0.76.

These results are relatively consistent with the results from the original Nevada analysis.

Table A-21. Nevada Models for Total Crashes at Signalized Intersections With and Without DSWF.

Covariate	With DSWF Coefficient (Robust SE)	Without DSWF Coefficient (Robust SE)
Total AADT	3.26E-01 (1.58E-01)	-6.31E-01 (1.77E-01)

Missing Minor	0.3255 (0.1578)	-0.6305 (0.1774)
Constant	0.4082 (0.1878)	1.2424 (0.2486)
Alpha	0.2518 (0.0804)	0.3459 (0.1093)
Log pseudo likelihood	-364.7	-439.5
Wald chi2	232.52	36.07
Prob > chi2	0.0000	0.0000

Virginia Analysis (Variation 1)

There were 335 site-years included in the model of signals without DSWF, representing 892 total crashes. There were 117 site-years included in the model of signals with DSWF, representing 309 total crashes. The model includes similar variables to the original Virginia model, minus the indicator for DSWF. Again, a clustered robust standard error is used to account for repeated observations.

Table A-22 and presents the models for total crashes at signalized intersections with and without DSWF. Taking the "with divided by without" approach for developing CMFs, the two regression equations are applied to the dataset of signalized intersections. Specifically, the ratio of expected crashes with and without DSWF is computed for each site and the results are averaged over various groups to determine the overall CMF.

- Averaging over all signalized intersections, the CMF is 0.861.
- Averaging over all signalized intersections with DSWF, the CMF is 0.842.
- Averaging over all signalized intersections without DSWF, the CMF is 0.877.

These results are consistent with the results from the original Virginia analysis.

Table A-22. Virginia Models for Total Crashes at Signalized Intersections With and Without DSWF.

Covariate	With DSWF Coefficient (Robust SE)	Without DSWF Coefficient (Robust SE)
LN(Major AADT)	1.0242 (0.2574)	0.5195 (0.1791)
LN(Minor AADT)	-0.0316 (0.0123)	0.0665 (0.0332)
Speed (1 = 55+ mph)	0.2962 (0.2136)	0.1446 (0.2425)
Approaches (1 = 4-legged)	-0.0773 (0.2464)	-0.0530 (0.2478)
Constant	-9.0338 (2.5982)	-4.4962 (1.7513)
Alpha	0.0745 (0.0672)	0.4194 (0.1008)
Log pseudo likelihood	-217.5	-686.9
Wald chi2	46.84	12.85
Prob > chi2	0.0000	0.0120

Variation 2

This analysis includes all years of data for all sites, including the years when sites were unsignalized. The coefficients illustrate the relative effects of a traffic signal versus DSWF. The purpose of this model is to check that the coefficients are intuitive. Several models were developed, including models for total crashes and individual crash types and severity.

This set of models was only developed for Virginia because the Nevada dataset does not include any site-years when sites are stop-controlled. There were 572 site-years included in the analysis, representing 1493 total crashes. The model includes similar variables to the original Virginia model, but the individual terms for AADT are replaced by a single term for total AADT and an additional indicator variable was included to identify whether the intersection is signalized or stop-controlled. Again, the models used a clustered robust standard error to adjust for repeated measures (i.e., same sites over multiple years).

Table A-23 summarizes the results of the analysis and Table A-24 presents the results for total crashes and individual crash types. The model for total crashes indicates a reduction in crashes based on the presence of a signal and DSWF, although the effects are insignificant. The results for individual crash types are also intuitive. The models indicate that a signal is expected to increase rear-end crashes, reduce angle crashes, and reduce injury crashes. The DSWF is expected to reduce all three crash types (rear-end, angle, and injury). Many of the effects are significant, but become insignificant when the adjustment is made to account for repeated measures. The sample size is too small to estimate a model for heavy vehicle crashes.

Table A-23. Summary of Results for Virginia Models (Variation 2).

Model	Effect of Signal	Effect of DSWF	Sample Size
Total	Insignificant Decrease	Insignificant Decrease	1493 crashes
Rear-end	Significant Increase	Insignificant Decrease	738 crashes

Angle	Significant Decrease	Insignificant Decrease	516 crashes
Injury	Insignificant Decrease	Insignificant Decrease	615 crashes

Table A-24. Virginia Models for Crashes at Signalized and Stop-Controlled Intersections.

	Total	**Rear-end**	**Angle**	**Injury**
Covariate	**Coefficient (Robust SE)**	**Coefficient (Robust SE)**	**Coefficient (Robust SE)**	**Coefficient (Robust SE)**
LN(Total AADT)	-1.49E-01 (2.31E-01)	7.53E-01 (3.16E-01)	0.3765 (0.2815)	0.6023 (0.1976)
Traffic Control (1 = Signal)	-0.1487 (0.2314)	0.7526 (0.3157)	-0.8535 (0.2740)	-0.1487 (0.2314)
DSWF (1 = DSWF)	-0.1317 (0.1818)	-0.2772 (0.2042)	-0.0154 (0.2834)	-0.1317 (0.1818)
Speed (1 = 55+ mph)	0.1602 (0.1704)	0.0684 (0.2019)	0.2351 (0.2277)	0.1602 (0.1704)
Approaches (1 = 4-legged)	-0.1322 (0.1719)	-0.4812 (0.2140)	0.2768 (0.2240)	-0.1322 (0.1719)
Constant	-4.8010 (1.9431)	-10.7240 (2.3791)	-3.4327 (2.7490)	-4.8010 (1.9431)
Alpha	0.4559 (0.0876)	0.4323 (0.1148)	0.0590 (0.2002)	-0.7855 (0.1922)
Log pseudo likelihood	-1177.8	-829.1	-730.9	-1177.9
Wald chi2	11.74	26.78	14.75	11.74
Prob > chi2	0.0385	0.0001	0.0115	0.0385

Before-After Analysis with Comparison Group

A before-after with comparison group method was employed as a separate analysis method to estimate the safety effects of DSWF. Ideally, the installation of DSWF would be at some point after the signal was installed and the crash experience before and after the DSWF installation would be compared. However, it was identified during the data collection process that many of the DSWF in Nevada and Virginia were installed at the same time as the signal. Comparing the before-after crash experience for this type of treatment would provide an estimate of the dual effect of signals and DSWF. Rather than estimating the effect of DSWF directly, it is necessary to estimate two separate effects 1) the effect of installing a signal only at stop-controlled intersections, and 2) the effect of installing a signal and DSWF simultaneously at stop-controlled intersections. The results from these two analyses can be compared to glean an approximation of the effect of DSWF.

Dataset

The Virginia data provided an opportunity to conduct this type of comparative before-after analysis. There were 7 sites included in the first treatment (i.e., install signal only at stop-controlled intersection). The signals were installed during various years throughout the study period. Crash and traffic data were obtained for each year from 1998 through 2008 and assigned to the before and after period for each site. The year in which the signal was installed was removed from the dataset for each site. There were a total

of 38 site-years in the before period, representing 90 crashes. There were 32 site-years in the after period, representing 79 crashes. The average major and minor road traffic volumes in the before period were 17,132 and 2,216 vehicles per day, respectively. The average major and minor road traffic volumes in the after period were 18,754 and 2,645 vehicles per day, respectively.

There were 8 sites included in the second treatment (i.e., install signal and DSWF at stop-controlled intersection). The combination treatment (signal+DSWF) was installed at the sites during various years throughout the study period. Crash and traffic data were obtained for each year from 1998 through 2008 and assigned to the before and after period appropriately. The year in which the combination treatment was installed was removed from the dataset for each site. There were a total of 29 site-years in the before period, representing 69 crashes. There were 51 site-years in the after period, representing 111 crashes. The average major and minor road traffic volumes in the before period were 14,534 and 1,967 vehicles per day, respectively. The average major and minor road traffic volumes in the after period were 15,371 and 1,849 vehicles per day, respectively.

The comparison group included 23 signalized intersections in Virginia from the same regions as the treatment sites. The comparison sites are in close proximity to treatment sites to account for factors such as weather, crash reporting practices, and demographics from one year to the next. Crash and traffic volume data were obtained for 1998 to 2008 for each of the comparison sites. The comparison sites were signalized throughout the entire study period and no major improvements were implemented during that time.

Overall, the characteristics of the sites with signal only and signal plus DSWF are quite similar. All sites are located in rural areas with a mix of 3-legged and 4-legged intersections. The range of major and minor road AADT is similar among the sites, but the average major and minor road AADT is slightly greater for the sites with signal only. The speed limit is 45 mph for the majority of the sites, but a few of the sites with signal plus DSWF are posted at 55 or 60 mph. Nearly all sites have two lanes on the major road and all have left-turn lanes from the major road. The number of total lanes varied among the sites (i.e., total left, through, and right lanes on all approaches), but the range was similar for the sites with signal only and the sites with signal plus DSWF.

Analysis

The analysis method presented by Hauer was employed for the before-after with comparison group evaluation (Hauer, 1997) as described on pages A-6 to A-10 of this report. A specific comparison group was identified for each specific treatment site due to the fact that the treatments were installed in different years. The crashes in the before period were adjusted for each comparison group to account for the changes in traffic volume *at the comparison sites* from the before to the after period.

Once a suitable (and adjusted) comparison group was identified for each treatment site, values of ω and Var(ω) were computed for each site. The expected crashes in the after period (π) was calculated based on the observed crashes in the before period, the ratio of before and after crashes at the comparison sites, and an adjustment for the change in traffic volume from the before to the after period *at the treatment sites.* The analysis was undertaken separately for the two treatments as described previously.

Results

Table A-25 compares the results for treatment 1 (signal only) and treatment 2 (signal + DSWF) related to total crashes. The CMF for “signal only” is 0.473 (a 53 percent reduction in crashes). The CMF for

"signal plus DSWF" is 0.266 (a 73 percent reduction in crashes). From these results, it appears that DSWF provides an additional safety improvement at signalized intersections.

Table A-25. Comparison of Total Crash CMF for Signal Only and Signal + DSWF.

	Treatment 1 (Signal Only)	Treatment 2 (Signal + DSWF)	Effect of DSWF
CMF	0.359	0.293	0.816
Standard Error	0.28	0.21	

The next questions are:

1. How much of an improvement does DSWF provide in addition to the signal?
2. Is it more appropriate to subtract the CRFs or divide the CMFs to estimate the individual effect of the DSWF?

Subtracting the CRFs provides an estimate of the effect of DSWF on the original number of crashes (without signal or DSWF). However, the DSWF would not be implemented without a signal already in place or at least at the same time. As such, the effect of the DSWF should be presented to indicate the additional effect after signal installation. This is accomplished by dividing the CMF for the combination treatment (i.e., signal + DSWF) by the CMF for signal only. The rationale for dividing the CMFs is also based on the procedure for combining multiple CMFs as outlined in the draft Highway Safety Manual. Assuming that the individual treatments are independent, the CMFs from multiple treatments can be multiplied to estimate the aggregate effect. For example, the combined effect of a signal plus the DSWF would be calculated by Equation 23.

$$\text{CMF(signal+DSWF)} = \text{CMF(signal)} * \text{CMF(DSWF)} \tag{23}$$

If Equation 23 holds, the CMF for DSWF is calculated by Equation 24.

$$\text{CMF(DSWF)} = \text{CMF(signal+DSWF)} / \text{CMF(signal)} \tag{24}$$

Using Equation 24 and the results of the comparison group analysis, the CMF for DSWF is 0.293/0.359 = 0.816 and the corresponding CRF is 100*(1-0.816) = 18.4. These results are consistent with the effect estimated from the cross-sectional analysis (CMF = 0.814). However, based on the larger sample size, the CMF from the cross-sectional analysis is associated with a smaller standard error.

The results of the before-after with comparison group corroborate the results from the cross-sectional analysis for total crashes. A similar analysis and comparison is undertaken for rear-end crashes. Table A-26 compares the results for treatment 1 (signal only) and treatment 2 (signal + DSWF) related to rear-end crashes. The CMF for "signal only" is 0.224 (a 77.6 percent reduction in crashes). The CMF for "signal plus DSWF" is 0.139 (an 86.1 percent reduction in crashes). Using Equation 19 and the results of the comparison group analysis, the CMF for DSWF related to rear-end crashes is 0.139/0.224 = 0.621 and the corresponding CRF is 100*(1-0.621) = 37.9. These results are relatively consistent with the effect estimated from the cross-sectional analysis (CMF = 0.792).

Table A-26. Comparison of Rear-end Crash CMF for Signal Only and Signal + DSWF.

	Treatment 1 (Signal Only)	Treatment 2 (Signal + DSWF)	Effect of DSWF
CMF	0.224	0.139	0.621
Standard Error	0.21	0.13	

It should be noted that the results for rear-end crashes are inconsistent with previous research and the results from the cross-sectional validation. Several previous studies have found that the installation of a signal is expected to increase rear-end crashes (Harkey et al., 2008). It is likely that the results of individual crash types from the before-after study are based on too small a sample to be reliable. This is also evidenced by the relatively large standard error associated with the results for rear-end crashes. This provides further support for the use of a cross-sectional study design for this evaluation.

Before-After Analysis with Empirical Bayes

Unlike with the treatment sites in Nevada and Virginia, the treatment sites in North Carolina were all signalized prior to the installation of DSWF. Therefore, the study team was able to utilize the EB method with the North Carolina data to estimate CMFs for total, rear-end, angle, and injury crashes.

Dataset

The treatment group was comprised of 14 four-legged, predominately rural signalized intersections. The reference group was comprised of 63 four-legged, signalized intersections in urban and rural areas in North Carolina.

Analysis

As outlined in the *Methodology* section, a crucial component of the EB method was the generation of suitable SPFs. To this end, the study team sought to develop a separate SPF for total, injury, rear-end, and angle crashes assuming a Negative Binomial error structure using the GENMOD procedure in SAS. To develop the four SPFs, a two-stage process was adopted. In the first stage, a preliminary SPF model, containing only traffic volume parameters, was generated and assessed. The following preliminary SPF model forms were evaluated:

1. Separate terms for major and minor AADT.
2. Separate terms for the natural log of major and minor AADT.
3. Total entering AADT.
4. Natural log of total entering AADT.

To help judge between the preliminary SPF models, the following goodness-of-fit (GOF) measures were used:

1. Mean Absolute Deviance (MAD).
2. Mean Squared Prediction Error (MSPE).
3. Freeman-Tukey R^2 (R_{FT}^2).
4. Log Likelihood.

In addition to comparing the GOF measures shown above, a cumulative residual (CURE) plot was created for each model based on the procedure developed by Hauer and Bamfo (1997) in order to assist with the

selection process. Ideally, the CURE plot should show cumulative residuals that oscillate about zero and are relatively close to zero. Cumulative residuals that fall outside two standard deviations may raise a concern.

Once the preliminary model form was selected, the second stage in the SPF development process was to add other parameters into the models which were statistically significant and or beneficial to model fit. Occasionally, a parameter was found to be statistically significant, but had a counterintuitive value (e.g., the provision of an exclusive right-turn lane increasing crash frequency). In such cases, the parameter was not retained in the model.

As noted in the *Data Collection* section, the reference and treatment groups included both rural and urban sites. To ensure that the SPFs, which were generated from a mix of rural and urban reference sites, would be applicable to the predominately rural treatment sites, area type was added as a parameter to every model irrespective of statistical significance. Afterward, other variables were considered including the following:

- Presence of left-turn lane(s) on major road.
- Presence of right-turn lane(s) on major road.
- Divided vs. undivided median type.
- Ratio of minor road AADT to total entering volume.
- Interaction between area type and traffic volume (major, minor, or total).

Based on this procedure, the following SPF model was selected for total crashes:

$$k_{Tot} = e^{0.9496+0.4508(Totvolscld)-0.0612(Areatype)} \tag{25}$$

Where,
k_{Tot} = total crashes per year.
Totvolscld = total entering volume ($AADT_{Total}$) scaled = $\frac{AADT_{Total}}{10{,}000}$.
Areatype = area type (1 if urban, 0 if rural).

Table A-27 displays the SAS output from the GENMOD procedure for the total crash SPF model. The regression output indicates that crash frequency increases as total entering volume increases and sites in urban areas tend to have a lower crash frequency than sites in rural areas. The CURE plot for this model is presented in Figure A-11. The plot shows that the cumulative residuals oscillate about zero, and are generally within two standard deviations. However, the plot suggests that the model underestimates total crash frequency for total entering volumes between 0 and 10,000 vehicles per day and 20,000 and 50,000 vehicles per day. The plot also suggests that the model overestimates crash frequency for total entering volumes between 10,000 and 20,000 vehicles per day.

Table A-27. Total Crash SPF Model for EB Analysis Based on North Carolina Data.

The GENMOD Procedure

Analysis Of Initial Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	0.9496	0.0659	0.8204	1.0788	207.55	<.0001
totvolscld	1	0.4508	0.0277	0.3965	0.5050	265.39	<.0001
areatype	1	-0.0612	0.0654	-0.1894	0.0670	0.88	0.3495
Dispersion	1	0.4474	0.0293	0.3936	0.5086		

NOTE: The negative binomial dispersion parameter was estimated by maximum likelihood.

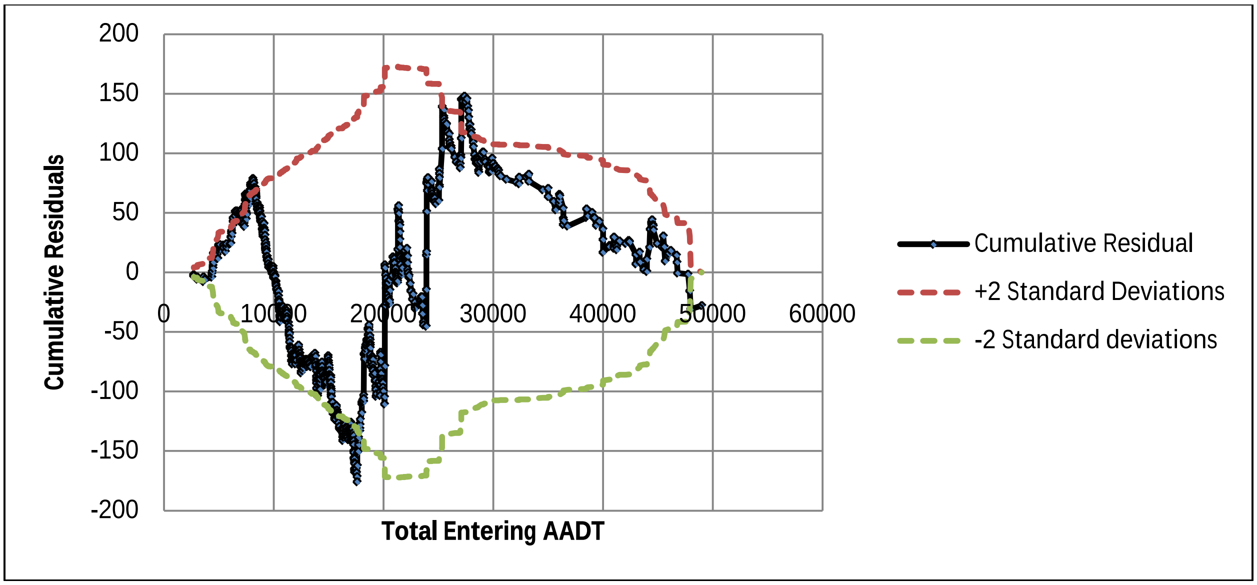

Figure A-11. CURE Plot for Total Crash SPF Model Based on North Carolina Data.

The following SPF model was selected for injury crashes:

$$k_{FI} = e^{0.2188+0.3950(majaadtscld)+0.2409(minaadtscld)-0.1255(areatype)} \tag{26}$$

Where,

k_{FI} = injury crashes per year.
majaadtscld = major road AADT ($AADT_{Maj}$) scaled = $AADT_{Maj}/10000$
minaadtscld = minor road AADT ($AADT_{Min}$) scaled = $AADT_{Min}/10000$.
Areatype = area type (1 if urban, 0 if rural).

Table A-28 presents the GENMOND output for the injury crash model. Once again, urban sites are shown to have a lower crash frequency when compared to rural sites. Figure A-12 shows the CURE plot for this model. The cumulative residuals oscillate about zero and remain within two standard deviations. The plot suggests that the model underestimates injury crash frequency for major AADTs between 0 and 5,000

vehicles per day, and between 18,000 and 40,000 vehicles per day. The plot also suggests that the model overestimates injury crash frequency for major AADTs between 5,000 and 18,000 vehicles per day.

Table A-28. Injury Crash SPF Model for EB Analysis Based on North Carolina Data.

The GENMOD Procedure

Analysis Of Initial Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	0.2188	0.0785	0.0650	0.3726	7.78	0.0053
majaadtscld	1	0.3950	0.0458	0.3054	0.4847	74.54	<.0001
minaadtscld	1	0.2409	0.0799	0.0843	0.3975	9.09	0.0026
areatype	1	-0.1255	0.0775	-0.2775	0.0265	2.62	0.1056
Dispersion	1	0.4287	0.0433	0.3516	0.5226		

NOTE: The negative binomial dispersion parameter was estimated by maximum likelihood.

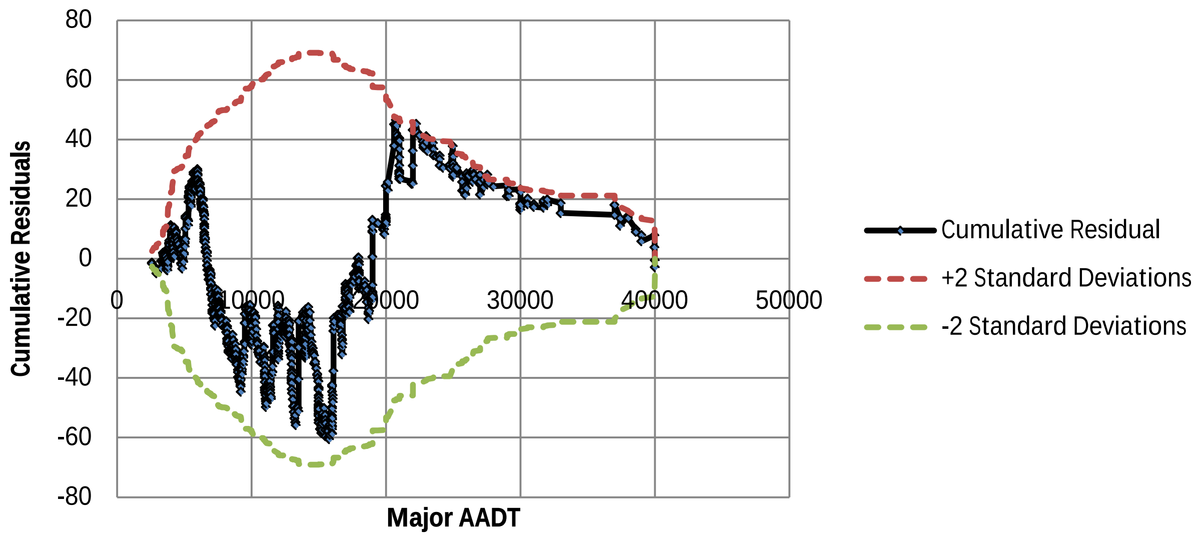

Figure A-12. CURE Plot for Injury Crash SPF Model Based on North Carolina Data.

The following SPF model was selected for rear-end crashes:

$$k_{RE} = e^{-0.5690+0.5089(totvolscld)-0.1079(areatype)-0.9799(minorratio)} \qquad (27)$$

Where,
k_{RE} = rear-end crashes per year.
totvolscld = total entering volume ($AADT_{Total}$) scaled = $AADT_{Total}/10000$
areatype = area type (1 if urban, 0 if rural).
minorratio = ratio of minor road AADT to total entering AADT.

Table A-29 presents the GENMOD output for the rear-end crash model. Once again, urban sites are shown to have a lower crash frequency relative to rural sites. The regression output also indicates that

crash frequency decreases as the ratio of minor road to total entering AADT increases. There are multiple possible explanations for this outcome. For example, a driver may be more alert and better prepared to stop when approaching an intersection with a large cross-street as opposed to a small side road. Such intersections are more likely to have other features or devices that can serve as cues for approaching drivers. Figure A-13 provides the CURE plot for this model. The cumulative residuals oscillate about zero and remain within two standard deviations. The plot suggests that the model underestimates rear-end crash frequency for total entering volumes between 0 and 10,000 vehicles per day, and between 18,000 and 35,000 vehicles per day. The plot also suggests that the model overestimates rear-end crash frequency for total entering volumes between 10,000 and 18,000 vehicles per day, and between 35,000 and 50,000 vehicles per day.

Table A-29. Rear-end Crash Model for EB Analysis Based on North Carolina Data.

The GENMOD Procedure

Analysis Of Initial Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	-0.5690	0.1139	-0.7923	-0.3457	24.94	<.0001
totvolscld	1	0.5089	0.0395	0.4314	0.5864	165.60	<.0001
areatype	1	-0.1079	0.1027	-0.3091	0.0934	1.10	0.2934
minorratio	1	-0.9799	0.2864	-1.5411	-0.4186	11.71	0.0006
Dispersion	1	0.5276	0.0763	0.3974	0.7004		

NOTE: The negative binomial dispersion parameter was estimated by maximum likelihood.

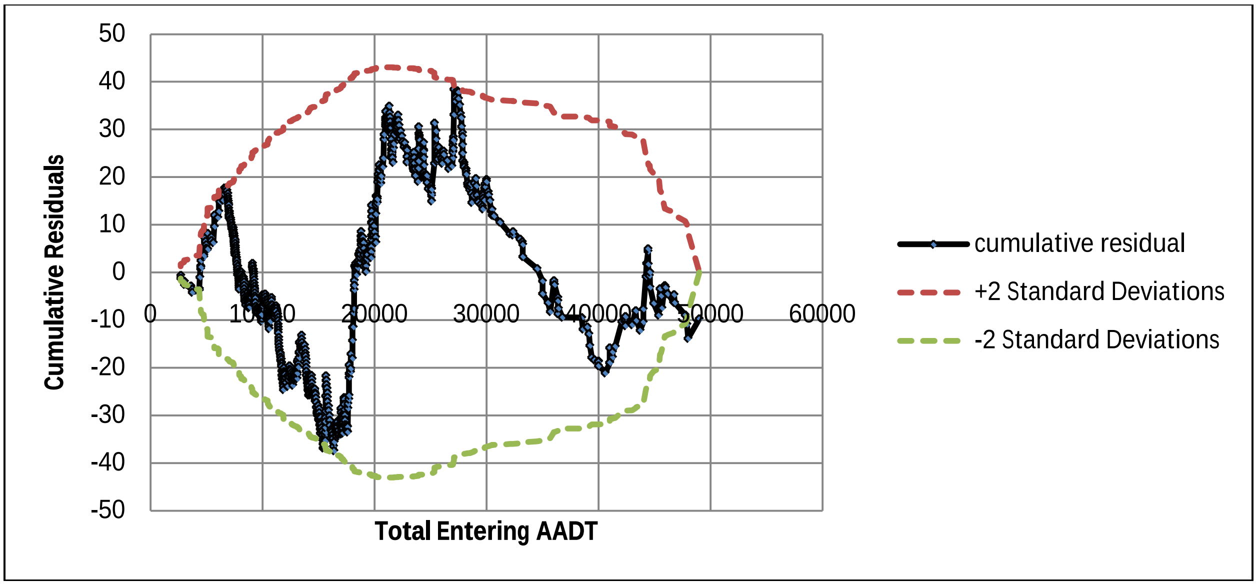

Figure A-13. CURE Plot for Rear-end Crash Model Based on North Carolina Data.

Lastly, the following SPF model was selected for angle crashes:

$$k_{Angle} = e^{-4.2176+0.3774(lnmajor)+0.1910(lnminor)-0.2257(areatype)-0.5689(left\_maj)} \quad (28)$$

Where,
k_{Angle} = angle crashes per year.
lnmajor = natural log of major road AADT.
lnminor= natural log of minor road AADT.
areatype = area type (1 if urban, 0 if rural).
left_maj = presence of left-turn lane(s) on major road (1 if yes, 0 if no).

Table A-30 presents the GENMOD output for the angle crash model. As with the other models, urban sites are shown to have a lower crash frequency relative to rural sites. The regression output also indicates that crash frequency decreases when one or more left-turn lanes are provided on the major road. Figure A-14 provides the CURE plot for this model. The cumulative residuals oscillate about zero and generally remain within two standard deviations. The plot suggests that the model underestimates angle crash frequency for major AADTs between 0 and 8,000 vehicles per day, and between 22,000 and 27,000 vehicles per day. The plot also suggests that the model overestimates angle crash frequency for major AADTs between 8,000 and 22,000 vehicles per day, and between 27,000 and 40,000 vehicles per day.

Table A-30. Angle Crash Model for EB Analysis Based on North Carolina Data.

The GENMOD Procedure

Analysis Of Initial Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square	Pr > ChiSq
Intercept	1	-4.2176	0.9034	-5.9881	-2.4471	21.80	<.0001
lnmajor	1	0.3774	0.1028	0.1760	0.5788	13.49	0.0002
lnminor	1	0.1910	0.0511	0.0908	0.2911	13.97	0.0002
areatype	1	-0.2257	0.1262	-0.4730	0.0216	3.20	0.0736
left_maj	1	-0.5689	0.1449	-0.8530	-0.2848	15.41	<.0001
Dispersion	1	1.2724	0.1132	1.0689	1.5147		

NOTE: The negative binomial dispersion parameter was estimated by maximum likelihood.

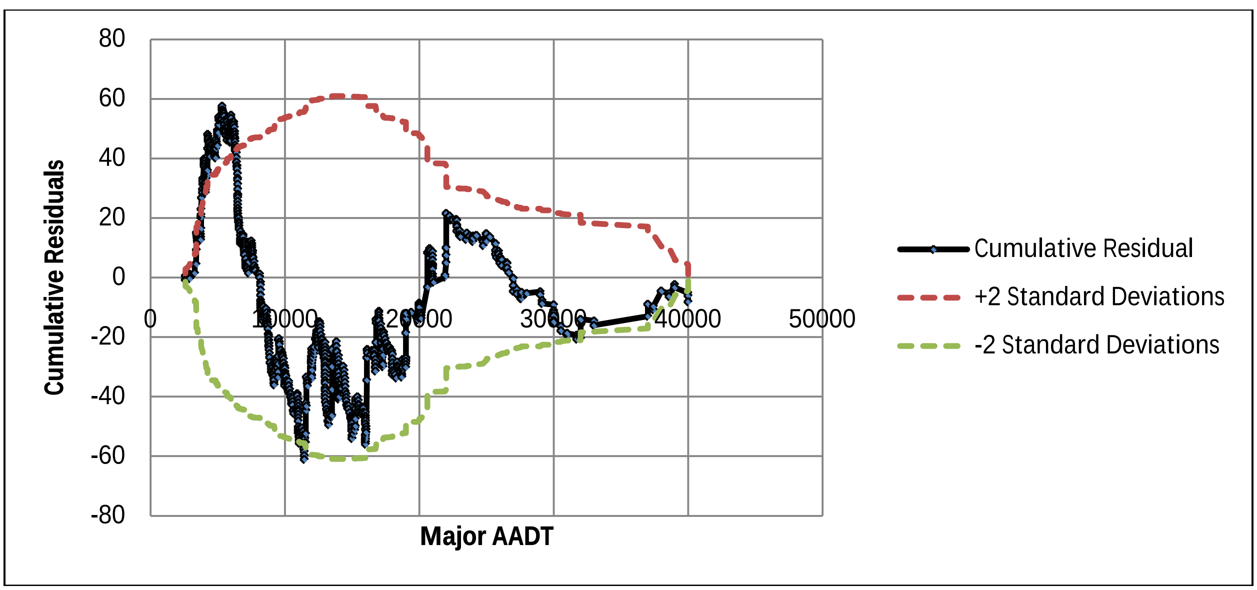

Figure A-14. CURE Plot for Angle Crash Model Based on North Carolina Data.

Table A-31 provides a summary of the goodness-of-fit statistics for the four selected models. With respect to mean absolute deviance and mean squared prediction error, the rear-end model is the best performing while the total crash model is the worst performing. Based on the Freeman-Tukey R^2 statistics, the total crash model performs best while the angle crash model performs the worst.

Table A-31. Goodness of Fit Statistics for SPF Models Selected for the EB Analysis.

	MAD	MSPE	R_{FT}^2
Total Crash Model	3.3115	29.6392	0.2275
Injury Crash Model	1.5332	4.7510	0.1099
Rear-end Crash Model	1.0126	1.8433	0.1398
Angle Crash Model	1.2894	3.6893	-0.0141*

*A negative value is possible using the Freeman-Tukey R^2 statistic.

Results

Table A-32 to A-34 summarize the results of the EB before-after analysis of DSWF. They present the CMFs for total, rear-end, angle, and injury crashes for three scenarios. Table A-32 shows the results for scenario 1, which is based on all 14 treatment sites including sites with less than one year of after data and sites with fully protected left-turn phasing. Table A-33 presents the results for scenario 2, which excludes the three treatment sites with less than one year of after data. Table A-34 presents the results for scenario 3, which excludes sites with less than one year of after data and those with fully protected left-turn phasing. The motivation for scenario 3 was discussed in the *Data Collection* section; the reference group from which the SPFs were derived for the EB analysis did not contain any sites with full left-turn protection while the treatment group did. Scenario 3 attempts to address this issue by excluding treatment sites with full left-turn protection from the EB analysis. Scenario 3 also excludes sites with less than one year of after data. Thus, scenario 3 represents the most restrictive scenario by screening out any questionable treatment sites. Consequently, it has the smallest sample size and largest standard errors.

Table A-32. EB Scenario 1 – CMFs for Installation of DSWF.
(All Sites)

	Total Crashes	**Rear-end**	**Angle**	**Injury**
CMF	0.9820	1.3883	0.5675	0.8869
Standard Error	0.0798	0.1329	0.0655	0.0697

Sites = 14

Table A-33. EB Scenario 2 – CMFs for Installation of DSWF.
(Excluding Sites with Less than 1 Year of After Data)

	Total Crashes	**Rear-end**	**Angle**	**Injury**
CMF	0.964	1.3684	0.5534	0.8494
Standard Error	0.0853	0.1394	0.0703	0.0726

Sites = 11

Table A-34. EB Scenario 3 – CMFs for Installation of DSWF.
(Excluding Sites with Less than 1 Year of After Data and Sites with Full Left-turn Protection)

	Total Crashes	Rear-end	Angle	Injury
CMF	0.6196	1.0360	0.4950	0.6361
Standard Error	0.0928	0.2091	0.0912	0.0993

Sites = 6

The results from all three scenarios suggest that the installation of DSWF at signalized intersections reduces total, injury, and angle crashes while increasing rear-end crashes. However, the statistical significance of the results varies depending on the scenario. Under the more inclusive scenarios (i.e. scenarios 1 and 2), the CMFs for total and injury crashes are not statistically significant at the five percent significance level while the CMFs for rear-end and angle crashes are statistically significant at the five percent level. In contrast, under the most restrictive scenario (i.e. scenario 3), the CMFs for total, angle, and injury crashes are statistically significant at the 5 percent level, while the CMF for rear-end crashes is not significant at the 5 percent level.

During the EB analysis, the study team made two observations which are important to consider when examining these results. First, one out of the fourteen treatment sites, site 2, had a dominant and perhaps inordinate level of influence on the overall results. When this site is excluded from the analysis, the CMFs for total and injury crashes are greater than 1.0, and the CMF for angle crashes is nearly 1.0. Site 2 also had the longest after period of any treatment site (15.18 years). Such a lengthy after period increases the chances that changes occurred at the site, which could influence the results. The second observation was that there were relatively large fluctuations in the results from site to site. While one site would indicate a relatively large reduction in crashes after the DSWF installation, another site would indicate a large increase in crashes. Such drastic variations were seen in virtually every crash type and scenario. Such results are not unprecedented. A study of DSWF installations in British Columbia found “the crash frequencies at each intersection individually indicated a wide range of results, from a 44 percent reduction to a 66 percent increase in accidents” (Sayed et al., 1999).

While these observations may reduce the confidence in the CMFs derived from the EB analysis, the results generally support the CMFs derived from the cross-sectional analysis. Normally, the results of an EB before-after analysis are given greater weight than those based on cross-sectional modeling. However, in light of the concerns with the North Carolina data, the study team placed greater weight on the results of the cross-sectional analysis. The one discrepancy between the cross-sectional models and the EB analysis is the CMF for rear-end crashes. The cross-sectional model indicates a reduction in crashes while the EB analysis shows an increase.

DISCUSSION AND CONCLUSIONS

The data collected and analyzed for this study clearly show a safety benefit for providing dynamic signal warning flashers (DSWF) at signalized intersections. The negative binomial regression models for Nevada, Virginia, and the two states combined show a consistent reduction in total crashes. It was also shown that DSWF may help to reduce rear-end, angle, injury, and heavy vehicle crashes, although the sample size was limited for many of the individual crash types. The results from the combined cross-sectional model are used to develop the suggested CMFs for DSWF. The CMFs along with the ideal standard errors and adjusted standard errors are presented in Table A-35. The adjusted standard errors are 2.0 times the ideal standard errors based on the procedures in the HSM for these types of cross sectional studies.

Table A-35. Summary of Suggested CMFs for DSWF.

	Total Crashes	Rear-end	Angle	Injury	Heavy Vehicle
CMF	0. 814	0.792	0. 745	0. 820	0. 956
S.E. of CMF (ideal)	0. 062	0.079	0.086	0. 083	0. 177
S.E. of CMF (adjusted)	0. 124	0.157	0.171	0. 165	0. 354

Several potential confounding factors were included in the analysis, including traffic volume, area type, number of approaches, and speed limit. Traffic volume is a strong predictor of crash frequency and may also influence the effect of DSWF. Interaction terms were explored during the cross-sectional analyses to further investigate the relationship between traffic volume and the effect of DSWF. While the interaction terms were not significant in the final combined model, the results from the Virginia analysis showed that DSWF may be more effective at lower traffic volumes. It is difficult to speculate why DSWF may be less effective as traffic volume increases, but the speed-flow-density relationship offers a potential explanation. As traffic flow increases, there is greater interaction among vehicles and vehicle speeds tend to decrease. If drivers are traveling at lower speeds, the DSWF would not provide as great a benefit.

The current state-of-the-art for developing CMFs is the empirical Bayes (EB) before-after method. However, employing the EB method to estimate the safety effect of DSWF using the Nevada and Virginia data was not possible because many of the DSWF treatments were installed at the same time as the traffic signal. Instead, a cross-sectional analysis, with negative binomial regression, was used to develop relationships between crashes and DSWF. To provide support for the cross-sectional models, several additional analysis techniques were employed. The additional techniques included variations of the cross-sectional analysis (e.g., different functional forms and subsets of data), a before-after with comparison group analysis, and an empirical Bayes before-after analysis. The results of the additional analyses are relatively consistent with the cross-sectional analysis and corroborate the results. The one discrepancy is for rear-end crashes; while the cross-sectional models indicate a reduction in rear-end crashes, the EB analysis indicates the potential for an increase in rear-end crashes.

An economic analysis were not been conducted in this effort, but could be conducted as part of future research. DSWF is identified in NCHRP 500 Series Report, *Volume 12: A Guide for Reducing Collisions at Signalized Intersections*, as a strategy to improve driver awareness of intersections and signal control (Antonucci et al., 2004). According to Objective 17.2 D, the costs will be low for most procedures to install or upgrade signs and signals to improve visibility and awareness of the traffic control devices. However, ongoing maintenance costs can add to the annual cost of this treatment. Based on the relatively

large effect of DSWF, particularly on fatal and injury crashes, this strategy is likely to be a cost-effective measure for reducing crashes and related severities at signalized intersections.

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NCHRP PROJECT 17-35

APPENDIX B:

Safety Effects of Converting Signalized Intersections to Roundabouts

INTRODUCTION

Intersections account for a small portion of the total highway system; however, in 2008, approximately 2.31 million intersection-related crashes occurred. Intersection crashes accounted for 40 percent of all reported crashes and 22 percent (7,421) of all fatal crashes (NHTSA, 2008). The disproportionately high percentage of intersection crashes is not surprising because intersections present more points of conflict than non-intersection locations. Crashes at signalized intersections represent about 51 percent (1.18 million) of all intersection-related crashes, of which 2,511 involved a fatality in 2008 (NHTSA, 2008).

The National Cooperative Highway Research Program (NCHRP) 500 Series Report, Volume 12, identifies safety issues related to signalized intersections and potential countermeasures to address the safety issues (Antonucci et al., 2004). Specifically, the report identifies geometric improvements as one method to reduce the frequency and severity of intersection conflicts. Objective 17.2 B identifies several strategies to reduce the frequency and severity of intersection conflicts through geometric improvements. A specific strategy listed in this section is “construct special solutions”, which includes conversion of signalized intersections to roundabouts.

This study investigates the safety effects of converting signalized intersections to roundabouts. Roundabouts have the potential to reduce both the frequency and severity of crashes compared to a similar signalized intersection. Regarding crash frequency, roundabouts have fewer potential conflict points than a signalized intersection. A signalized intersection of two two-lane roads has 32 potential vehicle-vehicle conflict points. A similar roundabout has just 8 potential conflict points as shown in Figure B-1. The types of crashes are also fundamentally different for roundabouts, which has the potential to reduce crash severity by eliminating typically serious severity crashes. Specifically, crashes related to crossing path and left-turn movements do not exist in a roundabout. The geometric design of a roundabout also encourages reduced speeds, which reduces the likelihood of injury if a crash occurs.

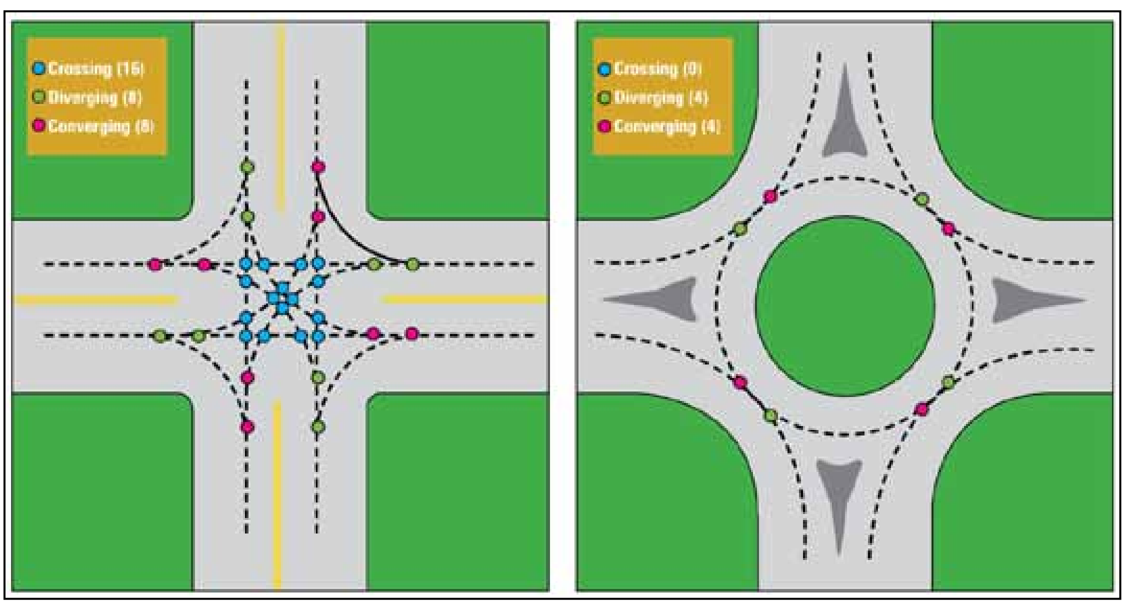

Figure B-1. Illustration of Conflict Points for a Signalized Intersection and Roundabout

The safety effects of converting signalized intersections to roundabouts are not well documented for conversions in the United States. The following provides an overview of the few studies that do exist both in the United States and internationally.

A comparison of predicted crash rates for conventional intersections in the United States and roundabouts in the United Kingdom revealed safety benefits for roundabouts at lower traffic volumes (less than 50,000 entering vehicles per day). For total entering volumes of 20,000 vehicles per day, the crash rate was 33 percent lower for roundabouts than for signalized intersections in urban/suburban areas and 56 percent lower in rural areas. For total entering volumes of 40,000 vehicles per day, the crash rate was 15 percent lower for roundabouts than for signalized intersections. The safety performance of roundabouts and signalized intersections is relatively comparable at higher volumes (Institute of Transportation Engineers, 1999).

A study of roundabouts in Maryland contained before-after comparisons of roundabout conversions that occurred between 1993 and 2000 (Cunningham, 2007). The study was comprised of two parts, 1) a before-after comparison of locations where a roundabout replaced a stop sign or intersection control beacon (15 single-lane roundabouts and three two-lane roundabouts), and 2) a more detailed analysis of the 15 single-lane roundabout conversions. For the single-lane roundabout locations, the total crash rate decreased from 1.36 crashes per million entering vehicles (MEV) in the before period to 0.27 crashes per MEV in the after period. The injury crash rate fell from 0.79 crashes per MEV to 0.09 crashes per MEV. According to the Morin Lower control limit test, these reductions in total and injury crash rates were statistically significant at a 95 percent confidence interval. For the two-lane roundabout conversions, there was a general reduction in both total and injury crashes, although one location had a 147 percent increase in total crash frequency. The more detailed analysis of the 15 single-lane roundabout conversions was based on crash rates in which one year of crash data, corresponding to the construction period, had been removed. For these sites, the total crash rate decreased by 60 percent, the fatal crash rate decreased by 100 percent, the injury crash rate decreased by 82 percent, and the property-damage-only (PDO) crash rate decreased by 27 percent. As for crash rates by crash type, right-angle, rear-end, opposite-direction, sideswipe, left-turn, nighttime, and wet surface crashes decreased by 91 percent, 11 percent, 100 percent, 75 percent, 95 percent, 5 percent, and 30 percent, respectively. In contrast, the fixed-object crash rate increased by 724 percent. Assuming a Poisson distribution, the reductions in total, fatal, injury, PDO, angle, opposite-direction, left-turn, and wet surface crash rates were statistically significant at a 95 percent confidence interval.

A before-after study of roundabout conversions in the United States employed the empirical Bayes (EB) methodology to control for regression-to-the-mean and other trends in crash occurrence (Persaud et al., 2001). The analyses used data from seven states—Colorado, Florida, Kansas, Maine, Maryland, South Carolina, and Vermont—where a total of 23 intersections were converted to modern roundabouts between 1992 and 1997. Of the 23 intersections studied, 19 were previously controlled by stop signs, and four were controlled by traffic signals. For the previously signalized intersections, the EB procedure estimated a 35 percent reduction for all crash severities combined and a 74 percent reduction for injury crashes. Three of these roundabouts had multilane circulation designs. A later study (Rodegerdts et al., 2007) applying the same methodology and using these same four sites with an additional five converted sites found a 48 percent reduction for all crash severities combined and a 78 percent reduction for injury crashes. When broken down by area type, the four suburban sites had a 67 percent reduction in all crash severities combined, but no results could be obtained for injury crashes due to a small sample size. The five urban sites had a statistically insignificant one percent reduction for all crash severities combined and a 60 percent reduction in injury crashes.

A Danish study found that at signalized dual carriageway intersections there was a 9.2 percent reduction in crashes for two-phase signals and a 7.8 percent reduction for three-phase signals (separate right-turn phase—left-hand driving) (Jørgensen and Jørgensen, 1994).

According to Ourston (1996), another study from the Netherlands investigated the effect of conversion of nine traffic signals to roundabouts. They found a 27 percent reduction in total crashes and a 33 percent reduction in casualties.

It is important to note that converting a signal to roundabout is relatively high cost and the timeframe for implementation is intermediate or long-term. This type of improvement may not be applicable for agencies focusing on low-cost, short-term solutions. As identified in NCHRP 500 Series Report, Volume 12, the following are key issues to consider for this strategy (Antonucci et al., 2004):

- Total entering traffic volumes.
- Turning movements.
- Operational characteristics.

OBJECTIVES

One objective was to estimate the general safety effectiveness of converting signalized intersections to roundabouts, as measured by crash frequency and severity. Target crash types included:

- All crash types (all severities).
- Property damage only crashes (all crash types).
- Fatal and injury crashes (all crash types).

The other key objective was to conduct a disaggregate analysis to identify circumstances (e.g., geometric and traffic conditions) under which conversion of signals to roundabouts may be more safety effective.

Meeting these objectives placed some special requirements on the data collection and analysis tasks, including the need to:

- Consider a supplemental cross-sectional study if data were insufficient for the preferred before-after analysis.
- Select a large enough sample size to detect, with statistical significance, what may be small changes in safety for target crash types.
- Carefully select comparison or reference sites.
- Properly account for traffic volume changes and the possibility of regression-to-the-mean.
- Pool data from multiple jurisdictions to improve reliability of the results and facilitate broader applicability of the research products.

Roadway, traffic volume, and crash data were acquired for Indiana, New York, Washington, Florida, Michigan, North Carolina, South Carolina, Colorado, Maryland, and Vermont to facilitate the analysis. The states identified treatment and reference sites. The states also provided crash data and information related to the installation of the strategy (i.e., location and date).

METHODOLOGY

The treatment of interest is the conversion of signalized intersections to roundabouts. The study team worked with several states to identify signalized intersections that have been converted to roundabouts in the recent past. The desired method for this evaluation is the state-of-the-art empirical Bayes (EB) methodology for observational before-after studies. However, several sites in Indiana were newly constructed roundabouts. As such, a regression-type analysis was also conducted to compare the safety performance of similar signalized intersections and roundabouts. The two methods are described below.

Empirical Bayes Analysis

The methodology applied was the EB before-after study, following the procedure outlined in Hauer (1997). The advantages of the EB approach are that it:

- Properly accounts for regression-to-the-mean.
- Overcomes the difficulties of using crash rates in normalizing for volume differences between the before and after periods.
- Reduces the level of uncertainty in the estimates of safety effect.
- Provides a foundation for developing guidelines for estimating the likely safety consequences of contemplated installations.
- Properly accounts for differences in crash experience and reporting practice in amalgamating data and results from diverse jurisdictions.
- Avoids the difficulties of conventional treatment-comparison experimental designs caused by possible spillover and/or migration effects to natural comparison groups.

In an EB evaluation, the change in safety for a given crash type at a treated intersection is given by Equation 1.

$$B - A \qquad (1)$$

where B is the expected number of crashes that would have occurred in the "after" period without the treatment and A is the number of reported crashes in the after period.

Due to changes in safety that may result from changes in traffic volume, regression-to-the-mean, and trends in crash reporting and other factors, the count of crashes before treatment by itself is not a good estimate of B, a reality that has now gained common acceptance. Instead, B is estimated from an EB procedure in which a safety performance function (SPF) is used to first estimate the number of crashes that would be expected in each year of the before period at locations with traffic volumes and other characteristics similar to the treated site being analyzed. The sum of these annual SPF estimates (P) is then combined with the count of observed crashes (x) in the before period at the treatment site to obtain an estimate of the expected number of crashes (m) before the treatment. The estimate of m is given by Equation 2.

$$m = w(P) + (1-w)(x) \qquad (2)$$

The weight w is estimated from Equation 3.

$$w = 1/(1 + kP) \qquad (3)$$

where k is the over-dispersion parameter of the negative binomial distribution that is assumed for the crash counts used in estimating the SPF. The value of k is estimated from the SPF calibration process with the use of a maximum likelihood procedure.

A factor is then applied to m to account for the length of the after period and differences in traffic volumes between the before and after periods. This factor is the sum of the annual SPF predictions for the after period divided by P, the sum of these predictions for the before period. The result, after applying this factor, is an estimate of B. The procedure also produces an estimate of the variance of B, the expected number of crashes that would have occurred in the after period without the treatment.

The estimate of B is then summed over all sites in a treatment group of interest (to obtain B_{sum}) and compared with the count of crashes during the after period in that group (A_{sum}). The variance of B is also summed over all sections in the group of interest.

The index of safety effectiveness (θ) is estimated by Equation 4.

$$\theta = \frac{A_{sum}/B_{sum}}{1 + \left(Var(B_{sum})/B_{sum}^2\right)} \tag{4}$$

The index of effectiveness is equivalent to the crash modification factor. The standard deviation of θ is given by Equation 5.

$$StDev(\theta) = \sqrt{\frac{\theta^2\left(\frac{A_{sum}}{A_{sum}^2} + \frac{B_{sum}}{B_{sum}^2}\right)}{\left(1 + \frac{Var(B_{sum})}{B_{sum}^2}\right)^2}} \tag{5}$$

The percent change in crashes is $100(1-\theta)$; thus a value of $\theta = 0.7$ with a standard deviation of 0.12 indicates a 30 percent reduction in crashes with a standard deviation of 12 percent.

Cross-Sectional Analysis

A cross-sectional analysis was employed to supplement the results of the EB analysis. This was necessary because the sample of conversions with sufficient after period data was somewhat lean for jurisdictions outside New York. A secondary objective of this supplemental investigation was to examine the comparability of results of before-after and cross-section studies, a subject of topical interest in CMF development, for which there is little research.

Negative binomial regression is a common method for developing relationships between crashes and roadway characteristics (e.g., traffic volume, area type, etc). The negative binomial regression model is applied in this evaluation framework to estimate the safety effects of roundabouts compared to signalized intersections. Specifically, an indicator variable is included in the models to represent the presence of a signal or roundabout. Once the models are estimated, the coefficient for the roundabout indicator is exponentiated to estimate the CMF. The general functional form of the model assumed for this analysis is shown in Equation 6.

$$Crashes\ /\ year = \exp\left(\alpha + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_n X_n\right) \qquad (6)$$

Where,
α and β_1 - β_n = parameters estimated in the model calibration process.
X_1 - X_n = covariates included in the model.

Average annual daily traffic (AADT) and the roundabout indicator are included in every cross-sectional model developed for this evaluation. Additional variables were considered based on available data and included in the models if the following conditions were met:

a) The variable significantly improved the model.
b) The effect of the variable was intuitive (e.g., crashes increase as number of approaches increases).

The following additional variables were considered in the model development.

- Number of intersection approaches (3-legged/4-legged indicator).
- Number of approach or roundabout lanes (single lane/multilane).
- Signalized intersection phasing (permissive/protected/protected-permissive indicators).

These variables entered the model form as adjustments to the base value of α in Equation 1. The base value of α was estimated for a particular baseline condition (e.g., three-legged, single lane intersection). When the condition of the intersection is anything other than the baseline, an adjustment was applied to the base value of α. The parameter values (β's) indicate the magnitude and direction of the adjustment to the base α value.

Common concerns related to cross-sectional analyses include:

- Misspecification of model functional form.
- Confounding effects.
- Interaction effects.
- Inconsistency among results from different studies of the same treatment.

The following techniques are used to address the common concerns associated with cross-sectional studies.

- Several functional forms are explored to test the sensitivity of the coefficients.
- Several covariates are considered for inclusion in the model.
- Interaction effects are tested and included as necessary.
- Models are developed for each state individually and compared for consistency.

DATA COLLECTION

This section provides a summary of the databases developed for each state. These include data for signalized intersections where no improvements were implemented, signalized intersections that were converted to roundabouts, and newly constructed roundabouts that were constructed in place of a signal. Table B-1 describes how the crash types analyzed were defined. Crash definitions are not identical between the states due to differences in crash reporting.

Table B-1. Definitions Used in Analyses by Jurisdiction.

Intersection-related	All crashes at or related to the intersection as provided by the state. In Indiana, intersection-related crashes were defined as those crashes at or within 250 feet of the intersection.
Property damage only	Defined as reported crashes not resulting in an injury or fatality.
Injury-related	Defined as crashes resulting in an injury or fatality.

Before-After Data

The data were structured differently and included different subsets of sites for the two analyses (cross-sectional and before-after). The datasets used for the before-after analyses are described below.

Indiana Data

The City of Carmel, Indiana is leading efforts to install roundabouts in place of traditional intersections where feasible. Reasons for roundabout installations in the City are numerous, including safety, cost, environmental, and operational benefits. The City of Carmel provided a list of all planned and constructed roundabouts, including the location, installation (or planned) date, and prior control. From this list, the study team identified sites where a signalized intersection was replaced with a roundabout. The City also provided a list of signalized intersections where no improvements were implemented for use as reference sites. The reference sites were selected from the same corridors where the roundabouts were installed based on similar geometry and traffic volume. For each treatment and reference site, the City provided traffic volumes, turning movement counts, and speed limit data when available. Crash data were obtained from the Indiana DOT for years 2003 through 2008. Three treatment sites were suitable for including in the before-after analysis.

New York Data

The New York State Department of Transportation (NYSDOT) has recently replaced several signalized intersections with roundabouts. The NYSDOT identified 11 sites where a signalized intersection was replaced with a roundabout. The NYSDOT provided data for each site, including the location, installation period, number of approaches, prior signal phasing, aerial photo, and total entering traffic volume. Crash data were also obtained from the NYSDOT for the before and after period. There were generally three years of before and after data provided for each site.

Washington Data

Several signalized intersections were recently replaced with roundabouts in various cities in Washington. The Washington State Department of Transportation (WSDOT) helped to identify two sites where this treatment was installed. Neither of the sites was selected for treatment based on safety reasons. Instead,

the goal was to improve operational efficiency, which was accomplished through the conversion to a roundabout. The study team worked with WSDOT and the individual cities to obtain data for each site, including the location, installation period, number of approaches, and traffic volume data. At least three years of crash data were obtained from WSDOT for both the before and after period. The crash data covered the period from 2001 through March 2009.

Florida Data

NCHRP Project 3-65 included a number of sites where signalized intersections were replaced with roundabouts in various states, including Florida. The locations were identified based on the NCHRP 3-65 project and the study team followed-up with the individual cities for additional details. The local agencies identified the installation date, basic geometric data, and traffic volume data. Crash data were obtained for the before and after periods from the database used for the NCHRP 3-65 project. Only one site had suitable data for inclusion in the current study.

Michigan Data

NCHRP Project 3-65 included a number of sites where signalized intersections were replaced with roundabouts in various states, including Michigan. The locations were identified based on the NCHRP 3-65 project and the study team followed-up with the Michigan Department of Transportation (MDOT) and local agencies for additional details. During discussions, an additional location was identified for inclusion in this study, which was not included in NCHRP 3-65. The installation date and traffic volume data were identified for each site. Crash data were obtained for the before and after periods from the database used for the NCHRP 3-65 project. For the newly identified site, crash data were obtained for years 2000 through 2009. In total, two sites were included from Michigan in this evaluation.

North Carolina Data

NCHRP Project 3-65 included a number of sites where signalized intersections were replaced with roundabouts in various states, including North Carolina. The locations were identified based on the NCHRP 3-65 project and the study team followed-up with the respective local agencies for additional details. The installation date, basic geometry, and traffic volume data were identified by the local agency. Crash data were obtained for the before and after periods using the state's Traffic Engineering Accident Analysis System (TEAAS). Crash data included years 1999 through 2009. A total of two treatment sites were included in the current before-after analysis.

Other State Data

NCHRP Project 3-65 included 3 sites in Colorado, 1 site in South Carolina, 2 sites in Maryland, and 1 site in Vermont where signalized intersections were replaced with roundabouts. These data were obtained and included in the present study.

Summary statistics for the treated sites used in the before-after study are provided in Table B-2.

Table B-2. Signal to Roundabout Conversions

Variable	Mean	Minimum	Maximum
Years before	3.92	1.83	13.00
Years after	3.12	1.00	5.00
Total crashes/site-year before	6.72	0.67	26.23
Total crashes /site-year after	6.55	0.33	34.00
PDO crashes/site-year before	5.32	0.00	24.04
PDO crashes/site-year after	6.06	0.00	34.00
Injury crashes/site-year before	1.40	0.00	5.00
Injury crashes/site-year after	0.49	0.00	3.33
Entering AADT before	18,529	5,322	43,123
Entering AADT after	20,952	5,322	52,541
Number of Sites = 28			

*where only one traffic count was available it was assumed that there was no change during the study period

All of the sites included in this analysis are located in urban or suburban areas with relatively low approach speeds. Before and after speed limit data were not available for each site, but the speed limit and/or advisory speed data were obtained for the 'after' condition along the major road for each of the study sites. The "associated speed" is based on the approach advisory speed when posted. Where no advisory speed is posted, the "associated speed" is based on the nearest upstream posted speed limit. For 14 locations (7 urban and 7 suburban), the speed associated with the major roundabout approaches was 20 mi/h or less. For the remaining 14 sites, the associated speed ranged from 25 to 35 mi/h. Table B-3 shows the distribution of sites by approach associated speed and area type.

Table B-3. Associated Speeds by Area Type for Roundabout Conversions

Associated Speed on Major Approach to Roundabout in the after period (mi/h)	**Number of Sites by Area Type**	
	Urban	**Suburban**
15	6	7
20	1	0
25	2	3
30	3	4
35	1	1

Safety Performance Functions for EB Analysis

Data for signalized intersections, similar to those converted to roundabouts, were sought for use in developing the SPFs required for the EB methodology. Unfortunately, such data were difficult to obtain for all states in which treatment sites were identified. Untreated reference sites were identified in Indiana, North Carolina, and New York. Crash, traffic volume, and roadway data were collected for the reference group. The data from Indiana and North Carolina were both used to directly calibrate SPFs for total and fatal+injury crashes separately for the two states. For all other locations, the SPFs previously used in NCHRP project 3-65 (Rodegerdts et al., 2007) were applied. As was done in that study, these SPFs were

recalibrated for use in the specific jurisdictions using data for the sample of roundabout conversions for the period immediately before conversion. Only the data in the one year immediately prior to roundabout construction were used for this purpose to guard against the possibility that a randomly high crash count in earlier years may have prompted the decision to install the roundabout and therefore provide functions that would overestimate safety improvements after conversion. Examination of annual crash trends in the before periods indicated that this decision was justified. The SPFs used for this evaluation are summarized below:

SPFs calibrated for NCHRP Project 3-65

- 4 Approaches
  - Acc/yr = exp(-9.00)(AADT)$^{1.029}$, k=0.20
  - InjAcc/yr = exp(-10.43)(AADT)$^{1.029}$, k=0.20

- 3 Approaches
  - Acc/yr = exp(-5.24)(AADT)$^{0.580}$, k=0.18
  - InjAcc/yr = exp(-6.51)(AADT)$^{0.580}$, k=0.18

New SPFs calibrated for Indiana data

- Acc/yr = exp(-13.0303)(AADT)$^{1.5324}$, k=0.2473
- InjAcc/yr = exp(-18.2655)(AADT)$^{1.8690}$, k=0.2506

New SPFs calibrated for North Carolina data

- Acc/yr = exp(-5.6522)(AADT)$^{0.7627}$, k=0.4659
- InjAcc/yr = exp(-4.9800)(AADT)$^{0.5943}$, k=0.4422

Where,
Acc/yr = expected number of total intersection crashes per year.
InjAcc/yr = expected number of fatal and injury intersection crashes per year.
AADT = the total entering volume.
k = over-dispersion parameter of the model used in the EB methodology.

Cross-Sectional Data

The datasets used for the cross-sectional analyses are described below.

Indiana Data

The Indiana data were categorized into two groups for the cross-sectional analysis. Tables B-4 and B-5 summarize the characteristics of the following groups.

1. Signalized intersections.
2. Roundabouts.

Table B-4. Signalized intersections in Indiana.

Variable	Mean	Minimum	Maximum
Years	5.87	2	6
Total crashes/site-year	7.38	3	171
PDO crashes/site-year	6.27	3	139
Injury crashes/site-year	1.11	0	32
Total Entering AADT	18,370	9,021	35,296
Number of Sites = 31			

Table B-5. Roundabouts in Indiana.

Variable	Mean	Minimum	Maximum
Years	4.07	2	6
Total crashes /site-year	5.41	0	117
PDO crashes/site-year	4.93	0	109
Injury crashes/site-year	0.48	0	8
Total Entering AADT	12,719	2,517	20,958
Number of Sites = 15			

New York Data

The same set of 11 intersections used from New York in the before after study were also used for the cross-sectional analysis. Table B-6 summarizes the characteristics of these intersections when they were signalized. Table B-7 is a summary of the characteristics of these intersections after they became roundabouts. Note that the total and PDO crashes per site year increase while the injury crashes per site year decrease when comparing the signal to roundabout years. Also, the traffic volume remains constant from the signal to roundabout conversion because it was assumed that there was no growth at the sites.

Table B-6. Characteristics of intersections in New York when they were signalized.

Variable	Mean	Minimum	Maximum
Years	3.73	3	6
Total crashes/site-year	2.76	2	21
PDO crashes/site-year	1.71	0	15
Injury crashes/site-year	1.05	2	7
Major road ADT	13,386	8,110	18,750
Minor road ADT	4,156	1,200	8,880
Number of Sites = 11			

Table B-7. Characteristics of intersections in New York after conversion to roundabouts.

Variable	Mean	Minimum	Maximum
Years	3.36	2	5
Total crashes /site-year	3.65	1	30
PDO crashes/site-year	3.22	1	27
Injury crashes/site-year	0.43	0	4
Major road ADT	13,386	8,110	18,750
Minor road ADT	4,156	1,200	8,880
Number of Sites = 11			

BEFORE-AFTER ANALYSIS

Aggregate Results

Table B-8 documents the results of the aggregate and disaggregate analysis. Over all 28 conversions it is seen from the first row of results that the safety benefit for injury crashes is substantial and is larger than the effect for all crashes combined. This general indication confirms the results of a recent study (Rodegerts et al., 2007) that was based on only nine conversions. For that study, the CMFs, which were based on four suburban and five urban conversions, were estimated as 0.522 and 0.223 for total and injury crashes, respectively. That study found overall effects of similar magnitude for converting stop controlled intersections to roundabouts.

A disaggregate analysis was conducted to identify circumstances under which conversion of signals to roundabouts may be more safety effective. The remainder of Table B-8 documents results disaggregated by state, number of circulating lanes, setting (urban versus suburban), number of approaches, and associated speed. Figures B-2 to B-5 illustrate the variability of the CMF with respect to traffic volume; trend lines are shown using a simple linear fit. (A linear trend line was selected after considering other forms since it provides a CMFunction that generally provides conservative predictions and also because it is more consistent with the cross-section analysis results presented later.)

The disaggregate results indicate that for all groups except Indiana, the CMF for injury crashes is smaller than for total crashes (i.e., a larger reduction in injury crashes). Similar to the results from the NCHRP study of nine signalized intersections (Rodegerts et al., 2007), the safety benefit for suburban conversions is larger than for urban conversions; however this result is dominated by the Colorado conversions, so caution is required in reading too much into this conclusion. The results also indicate that the CMFs for intersections with three approaches are larger than for intersections with four approaches. There is no clear pattern regarding the effectiveness of the roundabout as the ‘associated speed’ increases (as mentioned earlier, associated speed is the posted advisory speed or the nearest upstream posted speed limit on the major road during the ‘after’ period). The results do, however, indicate a substantial and statistically significant reduction in fatal/injury crashes for all categories of associated speeds. This is consistent with the other disaggregate analyses and confirms that roundabouts are effective at reducing the severity of crashes. There does appear to be a slight increase in total crashes for sites with 15 and 30 mi/h approach speeds, but these results are statistically insignificant. There is also a substantial increase in total crashes for sites with a 20 mi/h approach speed, the CMF for this category is based on just one site and is therefore suspect. The important note from these results is that there is a substantial and significant reduction in fatal/injury crashes in all scenarios.

As a note, the results presented in this report (i.e., CMFs) are not directly applicable to signal-to-roundabout conversions at rural high-speed locations from a strict CMF application standpoint. However, one of the primary functions of a roundabout is to reduce entry speeds (assuming the approach and deflection are designed appropriately). Previous research has shown that as speeds increase, there is an increased probability of a severe crash. One could hypothesize that roundabouts would be just as effective (if not more so) in rural high-speed areas compared to their urban/suburban counterparts because speeds are higher (i.e., greater chance of severity when crashes occur), but crossing-path and left-turn crashes (typically the more severe crash types) are still physically eliminated with the installation of a roundabout.

Table B-8. Disaggregate Before-After Results by State, Number of Lanes, and Area Type

Group	Number of Sites	EB Expected Total Crashes Without Roundabouts	Total Crashes Recorded in After Period	CMF for Total Crashes (standard error)	EB Expected Injury Crashes Without Roundabouts	Injury Crashes Recorded in After Period	CMF for Injury Crashes (standard error)
ALL	28	643.91	511	0.792 (0.050)	122.03	42	0.342 (0.058)
Colorado	3	269.52	75	0.276 (0.040)	24.52	1	0.038 (0.037)
Non-Colorado	25	374.39	436	1.162 (0.075)	97.51	41	0.424 (0.073)
Florida	1	6.0	11	1.499 (0.607)	1.91	3	1.342 (0.814)
Indiana	3	29.86	30	0.979 (0.233)	3.50	4	1.041 (0.559)
Maryland	2	41.90	72	1.684 (0.306)	19.93	11	0.535 (0.182)
Michigan	2	48.79	93	1.896 (0.237)	4.99	0	No after crashes
North Carolina	2	28.38	22	0.757 (0.194)	6.35	3	0.431 (0.258)
New York	11	110.84	135	1.210 (0.214)	32.28	16	0.490 (0.132)
South Carolina	1	49.90	9	0.167 (0.063)	10.25	0	No after crashes
Vermont	1	22.63	23	0.983 (0.264)	11.77	1	0.081 (0.080)
Washington	2	35,47	41	1.125 (0.249)	6.53	3	0.423 (0.252)
2-lane	16	485.51	394	0.809 (0.061)	82.59	24	0.288 (0.065)
1-lane	12	158.39	117	0.735 (0.086)	39.44	18	0.451 (0.115)
Suburban	15	403.00	233	0.576 (0.053)	68.58	18	0.259 (0.066)
Urban	13	240.91	278	1.150 (0.093)	53.46	24	0.445 (0.100)
3 approaches	6 (3 urban)	65.96	71	1.066 (0.163)	13.15	5	0.370 (0.172)
4	22	577.94	440	0.759	108.88	37	0.338

approaches	(10 urban)			(0.052)			(0.061)
Associated Speed 15 mi/h	13 (6 urban)	173.35	199	1.143 (0.108)	36.22	17	0.464 (0.121)
Associated Speed 20 mi/h	1 (urban)	29.35	68	2.293 (0.359)	2.50	0	No after crashes
Associated Speed 25 mi/h	5 (2 urban)	208.79	97	0.461 (0.062)	34.87	4	0.112 (0.057)
Associated Speed 30 mi/h	7 (3 urban)	105.62	110	1.033 (0.134)	35.19	18	0.504 (0.132)
Associated Speed 35 mi/h	2 (1 urban)	126.79	37	0.287 (0.060)	13.25	3	0.206 (0.123)

Figures B-2 to B-4 indicate that the CMF for total crashes increases (the safety benefit decreases) with increasing AADT for both urban and suburban sites. This is consistent with the conclusion from the before-after analysis in Rodegerts et al. (2007) that “The safety benefits appear to decrease with increasing AADT, irrespective of control type before conversion, number of lanes, and setting.” For all sites combined the trend line indicates that the CMF becomes larger than 1.0 at an AADT of around 18,000 vehicles per day. This “breakeven” point is approximately 14,000 and 22,000 vehicles per day, respectively for urban and suburban sites. For all levels of AADT in the comparable range, the CMF for suburban conversions is lower than for urban conversions, suggesting perhaps that the difference between the overall CMFs for urban and suburban sites evidenced in Table B-8 is not due to AADT differences, but possibly due to other reasons such as higher approach speeds in suburban areas. All of these findings with respect to the influence of AADT are revisited later in this appendix in the context of the findings from the cross-sectional analysis.

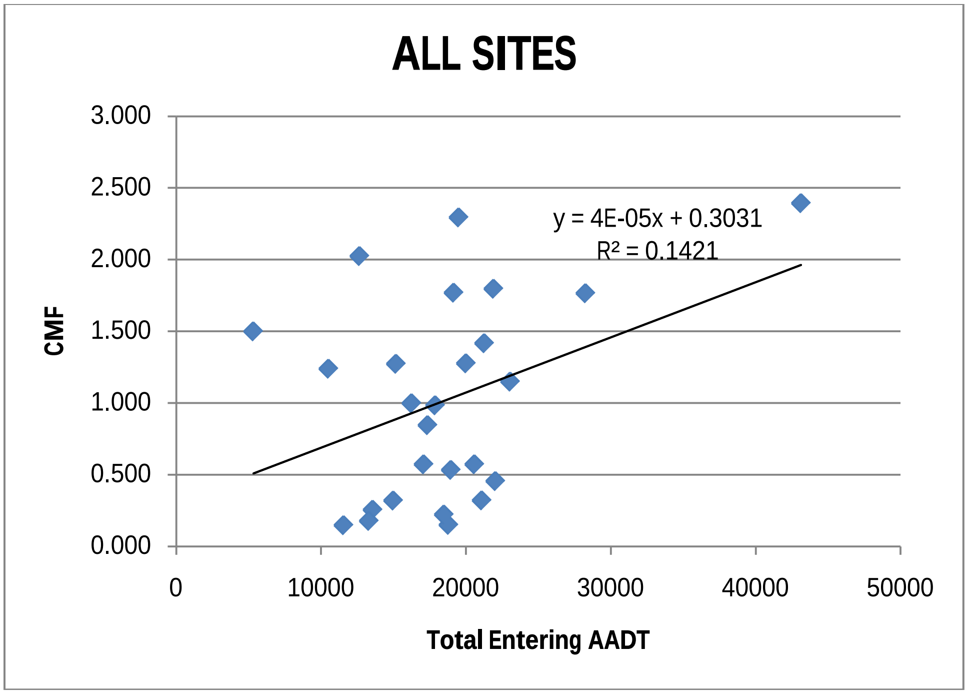

Figure B-2: Effect of Total Entering AADT on CMF for Total Crashes (All Sites)

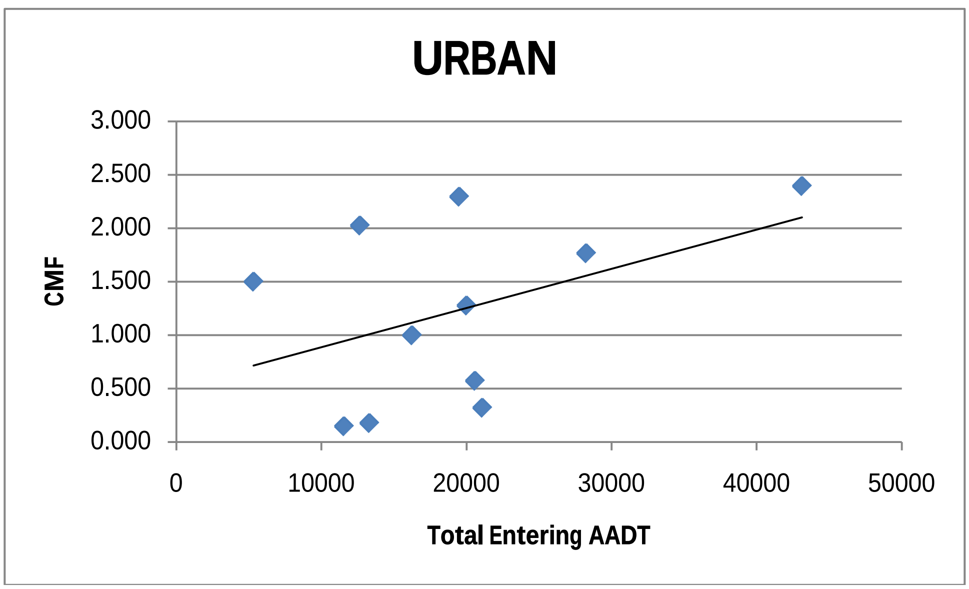

Figure B-3: Effect of Total Entering AADT on CMF for Total Crashes (Urban Sites)

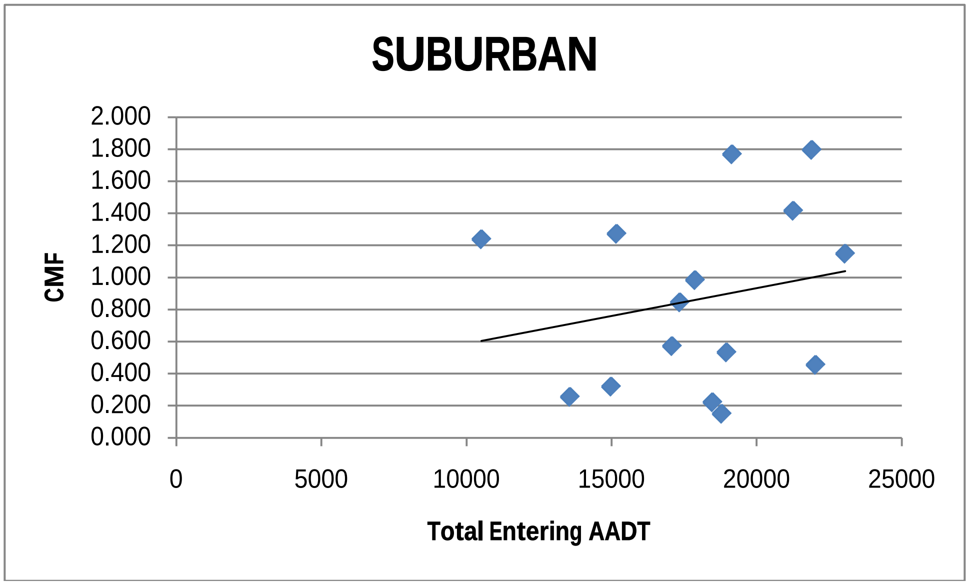

Figure B-4: Effect of Total Entering AADT on CMF for Total Crashes (Suburban Sites)

Figure B-5 indicates that the trend is relatively flat for injury crashes (i.e., there is little variation of the CMF with AADT), a result that is substantiated by the cross-sectional analysis presented later.

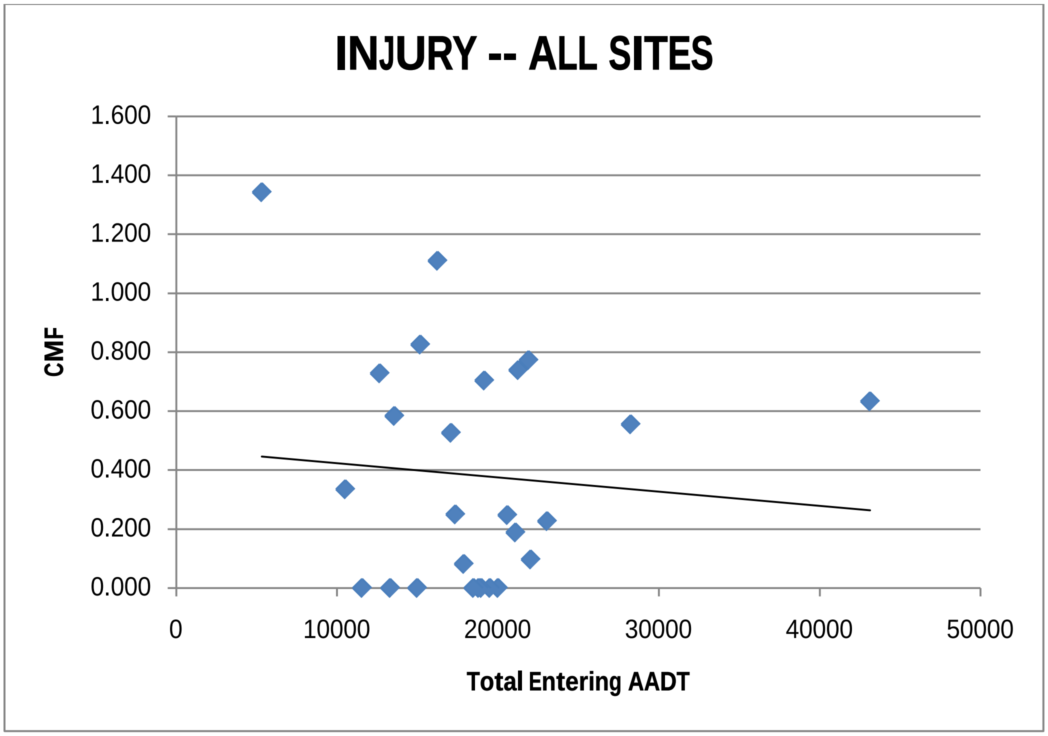

Figure B-5: Effect of Total Entering AADT on CMF for Injury Crashes (All Sites)

CROSS-SECTIONAL ANALYSIS

Cross-sectional models were developed individually for Indiana and New York using the negative binomial regression techniques described in the Methodology section. The regression analysis is based on

signalized intersections and roundabouts (i.e., no stop-controlled intersections were included). Models were developed for both states for total, PDO, and injury crashes. The data from the two states were also aggregated and combined models were developed for the same crash types. It is important to note that in New York, the cross sectional analysis was based on data from the same set of locations that were used in the before after evaluation, whereas in Indiana a different set of locations were identified for signalized intersections and roundabouts. In other words, in New York (unlike Indiana), the cross sectional analysis was based on 'before-after' data and hence could be characterized as a type of time-series cross-sectional analysis or panel data analysis. Since we had data from the same site over multiple years, it was necessary, however, to make an adjustment for the repeated observations. To accomplish this, the model fitting process used a clustered robust standard error, identifying individual sites as the cluster variable.

Preliminary models were developed using various forms of AADT, including:

1. Total entering AADT.
2. Natural log of total entering AADT.
3. Major and minor AADT (available only for New York).
4. Natural log of major and minor AADT (available only for New York).

The decision on which form to use was based on an evaluation of parameter estimates and other goodness of fit measures (i.e., log-likelihood and pseudo R-square). Other comparisons were made using the observed versus predicted values, including total predicted versus total observed crashes and the sum of squared residuals. For both the Indiana and New York models, as well as the combined model, the natural log of total entering AADT was the most appropriate form for the AADT term.

The following additional variables were considered in the model development. The interaction term considered for the final models is the product of the natural log of total AADT and the indicator for the presence of a roundabout.

- RAB (1/0 indicator for roundabout or signal, 1=roundabout).
- Approaches (1/0 indicator for number of approaches, 1=four or more approaches).
- Multilane (1/0 indicator for number of approach or roundabout lanes, 1=2 or more lanes).
- Permissive (1/0 indicator for signal phasing, 1=permissive).
- Protected (1/0 indicator for signal phasing, 1=protected or protected-permissive).
- Interaction (RAB x AADT term).
- State (1/0 indicator for state, 1=Indiana) – only included in combined analysis.

<u>Models for Total Crashes</u>

The models for total crashes are presented in Tables B-9 to B-11. For Indiana (Table B-9) and for the two states combined (Table B-11), the interaction term is significant, indicating a difference in the effect of traffic volume between signal and roundabout sites. As such, the interaction term remains in the model, but the model without interaction is presented for comparison. For New York (Table B-10), the interaction term has a low P-value of 0.855 but is consistent with the direction of the interaction for Indiana, so it was decided that the model with the interaction term could be considered for exploring the effect of AADT on the implied CMF.

Table B-9. Indiana Model for Total Crashes.

Based on 45 sites totaling 243 site-years, representing 1,673 total crashes.

WITH INTERACTION

```
Negative binomial regression                      Number of obs   =        243
Dispersion           = mean                       Wald chi2(3)    =      72.39
Log pseudo likelihood = -644.80984                Prob > chi2     =     0.0000
------------------------------------------------------------------------------
             |               Robust
totalcrashes |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
      lnAADT |   1.332461   .2955761     4.51   0.000     .7531429     1.91178
         rab |  -11.23326   4.466189    -2.52   0.012    -19.98683   -2.479686
 Interaction |   1.171605   .4602855     2.55   0.011     .2694618    2.073748
    constant |  -11.10755   2.858732    -3.89   0.000    -16.71056   -5.504534
-------------+----------------------------------------------------------------
       alpha |   .3280442   .0985645                      .1820452    .5911332
```

WITHOUT INTERACTION

```
Negative binomial regression                      Number of obs   =        243
Dispersion           = mean                       Wald chi2(2)    =      44.96
Log pseudo likelihood = -652.14955                Prob > chi2     =     0.0000
------------------------------------------------------------------------------
             |               Robust
totalcrashes |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
      lnAADT |   1.629662   .2447703     6.66   0.000     1.149921    2.109403
         rab |   .0825006   .2068749     0.40   0.690    -.3229667     .487968
    constant |  -14.01837   2.365046    -5.93   0.000    -18.65377   -9.382965
-------------+----------------------------------------------------------------
       alpha |   .3540962   .1041306                      .1989778    .6301413
```

Table B-10. New York Model for Total Crashes.

Based on 11 sites totaling 78 site-years, representing 248 total crashes.

WITH INTERACTION

```
Negative binomial regression                      Number of obs   =         78
Dispersion           = mean                       Wald chi2(4)    =     191.08
Log pseudo likelihood = -143.297                  Prob > chi2     =     0.0000
------------------------------------------------------------------------------
             |               Robust
       total |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
       lnadt |   1.903576   .6091084     3.13   0.002      .7097457    3.097407
         rab |   -.899476   6.215987    -0.14   0.885     -13.08259    11.28364
   multilane |   .6131477   .2391809     2.56   0.010      .1443617    1.081934
 interaction |   .1149168   .6302194     0.18   0.855     -1.120291    1.350124
    constant |  -17.87487   5.910987    -3.02   0.002     -29.46019    -6.28955
-------------+----------------------------------------------------------------
    /lnalpha |  -15.87619    1.14586                      -18.12204   -13.63035
-------------+----------------------------------------------------------------
       alpha |   1.27e-07   1.46e-07                       1.35e-08    1.20e-06
```

WITHOUT INTERACTION

```
Negative binomial regression                      Number of obs   =         78
Dispersion           = mean                       Wald chi2(3)    =      56.22
Log pseudo likelihood = -143.31084                Prob > chi2     =     0.0000
------------------------------------------------------------------------------
             |               Robust
       total |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
       lnadt |   1.965389    .809853     2.43   0.015      .3781062    3.552671
         rab |   .2328695    .135221     1.72   0.085     -.0321588    .4978978
   multilane |   .6126178   .2391457     2.56   0.010      .1439008    1.081335
    constant |  -18.48319    7.91539    -2.34   0.020     -33.99707   -2.969307
-------------+----------------------------------------------------------------
    /lnalpha |  -21.34061          .                              .           .
-------------+----------------------------------------------------------------
       alpha |   5.39e-10          .                              .           .
```

Table B-11. Combined Model for Total Crashes.

Based on 56 sites from Indiana and New York totaling 321 site-years, representing 1,921total crashes.

WITH INTERACTION

Negative binomial regression		Number of obs =	321
Dispersion	= mean	Wald chi2(4) =	103.96
Log pseudo likelihood	= -802.67722	Prob > chi2 =	0.0000

totalcrashes	Coef.	Robust Std. Err.	z	P>|z|	[95% Conf.	Interval]
lnaadt	1.390968	.2822448	4.93	0.000	.8377783	1.944157
rab	-10.54856	4.150873	-2.54	0.011	-18.68412	-2.412999
state	.9157908	.1574223	5.82	0.000	.6072488	1.224333
interaction	1.101419	.4252857	2.59	0.010	.2678739	1.934963
constant	-12.59671	2.751991	-4.58	0.000	-17.99052	-7.202912
/lnalpha	-1.215382	.2873031			-1.778486	-.6522784
alpha	.2965966	.0852131			.1688937	.5208577

WITHOUT INTERACTION

Negative binomial regression		Number of obs =	321
Dispersion	= mean	Wald chi2(3) =	65.85
Log pseudo likelihood	= -810.72709	Prob > chi2 =	0.0000

totalcrashes	Coef.	Robust Std. Err.	z	P>|z|	[95% Conf.	Interval]
lnaadt	1.699088	.2297192	7.40	0.000	1.248847	2.149329
rab	.1405135	.1491748	0.94	0.346	-.1518637	.4328907
state	.8504551	.1662	5.12	0.000	.5247091	1.176201
constant	-15.55824	2.259445	-6.89	0.000	-19.98667	-11.12981
/lnalpha	-1.133098	.2802066			-1.682293	-.5839034
alpha	.3220339	.0902361			.1859471	.5577171

With the inclusion of interaction terms, the safety performance of a roundabout design compared to a signalized intersection is not constant across all sites. In this case, the interaction term allows the estimated difference in safety performance to vary by total entering AADT.

Figures B-6 to B-8 show the model predicted values for signalized intersections and roundabouts, assuming a range of total AADT values. The range of AADT is based on the range of the data used to develop the negative binomial models. The predicted number of crashes increases with increasing total AADT for both signals and roundabouts; however, the expected crashes increase at a greater rate for roundabouts. This indicates that roundabouts may be safer (in terms of total crashes) than signals at lower AADTs, but the safety benefit of roundabouts appears to decrease as AADT increases. Figure B-9 further illustrates the point by showing the change in the CMF, implied by the difference in safety performance, as a function of total AADT. The implied CMF increases as total AADT increases, and actually exceeds 1.0 for AADTs greater than approximately 14,600 vehicles per day for Indiana and 14,400 vehicles per day for the two states combined. This is reasonably consistent with the results of the before-after analysis

which indicated "breakeven" AADTs of 14,000, 22,000, and 18,000 for urban, suburban and all sites, respectively. While Figure B-9 shows the breakeven point for New York (approximately 2,500 vehicles per day), this is well outside the range of applicable AADTs for the New York dataset (10,510 to 23,050 vehicles per day).

The general trend, particularly for Indiana, may be expected because there is greater "friction" (i.e., opportunity for vehicle conflicts) within a roundabout as circulating volumes increase. In fact, a similar trend is seen with respect to operations. Roundabouts have been shown to perform less efficiently than similar signalized intersections, with respect to operations, as traffic volumes exceed a given threshold. One study reports that single lane roundabouts are expected to operate sufficiently at traffic volumes below 18,000 vehicles per day and *may* operate sufficiently between 18,000 and 31,000 vehicles per day (Rice, 2010). Another study reports that the maximum daily service volume for a four-leg, single-lane roundabout is approximately 20,000 to 27,000 vehicles depending on the percentage of left-turn traffic (Robinson et al., 2000). This study indicates that roundabout capacity is approached as the vehicular volume in the circulating roadway approaches 1,800 motor vehicles per hour for a single lane roundabout.

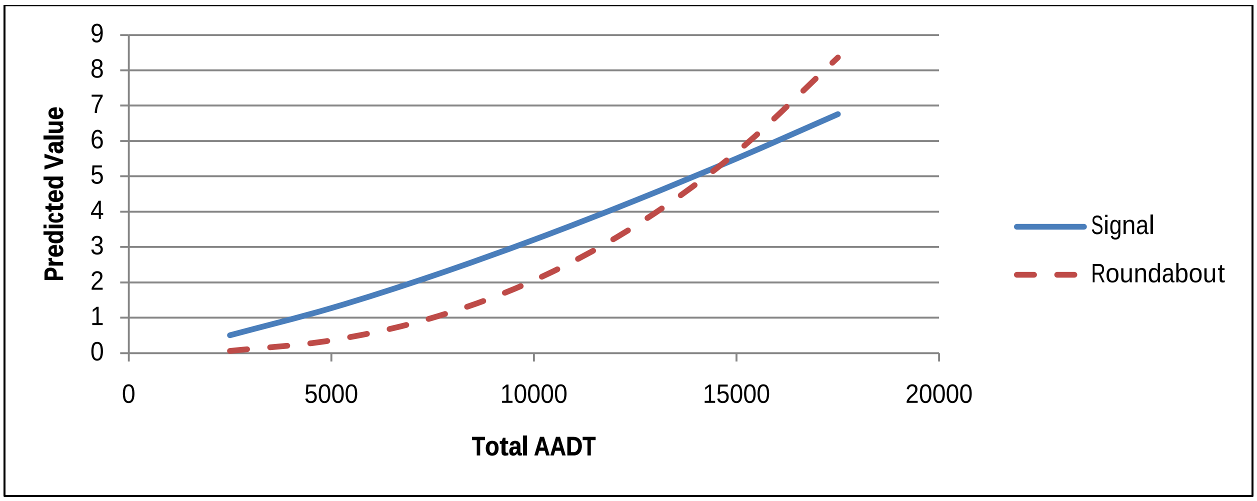

Figure B-6. Interaction Effect of Total Entering AADT – Indiana Model

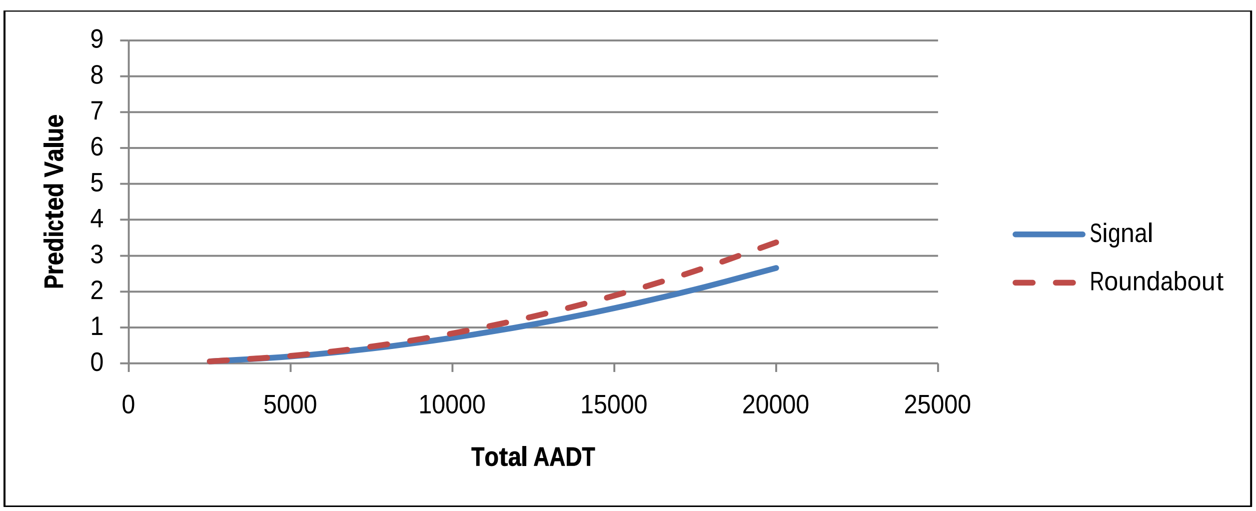

Figure B-7. Interaction Effect of Total Entering AADT – New York Model

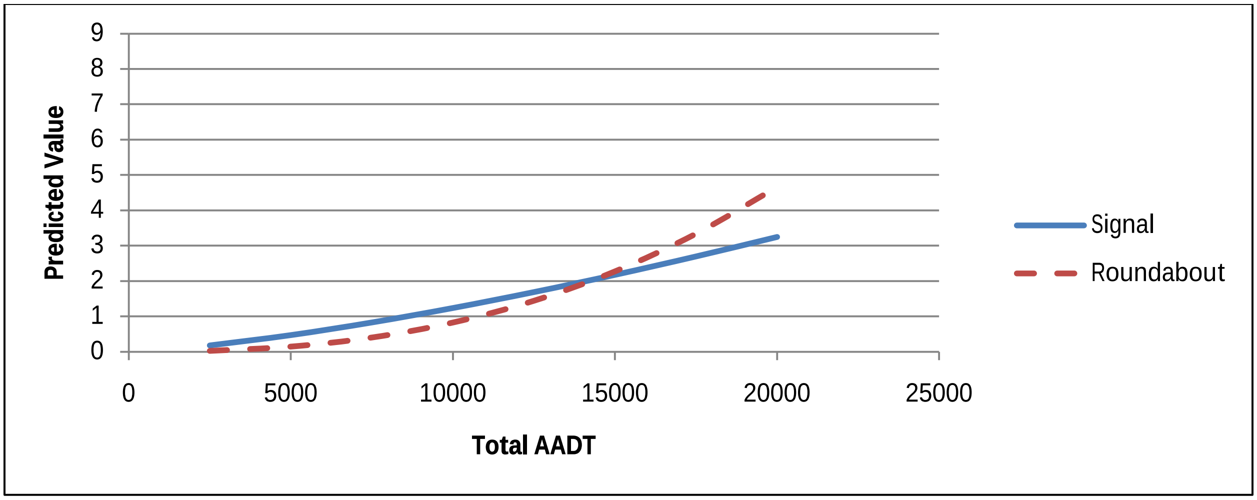

Figure B-8. Interaction Effect of Total Entering AADT – Combined Model

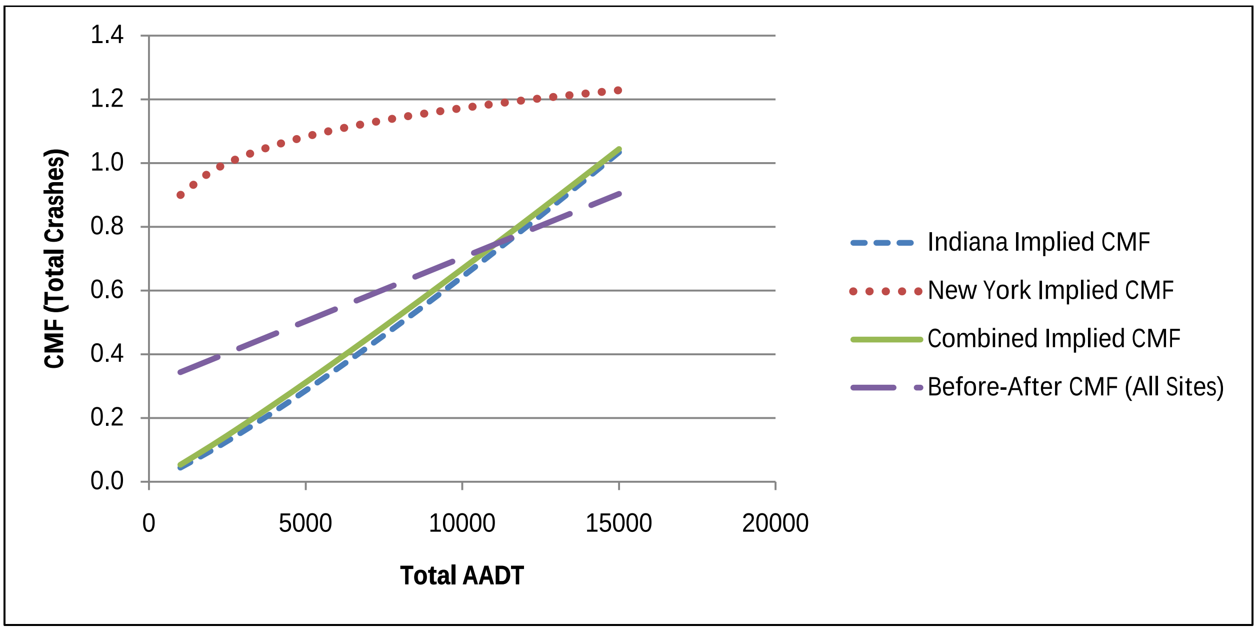

Figure B-9. Change in CMF Implied from Interaction of Total Entering AADT

The key point of the analysis with interaction is that a single CMF value may not accurately represent the safety impact of a treatment. Rather, it is conceivable that the CMF could vary over the range of values for specific variables (e.g., AADT). If there is evidence of a significant interaction, as is the case here, the relationship is better defined by a crash modification function (CMFunction) rather than a CMF. Below are the CMFunctions for total crashes for the three datasets.

- Indiana: CMFunction = exp(-11.2333 + 1.1716*LN(AADT)).
- New York (with insignificant interaction): CMFunction = exp(-0.8995+0.1149*LN(AADT)).
- New York (without interaction): CMF = exp(0.2329) = 1.262.
- Two States Combined: CMFunction = exp(-10.5486+1.1014*LN(AADT)).

Models by Severity

Separate models were developed for each state for PDO and injury crashes. These models were based on the same functional form and the same datasets used to develop the models for total crashes. The severity models were relatively consistent between the two states and, as such, were aggregated in a combined model. The models employ an indicator variable to identify the state (1 = Indiana). The results of the combined model are presented in Table B-12. At the bottom of the table, the CMF or CMFunction is presented for each severity, including the related sample size.

Table B-12. Combined Model for Individual Crash Types.

Based on 56 sites totaling 321 site-years, representing 1,631 PDO crashes and 290 injury crashes.

	Injury (without interaction)	**Injury (with interaction)**	**PDO**
Covariate	**Coefficient (Robust SE)**	**Coefficient (Robust SE)**	**Coefficient (Robust SE)**
LN(Total AADT)	1.6034 (0.3335)	1.5514 (0.3919)	1.3470 (0.2768)
RAB (1 = roundabout)	-0.5197 (0.1807)	-3.9068 (5.2618)	-11.8796 (4.4702)
State (1 = Indiana)	0.0735 (0.1614)	0.0898 (0.1605)	1.1137 (0.1736)
Interaction	Not Significant	0.3477 (0.5437)	1.2519 (0.4562)
Constant	-15.7455 (3.2707)	-15.2459 (3.8273)	-12.5347 (2.6995)
Alpha	0.3666 (0.1733)	0.3644 (0.1711)	0.3140 (0.0881)
Log pseudo likelihood	-381.8	-381.6	-760.4
Wald chi2	32.89	56.07	119.27
Prob > chi2	0.0000	0.0000	0.0000
CMF/CMFunction	0.595	exp(-3.9068 + 0.3477*LN(AADT))	exp(-11.8796 + 1.2519*LN(AADT))
Sample Size (crashes)	290	290	1,631

Roundabouts are expected to have fewer injury crashes when compared to similar signalized intersections (CMF = 0.595). This is consistent with expectations and previous studies of converting unsignalized intersections to roundabouts. This is also consistent with the empirical Bayes before-after analysis presented in this appendix (CMF = 0.342). (The smaller CMF is as typically expected from before-after studies on strictly logical and theoretical grounds.) With roundabouts, the crash types that are typically more severe (broadside and head-on crashes) are physically eliminated. The injury model with and

without interaction is shown for comparison purposes. Note that the interaction term is statistically insignificant at any reasonable level of significance. This indicates that the CMF is relatively constant over the range of AADTs included in this study. This is consistent with the results of the disaggregate analysis from the empirical Bayes before-after study where the trend line for the CMF is relatively flat over the range of AADTs.

The trend for PDO crashes is similar to the trend for total crashes. The implied CMF increases as total AADT increases, and actually exceeds 1.0 for AADTs greater than approximately 13,200 vehicles per day for the two states combined. This indicates that roundabouts may experience fewer PDOs than signals at lower AADTs, but may experience more PDOs than signals at higher AADTs.

Summary of Cross-Sectional Analyses

Table B-13 provides a summary of the CMFs/CMFunctions from the Indiana, New York, and combined cross-sectional analyses. It should be noted that the New York models are based on much smaller sample sizes than the Indiana models. Yet, the results are relatively consistent among the two states, at least in terms of the direction of effect. Both states show a general increase in total crashes at roundabouts, particularly for higher AADTs. A similar trend is shown for PDO crashes, where roundabouts are expected to experience fewer PDO crashes than signals, but only at lower AADTs. Injury crashes are expected to decrease where roundabouts are installed.

Table B-13. Summary of CMFs and CMFunctions from Cross-Sectional Analysis

State (crash severity)	Sample Size (crashes)	CMF/CMFunction
Indiana (total crashes)	1673	exp(-11.2333+1.1716*LN(AADT))
New York (total crashes)	248	1.262
Two States Combined (total crashes)	1921	exp(-10.5486+1.1014*LN(AADT))
Two States Combined (PDO crashes)	1631	exp(-11.8796+1.2519*LN(AADT))
Two States Combined (injury crashes)	290	0.595

DISCUSSION AND CONCLUSIONS

The data collected and analyzed for this study show a safety benefit for converting signalized intersections to roundabouts. The EB before-after analysis indicates a reduction in both total and injury crashes, with more consistent results for injury crashes for which the larger benefits were observed. The cross-sectional analysis, using negative binomial regression models, show a consistent increase in total and PDO crashes and a reduction in injury crashes for Indiana, New York, and the two states combined.

Several potential confounding factors were included in the analysis, including traffic volume, area type, number of approaches, and number of approach/roundabout lanes. The disaggregate analysis suggested that the safety benefit is larger for suburban than for urban conversions and for intersections with four approaches compared to those with three. Perhaps the most apparent and telling result of the disaggregate analysis is that the reduction in fatal and injury crashes is substantial and highly significant in all scenarios. This is a result of the basic configuration of a roundabout, where crossing-path and left-turn crashes are physically eliminated.

Traffic volume is a strong predictor of crash frequency and may also influence the effect of roundabouts. Interaction terms were explored during the cross-sectional analyses to further investigate the relationship between traffic volume and the effect of roundabouts. Interaction terms were significant in several of the cross-sectional models, indicating a change in effect for different volumes. Specifically, the safety benefit of roundabouts appears to decrease as traffic volumes increase, at least with respect to total crashes. This trend was supported by the disaggregate results from the before-after analysis. It is difficult to speculate why roundabouts may be less effective with respect to total crashes as traffic volumes increase, but one explanation is the increased friction (i.e., potential for conflict) within the roundabout. Other studies have also shown that signals perform better than roundabouts with respect to operations for traffic volumes that exceed a given threshold. The more important finding is that the effect on fatal and injury crashes is substantial and sustained, even as traffic volumes increase.

The current state-of-the-art for developing CMFs is the EB before-after method. While the study team employed the EB method to estimate the safety effects of converting signals to roundabouts, a cross-sectional analysis, employing negative binomial regression, was conducted to provide support for the EB analysis. The cross-sectional analyses included several variations of functional forms and subsets of data. The results of the cross-sectional analysis are relatively consistent with the EB analysis and corroborate the results, in particular the variation of the CMF with AADT for total crashes, and the lack of such variation for injury crashes. Thus, a CMFunction for total crashes is recommended as follows (based on the EB analysis):

$$\text{CMFunction} = 0.00004*\text{AADT} + 0.303$$

For injury (KAB) crashes, the recommended CMF = 0.342 (standard error = 0.058)

The applicable AADT range is 5,300 to 43,000 vehicles per day. Variations by setting (urban or suburban) and number of approaches should be considered with caution, at least subjectively, and on a case by case basis, using the results of the EB analysis. The EB analysis suggested that the safety benefit is larger for suburban than for urban conversions and for intersections with four approaches than for those with three.

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NCHRP PROJECT 17-35

APPENDIX C:

Safety Effects of Increasing the Clearance Interval

INTRODUCTION

Intersections account for a small portion of the total highway system; however, in 2008, approximately 2.31 million intersection-related crashes occurred. Intersection crashes accounted for 40 percent of all reported crashes and 22 percent (7,421) of all fatal crashes (NHTSA, 2008). The disproportionately high percentage of intersection crashes is not surprising because intersections present more points of conflict than non-intersection locations. Crashes at signalized intersections represent about 51 percent (1.18 million) of all intersection-related crashes, of which 2,511 involved a fatality in 2008 (NHTSA, 2008).

The National Cooperative Highway Research Program (NCHRP) 500 Series Report, Volume 12, identifies safety issues related to signalized intersections and potential countermeasures to address the safety issues (Antonucci et al., 2004). Specifically, the report identifies traffic control and operational improvements as one method to reduce the frequency and severity of intersection conflicts. Objective 17.2 A identifies several strategies to reduce the frequency and severity of intersection conflicts through traffic control and operational improvements. A specific strategy listed in this section is “optimize clearance intervals”.

This study investigates the safety effects of modifying the change interval at signalized intersections. The change interval is the time allocated for the yellow and all red phases for a given approach. The yellow phase allows drivers to either stop or proceed through the intersection depending on their proximity to the intersection; vehicles that are within one safe stopping distance of the stop bar should be able to enter the intersection legally during the yellow phase when traveling at the approach speed. The all red phase provides time for a vehicle to clear the intersection, assuming that it entered the intersection at the end of the yellow phase and is traveling at the approach speed.

Improper yellow and all red phases can lead to safety issues.

- Short yellow phase: yellow phases that are too short lead to a situation called the dilemma zone (as illustrated in Figure C- 1). The dilemma zone is the distance between points A and B in Figure C-1. Point A is the point along the approach where a driver traveling at the approach speed cannot stop at the stop line during the yellow phase (i.e., will enter the intersection illegally). Point B is the point along the approach where a driver traveling at the approach speed cannot clear the intersection prior to the end of yellow.
- Long yellow phase: yellow phases that are too long may result in inconsistent driver behavior (e.g., some drivers may stop during the yellow phase while other drivers try to beat the light), which can result in rear-end crashes.
- Short red phase: all red phases that are too short may not provide sufficient time for a vehicle to clear the intersection before the vehicles on the cross street begin to enter the intersection, which can result in angle type crashes.
- Long red phase: lengthy all red phases can lead to inconsistent driver behavior and may encourage drivers to enter the intersection during the all red phase if they are accustomed to the clearance time.

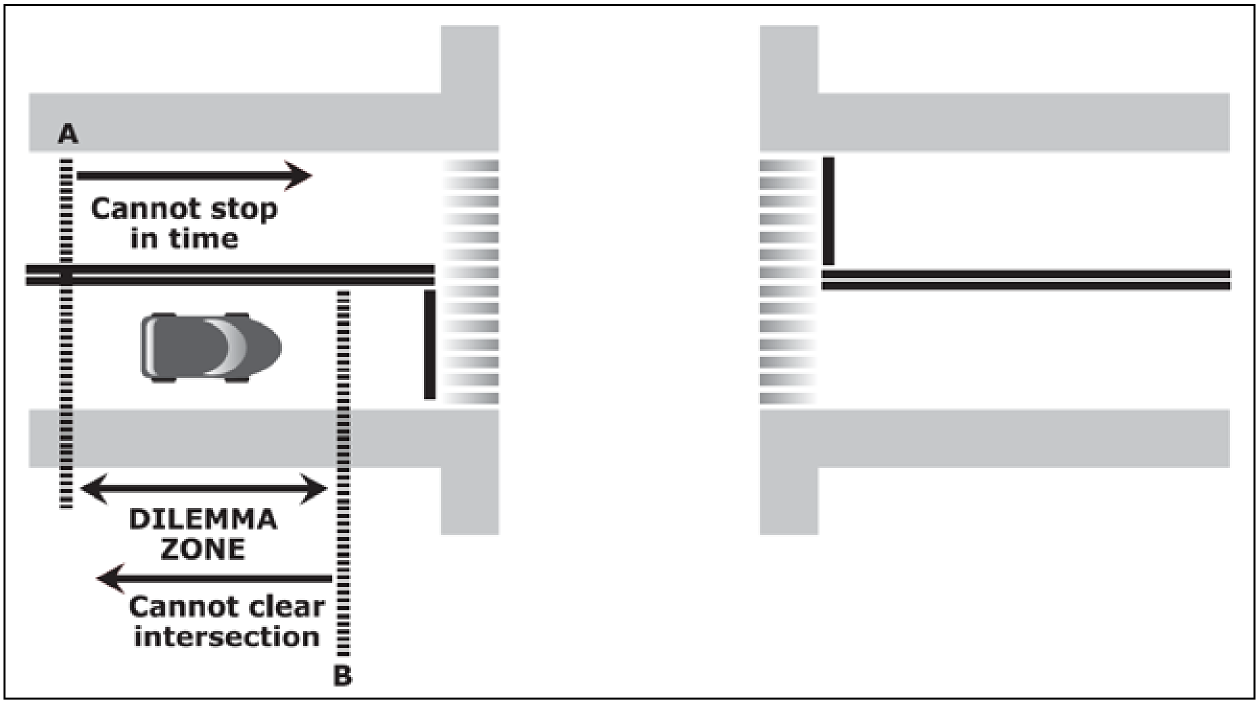

Figure C- 1. **Illustration of Dilemma Zone for a Signalized Intersection**

There is the potential to reduce the frequency and severity of crashes at signalized intersections by identifying an appropriate change interval and modifying the signal timing accordingly. Lengthening the yellow and/or all red time, however, will often result in an increase in the total cycle length. There is anecdotal evidence that suggests that widespread use of longer change intervals and cycle lengths may lead to red-light violations as drivers learn that the change interval is long (i.e., can accommodate late entry into the intersection and will result in longer delays if they stop). As such, establishment of a policy for determining appropriate change intervals is necessary to provide consistency within and among jurisdictions.

Identifying an appropriate change interval, however, is not a trivial task as there are multiple methods for calculating the yellow and all red phases as well as several assumptions that may need to be made during the calculations. The Institute of Transportation Engineers (ITE) presents one method for calculating the yellow phase (Equation C- 1) and all red phase (Equation C- 2) (ITE, 1985). The all red interval in Equation Y assumes that no pedestrians are present at the intersection; the equation is modified in situations where pedestrians are expected.

Equation C- 1

Equation C- 2

Where,

y	=	length of yellow phase (s).
t	=	driver reaction time (s).
S_{85}	=	85^{th} percentile speed of approaching vehicles, or speed limit (mi/h).

a = vehicle deceleration rate (ft/s^2).
G = approach grade (%).
g = acceleration due to gravity (32.2 ft/s^2).
ar = length of all red phase (s).
w = width of intersection; distance from departure stop line to the far side of the farthest conflicting traffic lane (ft).
L = standard vehicle length (ft).
S_{15} = 15^{th} percentile speed of approaching vehicles, or speed limit (mi/h).

The 2009 Manual on Uniform Traffic Control Devices (MUTCD) includes a new requirement that "durations of yellow change and red clearance intervals shall be determined using engineering practices" (FHWA, 2009). While specific standards are not set forth in the MUTCD for calculating yellow and all red times, there are references provided to ITE's "Traffic Control Devices Handbook" (Fullerton and Kell, 2001) and "Manual of Traffic Signal Design" (Kell and Fullerton, 1998) for calculating the yellow and all red interval. It should be noted that these are only provided as "support" in the MUTCD and are not "standards". As "guidance", the MUTCD indicates that the yellow interval should be a minimum of three seconds and a maximum of six seconds with the longer times reserved for approaches with higher speeds. The guidance also indicates that the all red interval should not exceed six seconds unless clearing a one-lane, two-way facility or when clearing an exceptionally wide intersection.

Other methods include "rule-of-thumb" or using a constant yellow and all red time for an entire jurisdiction. Utilizing a constant yellow and all red time may create a certain level of driver expectation, but there are two notable drawbacks to this method. First, agencies run the risk of creating a dilemma zone unless the timings are based on the intersections with the greatest approach speed and width within the jurisdiction. If approach speeds and intersection widths differ within the jurisdiction and the change interval is based on the worst case scenario, the agency has added unnecessary delay to intersections with lower approach speeds or narrower widths.

Literature Review

Several studies have investigated the safety effects of modifying the change interval over the past 50 years. In general, the research efforts can be categorized as one of the following: effects of change interval timing on red-light running and late exits; effects of change interval timing on crashes; or, crash effects associated with all-red intervals. This review identifies a number of relevant and available reports, providing a synthesis of research methods and the range of findings.

Effects of Change Interval Timing on Red-light Running and Late Exits

Several studies have examined the effect of increasing the duration of the yellow clearance interval to a value calculated by ITE guidelines on the occurrence of red-light running. Retting et al. (2008) evaluated the effect of lengthening the yellow duration on red-light running at six intersection approaches in Philadelphia, PA. The study concluded that increasing the duration of the yellow clearance interval by approximately one second reduced red-light running by approximately 36 percent. A similar study conducted by Retting and Greene (1997) at 20 intersection approaches in New York found that increasing the yellow duration decreased the likelihood of red-light running; however, increasing the all-red change interval duration showed no change. The study also examined the effect on late exists, indicating that increasing the yellow or all-red interval duration reduced the number of late exits. Studies by Bonneson and Zimmerman (2004), Harders (1981), Munro and Marshall (1982), Van der Horst and Wilmink (1986), and Wortman et al. (1985) also found that increasing the yellow interval by approximately one second reduced the occurrence of red-light running. Croft and Traudinger (1983), Schattler et al. (2002),

and Schattler (2003) also indicated that red-light running events decreased after modifications to change interval durations had been made. A study by Datta et al. (1999), comparing sites with all-red change interval versus sites without, concluded that red-light running was less frequent with the presence of all-red intervals.

Effects of Change Interval Timing on Crashes

Reductions in the number of crashes have been shown to be associated with modifications to the change interval. Specifically, past studies have shown favorable safety effects when the change interval is modified to better match the yellow and all red times based on the ITE procedure as detailed below and summarized in Table C- 1. Pilko and Bared (2010) concluded that the greatest intersection safety benefits are achieved when yellow and all-red interval durations are timed according to ITE guidelines. Retting et al. (2002) evaluated 40 sites requiring increases in the duration of change interval timings based on ITE guidelines compared to 56 control sites. The study results showed a statistically significant 12 percent reduction in all injury crashes at the experimental sites compared to the control sites. The same study showed an insignificant effect on angle and rear-end crashes, but noted a 37-percent reduction in crashes involving pedestrians or bicyclists. Abraham (2006) indicated that timing the yellow and all-red intervals in accordance with ITE guidelines at 20 high-crash intersections reduced the total number of crashes at a majority of the sites. Zador et al. (1985) performed a cluster analysis of 91 intersections, concluding that those with the least adequate change interval durations based on ITE guidelines exhibited higher crash rates than those with longer change intervals. A study by Schattler et al. (2003) examined revising clearance interval timings at three sites per ITE guidelines, resulting in shorter yellow and longer all-red intervals. The study indicated a decline in the average annual total number of crashes and a small reduction in the average annual number of rear-end and right angle crashes.

Table C- 1. Summary of Safety Effects of Change Intervals

Crash Type	Study	Treatment	Sample Size	Study Method	Results
Total	Schattler et al. (2003)	Adjust clearance intervals per ITE procedure	3 intersections	Before-after	Reduction in average crash frequency
	Zador et al. (1985)	Adjust clearance intervals per ITE procedure	91 intersections	Cluster analysis	Higher rates at sites with most inadequate timing compared to ITE guidelines
Injury	Retting et al. (2002)	Adjust clearance intervals per ITE procedure	40 treatment and 56 control intersections	Before-after	12 percent reduction
Red-light running crashes	Abraham (2006)	Adjust clearance intervals per ITE procedure	20 intersections	Before-after	Reduction in crashes at a majority of sites
	Pilko and Bared (2010)	Adjust clearance intervals per ITE procedure	Not Reported	Not Reported	Maximum benefit when yellow and all-red match ITE guidelines
Angle	Retting et al. (2002)	Adjust clearance intervals per ITE procedure	40 treatment and 56 control intersections	Before-after	Insignificant effect
	Schattler et al. (2003)	Adjust clearance intervals per ITE procedure	3 intersections	Before-after	Slight reduction in average crash frequency

Crash Type	Study	Treatment	Sample Size	Study Method	Results
Rear-end	Retting et al. (2002)	Adjust clearance intervals per ITE procedure	40 treatment and 56 control intersections	Before-after	Insignificant effect
	Schattler et al. (2003)	Adjust clearance intervals per ITE procedure	3 intersections	Before-after	Slight reduction in average crash frequency
Pedestrian and bicycle	Retting et al. (2002)	Adjust clearance intervals per ITE procedure	40 treatment and 56 control intersections	Before-after	37 percent reduction

Crash Effects Associated with All-Red Intervals

Research on the crash effects associated with the installation of all-red change intervals has shown a variety of results as detailed below and summarized in Table C- 2. Conradson and Bunker (1972) evaluated 17 intersections in Michigan where all-red intervals were installed. A before-after analysis of 814 crashes indicated a statistically significant 10 percent reduction in total crashes and 28 percent reduction in injury crashes. Additionally, right angle crashes decreased 46 percent; however, rear end crashes increased 36 percent. Benioff et al. (1980) concluded that the installation of an all-red interval reduced several crash types of varying severities. A study utilizing three statistical tests by Roper et al. (1991) of 25 intersections in Indiana showed that the installation of all-red intervals had no significant impact on the occurrence of crashes. Similarly, Souleyrette et al. (2004) collected data at 104 signals in Minneapolis, MN where all-red intervals were not installed. A cross-sectional analysis showed no intersection safety benefit of installing all-red change intervals. A before-after analysis indicated a 25 percent immediate reduction in several crash types; however, no long-term reductions were noted.

Researchers have also attempted to evaluate the time required to complete a left-turn movement in comparison to all-red interval durations. Yu et al. (2004) found that sites with little variance between the time required for a left-turn vehicle to complete the maneuver and the duration of the all-red change interval experienced fewer left-turn crashes than sites with a larger variance between the two times.

Table C- 2. Summary of Safety Effects of All Red Intervals

Crash Type	Study	Treatment	Sample Size	Study Method	Results
Total	Conradson and Bunker (1972)	Install all-red interval	17 intersections	Before-after	10 percent reduction
	Roper et al. (1991)	Install all-red intervals	25 intersections	Wilcoxon Signed Ranked test; Student t-test; Chi-square test	No significant impact
Intersection related	Benioff et al. (1980)	Install all-red intervals	3 relevant studies	Comprehensive literature review	Significant reductions in total, property damage only, angle, and “other”. Some reductions in rear-end crashes.
Injury	Conradson and Bunker (1972)	Install all-red interval	17 intersections	Before-after	28 percent reduction
Angle	Conradson and Bunker (1972)	Install all-red interval	17 intersections	Before-after	46 percent reduction
Rear-end	Conradson and Bunker (1972)	Install all-red interval	17 intersections	Before-after	36 percent increase

Crash Type	Study	Treatment	Sample Size	Study Method	Results
Head-on Rear-end Angle Left-turn Right-turn Sideswipe	Souleyrette et al. (2004)	Install all-red intervals	104 intersections	Cross-sectional	No change
	Souleyrette et al. (2004)	Install all-red intervals	104 intersections	Before-after	25 percent short-term reduction, but no long-term reduction
Left-turn	Yu et al. (2004)	Examination of all-red duration versus time to complete left-turn	21 intersections	Direct comparison	Sites with small variance between all-red duration and actual turn time experienced fewer crashes than sites with large variance

Optimizing or adjusting the change interval is relatively low-cost and the timeframe for implementation is short-term. As identified in NCHRP 500 Series Report, Volume 12, the following are key issues to consider for this strategy (Antonucci et al., 2004):

- A change interval should not be so long as to encourage disrespect in drivers for the interval, thereby contributing to red-light running and even more severe crashes.
- A change interval should not be so short as to violate driver expectancy regarding the length of the interval, resulting in abrupt stops and possible rear-end crashes.

OBJECTIVES

One objective was to estimate the general safety effectiveness of adjusting the change interval at signalized intersections, as measured by crash frequency and severity. Target crash types included:

- All crashes (all types and severities).
- Fatal and injury crashes (all crash types).
- Angle crashes (all severities).
- Rear-end crashes (all severities).

The other key objective was to conduct a disaggregate analysis to identify circumstances (e.g., geometric and traffic conditions) under which change interval modifications may be more safety effective. Meeting these objectives placed some special requirements on the data collection and analysis tasks, including the need to:

- Consider a supplemental cross-sectional study if data were insufficient for the preferred before-after analysis.
- Select a large enough sample size to detect, with statistical significance, what may be small changes in safety for target crash types.
- Carefully select comparison or reference sites.
- Properly account for traffic volume changes and the possibility of regression-to-the-mean.
- Pool data from multiple jurisdictions to improve reliability of the results and facilitate broader applicability of the research products.

Roadway, traffic volume, signal, and crash data were acquired for several local jurisdictions in California, Maryland, and North Carolina to facilitate the analysis. Treatment and reference sites were identified where possible. The states provided crash data and information related to the signal timing.

METHODOLOGY

The treatment of interest is modifying the change interval at signalized intersections. As part of a recent project, the study team worked with several states to identify signalized intersections where red light cameras were installed in the recent past. As part of that study, the study team identified untreated signalized intersections to be used as a reference group in the empirical Bayes (EB) analysis. Detailed data were collected for both treated and untreated sites. These data were revisited as part of this study.

The desired method for this evaluation is the state-of-the-art EB methodology for observational before-after studies. However, several sites were identified where the signal timing did not change over the study period. As such, a cross-sectional analysis was also conducted to compare the safety performance of signalized intersections that had various combinations of yellow and all red phases. The two methods are described below.

Empirical Bayes Analysis

The methodology applied was the EB before-after study, following the procedure outlined in Hauer (1997). The advantages of the EB approach are that it:

- Properly accounts for regression-to-the-mean.
- Overcomes the difficulties of using crash rates in normalizing for volume differences between the before and after periods.
- Reduces the level of uncertainty in the estimates of safety effect.
- Provides a foundation for developing guidelines for estimating the likely safety consequences of contemplated installations.
- Properly accounts for differences in crash experience and reporting practice in amalgamating data and results from diverse jurisdictions.
- Avoids the difficulties of conventional treatment-comparison experimental designs caused by possible spillover and/or migration effects to natural comparison groups.

In an EB evaluation, the change in safety for a given crash type at a treated intersection is given by Equation C- 3.

$$B - A$$

Equation C- 3

where *B* is the expected number of crashes that “would have occurred” in the after period without the treatment and *A* is the actual number of reported crashes in the after period.

The count of crashes before treatment by itself is not a good estimate of *B* due to changes in safety that may result from changes in traffic volume, regression-to-the-mean, and trends in crash reporting and other factors, a reality that has now gained common acceptance. Instead, *B* is estimated from an EB procedure in which a safety performance function (SPF) is used to first estimate the number of crashes that would be expected in each year of the before period at locations with traffic volumes and other characteristics similar to the treated site being analyzed. The sum of these annual SPF estimates (*P*) is then combined with the count of observed crashes (*x*) in the before period at the treatment site to obtain an estimate of the expected number of crashes (*m*) before the treatment. The estimate of *m* is given by Equation C- 4.

$$m = w(P) + (1-w)(x)$$

Equation C- 4

The weight w is estimated from Equation C- 5.

$$w = 1/(1 + kP)$$

Equation C- 5

where k is the over-dispersion parameter of the negative binomial distribution that is assumed for the crash counts used in estimating the SPF. The value of k is estimated from the SPF calibration process with the use of a maximum likelihood procedure.

A factor is then applied to m to account for the length of the after period and differences in traffic volumes between the before and after periods. This factor is the sum of the annual SPF predictions for the after period divided by P, the sum of these predictions for the before period. The result, after applying this factor, is an estimate of B. The procedure also produces an estimate of the variance of B, the expected number of crashes that would have occurred in the after period without the treatment.

The estimate of B is then summed over all sites in a treatment group of interest (to obtain B_{sum}) and compared with the count of crashes during the after period in that group (A_{sum}). The variance of B is also summed over all sections in the group of interest.

The index of safety effectiveness (θ) is estimated by Equation C- 6.

$$\theta = \frac{A_{sum}/B_{sum}}{1 + \left(Var(B_{sum}) / B_{sum}^2 \right)}$$

Equation C- 6

The index of effectiveness is equivalent to the crash modification factor. The standard deviation of θ is given by Equation C- 7.

$$StDev(\theta) = \sqrt{\frac{\theta^2 \left(\frac{A_{sum}}{A_{sum}^2} + \frac{B_{sum}}{B_{sum}^2} \right)}{\left(1 + \frac{Var(B_{sum})}{B_{sum}^2} \right)^2}}$$

Equation C- 7

The percent change in crashes is $100(1-\theta)$; thus a value of $\theta = 0.7$ with a standard deviation of 0.12 indicates a 30 percent reduction in crashes with a standard deviation of 12 percent.

Cross-Sectional Analysis

A cross-sectional analysis was employed to supplement the results of the EB analysis. This was necessary because several sites did not have changes to the yellow or all red phases during the study period. Also, some of the sites with modifications to the change interval had limited after period data. A secondary objective of this supplemental investigation was to examine the comparability of results of before-after

and cross-section studies, a subject of topical interest in CMF development, for which there is little research.

Negative binomial regression is a common method for developing relationships between crashes and roadway characteristics (e.g., traffic volume, area type, etc). The negative binomial regression model is applied in this evaluation framework to estimate the safety effects of various change intervals at signalized intersections. The general functional form of the model assumed for this analysis is shown in Equation C- 8.

$$Crashes\ /\ year = \exp\left(\alpha + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_n X_n\right)$$

Equation C- 8

Where,
α and β_1 - β_n = parameters estimated in the model calibration process.
X_1 - X_n = covariates included in the model.

Average daily traffic (ADT) and variables representing the change interval were included in every cross-sectional model developed for this evaluation. Once the models are estimated, the coefficient for the variable of interest may be exponentiated to estimate the CMF. Additional variables were considered based on available data and included in the models if the following conditions were met:

a) The variable significantly improved the model.
b) The effect of the variable was intuitive (e.g., crashes increase as number of approaches increases).

The following additional variables were considered in the model development. These variables entered the model form as adjustments to the base value of α in Equation 1. The base value of α was estimated for a particular baseline condition (e.g., three-legged, permissive phasing). When the condition of the intersection is anything other than the baseline, an adjustment was applied to the base value of α. The parameter values (β*'s*) indicate the magnitude and direction of the adjustment to the base α value.

- Number of intersection approaches (3-legged/4-legged indicator).
- Signalized intersection phasing (permissive/protected/protected-permissive indicators).
- Area type (all sites were urban in nature).
- Posted speed on major and minor approach.
- Number of approach lanes.
- Width of approach lanes.
- Presence and width of median on approach.

Several of the variables listed above are used directly to calculate the length of yellow and all red phases based on the ITE procedure. Specifically, the yellow time is based on speed limit and the all red phase is based on speed limit and intersection width (as shown previously in Equation C- 1 and Equation C- 2). Rather than including all variables in the model, it was possible to calculate the "desired" yellow and all red phases per the ITE procedure. In this way, the actual yellow and all red times could be compared to the "desired" ITE times to investigate potential differences in safety effectiveness.

The ITE procedure was used to calculate the approximate "desired" yellow and all red times (from Equation C- 1 and Equation C- 2). The calculation of all red time is approximate because assumptions were made to estimate intersection width. Specifically, the functional intersection width for use in Equation C- 2 depends not only on the width of the cross-street, but also depends on the presence of

pedestrians and the location of stop bars and crosswalks. Data related to pedestrians, pedestrian facilities, and stop bar location were not available in the database. Instead, the functional width of the intersection was based on the maximum width of the cross-street approaches. Approach width was estimated from the number of lanes and median width on the cross-street. Lane width was not available for all approaches so a 12-ft lane width was assumed for all lanes.

The results from the ITE procedure were then compared to the actual yellow and all red times. This was captured in the model through various methods as follows:

- Difference between ITE yellow time and actual yellow time: a continuous variable calculated as the actual yellow minus the ITE yellow time.
- Difference between ITE all red time and actual all red time: a continuous variable calculated as the actual all red minus the ITE all red time.
- Difference between total ITE change interval and actual change interval: a continuous variable calculated as the actual total change interval (actual yellow plus all red) minus the ITE change interval (ITE yellow plus all red).
- Indicator variables: several indicator variables were created to represent various situations based on the comparison of ITE and actual yellow and all red times.
    - Greater than, less than, or equal to ITE yellow: three indicator variables were created to indicate if the actual yellow time is greater than, less than, or equal to the ITE yellow time. It should be noted that yellow times set from the ITE procedure may be rounded up to be conservative or to avoid decimals. As such, an actual yellow time was considered equal to the ITE yellow time if the difference was greater than zero, but less than 1.0 second (difference calculated as actual yellow minus the ITE yellow).
    - Greater than, less than, or equal to ITE all red: three indicator variables were created to indicate if the actual all red time is greater than, less than, or equal to the ITE all red time. It should be noted that all red times set from the ITE procedure may be rounded up in practice to be conservative. As such, an actual all red time was considered equal to the ITE all red time if the difference was greater than zero, but less than 0.5 seconds (difference calculated as actual all red minus the ITE all red).
    - Both yellow and all red match ITE: an indicator was created to indicate intersections where both the yellow and all red times "match" the ITE yellow and all red times. The actual timing "matches" the ITE timing when the actual yellow and all red times are equal to the ITE yellow and all red times as defined in the previous two bullets.
    - Total yellow plus all red matches ITE: an indicator was created to indicate intersections where the total change interval (yellow plus all red) "matches" the ITE change interval (yellow plus all red). The actual change interval "matches" the ITE change interval if the difference is less than 0.5 seconds.

Common concerns related to cross-sectional analyses include:

- Misspecification of model functional form.
- Confounding effects.
- Interaction effects.
- Inconsistency among results from different studies of the same treatment.

The following techniques were employed to address the common concerns associated with cross-sectional studies.

- Several functional forms were explored to test the sensitivity of the coefficients.
- Several covariates were considered for inclusion in the model.
- Interaction effects were tested and included as necessary.

DATA COLLECTION

This section provides a summary of the databases developed for each jurisdiction. Data were obtained from a previous study that collected detailed information related to the signal timing, traffic volume, roadway geometry, and crash history at signalized intersections in several jurisdictions. The sites include data for two types of signalized intersections, 1) signalized intersections where the change interval was modified during the study period, and 2) signalized intersections where the change interval was not modified during the study period. If there were major changes to the geometry or operations during the study period, the sites were excluded. Examples of major changes include signal upgrade or modification, changes to phasing, lane use change, widening, and installation of lighting. Minor signal modifications such as relocating detectors were not counted as major changes and the sites could still be included.

Table C- 3 describes how the crash types analyzed were defined. Crash definitions are not identical between the jurisdictions due to differences in crash reporting. Note that "perpendicular approaches" was defined using the compass directions of travel for each involved vehicle, a variable that was present in the data for all jurisdictions.

Table C- 3. Definitions Used in Analysis

Total	San Diego and San Francisco, CA	All crashes (other than rear-end) at or within 20 feet of intersection; rear-end crashes within 150 feet.
	Howard County and Montgomery County, MD	All crashes within 158 feet and identified as "intersection" or "intersection-related."
Injury-related	San Diego and San Francisco, CA	All "intersection-related" crashes that resulted in a K, A, B, or C-type injury as defined by the KABCO scale.
	Howard County and Montgomery County, MD	All "intersection-related" crashes that resulted in a K, A, B, or C-type injury as defined by the KABCO scale.
Angle	San Diego and San Francisco, CA	Broadside, head-on, or sideswipe crashes where vehicles approach intersection from perpendicular directions.
	Howard County and Montgomery County, MD	Any category of head-on, head-on left-turn, opposite direction sideswipe, straight movement angle, angle meets right-turn, angle meets left-turn, or angle meets left head-on where vehicles approach intersection from perpendicular directions.
Rear-end	San Diego and San Francisco, CA	All rear-end crashes within 150 feet of intersection.
	Howard County and Montgomery County, MD	All rear-end crashes within 158 feet and identified as "intersection" or "intersection-related."

The following summarizes the data collection efforts in each jurisdiction. The primary jurisdictions include Howard County (MD), Montgomery County (MD), San Francisco (CA), and San Diego (CA). While data were available for additional jurisdictions, the data were incomplete for one or more of variables of interest (e.g., signal timing).

Howard County, Maryland

Crash Data

Crash data were obtained from the National Study Center for Trauma and MS at the University of Maryland – Baltimore. The Center maintains Maryland crash data as part of the CODES project. Since

there is no “milepost” system in either the original intersection inventory data or these crash files, the crash data were linked to the pertinent intersection using the names of the crossing streets. Searches for alternative spellings of street names were conducted in this data linking effort.

Volume Data

The primary source of volume data for the study intersections were turning movement counts maintained by the Howard County Traffic Division. Partial day turning movement counts were converted to average daily traffic (ADT) using factors obtained from the Maryland State Highway Administration (MDSHA). If a turning movement count was not available for an intersection, midblock volumes were used. For intersections on state-maintained roadways, both turning movement counts and midblock ADTs maintained by MDSHA were used to estimate traffic volumes. A more detailed description of the conversion process is provided following the summary of data collection efforts.

Geometric Data

Howard County signal plans were used to determine the geometry of signalized intersections at non-state-maintained intersections. The signal plans were provided to the project team as image files. When the images were scanned in, the scale was not retained. Therefore, information on pavement and median widths was difficult to determine. Additionally, some signal plans were outdated. Aerial photographs were used to supplement this information. Changes in geometry at these signalized intersections were determined from superseded signal plans, change notations on the current signal plans, or from a log maintained by Howard County Traffic Division staff.

MDSHA signal plans were used to determine the geometry at state-maintained signalized intersections. These signal plans were detailed and to scale. Changes in geometry were determined from the change notations on these signal plans.

Operations Data

Signal phasing data were obtained from files maintained by the Howard County Traffic Division for non-state-maintained intersections. These files include information on changes in the phasing. For state-maintained intersections, present and historical signal timing cards were obtained from MDSHA. Signal modifications (e.g., installation of LEDs or backplates) were obtained from a detailed log maintained by a member of Howard County Traffic Division staff.

Montgomery County, Maryland

Crash Data

As with Howard County, crash data were obtained from the National Study Center for Trauma and MS at the University of Maryland – Baltimore. The Center maintains Maryland crash data as part of the CODES project. The crashes were referenced to intersections by street names.

Volume Data

The Montgomery County Department of Public Works maintains a database of intersection ADTs. This database was provided to the project staff along with photocopies of various turning movement counts. Approach volume data were obtained from the database and the turning movement counts. Partial day turning movement counts were extrapolated to ADTs using factors provided by Montgomery County.

Geometric Data

Signal plans were primarily used to determine the geometry of signalized intersections. Aerial photographs were used to supplement these data.

Operations Data

Montgomery County maintains two files for every signal. One file contains information on the current signal phasing at the intersection. The other file contains information on historical signal phasing. The project staff used the two files to determine the current signal timing and any changes in signal timing during the study period.

San Diego, California

Crash Data

Crash data were obtained from the California Highway Patrol's database, SWITRS. Since there is no "milepost" system available, the crash data were manually referenced to the pertinent intersection by the names of the primary street and cross street. These locations were then matched with the study intersections.

Volume Data

The city of San Diego maintains a fairly comprehensive database of midblock counts throughout the city. This database is based on machine counts. These midblock counts were used as approach volumes to the study intersections. These data were supplemented with photocopies of turning movement counts.

Geometric Data

The San Diego Traffic Engineering Division had a limited number of signal plans on file, many of which were outdated. When possible, these signal plans were used to determine the geometry at signalized intersections. This information was supplemented with aerial photographs. Information on changes in geometry was obtained, when available, from the change notations on signal plans and copies of work orders at those intersections. The work orders documented any significant changes at the intersections during the study period.

Operations Data

Signal timing information was obtained from the Traffic Engineering Division's current and historical signal timing files for signalized intersections.

San Francisco, California

Crash Data

As with San Diego, crash data for the study intersections were obtained from the California Highway Patrol's database, SWITRS. The crash data were referenced to the pertinent intersections by project staff using the names of the primary street and cross street.

Volume Data

Volume data were obtained from the San Francisco Department of Parking and Traffic (DPT), which maintains files of ADT for some locations. These ADTs were often collected at midblock locations and were the primary source of volume data. These ADTs were supplemented with photocopies of intersection turning movement counts. DPT also provided an electronic copy of ADTs collected by the State.

Geometric Data

DPT provided the project team with electronic computer aided design files of some signal plans and pavement marking drawings. The geometry for a small proportion of intersections was available from these electronic files. The project team also photocopied some rudimentary signal plans from DPT. These signal plans illustrate the signal phasing, but not necessarily the lane configurations. Because of the detailed information needed for the roadway geometry, the project team sought another source of data. The San Francisco Department of Telecommunications and Information Services maintains the Enterprise Geographic Information Systems (GIS). The GIS system contains detailed aerial photographs of the city. These aerial photographs were used to obtain geometric data for the majority of the study intersections. Changes in geometry during the study period are not discernable from aerial photographs. Therefore, the majority of the study intersections did not have information on changes in geometry.

Operations Data

Signal timing information was obtained from the DPT's current and historical signal timing files for signalized intersections. DPT maintains in depth records of operational changes at signalized intersections. These were reviewed by the project team. All relevant changes were photocopied.

DATA PREPARATION

Volume Data Extrapolation

Whenever possible, the project team selected intersections for this study that had sufficient data available. In some cities, however, there were not a sufficient number of intersections that had traffic volume counts in both the before and after periods. Often, this meant that project staff had to locate other data sources or extrapolate in order to obtain the necessary volume data for the study. Extrapolation took on different forms, depending on the availability of the data as discussed below.

Converting Partial Day Counts to Average Daily Traffic (ADT)

The standard convention for reporting the volume of a roadway is by the average daily traffic (ADT). The ADT represents the typical daily traffic volumes on a roadway over the course of 24 hours. Most counts used for this study were for periods less than 24 hours. It is not economical to count traffic volumes for long periods of time and as such, most ADTs are based on partial day counts. A factor is applied to the partial day count to obtain the ADT. Some agencies have developed their own factors for converting partial day counts to ADT. These factors represent the unique traffic characteristics of the region. When available, local factors were used to convert the volumes to ADT. In the absence of local data, state or national factors were used.

The turning movement counts, identified for this study, were generally counted for 11 to 13 hours. Traffic volumes from longer count durations are more desirable because they are more representative of

the actual ADT. The project team made every attempt to use volumes from long duration counts. However, at some intersections, the only volumes available were from short duration counts (i.e., 4 to 6 hours).

Midblock Volumes

When available, traffic counts collected at the intersection (e.g., turning movement counts) were used for approach volumes. However, turning movement counts or similar were not available for all intersections in all cities. Some jurisdictions collect traffic volumes on roadway segments. These volumes are usually collected at the midblock of the roadway and may be directional or non-directional. In the absence of turning movement counts, these midblock volumes were used to estimate approach volumes. The primary assumption was that the volume remained relatively constant between the data collection point and the intersection (i.e., any traffic that left or entered the roadway between the collection point and the intersection was negligible). For non-directional midblock volumes, the project team assumed that the directional split was 50 percent and used one half of the midblock volume as the approach volume.

Mirroring Approach Volumes

Generally, the ADT from opposing directions of the major movement are similar and opposing directions of the minor movement are similar. This is not true, however, at intersections with a large percentage of turning movements. At some intersections, one or more approach volumes were not available during the study period. In the absence of one or more approach volumes, volumes were estimated by assuming that the opposing approach volumes were equal. This is referred to as “mirroring” approach volumes.

Estimating Volumes for Missing Years

Traffic counts were available for most sites for at least one year per approach during the study period; however, these data were not available for all years of the study period for each site. As such, it was necessary to estimate the traffic counts for those years that were missing. In cases where traffic counts were available for multiple years, the missing years were estimated by simple linear interpolation for those years in between the available data. For the years on either end of the available data, a constant growth factor of 1.0 was assumed. This was completed for each site by approach before the data were aggregated for use in the analysis. For example, the study period covers years 1993 to 2002 and for site #1 the following traffic counts were available.

Approach	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002
NB	-	8500	8500	8500	-	-	-	7700	-	7700
EB	4268	4550	4150	4250	-	3708	3675	3675	4100	4100
SB	8524	8500	8500	8500	-	8565	7204	7700	7600	7700
WB	-	4550	4150	4250	-	-	-	3675	-	4100

For the northbound approach, traffic counts were missing for 1993, 1997-1999, and 2001. The traffic count for 1993 was estimated from the available 1994 data, assuming a constant growth factor of 1.0, while the traffic counts for 1997-1999 and 2001 were estimated from linear interpolation as illustrated below.

1993 traffic count	=	1994 traffic count * 1.0
	=	8500
1997 traffic count	=	1996 traffic count + [2000 traffic count – 1996 traffic count]/4
	=	8500 + [7700-8500]/4
	=	8300

Before-After Data

The data were structured differently and included different subsets of sites for the two analyses (before-after and cross-sectional). Sites were included in the before-after analysis if distinct changes in the yellow and/or all red phase occurred during the study period. Summary statistics for the treated sites used in the before-after study are provided in Table C- 4.

Table C- 4. Sites with Change Interval Modifications during the Study Period

Variable	Mean	Minimum	Maximum
Years before	5.7	3	8
Years after	1.8	1	4
Total crashes/site-year before	5.9	0	30
Total crashes /site-year after	9.6	0	88
Injury crashes/site-year before	4.2	0	17
Injury crashes/site-year after	4.3	0	26
Rear-end crashes/site-year before	2.0	0	13
Rear-end crashes/site-year after	3.5	0	38
Angle crashes/site-year before	2.0	0	12
Angle crashes/site-year after	2.9	0	26
Major ADT before	17,417	5,950	31,600
Major ADT after	16,721	6,503	32,810
Minor ADT before	8,484	2,650	20,225
Minor ADT after	7,707	2,650	19,200
Number of sites = 31	Howard County, MD = 2 sites Montgomery County, MD = 6 sites San Diego, CA = 16 sites San Francisco, CA = 7 sites		

Data for signalized intersections, similar to those where the change interval was modified, were sought for use in developing the SPFs required for the EB methodology. Untreated reference sites (i.e., sites where no modifications were made to the change interval during the study period) were identified in each jurisdiction. Crash, traffic volume, and roadway data were obtained for the reference group. The data were used to directly calibrate SPFs for total, injury, rear-end, and angle crashes separately for each jurisdiction. The SPFs used for this evaluation are summarized below:

SPFs for Howard County, MD

Total/year = $\exp(-9.179)(\text{MajorADT})^{0.740}(\text{MinorADT})^{0.494}$, k=0.482
Injury/year = $\exp(-9.348)(\text{MajorADT})^{0.696}(\text{MinorADT})^{0.396}$, k=0.685
Rear-end/year = $\exp(-14.123)(\text{MajorADT})^{1.236}(\text{MinorADT})^{0.427}$, k=0.756
Angle/year = $\exp(-7.768)(\text{TotalADT})^{0.805}$, k=0.911

SPFs for Montgomery County, MD

Total/year = $\exp(-8.431 - 0.646*\text{Protected})(\text{MajorADT})^{0.640}(\text{MinorADT})^{0.492}$, k=0.397
Injury/year = $\exp(-9.026 - 0.676*\text{Protected})(\text{MajorADT})^{0.583}(\text{MinorADT})^{0.529}$, k=0.502
Rear-end/year = $\exp(-8.914 + 0.022*\text{MaxSpeed})(\text{MajorADT})^{0.707}(\text{MinorADT})^{0.233}$, k=0.458
Angle/year = $\exp(-5.275 + 0.679*\text{Fourleg})(\text{TotalADT})^{0.507}$, k=0.656

SPFs for San Diego and San Francisco, CA

Total/year = $\exp(-9.028 + 0.225*SF - 0.522*Speed40)(TotalADT)^{1.023}$, k=0.272
Injury/year = $\exp(-9.002 + 0.046*SF - 0.527*Speed40)(TotalADT)^{1.018}$, k=0.281
Rear-end/year = $\exp(-11.221 – 0.223*SF)(MajorADT)^{0.680}(MinorADT)^{0.551}$, k=0.228
Angle/year = $\exp(-9.066 + 0.097*SF – 0.573*Protected - 0.977*Speed40)(TotalADT)^{0.951}$, k=0.582

Where,
Total/year = predicted number of total intersection crashes per year.
Injury/year = predicted number of injury-related intersection crashes per year.
Rear-end/year = predicted number of rear-end intersection crashes per year.
Angle/year = predicted number of angle intersection crashes per year.
MajorADT = average daily traffic on major road.
MinorADT = average daily traffic on minor road.
TotalADT = total entering volume.
Protected = indicator for left-turn phasing on major road (1 = fully protected; 0 otherwise).
MaxSpeed = maximum posted approach speed (mph).
Fourleg = indicator for number of approaches (1 = 4-legged; 0 otherwise).
SF = indicator for jurisdiction in California (1 = San Francisco; 0 otherwise).
Speed40 = indicator for maximum posted approach speed (1 = 40+ mph; 0 otherwise).
k = over-dispersion parameter of the model.

Cross-Sectional Data

The datasets used for the cross-sectional analyses are described below. Each observation in the database represents a site-year (i.e., when multiple years were available for a site, an observation was created for each year). Each site-year reflects the specific characteristics during the year (e.g., yellow and all red time, traffic volume, etc). The treatment site data from the before-after analysis were also included in the cross-sectional analysis. As with the other sites, the specific yellow and all red times were associated with the respective site-year.

Howard County Data

Table C- 5 summarizes the characteristics of the Howard County data used for the cross-sectional analysis.

Table C- 5. Signalized intersections in Howard County.

Variable	Mean	Minimum	Maximum
Years	8.9	2	11
Total crashes/site-year	3.96	0	30
Injury crashes/site-year	1.12	0	10
Rear-end crashes/site-year	1.68	0	15
Angle crashes/site-year	0.83	0	12
Major Road ADT	9,650	3,953	24,725
Minor Road ADT	2,924	574	10,228
Number of Sites = 40			

Montgomery County Data

Table C- 6 summarizes the characteristics of the Montgomery County data used for the cross-sectional analysis.

Table C- 6. Signalized intersections in Montgomery County.

Variable	Mean	Minimum	Maximum
Years	8.5	3	11
Total crashes/site-year	6.54	0	31
Injury crashes/site-year	2.78	0	17
Rear-end crashes/site-year	2.47	0	17
Angle crashes/site-year	1.35	0	11
Major Road ADT	22,520	8,900	46,267
Minor Road ADT	7,147	630	19,048
Number of Sites = 42			

San Diego Data

Table C- 7 summarizes the characteristics of the San Diego data used for the cross-sectional analysis.

Table C- 7. Signalized intersections in San Diego.

Variable	Mean	Minimum	Maximum
Years	7.2	6	10
Total crashes/site-year	3.69	0	13
Injury crashes/site-year	3.54	0	13
Rear-end crashes/site-year	1.56	0	8
Angle crashes/site-year	0.96	0	5
Major Road ADT	18,061	9,850	31,600
Minor Road ADT	8,001	3,280	20,225
Number of Sites = 17			

San Francisco Data

Table C- 8 summarizes the characteristics of the San Francisco data used for the cross-sectional analysis.

Table C- 8. Signalized intersections in San Francisco.

Variable	Mean	Minimum	Maximum
Years	6.1	3	11
Total crashes/site-year	7.39	0	22
Injury crashes/site-year	6.49	0	20
Rear-end crashes/site-year	1.46	0	6
Angle crashes/site-year	4.18	0	18
Major Road ADT	19,706	8,240	35,471
Minor Road ADT	7,652	3,150	14,889
Number of Sites = 11			

RESULTS

Before-After Analysis

Table C- 9 documents the results of the EB before-after analysis for total and injury crashes. Similarly, Table C- 10 documents the results for rear-end and angle crashes. It should be noted that the modifications to the yellow and all red time were not equivalent for all sites. This applies to both the existing conditions and the increase in the yellow and/or all red intervals. For example, several of the intersections did not include an all red phase in the before condition.

The results are first presented based on the specific change in signal timing as follows:

- Increase Yellow and All Red: The analysis shows a minimal change in crashes when the change interval is increased as a result of increasing both the yellow and all red phases. Specifically, there appears to be a slight reduction in total and angle crashes and a slight increase in injury and rear-end crashes. All results, however, are statistically insignificant at the 5 percent significance level. Note that the yellow phase was generally less than or equal to the ITE recommended practice during the before period. In all cases, there was no all red phase in the before period. After modifications, the yellow phase for each site was greater than the ITE recommended practice and each site had an all red phase, although still less than the ITE recommended practice in some cases. The average increases in the yellow and all red times were 0.8 seconds and 1.2 seconds, respectively.
- Increase All Red Only: A similar trend appears when only the red interval is increased. Specifically, there appears to be a consistent safety benefit (i.e., CMF less than 1.0) for total, injury, rear-end, and angle crashes (all target crash types). Note that the all red phase was either less than the ITE recommended practice or non-existent during the before period. After modifications, each site had an all red phase, although some were still less than the ITE recommended practice. The average increase in the all red time was 1.1 seconds. The reduction in total crashes is significant at the 5 percent significance level. The results for injury, rear-end, and angle crashes are not statistically significant at the 10 percent significance level.
- Increase Yellow Only: The results are highly insignificant for locations where only the yellow interval was increased. Note that the yellow phase was generally less than or equal to the ITE recommended practice during the before period. After modifications, the yellow phase for each site was equal to or greater than the ITE recommended practice. The average increase in the yellow time was 1.0 seconds. The sample size was relatively limited for this group, representing just five sites.

The applicable conditions for the above results (i.e., number of intersections without an all red phase before changes and the average increase in the respective intervals) are presented in Table C- 11.

The results are then presented based on compliance with the ITE procedure for calculating yellow and all red intervals. Similar to the cross-sectional analysis, the yellow and all red intervals were estimated based on the ITE procedure as described in the *Methodology for Cross-Sectional Analysis*. The sample size was too small to compare the actual yellow and all red phases to those obtained from the ITE procedure. Instead, the total change interval was computed by summing the actual yellow and all red times and this value was compared to the sum of the yellow and all red times from the ITE recommended practice. The total change interval was increased at 27 sites. For all 27 sites, the total change interval (actual yellow plus all red) was initially less than the total change interval from the ITE recommended practice (ITE recommended yellow plus all red). After modifications, 12 sites had a longer change interval, but it was

still less than the ITE recommended practice [shown as “Increase Change Interval (< ITE)” below]. The change interval for the remaining 15 sites was increased and exceeded the ITE recommended practice [shown as “Increase Change Interval (> ITE)” below]. The results are discussed as follows:

- Increase Change Interval (< ITE): While the change interval at these 12 sites did not meet the ITE recommended practice, the increase resulted in a reduction in total (27.2 percent) and injury (33.8 percent) crashes. These reductions are statistically significant at the five percent significance level. The changes in both rear-end and angle crashes were statistically insignificant at the 10 percent significance level. The average increase in the total change interval was 0.9 seconds.
- Increase Change Interval (> ITE): The results for these 15 sites are generally consistent with the previous results, showing a reduction in total, injury, and rear-end crashes. There are, however, a few notable differences. First, the reductions in total (7.8 percent) and injury (6.3 percent) crashes are statistically insignificant (i.e., CMF closer to 1.0), even at the 10 percent significance level. Second, the reduction in rear-end crashes (35.7 percent) is statistically significant at the five percent significance level. Note the change in angle crashes is statistically insignificant, similar to the previous results. The average increase in the total change interval was 1.6 seconds.

The applicable conditions for the above results (i.e., number of intersections without an all red phase before changes and the average increase in the total change interval) are presented in Table C- 12.

It was surprising that the results showed more favorable crash reductions when the total change interval was increased, but still less than the ITE recommended practice. There are several points of discussion as follows:

- The safety performance in the before period may have an influence on the results. The change interval was increased for all 27 sites and all sites did not meet the ITE recommended practice in the before period. However, the difference in the actual and ITE change interval was not equal for all sites. For the 12 sites that did not meet the ITE recommended practice in the after period, the average difference in the before period was 1.4 seconds. For the 15 sites that exceeded the ITE recommended practice in the after period, the average difference in the before period was 0.7 seconds (approximately half that of the other group). So, there may have been more potential for improvement at those sites with a greater difference in the before period.
- It should also be noted that the increase in the change interval was also not equal for the two groups. Specifically, the average increase in the total change interval was 0.9 seconds for the 12 sites that did not meet the ITE recommended practice in the after period. For the 15 sites that exceeded the ITE recommended practice in the after period, the average increase in the change interval was 1.6 seconds (approximately 70 percent greater than the other group).
- Based on the previous discussion, these results suggest that there may be an optimal change interval (i.e., there is not a linear increase or decrease in safety as yellow and all red time are increased). Instead, there appears to be a safety benefit as the change interval is increased to a certain point, after which the safety benefit may level-off or decrease. Whether or not this optimal point is proximate to the ITE recommended practice is a point for further research.

Table C- 9. Before-After Results for Total and Injury Crashes by ITE Compliance

Group[1]	Sites	EB Expected Total Crashes Without Timing Change	Observed Total Crashes With Timing Change	CMF for Total Crashes (SE)	EB Expected Injury Crashes Without Timing Change	Observed Injury Crashes With Timing Change	CMF for Injury Crashes (SE)
Increase Yellow and All Red	11	59.2	59	0.991 (0.146)	52.69	54	1.020 (0.156)
Increase All Red Only	14	228.7	183	0.798 (0.074)	96.8	84	0.863 (0.114)
Increase Yellow Only	5	56.4	65	1.141 (0.177)	34.0	37	1.073 (0.216)
Increase Change Interval (< ITE)	12	217.2	159	0.728 (0.077)	98.9	66	0.662 (0.099)
Increase Change Interval (> ITE)	15	177.3	164	0.922 (0.089)	102.0	96	0.937 (0.114)

[1] See "Change interval compliance", "Difference between yellow intervals", and "Difference between all red intervals" above for definitions.

Table C- 10. Before-After Results for Rear-end and Angle Crashes by ITE Compliance

Group[2]	Sites	EB Expected Rear-end Crashes Without Timing Change	Observed Rear-end Crashes With Timing Change	CMF for Rear-end Crashes (SE)	EB Expected Angle Crashes Without Timing Change	Observed Angle Crashes With Timing Change	CMF for Angle Crashes (SE)
Increase Yellow and All Red	11	15.09	17	1.117 (0.288)	26.69	26	0.961 (0.217)
Increase All Red Only	14	70.1	57	0.804 (0.135)	54.3	53	0.966 (0.164)
Increase Yellow Only	5	22.1	21	0.934 (0.237)	19.7	22	1.076 (0.297)
Increase Change Interval (< ITE)	12	80.2	69	0.848 (0.142)	32.7	28	0.840 (0.195)
Increase Change Interval (> ITE)	15	50.7	33	0.643 (0.130)	69.6	75	1.068 (0.156)

Table C- 11. Applicable Conditions for Changes to Yellow and All Red Intervals

Group	Sites	Number of Sites without All Red Interval Before	Average Increase in Yellow Interval (min, max)	Average Increase in All Red Interval (min, max)
Increase Yellow and All Red	12	12	0.8 (0.5, 1.6)	1.2 (1.0, 2.0)
Increase All Red Only	14	10	--	1.1 (1.0, 2.0)
Increase Yellow Only	5	1	1.0 (1.0, 1.0)	--

Table C- 12. Applicable Conditions for Changes to Total Change Interval

Group	Sites	Number of Sites without All Red Interval Before	Average Increase in Total Change Interval (min, max)
Increase Change Interval (< ITE)	12	11	0.9 (0, 1.5)
Increase Change Interval (> ITE)	15	10	1.6 (1.0, 3.0)

[2] See “Change interval compliance”, “Difference between yellow intervals”, and “Difference between all red intervals” above for definitions.

Cross-Sectional Analysis

The before-after analysis was useful for investigating the safety effects of basic changes to the yellow and all red phases. Specifically, the analysis focused on increases in the yellow, all red, or both phases. In most cases, the yellow and/or all red phases were increased to either meet or exceed the ITE recommended practice; however, there were some cases in which the yellow and/or all red time were increased, but still did not meet the ITE recommended practice. Due to the relatively small sample, it was only possible to investigate the safety effects of modifications to the *total* change interval with respect to the ITE recommended practice. To investigate the individual yellow and all red phases with respect to the ITE recommended practice, a cross-sectional study was employed. The cross-sectional analysis included a larger sample of signalized intersections with various combinations of yellow and all red phases.

Cross-sectional models were developed using the negative binomial regression techniques described in the *Methodology* section. The regression analysis is based on signalized intersections with various yellow and all red times. The data from the two states were aggregated and models were developed for total, injury, rear-end, and angle crashes. It is important to note that multiple years of data were used for most sites and data were included from the before-after analysis. In other words, the cross-sectional analysis could be characterized as a type of time-series cross-sectional analysis or panel data analysis. The before-after data were included in the cross-sectional analysis to enhance the sample size. These sites are not expected to significantly influence the results as there were relatively few sites to include from the before-after study and potential regression-to-the-mean effects were negligible. However, it was necessary to make an adjustment for the repeated observations (i.e., same site over multiple years). To accomplish this, the model fitting process used a clustered robust standard error, identifying individual sites as the cluster variable.

Preliminary models were developed using various forms of ADT, including:

1. Total entering ADT.
2. Natural log of total entering ADT.
3. Major and minor ADT.
4. Natural log of major and minor ADT.

The decision on which form to use was based on an evaluation of parameter estimates and other goodness of fit measures (i.e., log-likelihood and pseudo R-square). In all cases, the natural log of major and minor ADT was the most appropriate form for the ADT term.

As defined in the *Methodology*, several variations were explored to represent the yellow and all red phases. The following provides a brief summary of the form selected for the final models:

- DiffChange: a continuous variable representing the difference between the ITE total change interval and actual total change interval (calculated as actual minus ITE).
- DiffY: a continuous variable representing the difference between the ITE yellow time and actual yellow time (calculated as actual yellow minus ITE yellow time).
- DiffR: a continuous variable representing the difference between ITE all red time and actual all red time (calculated as actual all red minus ITE all red time).

It was hypothesized that the safety effect of yellow and all red times may not be linear. As such, it was necessary to specify a functional form that would allow for a non-linear trend. There are several ways to capture this type of trend. The squared and natural log forms of DiffChange, DiffY, and DiffR were

considered in addition to a linear term. It was determined that the squared term of DiffChange, DiffY, and DiffR was most appropriate. This model form allows for differences that result in a negative value; using the natural log would not allow for zero or negative differences.

The variables of interest were included in all cross-sectional models, regardless of significance. While many of the coefficients were statistically insignificant, it was decided to keep the variables in the model to allow for general comparisons with the results of the before-after analysis. The following additional variables were considered in the model development.

- Approaches: indicator for number of approaches, 1 = four approaches.
- Protected: indicator for left-turn phasing on major road, 1 = protected.
- State: indicator for state, 1 = California.

There was some concern that endogeneity may be an issue in the estimation of the model. Specifically, left-turn protection will generally help to reduce left-turn crashes, but left-turn protection may be implemented due to a safety issue. As such, the indicator for left-turn protection may be correlated with the error term when predicting crashes. If the variable for left-turn protection is left-out of the model, omitted variable bias will become an issue. If the variable is included, it may result in inconsistent and biased estimates. To overcome these issues, an instrumental variable can be used in place of the left-turn phasing. An instrumental variable is essentially the result of a separate regression analysis where the dependent variable is specified as the variable in question (i.e., left-turn protection). In this case, the indicator is a binary (1/0) variable so binary logistic regression was used to estimate an equation, using the natural log of major and minor ADT, number of approaches, posted speed on the major road, yellow and all red time, and major road width as predictors. The result of the binary logistic regression model was then used as a predictor (i.e., instrument) in the negative binomial models.

Model for Total Crashes

The two models for total crashes are presented in Table C- 13. The first model presents the results for the difference in yellow (DiffY) and the difference in all red (DiffR), while the second model presents the results for the difference in the overall change interval (DiffChange). Based on the first model, the terms related to yellow time are insignificant, but result in an inverted u-shape trend as the difference in yellow time increases from -2.0 to 2.0 as shown in Figure C- 2. The terms related to all red time are also insignificant, but indicate a general safety benefit as the difference between the actual and ITE recommended all red time approaches zero as shown in Figure C- 3. These results are consistent with previous studies, which indicate a decrease in crashes as the actual all red time approaches the all red time computed from the ITE procedure. As discussed previously, it is difficult to make comparisons with the results from the EB analysis because the yellow and all red time were not evaluated with respect to the ITE recommended practice; it was only possible to investigate the total change interval with respect to the ITE practice using the EB analysis.

The results from the two analyses are relatively consistent when comparing the difference in the total change interval. Based on the results of the cross-sectional analysis, there is a u-shaped trend in the expected CMF as shown in Figure C- 4. Specifically, there appears to be a safety benefit as the difference between the actual and ITE recommended change interval approaches zero. The results from the EB analysis similarly indicated a safety benefit as the total change interval is increased and approaches the ITE recommended practice and a less pronounced safety benefit as the total change interval is increased and exceeds the ITE recommended practice (i.e., a u-shaped trend). It should be noted that while the general trends are consistent with the EB analysis, the coefficients are highly insignificant.

Table C- 13. Models for Total Crashes

Based on 110 sites totaling 903 site-years, representing 4,691 total crashes.

Model with separate terms for yellow and all red

```
Negative binomial regression                      Number of obs   =        903
Dispersion          = mean                        Wald chi2(8)    =      75.69
Log pseudolikelihood = -2390.2323                 Prob > chi2     =     0.0000

                              (Std. Err. adjusted for 110 clusters in studyid)
------------------------------------------------------------------------------
             |               Robust
       Total |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
  California |  -.2449036   .1531655    -1.60   0.110    -.5451024    .0552953
   ln(MajADT)|   .3861918   .1069307     3.61   0.000     .1766115     .595772
   ln(MinADT)|   .4397945   .0833817     5.27   0.000     .2763694    .6032196
  Instrument |  -.4190079   .2986419    -1.40   0.161    -1.004335    .1663196
       DiffY |  -.1389192   .1400208    -0.99   0.321     -.413355    .1355166
       DiffR |   .0765382   .0867615     0.88   0.378    -.0935111    .2465876
DiffYSquared |  -.1623213   .1736764    -0.93   0.350    -.5027208    .1780782
DiffRSquared |   .0299189   .0677055     0.44   0.659    -.1027814    .1626193
    Constant |  -5.591209    1.28339    -4.36   0.000    -8.106607    -3.07581
-------------+----------------------------------------------------------------
   ln(alpha) |     .08852   .1103712                     -.1278035    .3048435
-------------+----------------------------------------------------------------
       alpha |   1.092556   .1205867                      .8800263    1.356413
```

Model for overall change interval

```
Negative binomial regression                      Number of obs   =        903
Dispersion          = mean                        Wald chi2(6)    =      61.31
Log pseudolikelihood = -2393.9109                 Prob > chi2     =     0.0000

                              (Std. Err. adjusted for 110 clusters in studyid)
------------------------------------------------------------------------------
             |               Robust
       Total |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
  California |  -.2967663   .1488449    -1.99   0.046     -.588497   -.0050355
   ln(MajADT)|   .4605744   .1054567     4.37   0.000      .253883    .6672657
   ln(MinADT)|   .4852506   .0870849     5.57   0.000     .3145673    .6559339
  Instrument |  -.5642603   .2855242    -1.98   0.048    -1.123878   -.0046431
  DiffChange |   .0476341   .0856071     0.56   0.578    -.1201528     .215421
DiffChangeSq |    .027716   .0717518     0.39   0.699     -.112915     .168347
    Constant |  -6.710343   1.229266    -5.46   0.000     -9.11966   -4.301026
-------------+----------------------------------------------------------------
   ln(alpha) |   .1005929   .1096533                     -.1143237    .3155095
-------------+----------------------------------------------------------------
       alpha |   1.105826   .1212576                      .8919692    1.370958
```

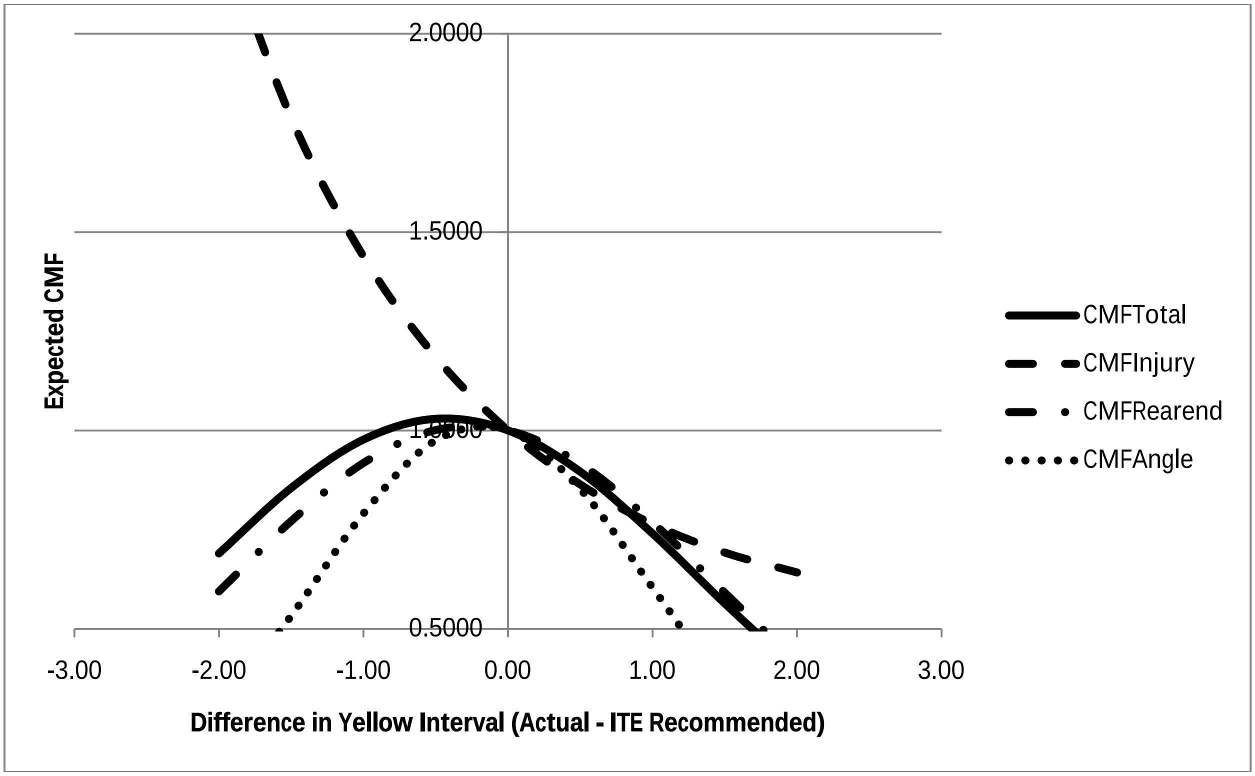

Figure C- 2. CMF Trend with Respect to the Difference in Acutal and ITE Yellow Interval

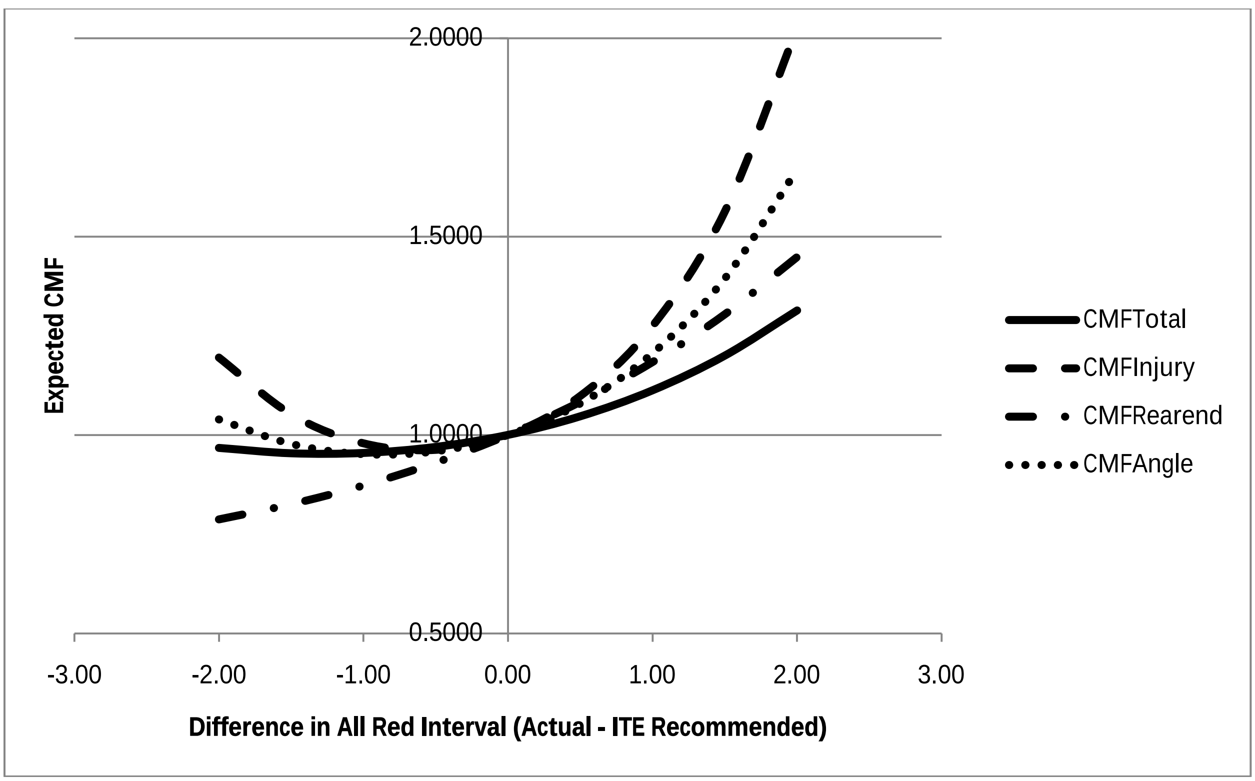

Figure C- 3. CMF Trend with Respect to the Difference in Acutal and ITE All Red Interval

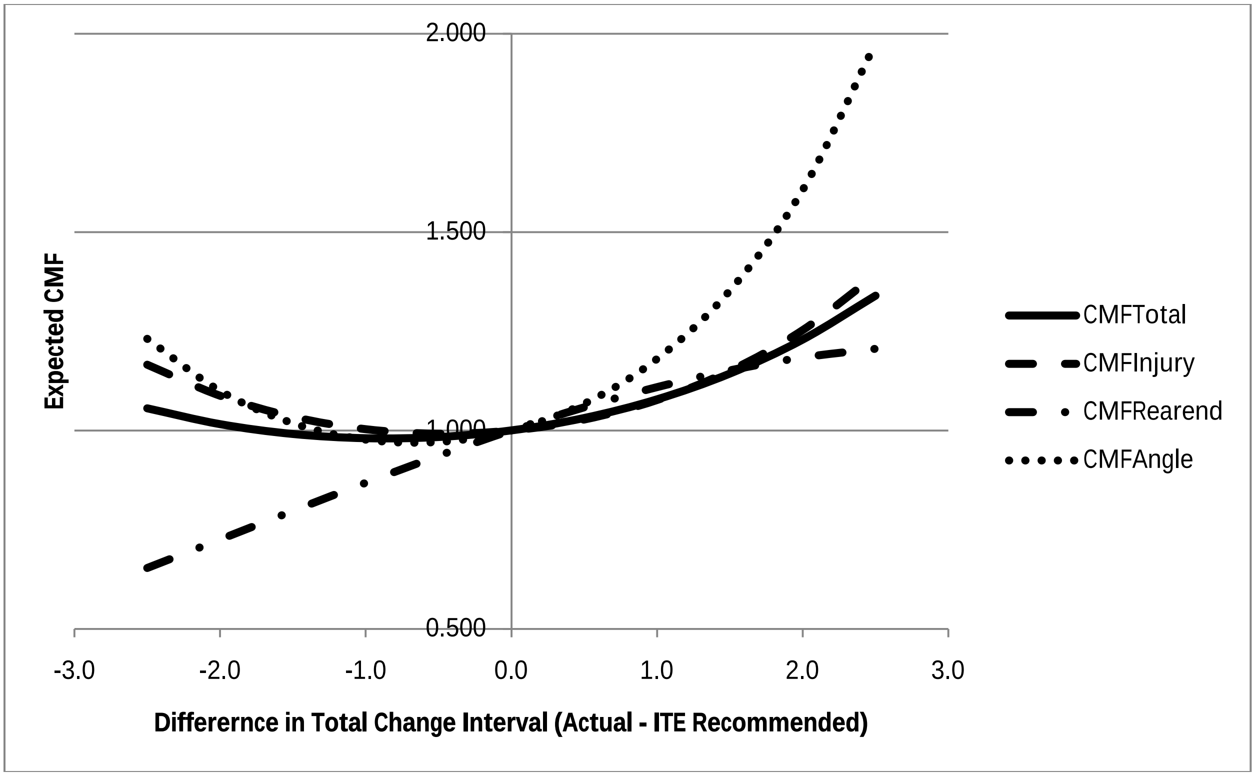

Figure C- 4. CMF Trend with Respect to the Difference in Acutal and ITE Total Change Interval

The key point of the analysis is that a constant CMF value may not accurately represent the safety impact of a treatment. Rather, it is conceivable that the safety impact could change over the range of values for a given treatment. If there is evidence of a significant non-linear trend, as is the case here, the relationship is better defined by a series of CMFs or a crash modification function (CMFunction) rather than a single CMF.

Models by Severity and Crash Type

Separate models were developed for injury, rear-end, and angle crashes. These models were based on the same functional form and the same dataset used to develop the models for total crashes. The results for injury, rear-end, and angle crashes are presented in Table C- 14 through Table C- 16, respectively. Within each table, the first model presents the results for the difference in yellow (DiffY) and the difference in all red (DiffR), while the second model presents the results for the difference in the overall change interval (DiffChange). The results for all four models (total, injury, rear-end, and angle crashes) are illustrated previously in Figure C- 2 to Figure C- 4.

For injury crashes, the term related to the difference in yellow time is significant at the five percent significance level, although the squared term is highly insignificant. The CMF trend is decreasing at a decreasing rate, indicating a general reduction in expected crashes as the actual yellow time approaches and exceeds the ITE yellow time. The terms related to all red time are marginally significant (fifteen percent significance level) and indicate a general reduction in crashes as the actual all red time approaches the ITE all red time. These results are relatively consistent when compared to the second injury-related cross-sectional model, which investigates the difference in the total change interval. The second injury-related model indicates a minor safety benefit when the total change interval approaches the ITE recommended practice; however, the results are highly insignificant.

Based on the results of the cross-sectional analysis, there is a u-shaped trend in the expected CMF. Specifically, there appears to be a safety benefit as the difference between the actual and ITE recommended change interval approaches zero. The results from the EB analysis indicated a safety benefit as the total change interval is increased and approaches the ITE recommended practice and a less pronounced safety benefit as the total change interval is increased and exceeds the ITE recommended practice (i.e., a u-shaped trend).

Table C- 14. Models for Injury Crashes

Based on 110 sites totaling 903 site-years, representing 2,260 total crashes.

Model with separate terms for yellow and all red

```
Negative binomial regression                      Number of obs   =        903
Dispersion          = mean                        Wald chi2(8)    =     157.06
Log pseudolikelihood = -1751.8476                 Prob > chi2     =     0.0000

                              (Std. Err. adjusted for 110 clusters in studyid)
------------------------------------------------------------------------------
             |               Robust
      Injury |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
  California |   .7187651   .1487627     4.83   0.000     .4271956    1.010335
   ln(MajADT)|   .5663708   .1371185     4.13   0.000     .2976234    .8351182
   ln(MinADT)|   .3882956   .0873993     4.44   0.000     .2169961     .559595
  Instrument |  -.5266893   .3222164    -1.63   0.102    -1.158222    .1048432
       DiffY |  -.3162861   .1441488    -2.19   0.028    -.5988126   -.0337595
       DiffR |   .1319422   .0916282     1.44   0.150    -.0476458    .3115301
DiffYSquared |    .047386   .1708843     0.28   0.782     -.287541     .382313
DiffRSquared |   .1104777    .067521     1.64   0.102     -.021861    .2428164
    Constant |  -7.968908   1.596877    -4.99   0.000    -11.09873   -4.839087
-------------+----------------------------------------------------------------
   ln(alpha) |  -.1841585   .1310224                     -.4409577    .0726408
-------------+----------------------------------------------------------------
       alpha |    .831804    .108985                      .6434199    1.075344
```

Model for overall change interval

```
Negative binomial regression                      Number of obs   =        903
Dispersion          = mean                        Wald chi2(6)    =     133.53
Log pseudolikelihood = -1764.0619                 Prob > chi2     =     0.0000

                              (Std. Err. adjusted for 110 clusters in studyid)
------------------------------------------------------------------------------
             |               Robust
      Injury |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
  California |   .6234605   .1495652     4.17   0.000      .330318     .916603
   ln(MajADT)|   .6819712   .1363583     5.00   0.000     .4147138    .9492286
   ln(MinADT)|     .45887   .0974454     4.71   0.000     .2678806    .6498594
  Instrument |  -.7565715   .2957295    -2.56   0.011    -1.336191   -.1769524
  DiffChange |   .0353799   .0935742     0.38   0.705    -.1480222    .2187819
 DiffChangeSq|   .0387181   .0795957     0.49   0.627    -.1172866    .1947227
    Constant |  -9.594148   1.463042    -6.56   0.000    -12.46166   -6.726639
-------------+----------------------------------------------------------------
   ln(alpha) |  -.1279947   .1319806                      -.386672    .1306825
-------------+----------------------------------------------------------------
       alpha |    .879858   .1161242                      .6793139    1.139606
```

For rear-end crashes, the terms for yellow and all red time are insignificant. This is consistent with the EB analysis. Recall from the EB analysis, the change in rear-end crashes was insignificant in most cases (i.e., increase yellow and all red, increase all red only, increase yellow only, and increase total change interval to be less than the ITE recommended practice). Only when the total change interval was increased to exceed the ITE recommended practice was there a significant reduction in expected rear-end crashes.

Table C- 15. Models for Rear-end Crashes

Based on 110 sites totaling 903 site-years, representing 1,770 total crashes.

Model with separate terms for yellow and all red

```
Negative binomial regression                      Number of obs   =        903
Dispersion          = mean                        Wald chi2(8)    =      87.61
Log pseudolikelihood = -1615.8217                 Prob > chi2     =     0.0000
                              (Std. Err. adjusted for 110 clusters in studyid)
------------------------------------------------------------------------------
             |               Robust
    Rear-end |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
  California | -.4722822   .1761016    -2.68   0.007     -.817435   -.1271294
   ln(MajADT)|    .560058   .1612441     3.47   0.001     .2440253    .8760906
   ln(MinADT)|   .3014637   .1164282     2.59   0.010     .0732686    .5296587
  Instrument |   .3638348   .3600462     1.01   0.312    -.3418427    1.069512
       DiffY |    -.08892   .1738854    -0.51   0.609    -.4297291    .2518891
       DiffR |   .1523025   .1189743     1.28   0.200    -.0808828    .3854878
DiffYSquared |  -.1745485   .2284538    -0.76   0.445    -.6223098    .2732128
DiffRSquared |   .0164411   .0914649     0.18   0.857    -.1628268     .195709
    Constant |  -7.269799   1.773942    -4.10   0.000    -10.74666   -3.792936
-------------+----------------------------------------------------------------
   ln(alpha) |   .0843389   .1192616                     -.1494097    .3180874
-------------+----------------------------------------------------------------
       alpha |   1.087998   .1297564                      .8612162    1.374496
```

Model for overall change interval

```
Negative binomial regression                      Number of obs   =        903
Dispersion          = mean                        Wald chi2(6)    =      77.88
Log pseudolikelihood = -1618.9225                 Prob > chi2     =     0.0000
                              (Std. Err. adjusted for 110 clusters in studyid)
------------------------------------------------------------------------------
             |               Robust
    Rear-end |      Coef.   Std. Err.      z    P>|z|     [95% Conf. Interval]
-------------+----------------------------------------------------------------
  California |  -.5101089   .1701312    -3.00   0.003    -.8435599   -.1766579
   ln(MajADT)|   .6490247   .1375025     4.72   0.000     .3795248    .9185245
   ln(MinADT)|    .358187   .1210286     2.96   0.003     .1209753    .5953988
  Instrument |   .1551648   .3191337     0.49   0.627    -.4703258    .7806554
  DiffChange |   .1223684   .1099608     1.11   0.266    -.0931508    .3378876
DiffChangeSq |  -.0190702   .0834564    -0.23   0.819    -.1826417    .1445014
    Constant |  -8.598464   1.499696    -5.73   0.000    -11.53781   -5.659115
-------------+----------------------------------------------------------------
   ln(alpha) |   .1008642   .1187251                     -.1318328    .3335612
-------------+----------------------------------------------------------------
       alpha |   1.106126    .131325                      .8764875     1.39593
```

For angle crashes, the terms related to yellow and all red time are statistically insignificant. While the results from the cross-sectional analysis are statistically insignificant, this is generally consistent with the results from the EB analysis. Again, recall the results from the EB analysis; the change in angle crashes was statistically insignificant in all cases.

Table C- 16. Models for Angle Crashes

Based on 110 sites totaling 903 site-years, representing 1,175 total crashes.

Model with separate terms for yellow and all red

```
Negative binomial regression                      Number of obs   =        903
Dispersion           = mean                       Wald chi2(8)    =      47.27
Log pseudolikelihood = -1362.8557                 Prob > chi2     =     0.0000

                              (Std. Err. adjusted for 110 clusters in studyid)
```

Angle	Coef.	Robust Std. Err.	z	P>|z|	[95% Conf.	Interval]
California	.5297058	.2287684	2.32	0.021	.081328	.9780836
ln(MajADT)	.1385979	.1934087	0.72	0.474	-.2404762	.5176719
ln(MinADT)	.3908491	.1134562	3.44	0.001	.168479	.6132192
Instrument	-.8680396	.5270438	-1.65	0.100	-1.901026	.1649473
DiffY	-.1346974	.2208775	-0.61	0.542	-.5676093	.2982144
DiffR	.118376	.124164	0.95	0.340	-.1249809	.3617329
DiffYSquared	-.3698155	.241347	-1.53	0.125	-.842847	.103216
DiffRSquared	.0687971	.0916036	0.75	0.453	-.1107426	.2483369
Constant	-4.231353	2.131944	-1.98	0.047	-8.409886	-.0528193
ln(alpha)	.1801383	.1553099			-.1242635	.4845402
alpha	1.197383	.1859654			.8831471	1.623428

Model for overall change interval

```
Negative binomial regression                      Number of obs   =        903
Dispersion           = mean                       Wald chi2(6)    =      33.23
Log pseudolikelihood = -1368.2739                 Prob > chi2     =     0.0000

                              (Std. Err. adjusted for 110 clusters in studyid)
```

Angle	Coef.	Robust Std. Err.	z	P>|z|	[95% Conf.	Interval]
California	.4890206	.220438	2.22	0.027	.05697	.9210711
ln(MajADT)	.2493572	.1838048	1.36	0.175	-.1108936	.6096079
ln(MinADT)	.4549325	.1167587	3.90	0.000	.2260896	.6837754
Instrument	-1.05497	.5137883	-2.05	0.040	-2.061977	-.0479639
DiffChange	.0946528	.1265054	0.75	0.454	-.1532933	.3425989
DiffChangeSq	.0711	.1209959	0.59	0.557	-.1660476	.3082477
Constant	-5.89891	1.877765	-3.14	0.002	-9.579262	-2.218557
ln(alpha)	.2079597	.145073			-.0763782	.4922975
alpha	1.231163	.1786086			.9264658	1.636071

DISCUSSION AND CONCLUSIONS

The data collected and analyzed for this study show a general safety benefit for increasing the change interval at signalized intersections. The EB before-after analysis indicated a significant reduction in total, injury, and rear-end crashes under various scenarios. Specifically, the EB analysis indicated a significant reduction in total crashes as a result of 1) increasing the all red phase only and 2) increasing the total change interval to be less than the ITE recommended practice. Injury crashes were significantly reduced as a result of increasing the total change interval to be less than the ITE recommended practice. Rear-end crashes were significantly reduced as a result of increasing the total change interval to be greater than the ITE recommended practice. The change in angle crashes was statistically insignificant under all scenarios investigated.

The current state-of-the-art for developing CMFs is the EB before-after method. Due to a relatively small sample size, it was only possible to investigate the safety effects of modifications to the *total* change interval with respect to the ITE recommended practice, using the EB analysis. To investigate the safety effects of individual yellow and all red phases with respect to the ITE recommended practice, a cross-sectional study was employed, using negative binomial regression models. The cross-sectional analysis included a larger sample of signalized intersections with various combinations of yellow and all red phases. The coefficients from the cross-sectional models were generally insignificant for both yellow and all red time, but the direction of the effect, particularly for total and injury crashes, corroborate the results from the EB analysis.

Various forms of the variables were explored during the cross-sectional analyses to further investigate the relationship between the actual and ITE recommended yellow and all red phases. Specifically, a squared term was included for each variable to allow a non-linear trend. While most of the squared terms were insignificant, the resulting trends agreed well with the trends identified from the EB analysis. These trends indicate that the relationship between safety and yellow and all red time may not be linear. In other words, there may be an optimal yellow and all red time for a given scenario, above and below which crashes may increase.

Based on the relative strength of the EB method, and the general insignificant results of the cross-sectional analysis, the recommended CMFs for total, injury, and rear-end crashes are as follows (based on the EB analysis):

Treatment	**Crash Type**	**Severity**	**CMF**	**Standard Error**
Increase All Red Only	All	All	0.798	0.074
Increase Change Interval (< ITE)	All	All	0.728	0.077
Increase Change Interval (< ITE)	All	Injury	0.662	0.099
Increase Change Interval (> ITE)	Rear-end	All	0.643	0.130

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NCHRP PROJECT 17-35

APPENDIX D:

Safety Evaluation of Changing Left Turn Phasing at Signalized Intersections

INTRODUCTION

Background and Previous Work

The safety problems encountered by left turning vehicles could be addressed in different ways. One possible way is to use protected left turn phases. With protected left-turn phases, left turning vehicles do not yield to opposing through traffic hence eliminating this conflict. Many factors may need to be considered before protected left turn phasing is provided including turning volumes, opposing through volumes, pedestrian crossing volumes, approach speeds, sight distance, number of lanes, delay, type and nature of channelization, and crash experience.

Left turn phasing could be permissive, protected only, or protected-permissive (including both lead-protected-permissive and lag-protected-permissive). Protected-permissive phasing has several advantages, "the most important being the reduction in delay for left-turning vehicles achieved by permitting left turns while the opposing through movement has a green indication" (Antonucci et al., 2004). In addition, since less green time is allocated for protected left turns (and hence more time for other high priority movements) there may be potential for improved arterial progression. However, since the protected-permissive phase includes a permissive phase, it does not eliminate the conflict between left turns and opposing through vehicles.

Protected-only phasing may be the best option in terms of reducing or eliminating crashes involving left turn vehicles and opposing through vehicles. Antonucci et al. (2004) report on a study by Shebeeb (1995) which showed that "the left-turn signal phases that provide the greatest operational benefit to left-turning vehicles, with respect to stopped delay, increase the crash risk for left-turning vehicles the most". Antonucci et al. indicate that additional guidance on choosing a type of left-turn phasing is summarized in *NCHRP Synthesis 225: Left-Turn Treatments at Intersections* (Pline, 1996). Antonucci et al. (2004) in their review also discuss the issues related to lead-protected-permissive and lag-protected-permissive.

Hauer (2004) conducted a detailed critical review of 14 studies that were completed over a period of 24 years in several countries. Hauer notes that the CMF for changing from permissive to protected most likely depends on the number of opposing lanes, and that most of the other evidence is insufficient and contradictory. Based on the review of these studies, Hauer concludes that the CMF for left-turn crashes for changing to protected phasing from either permissive or protected-permissive is around 0.3 (i.e., a 70% reduction in left-turn crashes); for other crashes, the CMF is 1.0 (i.e., no effect). When changing from permissive to protected-permissive, Hauer estimates that the CMF is around 1.0 for both left-turn crashes and other crashes (i.e., no effect).

More recently, Lyon et al. (2005) used the empirical Bayes before-after study approach to evaluate the impact of flashing advance green (FAG) and left-turn green arrow (LTGA) treatments in Toronto, Canada, on injury and fatal left-turn crashes of all types and also, specifically, left-turn side impact crashes. Priority was a leading left-turn. In some cases, some form of left-turn priority existed before-hand and in others additional minor modifications were made. A total of 35 intersections from Toronto were included; 15 sites received the FAG, while 20 received the LTGA. Left-turn crashes decreased by 16 percent in the FAG sites and 17 percent at the LTGA sites. Left-turn side impact crashes decreased by 12 percent in the FAG sites and 25 percent at the LTGA sites. All results were statistically significant at the 95 percent confidence level.

Srinivasan et al. (2008) evaluated three types of left-turn phasing treatments using data from Winston-Salem, North Carolina. The first involved replacing a permissive left-turn phase with a protected-permissive phase at three sites. The second involved replacing a permissive left-turn phase with a fully protected phase at eight sites. The third type involved replacing a protected-permissive phase with a fully protected phase at four sites. The target crashes for these treatments were identified as those involving at least one left-turning vehicle on the treated roadway. For the three sites where the permissive phase was replaced by protected-permissive phase, there was very little change in the target as well as the total crashes. However, since the sample size was small, this result of no apparent effect of this treatment cannot be taken as definitive. For each of the other two

treatment types where the left turn phase was changed to fully protected phase (from either permissive or protected-permissive), left turn crashes were virtually eliminated, but there was very little change in total crashes. Since the left-turn crashes decreased substantially, and total crashes did not, it is evident that there must have been an increase in non-left-turn crashes of the same order as the decrease in left turn crashes. The authors noted that further research is necessary to determine the specific reasons for the effect on non-left-turn crashes and that it seems reasonable to speculate that introducing a protected left-turn phase will tend to increase mostly rear-end crashes because of the increased number of phases (and therefore dilemma zone opportunities) and the increase in queues that results from reduced green time available for all traffic not protected by the introduced phase. This identified void was part of the motivation for the current study.

OBJECTIVE

The objective was to estimate the general safety effects of changing left turn phasing at signalized intersection approaches. The original intent was to investigate the effects of different types of changes: permissive to protected-permissive, permissive to protected, protected-permissive to protected, and protected to permissive or protected-permissive. However, sufficient sites could be found only for the change from permissive to protected-permissive at approaches of intersections that had left turn lanes. In addition to investigating the change in left turn crashes, another goal was to investigate the effects on non-left-turn related crash types and look at the effects of traffic volume, left-turn volume, and number of opposing lanes on the estimated change in crashes.

The basic objective was to estimate the change in target crashes. Possible target crash types include:

- Total crashes
- Injury crashes
- Left-turn crashes
- Left-turn opposing crashes
- Rear-end crashes

Further questions of interest identified for possible consideration include:

- Do effects vary by level of traffic volumes, particularly left-turning volumes?
- Do effects vary by the number of lanes?

Meeting these objectives placed some special requirements on the data collection and analysis tasks, including the need to:

- Select a large enough sample size to detect, with statistical significance, what may be small changes in safety for some crash types.
- Carefully select comparison or reference sites.
- Properly account for traffic volume changes.
- Pool data from multiple jurisdictions to improve reliability of the results and facilitate broader applicability of the research products.

Intersection inventory, traffic volume and crash data were provided by the City of Toronto, Canada, and from various urban areas in North Carolina.

METHODOLOGY

The methodology applied was the empirical Bayes (EB) before-after study. The advantages of the EB approach are that it:

- Properly accounts for regression-to-the-mean
- Overcomes the difficulties of using crash rates in normalizing for volume differences between the before and after periods
- Reduces the level of uncertainty in the estimates of safety effect
- Provides a foundation for developing guidelines for estimating the likely safety consequences of contemplated installations
- Properly accounts for differences in crash experience and reporting practice in amalgamating data and results from diverse jurisdictions
- Avoids the difficulties of conventional treatment-comparison experimental designs caused by possible spillover and/or migration effects to natural comparison groups.

In the empirical Bayes evaluation of the effect of a treatment, the change in safety for a given crash type at a treated intersection is given by:

$$B - A, \tag{1}$$

where B is the expected number of crashes that would have occurred in the "after" period without the treatment and A is the number of reported crashes in the after period. Because of changes in safety that may result from changes in traffic volume, from regression-to-the-mean, and from trends in crash reporting and other factors, the count of crashes before a treatment by itself is not a good estimate of B, a reality that has now gained common acceptance. Instead, B is estimated from an empirical Bayes (EB) procedure in which a safety performance function (SPF) is used to first estimate the number of crashes that would be expected in each year of the "before" period at locations with traffic volumes and other characteristics similar to a treatment site being analyzed. The sum of these annual SPF estimates (P) is then combined with the count of crashes (x) in the before period at the treatment site to obtain an estimate of the expected number of crashes (m) before the treatment. This estimate of m is:

$$m = w_1(x) + w_2(P) \tag{2}$$

The weights w_1 and w_2 are estimated as:

$$w_1 = P/(P + 1/k) \tag{3}$$

$$w_2 = 1/k(P + 1/k), \tag{4}$$

where k is the overdispersion parameter of the negative binomial distribution that is assumed for the crash counts used in estimating the SPF. The value of k is estimated from the SPF calibration process with the use of a maximum likelihood procedure.

A factor is then applied to m from Equation 2 to account for the length of the after period and differences in traffic volumes between the before and after periods. This factor is the sum of the annual SPF predictions for the after period divided by P, the sum of these predictions for the before period. The result, after applying this factor, is an estimate of B. The procedure also produces an estimate of the variance of B, the expected number of crashes that would have occurred in the after period without the treatment.

The estimate of B is then summed over all sites in a treatment group of interest (to obtain B_{sum}) and compared with the count of crashes during the after period in that group (A_{sum}). The variance of B is also summed over all sections in the group of interest.

The index of safety effectiveness (θ) is estimated as:

$$\theta = (A_{sum}/B_{sum}) \,/\, \{1 + [Var(B_{sum})/B_{sum}^2]\}. \tag{5}$$

The standard deviation of θ is given by:

$$Stddev(\theta) = [\theta^2\{[Var(A_{sum})/A_{sum}^{\ 2}] + [Var(B_{sum})/B_{sum}^{\ 2}]\} \ / \ [1 + Var(B_{sum})/B_{sum}^{\ 2}]^2]^{0.5} \qquad 6)$$

The percent change in crashes is in fact $100(1-\theta)$; thus a value of $\theta = 0.7$ with a standard deviation of 0.12 indicates a 30 percent reduction in crashes with a standard deviation of 12%.

DATA COLLECTION

This section describes the sources and some summary statistics for the data used for the evaluation. Data were acquired from the City of Toronto, Canada, and from various urban areas in North Carolina for both treated and untreated signalized intersections. Following is a description of the data from the two locations.

Data from Toronto, Canada

The City of Toronto maintains a database of signalized intersections including many variables related to geometry (e.g. number of lanes by type by approach), traffic volumes and crash data. Crashes within 25 metres (about 82 feet) are identified as related to an intersection. Volume and crash data from 1999 to 2007 were collected for this evaluation.

This database was augmented by querying the crash data for specific crash types and adding left-turn AADTs. A separate database of intersection approaches was also created as it was desired to evaluate left-turn protection improvements at both the intersection-level and approach-level. Intersections at which only one approach had an improvement in left-turn protection were used for the approach-level analysis.

Treated sites were identified in a two-step process. First, an electronic file of work orders for signalized intersections was scanned to identify sites where a change in left-turn phasing was made. Using this list, a subsequent search of hard copy signal timing reports for these sites identified those where the left-turn phasing on at least one approach was changed to either protected-permissive or fully protected at any time of day. Only one site was found where the phasing was change to fully protected, and this site was dropped from the analysis.

Unfortunately the complexity of multiple phasing plans made the identification of sites with the identical before and after condition an impossible task. At some treated sites there were approaches that already had protected-permissive or protected phasing and others that did not. Additionally, some identified treatment sites had left-turn protection added for all times of the day while others had these phases added for only one or more peak periods. The result is that the group of treated sites represents a range of before and after conditions with regard to left-turn phasing options. Hence, approaches and intersections were classified based on the predominant phasing system.

A reference group of untreated signalized intersections was identified to match the treatment sites based on site characteristics, including number of approaches, presence of left-turn lanes and traffic volumes.

The definitions of crashes for both the intersection-level and approach-level analyses are provided below:

Crash Definitions for Intersection-Level Analysis
Total – Within 25 metres of intersection
Injury – Severity level fatal or non-fatal injury
Rear-end – Identified as impact type ‘rear-end’
Left-turn – Any vehicle involved in a crash making a left-turn
Left-Turn-Opposing – One vehicle making a left-turn with the second vehicle proceeding straight through from the opposing approach

Crash Definitions for Approach-Level Analysis
Total – Within 25 metres of intersection and one or more vehicles originating from treated or opposite approach
Injury – Severity level fatal or non-fatal injury
Rear-end – both vehicles originating from treated approach and either proceeding straight or making a left-turn
Left-turn – One or more vehicles originating from treated approach and making a left-turn
Left-Turn-Opposing – Left-turning vehicle is from treated approach and the second vehicle proceeding straight from the opposing approach

Tables D-1 through D-4 provide summary statistics for treatment and reference sites for both the intersection and approach level analyses.

Table D-1: Summary Statistics 59 Sites Used for the Intersection-Level Analysis (Toronto)
(Sites had 4-approaches, with one or more converted from permissive to protected-permissive operation)

Variable	Minimum	Maximum	Average
Years before	1.00	7.00	4.03
Years after	1.00	7.00	3.97
Number of treated Approaches	1	4	1.27
Left-turn lanes on major road	0	3	1.69
Left-turn lanes on minor road	0	1	0.88
Major road AADT before	14,489	74,990	35,253
Major road AADT after	11,504	73,697	35,094
Minor road AADT before	1,466	42,723	18,209
Minor road AADT after	1,466	37,491	18,636
Major road left-turn AADT before	28	9,830	3,351
Major road left-turn AADT after	146	11,110	3,563
Minor road left-turn AADT before	124	7,442	3,364
Minor road left-turn AADT after	124	8,586	3,433
Total crashes/year before	0.0	63.0	26.09
Total crashes/year after	0.0	54.2	25.7
Injury crashes/year before	0.0	22.0	9.0
Injury crashes/year after	0.0	14.3	6.5
Left-turn crashes/year before	0.0	25.0	7.2
Left-turn crashes/year after	0.0	17.7	6.9
Left-turn opposing crashes/year before	0.0	14.0	3.0
Left-turn opposing crashes/year after	0.0	5.8	2.1
Rear-end crashes/year before	0.0	30.3	10.0
Rear-end crashes/year after	0.0	24.8	9.7

Table D-2: Summary Statistics 626 Reference Sites Used for the Intersection-Level Analysis (Toronto)

Variable	Minimum	Maximum	Average
Number of approaches	3	4	3.96
Left-turn lanes on major road	0	3	1.31
Left-turn lanes on minor road	0	3	0.85
Major road AADT	11,080	63,596	29,270
Minor road AADT	1,190	44,967	9,510
Major road left-turn AADT	183	9,585	1,648
Minor road left-turn AADT	16	5,821	692
Total crashes/year	0.0	56.2	13.2
Injury crashes/year	0.0	18.1	3.8
Left-turn crashes/year	0.0	17.7	3.3
Left-turn opposing crashes/year	0.0	7.2	1.1
Rear-end crashes/year	0.0	34.9	4.7

Table D-3: Summary Statistics 46 Treatment Sites Used for the Approach-Level Analysis (Toronto)

Variable	Minimum	Maximum	Average
Left-turn lanes	0	1	0.80
Approach AADT before	5,536	27,880	13,522
Approach AADT after	4,690	28,160	13,787

Opposing AADT before	4,168	27,788	13,755
Opposing AADT after	4,632	26,092	13,497
Perpendicular AADT before	2,978	49,444	22,611
Perpendicular AADT after	3,462	49,766	22,703
Approach left-turn AADT before	589	4,580	1,847
Approach left-turn AADT after	704	4,580	1,880
Total crashes/year before	0.0	40.5	14.8
Total crashes/year after	3.3	32.0	14.2
Injury crashes/year before	0.0	14.5	5.1
Injury crashes/year after	0	7.8	4.0
Left-turn crashes/year before	0.0	7.0	2.3
Left-turn crashes/year after	0.0	11.0	2.2
Left-turn opposing crashes/year before	0.0	4.0	1.0
Left-turn opposing crashes/year after	0.0	2.7	0.7
Rear-end crashes/year before	0.0	6.0	1.5
Rear-end crashes/year after	0.0	5.0	1.1

Table D-4: Summary Statistics 552 Reference Sites Used for the Approach-Level Analysis (Toronto)

Variable	Minimum	Maximum	Average
Left-turn lanes	0	2	0.67
Approach AADT	630	36,974	10,641
Opposing AADT	9	32,158	9,206
Perpendicular AADT	1,460	61,646	19,935
Left-turn AADT on approach	5	6,412	598
Total crashes/year	0.1	39.0	8.1
Injury crashes/year	0.0	13.8	1.6
Left-turn crashes/year	0.0	7.6	1.1
Left-turn opposing crashes/year	0.0	5.0	0.4
Rear-end crashes/year	0.0	7.9	0.6

Data from North Carolina

In North Carolina, data were available for nineteen four-leg intersections that experienced a change in left turn phasing on at least one leg of the intersection. All these nineteen sites were in urban areas. The change in phasing was one of the following three categories:

- From Permissive to Protected-Permissive (12 intersections)
- From Permissive or Protected-Permissive to Protected (5 intersections)
- From Protected to Permissive or Protected-Permissive on at least 2 legs (2 intersections)

Since the number of intersections in the last two categories is limited (less than 10 intersections), results are provided here only for the first category of sites, i.e., for intersections where the phasing was changed from permissive to protected-permissive phasing in at least one leg of the intersection. All the treatment locations had a left turn on the major legs. Hence, for the before-after empirical Bayes analysis, only sites with turn lanes on the major legs were included in a reference group. In addition, the reference group of untreated signalized intersections was identified to match the treatment sites based on site characteristics, including number of approaches, and left turn phasing. Signal plans were reviewed in order to ensure that reference sites did not undergo significant changes in phasing and geometry during the study period. Data for the treatment and reference sites were compiled from 1993 to 2008.

The target crash types included:

- Total intersection crashes
- Total intersection injury and fatal crashes
- Total intersection rear end crashes
- Total intersection left-turn opposing through crashes

Crashes within 250 feet of an intersection were classified as intersection crashes. Unlike Toronto, crash data by approach were not available in North Carolina without a manual review of crash reports. So, the analysis of North Carolina focused only on intersection level crashes.

Table D-5 shows the summary statistics for the treatment sites used in North Carolina. Table D-6 shows the summary statistics for the reference sites from North Carolina used for the development of Safety Performance Functions. In North Carolina, data were not available on turning volumes, but major and minor road AADT were available.

Table D-5 Summary Statistics for 12 Treatment Sites Used in North Carolina
(Sites had 4-approaches, with one or more converted from permissive to protected-permissive operation)

Variable	Minimum	Maximum	Average
Years before	2.0	10.0	6.8
Years after	2.0	10.0	5.8
Number of treated Approaches	1	4	1.9
Number of approaches	4	4	4
Left-turn lanes on major road	2	2	2
Major road AADT before	4,857	18,766	12,302
Major road AADT after	6,000	22,000	15,017
Minor road AADT before	1,715	9,300	5,124
Minor road AADT after	2,388	12,000	6,512
Total crashes/year before	1.8	14.3	6.4
Total crashes/year after	3.0	15.7	7.0
Injury and fatal crashes/year before	0.9	5.6	2.7
Injury and fatal crashes/year after	1.1	5.3	2.6
Rear-end crashes/year before	0.8	5.3	2.0
Rear-end crashes/year after	0.9	5.7	2.5
Left-turn opposing crashes/year after	0.1	3.1	1.4
Left-turn opposing crashes/year before	0.0	3.7	1.4

Table D-6: Summary Statistics for 49 Reference Sites Used in North Carolina

Variable	Minimum	Maximum	Average
Number of approaches	4	4	4
Major road AADT	2,757	40,000	14,188
Minor road AADT	50	23,000	4,986
Total crashes/year	0	57	6.5
Injury and Fatal crashes/year	0	16	2.4
Rear end crashes/year	0	16	2.0
Left-turn opposing through crashes/year	0	13	1.2

DEVELOPMENT OF SAFETY PERFORMANCE FUNCTIONS (SPFs)

This section presents the safety performance functions (SPFs) that were developed. The SPFs are used in the EB methodology to estimate the safety effectiveness. Generalized linear modeling was used to estimate model coefficients and assuming a negative binomial error distribution, which is consistent with the state of research in developing these models.

A number of SPFs were calibrated as follows:

- SPFs were calibrated separately for Total, Injury, Left-turn, Left-turn-opposing and Rear-end crashes as defined in the previous section.
- SPFs at the intersection-level and approach-level were separately developed for Toronto. For North Carolina, SPFs were estimated at the intersection-level
- For the City of Toronto, separate models were also developed for intersections without and with one-way roads.

Safety Performance Functions from Toronto

<u>Intersection-Level Safety Performance Functions for Two-Way Roads</u>

The intersection-level models of intersections without one-way roads are shown in Table D-7. These models are of the form:

Crashes/year = $e^{(alpha)}(MajAADT)^{b}(MinAADT)^{c}e^{(inttype*d+ltmaj*e+ltmin*f+Intersection\_Class)}$

Where,
MajAADT = the major road AADT
MinAADT = the minor road AADT
ltmaj = 1 if one or more left-turn lanes are present on the major road
ltmin = 1 if one or more left-turn lanes are present on the minor road
inttype = 1 if four approaches; 0 if three approaches
Intersection_class – various categories (not all statistically different from each other but kept in as a multiplier)

Intersection_class defines the classes of roads intersecting as follows:
<u>4-legged intersections</u>

1. Private, Locals and Collectors/Private, Locals and Collector
2. Minor Arterial / Private, Local
3. Minor / Collector
4. Minor / Minor
5. Major / Private, Local
6. Major / Collector
7. Major / Minor
8. Major / Major
9. Expressway

<u>3-legged intersections:</u>

10. Private, Locals, Collectors and Minor Arterial / Private, Locals, Collectors and Minor Arterial
11. Major Arterial / Private, Local
12. Major Arterial / Collector
13. Major Arterial / Minor Arterial, Major Arterial

Table D-7: Intersection-Level Safety Performance Functions for Two-Way Roads

Crash Type	alpha (s.e.)	b (s.e.)	c (s.e.)	d (s.e.)	e (s.e.)	f (s.e.)	Intersection_Class	dispersion parameter (s.e.)
Total	-8.2165 (0.7370)	0.4480 (0.0729)	0.6037 (0.0464)	-0.0908 (0.0904)	0.2133 (0.0520)	0.1986 (0.0435)	2 0.1824 3 0.9302 5 0.7306 6 0.5383 8 0.5836 9 0.3608 10 -0.6393 12 0.4087 13 0.0000	0.1649 (0.0111)
Injury	-8.5132 (0.7297)	0.4630 (0.0717)	0.4894 (0.0514)	0.0179 (0.0981)	0	0.1321 (0.0468)	2 -0.7280 3 0.8986 5 0.6657 6 0.5754 8 0.5093 9 0.2280 10 -0.5442 12 0.3404 13 0.0000	0.1685 (0.0131)
Left-Turn	-12.6341 (0.9372)	0.7058 (0.0956)	0.6005 (0.0591)	0.1436 (0.1201)	0.2009 (0.0674)	0	2 -18.8367 3 1.1141 5 0.9857 6 1.0301 8 0.8364 9 0.6523 10 -0.1326 12 0.6418 13 0.0000	0.2501 (0.0189)
LTOPP	-15.7978 (1.2410)	0.8527 (0.1253)	0.5849 (0.0785)	0.8999 (0.1832)	0.1906 (0.0889)	0	2 -19.0407 3 1.1274 5 1.0131 6 1.1039 8 0.8281 9 0.5487 10 -0.3229 12 0.4822 13 0.0000	0.3673 (0.0337)
Rear-End	-12.8195 (0.8556)	0.8319 (0.0848)	0.5602 (0.0526)	0.0742 (0.1053)	0.1927 (0.0599)	0.1878 (0.0503)	2 0.8401 3 0.8810 5 0.5012 6 0.3348 8 0.2940 9 0.0853 10 -0.9425 12 0.1758 13 0.0000	0.1919 (0.0145)

Intersection-Level Safety Performance Functions for One-Way Roads

The intersection-level models for intersections including one-way roads are shown in Table D-8. These models are of the form:

Crashes/year = $e^{(alpha)}(MajAADT)^b(MinAADT)^c$

Where,
MajAADT = the major road AADT
MinAADT = the minor road AADT

Table D-8: Intersection-Level Safety Performance Functions for One-Way Roads

Crash Type	alpha (s.e.)	b (s.e.)	c (s.e.)	dispersion parameter (s.e.)
Total	-8.3990 (1.8985)	0.5486 (0.1796)	0.5835 (0.1092)	0.4319 (0.0622)
Injury	-9.8323 (1.7905)	0.7028 (0.1706)	0.4091 (0.1018)	0.3373 (0.0566)
Left-Turn	-12.3749 (2.3291)	0.8226 (0.2199)	0.5448 (0.1349)	0.5717 (0.0910)
LTOPP	-13.2508 (2.9484)	0.9466 (0.2803)	0.3423 (0.1676)	0.7710 (0.1508)
Rear-End	-13.2095 (2.0141)	0.8905 (0.1890)	0.6178 (0.1115)	0.4583 (0.0733)

Approach-Level Safety Performance Functions

The approach-level models are shown in Table D-9. These models are of the form:

Crashes/year = $e^{(alpha)}(AppAADT)^b(OppAADT)^c(PerpAADT)^d\exp(left*e)\,(LTAppAADT)^f$

Where,
Where,
AppAADT = the approach AADT
OppAADT = the AADT on the opposite approach
PerpAADT = the AADT on the perpendicular roadway
LTAppAADT = the left-turning AADT on the approach
left = 1 if a left-turn lane is present on the approach

Table D-9: Approach-Level Safety Performance Functions

Crash Type	alpha (s.e.)	b (s.e.)	c (s.e.)	d (s.e.)	e (s.e.)	f (s.e.)	dispersion parameter (s.e.)
Total	-10.8148 (0.4993)	0.5587 (0.0866)	0.3285 (0.0414)	0.4732 (0.0269)	-0.3271 (0.0538)	0.0811 (0.0453)	0.2530 (0.0167)
Injury	-6.6999 (.6038)	0.2790 (0.12112)	0.3170 (0.1060)	0.2742 (0.0303)	-0.1110 (0.0589)		0.0644 (0.0093)
Left-Turn	-12.7005 (0.6542)	0.5198 (0.1165)	0.2185 (0.0594)	0.4851 (0.0340)	-0.2736 (0.0728)	0.2528 (0.0600)	0.3235 (0.0293)
LTOPP	-12.8687	0.5854	0.4367	0	-0.3555	0.4391	0.6809

Crash Type	alpha (s.e.)	b (s.e.)	c (s.e.)	d (s.e.)	e (s.e.)	f (s.e.)	dispersion parameter (s.e.)
	(0.8355)	(0.2153)	(0.1301)		(0.1184)	(0.0982)	(0.0686)
Rear-End	-10.0311 (0.5963)	1.0207 (0.0650)	0	0	0		1.0607 (0.0857)

Safety Performance Functions from North Carolina

In North Carolina, the SPFs (intersection level) were of the following form:

Crashes/year = exp{a + b**ln*(MajAADT/10000) + c*(MinAADT/10000)}

Table D-10 shows the parameter estimates, standard errors, and the overdispersion parameter (k) for each SPF that was estimated.

Table D-10: Intersection Level SPFs from North Carolina

Crash Type	**a(s.e.)**	**b(s.e.)**	**c(s.e.)**	**k**
Total	1.3333 (0.04507)	0.7502 (0.05994)	0.5044 (0.07144)	0.4116
Injury and Fatal	0.4984 (0.05358)	0.6680 (0.07216)	0.2744 (0.08165)	0.4023
Rear End	0.1226 (0.06034)	0.8960 (0.08413)	0.4670 (0.08888)	0.4921
Left Turn Opposing Through	-0.3696 (0.06993)	0.5564 (0.09255)	0.6585 (0.09944)	0.5641

EVALUATION RESULTS AND DISCUSSION

Evaluation Results from Toronto

As noted earlier, the evaluation was done at both the intersection and approach levels in Toronto. Tables D-11 and D-12 show the results including the expected crashes in the after period based on the EB procedure, the observed crashes in the after period, the CMF, and its standard error. CMFs that are statistically different from 1.0 at the 5% level are shown in bold.

At both levels, the results indicate benefits for the target crash type, involving a left turn vehicle and a through vehicle from the opposing approach (LTOPP). As expected, the benefit at the intersection level is greater at intersections where more than one approach is treated. Among the 12 sites where more than 1 approach was treated, 2 approaches were treated in 9 sites, 3 approaches were treated in 2 sites, and 4 approaches were treated in site.

One of the fundamental questions the study was expected to answer was the extent to which the decrease in target crashes may be offset by a compensating increase in a non-target crash type such as rear-end. At both the intersection and approach levels, there were small percentage increases in rear-end crashes. This increase in rear end crashes was statistically significant (at the 5% level) for sites where only one approach was treated (see Table D-11). So, there is sufficient indication to suggest that rear end crashes may increase when left turn phasing is changed from permissive to protected-permissive.

Disaggregation of the effects by AADT, either total entering or left turn, did not reveal any trend. This may be because the intersections did not have a wide enough distribution of these variables

Table D-11: Intersection Level Evaluation Results for Toronto (Permissive to Protected-Permissive)

Crash Type	Grouping	No. Sites	EB Expected in the after period	Observed crashes in the after period	CMF (s.e.)
All	All sites	59	5,916	6,111	1.033 (0.023)
	1 treated approach	47	3,690	4,004	**1.085** (0.028)
	>1 treated approach	12	2,226	2,107	0.945 (0.040)
Injury and Fatal	All sites	59	1620	1553	0.958 (0.037)
	1 treated approach	47	1010	1016	1.005 (0.045)
	>1 treated approach	12	610	537	**0.878** (0.062)
LTOPP	All sites	59	568	489	**0.858** (0.056)
	1 treated approach	47	341	314	0.919 (0.069)
	>1 treated approach	12	227	175	**0.762** (0.088)
Rear end	All sites	59	2,166	2,304	1.063 (0.038)
	1 treated approach	47	1,266	1,383	**1.091** (0.046)
	>1 treated approach	12	900	921	1.021 (0.062)

Table D-12: Approach Level Aggregate Evaluation Results for Toronto (Permissive to Protected-Permissive) (46 sites)

Crash Type	EB Expected in the after period	Observed crashes in the after period	CMF (s.e.)
All	2307	2487	**1.077** (0.037)
Injury and Fatal	602	693	**1.150** (0.056)
LTOPP	147	114	**0.776** (0.098)
Rear end	177	197	1.103 (0.118)

Evaluation Results from North Carolina

Table D-13 shows the results of the before-after analysis from North Carolina. Consistent with the results from Toronto, there seems to be a reduction in left turn opposing through crashes and an increase in rear end crashes as a result of changing from permissive to protected-permissive phasing. None of the changes in the CMFs are statistically significant at the 5% percent significance level, probably because of the limited sample size.

Table D-13: Intersection Level Results for North Carolina

Crash Type	Grouping (number of approaches treated per site)	Number of sites	EB Expected in the after period	Observed crashes in the after period	CMF (S.E.)
Total	All sites	12	552.1	562	1.015 (0.066)
	1 treated approach	3	147.4	147	0.985 (0.132)
	>1 treated approach	9	404.7	415	1.023 (0.075)
Injury and Fatal	All sites	12	206.0	206	0.995 (0.100)
	1 treated approach	3	57.4	48	0.815 (0.172)
	>1 treated approach	9	148.6	158	1.057 (0.118)
Left Turn Opposing Through	All sites	12	131.9	115	0.865 (0.111)
	1 treated approach	3	29.8	29	0.928 (0.252)
	>1 treated approach	9	102.1	86	0.834 (0.120)
Rear end	All sites	12	159.7	199	1.236 (0.142)
	1 treated approach	3	43.5	51	1.128 (0.266)
	>1 treated approach	9	116.2	148	1.261 (0.162)

Combined Intersection Level Results from Toronto and North Carolina

Table D-14 shows the combined results from Toronto and North Carolina at the intersection level. Since Toronto had a much larger number of sites with more crashes, the combined results are somewhat dominated by the Toronto data. However, given that the results are generally similar between the two jurisdictions, it seems worthwhile to base the final conclusions on the combined results. It is clear that there is a reduction in left turn opposing through crashes when left turn phasing is changed from permissive to protected-permissive phasing. As expected, the benefits are larger if more than 1 approach is treated, compared to when only 1 approach is treated (17 out of the 21 sites in the ">1 treated approach group" had 2 approaches that were treated). The percentage increase in rear end crashes is smaller compared to the percentage reduction in left turn opposing through crashes.

Table D-14: Combined Intersection Level Results from Toronto and North Carolina

Crash Type	Grouping (number of approaches treated per site)	Number of sites	EB Expected in the after period	Observed crashes in the after period	CMF (S.E.)
Total	All sites	71	6468	6673	1.031 (0.022)
	1 treated approach	50	3837	4151	**1.081** (0.027)
	>1 treated approach	21	2631	2522	0.958 (0.036)
Injury and Fatal	All sites	71	1826	1758	0.962 (0.035)
	1 treated approach	50	1067	1063	0.995 (0.043)
	>1 treated approach	21	759	695	0.914 (0.055)
Left Turn Opposing Through	All sites	71	837	604	**0.862** (0.050)
	1 treated approach	50	370	343	0.925 (0.067)
	>1 treated approach	21	330	261	**0.787** (0.072)
Rear end	All sites	71	2326	2503	**1.075** (0.036)
	1 treated approach	50	1310	1434	**1.094** (0.045)
	>1 treated approach	21	1016	1069	1.050 (0.059)

CONCLUSIONS

Data from signalized intersections in Toronto and urban areas in North Carolina were used to examine the safety impact of changing the left turn phasing from permissive to protected-permissive. Fifty nine intersections from Toronto and 12 intersections from North Carolina were included in a before-after empirical Bayes evaluation. In Toronto, the analysis included the examination of different types of crashes at both intersection and approach levels. In North Carolina, the analysis was done only at the intersection level. At least one of the previous studies had suspected a possible increase in rear end crashes when left turn phasing is changed to protected or protected-permissive operation from permissive operation. So, one of the motivations of this study was to investigate if there was a change in rear end crashes along with the anticipated change in left turn crashes.

Results from each jurisdiction and the combined results indicated a substantial reduction (statistically significant at the 5% level) in intersection level left turn opposing through crashes. The reduction in left turn crashes was larger at intersections where more than 1 approach was treated. The combined results show a small percentage increase in rear end crashes, which was statistically significant at the 5% level.

Following are the CMFs for intersection level left turn opposing through crashes based on combined data from Toronto and North Carolina:

All sites: CMF = 0.862 with a standard error 0.050
Sites where 1 approach was treated: CMF = 0.925 with a standard error of 0.067
Sites where more than 1 approach was treated: CMF = 0.787 with a standard error of 0.072

The CMF for approach level turn opposing through crashes was 0.776 with a standard error of 0.098. This was based on just Toronto data.

In applying these CMFs to evaluate the effects of a potential treatment, the negative impact on the frequency of rear-end crashes also needs to be considered, along with the relative severity of the left turn opposed and rear-end crashes. For this, the results indicate a CMF for rear-end crashes for all sites is 1.075 with a standard error of 0.036.

Further research could investigate the specific safety effects of changing left turn phasing during particular times of day (e.g., peak versus off-peak) and days of the week (e.g., weekday versus weekend). Another area of research is to investigate the effect of combined left turn treatments: adding a left turn lane and changing the left turn phase at the same time. There were a few sites in North Carolina where such combined treatments were implemented, but they were not sufficient to conduct an evaluation.

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NCHRP PROJECT 17-35

APPENDIX E:

Evaluation of Flashing Yellow Arrow for Permissive Left Turn Movements at Signalized Intersections

INTRODUCTION

The use of a flashing yellow arrow (FYA) for permissive left-turn movements at signalized intersections is intended to avoid the confusion for drivers turning left on a permissive circular green signal. The concern is that drivers turning left on a permissive circular green signal indication might mistake that indication as implying the left turn has the right of way over opposing traffic, especially under some geometric conditions.

There is currently little published work evaluating the safety effects of flashing yellow arrows providing an estimate of effects on crash frequency. The Federal Highway Administration (FHWA) maintains a web page (http://mutcd.fhwa.dot.gov/resources/interim_approval/ia_10_flashyellarrow.htm) devoted to flashing yellow arrows with some information, including a citation from NCHRP report 493 (Brehmer et al., 2003) that flashing yellow arrows were the best alternative to circular green for a permissive signal for left-turn movement. In the research on which that report is based, a variety of analyses were performed, including engineering analyses, static and video-based driver comprehension studies, field implementation, video conflict studies, and crash analyses. Key findings of that research include:

- FYA was found to be the best overall alternative to the circular green as the permissive signal display for a left-turn movement
- FYA was found to have a high level of understanding and correct response by left-turn drivers
- FYA display in a separate signal face for the left-turn movement offers more versatility in field application. It is capable of being operated in any of the various modes of left-turn operation by time of day, and is easily programmed to avoid the "yellow trap" associated with some permissive turns at the end of the circular green display.

In light of these findings, the NCHRP Report 493 recommends that the FYA be allowed as an alternative to the circular green for permissive left-turn intervals.

Noyce et al. (2007) evaluated the safety effectiveness of FYA permissive left-turn indication considering variables such as signal phasing, vehicle flow rates, posted speed limits, and intersection geometry. The sites used represented diverse intersection characteristics. The study employed several evaluation methodologies, the most appropriate being an empirical Bayes before-after study for nineteen treated locations with the appropriate data available. Target crashes were left-turn crashes and the findings of this study, included:

- Safety was improved at intersections that already had protected-permissive left turn (PPLT) phasing prior. The authors indicated that the change in signal phasing had a more significant impact on safety than the change to FYA permissive indication.
- Safety was not improved at intersections that operated with protected only left-turn phasing prior
- Results were inclusive for intersections that operated with permissive only left-turn phasing prior, due to a minimal number of implementation sites and data.

These findings were largely qualitative, with the main results being numbers of treated intersections that had increases or decreases in crashes following FYA implementation, rather than crash modification factors. The authors recognized that “further evaluations can improve the strength of the statistical results by including a larger number of ‘after’ years in the data set and improving the completeness of the data available for analysis”. Another acknowledged limitation of the study was that “evaluation of individual sites, where known changes occurred apart from the implementation of the FYA indication, was not included”.

Following the publication of NCHRP Report 493, FHWA has approved additional experimentation with the FYA by numerous jurisdictions.

The following signal displays for FYA are prescribed on the FHWA website.

1. During a <u>protected left-turn movement</u>, the left-turn signal face shall display only a steady left-turn GREEN ARROW signal indication.
2. During a <u>permissive left-turn movement</u>, the left-turn signal face shall display only a flashing left-turn

YELLOW ARROW signal indication.

3. During a prohibited left-turn movement, the left-turn signal face shall display only a steady left-turn RED ARROW or a steady CIRCULAR RED.
4. A steady left-turn YELLOW ARROW signal indication shall be displayed following every steady left-turn GREEN ARROW signal indication.
5. A steady left-turn YELLOW ARROW signal indication shall be displayed following the flashing left-turn YELLOW ARROW signal indication if the permissive left-turn movement is being terminated and the left-turn signal face will subsequently display a steady red signal indication. The signal section that displays the steady left-turn YELLOW ARROW signal indication during change intervals shall not be used to display the flashing left-turn YELLOW ARROW signal indication for permissive left turns.
6. When a permissive left-turn movement is changing to a protected left-turn movement, a steady left-turn GREEN ARROW signal indication shall be displayed immediately upon termination of the flashing left-turn YELLOW ARROW signal indication. A steady left-turn YELLOW ARROW signal indication shall not be displayed between the display of the flashing left-turn YELLOW ARROW signal indication and the display of the steady left-turn GREEN ARROW signal indication.
7. During flashing mode operation the display of a flashing left-turn YELLOW ARROW signal indication shall be only from the signal section that displays a steady left-turn YELLOW ARROW signal indication during steady mode (stop-and-go) operation.

Figure E-1 illustrates an installation of FYA.

Figure E-1 Illustration of FYA Installation

Source: http://www.newsline.dot.state.mn.us/images/06/sept/20-yellowlight.jpg

OBJECTIVE

The objective of this study was to supplement the work of Noyce et al. (2007), using a much larger database, with longer analysis periods, to estimate the effects on crashes of changing the permissive left-turn phasing guidance provided to drivers to the FYA schema at signalized intersection approaches in a number of jurisdictions in Oregon, Washington, and North Carolina. In particular, the study sought to develop crash modification factors, where possible, disaggregated by intersection characteristics and left turn protection scheme before FYA implementation.

For the treated sites, some approaches previously had permissive, protected-permissive or protected left-turn phasing. The after condition in all the treated approaches of the sites investigated was a FYA protected-permissive phasing system.

The phasing may also vary by time of day or day of week. Thus, the treated sites represent a range of specific timing characteristics. The commonality for treated approaches is, however, that a FYA scheme was applied to the permissive phasing when it was in use.

The basic objective was to estimate the change in target crashes for FYA treated intersections grouped in various ways. Possible target crash types include:

- Total crashes
- Injury crashes
- All left-turn crashes at the intersection
- All left-turn crashes from the treated FYA approach

Meeting these objectives placed some special requirements on the data collection and analysis tasks, including the need to:

- Select a large enough sample size to detect, with statistical significance, what may be small changes in safety for some crash types.
- Carefully select comparison or reference sites.
- Properly account for traffic volume changes.
- Pool data from multiple jurisdictions to improve reliability of the results and facilitate broader applicability of the research products.

DATA COLLECTION

This section describes the sources of data used for the evaluation and some summary statistics for the sites used in the evaluation.

Kennewick, Washington

Data were acquired from the City of Kennewick, Washington, for thirty-three signalized intersections where FYA was installed. Six of these sites were converted between 2004 and 2006 while the remaining sites were converted in 2008. One of these six sites was not used due to other changes that occurred. Following a recommendation from the City of Kennewick, the first year of data after treatment was not used because there was an adjustment period. The initial six locations were selected because either new left-turn phases were scheduled due to growth or improved coordination was needed due to heavy commercial vehicle volumes. The remaining locations were selected primarily in anticipation of safety benefits of converting all existing protected-permissive left-turn phases to FYA and also with the aim to improve operations.

The City was able to provide many variables related to geometry (e.g. number of lanes by type and approach), traffic volumes in the form of major and minor road AADTs and peak hour left-turn movements, and crash data. The definitions of crashes are provided below:

Crash Definitions

Total – Crashes occurring at or related to the intersection
Left-Turn - All crashes that occurred at an intersection that involved a left-turning vehicle.
FYA Left-Turn - Crashes that involved a vehicle making a left-turn from an approach where a FYA display was installed.

Crash data were only available up to May, 2009, with the result that little data were available for the majority of sites, since most sites were converted in 2008. A decision was made to only analyze the six locations converted between 2004 and 2006 and to use the remaining locations as a comparison group, using only data up to and including 2007 (i.e., before FYA implementation). Because these remaining locations were converted as part of a policy to provide FYA in general, regression-to-the-mean should not be an issue and their use as a comparison group could be justified.

Tables E-1 and E-2 provide summary statistics for the treated and comparison locations.

Table E-1: Summary Statistics for 5 treatment sites in Kennewick, Washington

Variable	Minimum	Maximum	Average	Frequency
Years before	4.0	5.0	4.4	
Years after	0.1	1.8	0.9	
Phasing before[1]				Permissive - 1 Protected-permissive - 4 Protected - 0
Number of treated Approaches	2.0	4.0	2.8	
Number of approaches	4.0	4.0	4.0	
Major road AADT before	11,443	22,756	18,568	
Major road AADT after	11,392	26,630	22,137	
Minor road AADT before	3,020	11,765	6,729	
Minor road AADT after	2,476	13,745	6,926	
Major road left-turn AADT before	1,040	4,150	2,410	
Major road left-turn AADT after	813	5,260	2,685	
Minor road left-turn AADT before	670	3,120	2,016	

Minor road left-turn AADT after	840	3,333	2,265	
Total crashes/year before	3.5	12.6	8.5	
Total crashes/year after	2.3	12.0	7.0	
Left-turn crashes/year before	1.8	8.6	4.4	
Left-turn crashes/year after	1.7	12.0	5.4	
FYA Left-turn crashes/year before	1.8	8.0	3.4	
FYA Left-turn crashes/year after	0.0	4.0	2.0	

[1]Refers to the treated approach(es)

Table E-2: Summary statistics for 27 reference sites in Kennewick, Washington

Variable	Minimum	Maximum	Average
Number of approaches	3.0	4.0	3.9
Phasing[1]			Permissive - 4 Protected-permissive - 21 Protected – 2
Major road AADT	4,850	23,320	15,201
Minor road AADT	2,310	12,666	7,078
Major road left-turn AADT	178	4,050	1,624
Minor road left-turn AADT	470	7,738	2,756
Total crashes/year	1.4	11.6	4.7
Left-turn crashes/year	0.3	4.4	1.7
FYA Left-turn crashes/year[2]	0.0	4.4	3.9

[1]Refers to the approaches that were eventually treated
[2]Refers to the approaches that were eventually treated

Oregon Data

Data were collected for four cities in Oregon; Beaverton, Gresham, Oregon City and Portland. A brief summary of the data available in each city is provided, followed by summary statistics.

Beaverton

The City of Beaverton provided data for 15 sites with FYA, which previously were under signalized control, and for which sufficient data existed. Data were however not available for comparison sites. For 3 of the 15 treated sites traffic volumes were only available for one of the before and after time periods. For these sites it was assumed that the traffic volumes did not change. The first year of data after treatment was not used to allow for an adjustment period as was done for the data from Kennewick, WA.

Gresham

The City of Gresham provided data for 6 sites with FYA, which previously were under signalized control, and for which sufficient data existed. Data were also provided for 20 signalized intersections selected to serve as a comparison group. Only one traffic volume count was available for each site. It was necessary to assume that the traffic volumes did not change. The first year of data after treatment was not discarded due to a small number of crashes in the data.

Oregon City

The City of Oregon City provided data for 3 sites with FYA, which previously were under signal control, and for which sufficient data existed. Data were, however, not available for comparison sites. Traffic volumes were only available for one of the before and after time periods. It was necessary to assume that the traffic volumes did not change. The first year of data after treatment was not discarded due to a small number of crashes in the data.

Portland

The City of Portland provided data for 10 sites with FYA, which previously were under signalized control, and for which sufficient data existed. Data were also provided for 25 signalized intersections selected to serve as a comparison group. Only one traffic volume count was available for each site. It was therefore necessary to assume that the traffic volumes did not change. The first year of data after treatment was not discarded due to a small number of crashes in the data.

The 4 cities were able to provide many variables related to geometry (e.g. number of lanes by type by approach) and crash data.

The definitions of crashes are provided below:

Crash Definitions

Total – Crashes occurring at or related to the intersection
Left-Turn - All crashes that occurred at an intersection that involved a left-turning vehicle.
FYA Left-Turn - Crashes that involved a vehicle making a left-turn from an approach where a FYA display was installed.

Tables E-3 and E-4 provide summary statistics for the treated and comparison locations in Oregon. Left-turning volumes were not available for the sites in Oregon.

Table E-3: Summary Statistics for 34 treatment sites in Oregon

Variable	Minimum	Maximum	Average	Frequency
Years before	3.25	6.98	9.25	
Years after	0.67	6.67	2.88	
Phasing before[1]				Permissive - 3 Protected-permissive - 3 Protected – 24 Prohibited - 4
Number of treated Approaches	1	4	1.82	
Number of approaches	3	4	3.79	
Major road AADT before	8,260	32,350	22,490	
Major road AADT after	8,260	32,600	22,654	
Minor road AADT before	780	10,620	3,455	
Minor road AADT after	780	10,984	3,477	
Total crashes/year before	0.00	7.34	2.78	
Total crashes/year after	0.00	7.90	2.25	
Left-turn crashes/year before	0.00	4.43	0.92	
Left-turn crashes/year after	0.00	3.60	1.07	
FYA Left-turn crashes/year before	0.00	3.60	0.49	
FYA Left-turn crashes/year after	0.00	4.13	0.42	

[1]Refers to the treated approach(es)

Table E-4: Summary statistics for 45 reference sites in Oregon

Variable	Minimum	Maximum	Average
Number of approaches	3	4	3.87
Major road AADT	7,826	33,188	17,857
Minor road AADT	460	26,960	6,593
Total crashes/year	0.00	13.20	3.50

Left-turn crashes/year	0.00	2.70	0.72

Data from North Carolina

Data from sixteen four leg signalized intersections in urban areas from North Carolina were used to evaluate the safety impacts of installing flashing yellow arrow. In all the intersections, flashing yellow arrow (FYA) was introduced in two out of the four legs. The changes can be divided into the following three categories:

- Change from protected phasing to FYA protected-permissive in 2 legs of the intersection (5 intersections)
- Change from doghouse (conventional protected-permissive) to FYA protected-permissive in 1 leg and from permissive to FYA protected-permissive in another leg (5 intersections)
- Change from doghouse (conventional protected-permissive) to FYA protected-permissive in 2 legs of the intersection (6 intersections)

All the treatment intersections had left turn lanes on the major road. Hence, a reference group of signalized intersections with at least one left turn lane on the major road were used to develop SPFs. Turning volumes were not available at the treatment or reference sites. However, data on major and minor road AADT were available for the treatment and reference site. The target crash types included:

- Total intersection crashes
- Total intersection injury and fatal crashes
- Total intersection rear end crashes
- Total intersection left-turn crashes
- Total intersection left-turn opposing through crashes

Crashes within 250 feet of an intersection were classified as intersection crashes. Crash data by approach were not available in North Carolina without a manual review of crash reports. So, the analysis of North Carolina focused only on intersection level crashes.

Summary statistics from the North Carolina sites are presented in Tables E-5 and E-6.

Table E-5: Summary Statistics for 16 treatment sites in North Carolina

Variable	Minimum	Maximum	Average	Frequency
Years before	0.5	3.0	2.7	
Years after	1.2	3.8	2.1	
Phasing before[1]				Protected - 5 Protected-Permissive - 6 Combination of Permissive and Protected-Permissive - 5
Number of treated Approaches	2	2	2	
Number of approaches	4	4	4	
Major road AADT before	9,100	43,000	24,206	
Major road AADT after	9,700	38,000	22,346	
Minor road AADT before	600	11,350	5,048	
Minor road AADT after	600	10,000	5,007	
Total crashes/year before	2.7	20.3	11.5	
Total crashes/year after	4.2	31.6	10.8	
Injury and Fatal crashes/year before	1.0	10.7	5.1	

Injury and Fatal crashes/year after	1.2	11.4	4.1	
Rear end crashes/year before	0.7	7.0	3.4	
Injury crashes/ year before	0.4	13.3	3.8	
Left-turn crashes/year before	0.0	9.8	3.9	
Left-turn crashes/year after	0.6	12.6	3.0	
Left turn opposing through crashes/year before	0.0	9.8	3.5	
Left turn opposing through crashes/year after	0.6	8.8	2.2	

Table E-6: Summary statistics for 49 reference sites in North Carolina

Variable	Minimum	Maximum	Average
Number of approaches	4	4	4
Major road AADT	2,757	40,000	14,188
Minor road AADT	50	23,000	4,986
Total crashes/year	0	57	6.5
Injury and Fatal crashes/year	0	16	2.4
Rear end crashes/year	0	16	2.0
Left-turn crashes/year	0	14	2.0
Left-turn opposing through crashes/year	0	13	1.2

METHODOLOGY

Methodology for Oregon and Washington Sites

Because of the limited data available for reference sites in most of the jurisdictions the empirical Bayes methodology could not be applied with the required rigor.

Because the EB method could not be applied, it was crucial to ensure that regression-to-the-mean was not a concern for the treated locations. The cities did say that the sites were not selected based on crash history but some evidence of an absence of regression-to-the-mean was still desired. The investigation of potential regression-to-the-mean involved aggregating the crash data over all treatment sites and plotting the totals for each year before treatment (e.g., 1 year before treatment, 2 years before treatment, 3 years before treatment, etc.). This test was conducted for each city separately and for each it was concluded that there was no evidence for regression-to-the-mean notwithstanding the natural randomness of crash counts.

The methodology applied combines some aspects of the EB and C-G approaches. Similar to the empirical Bayes evaluation of the effect of a treatment, the change in safety for a given crash type at a treated intersection is given by:

$$B–A,$$

where *B* is the expected number of crashes that would have occurred in the "after" period without the treatment and *A* is the number of reported crashes in the after period.

The estimate of *B* is as follows:

$$B = OBS_b \left(\frac{Years_a}{Years_b} \right) (ADJ_{AADT})(ADJ_{Trend})$$

$$Var(B) = \left[\left(\frac{Years_a}{Years_b} \right) (ADJ_{AADT})(ADJ_{Trend}) \right]^2 OBS_b$$

Where,
OBS_b = the observed crash count in the before period
$Years_b$ = number of years of before period crash data
$Years_a$ = number of years of after period crash data
ADJ_{AADT} = adjustment for changes in AADT between the after and before periods
ADJ_{Trend} = adjustment for trends unrelated to treatment between the after and before periods

The adjustment for changes in AADT, ADJ_{AADT}, was estimated by using a Safety Performance Function (SPF) calibrated for the one jurisdiction (Kennewick, WA) that had sufficient data for this purpose. This adjustment was estimated by dividing the SPF estimate using the after period AADT by the SPF estimate using the before period AADT. The adjustment accounts for changes in expected crash frequency due to traffic volume changes while not assuming that crash frequencies increase linearly with traffic volume. For locations where only one traffic count was available the value of ADJ_{AADT} is equal to 1.

The adjustment for time trends, ADJ_{Trend}, is determined using a group of comparison sites. The sum of SPF predictions per year for the after period is divided by the sum of SPF predictions per year in the before periods for the comparison group. Where only one traffic volume was available for the entire analysis period, the factor was simply calculated as the sum of crashes per year in the after period divided by the sum of crashes per year in the before period for the comparison group sites. Since not all treatments were in the same year there were several combinations of before and after periods used when calculating the time trend adjustment factors.

The estimate of B is then summed over all sites in a treatment group of interest (to obtain B_{sum}) and compared with the count of crashes during the after period in that group (A_{sum}). The variance of B is also summed over all sections in the group of interest.

The index of safety effectiveness (θ) is estimated as:

$$\theta = (A_{sum}/B_{sum}) \,/\, \{1 + [Var(B_{sum})/B_{sum}^2]\}.$$

The standard deviation of θ is given by:

$$Stddev(\theta) = [\theta^2\{[Var(A_{sum})/A_{sum}^2] + [Var(B_{sum})/B_{sum}^2]\} \,/\, [1 + Var(B_{sum})/B_{sum}^2]^2]^{0.5}$$

Some notes pertaining to the application of the methodology for each city are given below.

Kennewick

An SPF was used to account for traffic volume changes at the treated sites. Only one traffic volume count was available for the reference sites so comparison site crash counts were used to account for time trends.

Beaverton sites

An SPF was used to account for traffic volume changes at the treated sites. No comparison group was available so the comparison groups from Gresham and Portland were combined and used. A test of comparability as outlined in Hauer (1997) confirmed that this comparison group was adequate.

Gresham, Oregon

Only one traffic volume count was available so no correction for traffic volume changes at the treated sites was possible. Crash counts in the comparison group were used to account for time trends.

Oregon City, Oregon

Only one traffic volume count was available so no correction for traffic volume changes at the treated sites was possible. No comparison group was available so the comparison groups from Gresham and Portland were combined and used. A test of comparability as outlined in Hauer (1997) confirmed that this comparison group was adequate.

Portland, Oregon

Only one traffic volume count was available so no correction for traffic volume changes at the treated sites was possible. Crash counts in the comparison group were used to account for time trends.

Methodology for North Carolina Sites

In North Carolina, the state of the art empirical Bayes method could be applied. In the empirical Bayes evaluation of the effect of a treatment, the change in safety for a given crash type at a treated intersection is given by:

$$B – A, \qquad 1)$$

where B is the expected number of crashes that would have occurred in the "after" period without the treatment and A is the number of reported crashes in the after period. Because of changes in safety that may result from changes in traffic volume, from regression-to-the-mean, and from trends in crash reporting and other factors, the count of crashes before a treatment by itself is not a good estimate of B, a reality that has now gained common

acceptance. Instead, B is estimated from an empirical Bayes (EB) procedure in which a safety performance function (SPF) is used to first estimate the number of crashes that would be expected in each year of the "before" period at locations with traffic volumes and other characteristics similar to a treatment site being analyzed. The sum of these annual SPF estimates (P) is then combined with the count of crashes (x) in the before period at the treatment site to obtain an estimate of the expected number of crashes (m) before the treatment. This estimate of m is:

$$m = w_1(x) + w_2(P) \qquad 2)$$

The weights w_1 and w_2 are estimated as:

$$w_1 = P/(P + 1/k) \qquad 3)$$

$$w_2 = 1/k(P + 1/k), \qquad 4)$$

where k is the overdispersion parameter of the negative binomial distribution that is assumed for the crash counts used in estimating the SPF. The value of k is estimated from the SPF calibration process with the use of a maximum likelihood procedure.

A factor is then applied to m from Equation 2 to account for the length of the after period and differences in traffic volumes between the before and after periods. This factor is the sum of the annual SPF predictions for the after period divided by P, the sum of these predictions for the before period. The result, after applying this factor, is an estimate of B. The procedure also produces an estimate of the variance of B, the expected number of crashes that would have occurred in the after period without the treatment.

The estimate of B is then summed over all sites in a treatment group of interest (to obtain B_{sum}) and compared with the count of crashes during the after period in that group (A_{sum}). The variance of B is also summed over all sections in the group of interest.

The index of safety effectiveness (θ) is estimated as:

$$\theta = (A_{sum}/B_{sum}) / \{1 + [Var(B_{sum})/B_{sum}^2]\}. \qquad 5)$$

The standard deviation of θ is given by:

$$Stddev(\theta) = [\theta^2\{[Var(A_{sum})/A_{sum}^2] + [Var(B_{sum})/B_{sum}^2]\} / [1 + Var(B_{sum})/B_{sum}^2]^2]^{0.5} \qquad 6)$$

The percent change in crashes is in fact $100(1-\theta)$; thus a value of $\theta = 0.7$ with a standard deviation of 0.12 indicates a 30 percent reduction in crashes with a standard deviation of 12%.

DEVELOPMENT OF SAFETY PERFORMANCE FUNCTIONS (SPFs)

Safety Performance Functions from Oregon and Washington

For the sites in Kennewick, WA and Beaverton, OR for which traffic volume counts for both the before and after treatment periods were available, it was desired to account for the impact of volume changes on the expected number of crashes without treatment. To accomplish this an SPF was developed using reference site data from Kennewick. Generalized linear modeling was used to estimate model coefficients and assuming a negative binomial error distribution, which is consistent with the state of research in developing these models.

Only an SPF for total crashes was successfully developed. The SPF for total crashes was the following:

Crashes/year = $\exp(a)\{(MajAADT)^b (MinAADT)^c\}$

It was assumed that the coefficients for the major (MajAADT) and minor road (MinAADT) AADTs could be used for all crash types. Although this was not ideal it was felt that this approach would be more accurate than simply dividing the after period AADT by the before period AADT and in so doing incorrectly assuming a linear relationship between total entering traffic volumes and crashes.

Table E-7 shows the parameter estimates, standard errors, and the overdispersion parameter for the SPF that was estimated for total crashes

Table E-7: Intersection Level SPF from Kennewick

Crash Type	a(s.e.)	b(s.e.)	c(s.e.)	k
Total	-10.7112 (1.7507)	0.4945 (0.1744)	0.8458 (0.1268)	0.0494 (0.0212)

Safety Performance Functions from North Carolina

In North Carolina, the following form of the SPF was found to be the best:

Crashes/year = $\exp\{a + b*\ln(MajAADT/10000) + c*(MinAADT/10000)\}$

Table E-8 shows the parameter estimates, standard errors, and the overdispersion parameter (k) for each SPF that was estimated.

Table E-8: Intersection Level SPFs from North Carolina

Crash Type	a(s.e.)	b(s.e.)	c(s.e.)	k
Total	1.3333 (0.04507)	0.7502 (0.05994)	0.5044 (0.07144)	0.4116
Injury and Fatal	0.4984 (0.05358)	0.6680 (0.07216)	0.2744 (0.08165)	0.4023
Rear End	0.1226 (0.06034)	0.8960 (0.08413)	0.4670 (0.08888)	0.4921
Left Turn	0.07664 (0.05800)	0.6702 (0.07561)	0.6261 (0.08373)	0.4120
Left Turn Opposing Through	-0.3696 (0.06993)	0.5564 (0.09255)	0.6585 (0.09944)	0.5641

EVALUATION RESULTS AND DISCUSSION

The evaluation results are presented in Tables E-9, E-10, and E-11.

TABLE E-9: Results from Oregon and Washington

Group (number of sites) (number of legs treated)	Crash Type	Expected Crashes in the After Period	Observed Crashes in the After Period	CMF (S.E.)
Protected Before (24 sites) (46 legs treated)	Total	120.2	143	1.187 (0.115)
	Left-Turn	36.9	76	**2.043** (0.291)
	FYA Left-Turn	17.1	36	**2.073** (0.423)
Permissive before (4 sites) (10 legs treated)	Total	21.9	9	**0.404** (0.142)
	Left-Turn	6.5	4	0.580 (0.304)
	FYA Left-Turn	5.3	3	0.529 (0.314)
Protected/ Permissive before (7 sites) (15 legs treated)	Total	46.8	40	0.838 (0.174)
	Left-Turn	23.9	19	0.762 (0.225)
	FYA Left-Turn	18.6	11	**0.563** (0.202)

TABLE E-10: Results from North Carolina

Group (number of sites) (number of legs treated)	Crash Type	EB Expected Crashes in the After Period	Observed Crashes in the After Period	CMF (S.E.)
Protected on 2 legs in the before period (5 sites) (10 legs treated)	Total	102.8	156	**1.509** (0.165)
	Injury and Fatal	36.0	54	1.479 (0.265)
	Rear end	30.3	54	**1.752** (0.319)
	Left Turn	14.0	39	**2.683** (0.646)
	Left Turn Opposing Through	7.4	29	**3.696** (1.101)
Protected-Permissive on 1 leg and Permissive on 1 leg in the before period (5 sites) (10 legs treated)	Total	99.7	83	0.827 (0.112)
	Injury and Fatal	45.6	42	0.911 (0.170)
	Rear end	23.1	25	1.058 (0.258)
	Left Turn	45.0	29	**0.638** (0.135)
	Left Turn Opposing Through	38.8	23	**0.586** (0.137)
Protected-Permissive on 2 legs in the before period (6 sites) (12 legs treated)	Total	98.9	95	0.954 (0.127)
	Injury and Fatal	42.0	32	0.752 (0.156)
	Rear end	26.2	36	1.344 (0.297)
	Left Turn	33.6	28	0.820 (0.182)
	Left Turn Opposing Through	29.5	21	0.700 (0.173)

Results from Oregon and Washington appear to indicate that there is a benefit with the implementation of FYA at locations with some kind of permissive left turn operation before, and a disbenefit where there was protected only operation before. This is consistent with the results of Noyce et al. (2007). The benefit is larger for those sites with permissive only operation before compared to sites with protected-permissive left turn operation before the implementation of FYA. This is also consistent with the findings of Noyce et al. (2007).

The results from North Carolina were quite similar. There was a disbenefit when there was protected only operation before, and a benefit at locations when some kind of permissive left operation before the implementation of FYA. The group of intersections that had permissive operation in one leg and protected-permissive operation in one leg before the implementation of FYA experienced a larger reduction in left turn

and left turn opposing through crashes compared to the group of intersections that had protected-permissive operation in two legs before the implementation of FYA. Again, these results are consistent with the findings from Noyce et al. (2007).

That the results for North Carolina and the West Coast States were comparable made it possible to combine the results for the 3 States. Table E11 has these combined results. These results pertain to total crashes and total left turn crashes, the crash types that were common to the sets of two databases analyzed.

Table E11 shows the results for three types of changes:

- Intersections where the converted legs had either permissive or protected-permissive phasing in the before period, with at least one of the legs had permissive phasing. This group includes 9 four-leg intersections (total of 36 legs). A total of 20 legs were treated with FYA: 15 of the treated legs had permissive phasing in the before period while 5 of the treated legs had protective-permissive phasing in the before period.
- Intersections where the converted legs only had protected-permissive phasing in the before period. This group included 1 3-leg and 12 four-leg intersections (total of 51 legs). A total of 27 legs were treated with FYA; all of them had protected-permissive phasing in the before period.
- Intersections where the converted legs only had protected only phasing the before period. This group included 5 3-leg intersections and 24 4-leg intersections (total of 111 legs). A total of 56 legs were treated with FYA; all of them had protected only phasing in the before period.

Intersections in the first group experienced reductions in total intersection crashes and total intersection left turn crashes that were statistically significant at the 0.05 level. Intersections in the second group experienced a smaller reduction that was not statistically significant at the 0.05 level. As expected on the basis of individual results and those in Noyce at at. (2007), intersections in the third group (with protected only phasing in the before period) experienced significant increases in total and left turn crashes. As Noyce at al. commented, the change in signal phasing may have had a more significant impact on safety than the change to FYA permissive indication. Collectively, these results indicate that the largest benefit due can be found at sites where at least one of the converted legs had permissive only operation before the FYA was implemented with protected-permissive operation.

Further attempts to disaggregate the results to, e.g., see if a benefit may actually be achieved at some sites converted from protected operation, were unsuccessful. Left turn AADTs, a crucial variable for such an analysis was not available for a sufficient number of sites. Most of the sites had 2 legs that were converted (except for the few 3-leg intersections in the sample). So, it was not possible to specifically investigate the relationship between the number of legs that are treated and the associated safety benefits for left turn crashes.

Table E-11: Combined Results from Oregon, Washington, and North Carolina

Left Turn Phasing Before (sites) (legs treated)	Crash Type	Expected crashes in the after period	Observed crashes in the after period	CMF (S.E.)
Permissive or combination of permissive and protected-permissive (at least 1 converted leg was permissive in the before period) (9 sites) (20 legs treated)	Total Intersection Crashes	121.7	92	**0.753** (0.094)
	Total Intersection Left turn crashes	51.5	33	**0.635** (0.126)
Protected-Permissive (all converted legs had protected-permissive in the before period) (13 sites) (27 legs treated)	Total Intersection Crashes	145.7	135	0.922 (0.104)
	Total Intersection Left turn crashes	57.6	47	0.806 (0.146)

Protected (all converted legs had protected in the before period) (29 sites) (56 legs treated)	Total Intersection Crashes	223.0	299	**1.338** (0.097)
	Total Intersection Left turn crashes	50.9	115	**2.242** (0.276)

CONCLUSIONS

Data from 51 signalized intersections from Oregon, Washington, and North Carolina, were used to examine the safety impacts of implementing FYA protected-permissive phasing for left turn movements. Most of the intersections had 2 legs that were converted to FYA operation (except for the few 3-leg intersections in the sample).

Results indicate that there is a benefit with the implementation of FYA at locations with some kind of permissive left turn operation before, and a disbenefit where there was protected only operation before. The benefit is larger for those sites where at least one of the converted legs had permissive only operation before compared to sites where all the converted legs had protected-permissive left turn operation before the implementation of FYA.

Here are the CMFs for total intersection and total intersection left turn crashes.

Left Turn Phasing Before (sites) (legs treated)	Crash Type	CMF (S.E.)	Significant at 5% level?
At least 1 treated leg had permissive only (9 sites)(20 legs treated)	Total Intersection Crashes	0.753 (0.094)	Yes
	Total Intersection Left turn crashes	0.635 (0.126)	Yes
All treated legs had protected-permissive (13 sites)(27 legs treated)	Total Intersection Crashes	0.922 (0.104)	No
	Total Intersection Left turn crashes	0.806 (0.146)	No
All converted legs had protected (29 sites)(56 legs treated)	Total Intersection Crashes	1.338 (0.097)	Yes
	Total Intersection Left turn crashes	2.242 (0.276)	Yes

Future research could examine the relationship between the number of legs that are treated and the associated safety impacts for left turn crashes. Another area of research is to investigate the effect of left turn volume and opposing through volume on the safety impacts.

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