SAFETY EVALUATION OF TWO-LANE TO FOUR-LANE CONVERSIONS IN WISCONSIN

by

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Abstract

For this study, 12 two-lane to four-lane conversions constructed in Wisconsin within the last decade were analyzed. Five years of before crash data and a range of one to five years of after crash data (depending on the construction year) were collected, as well as geometric and volume data. A simple before-and-after analysis was completed to analyze specific types of injury crashes and manners of collision for each two-lane to four-lane conversion. An Empirical Bayes (EB) analysis was also used to determine the expected average crash volume per year to evaluate the safety benefits of the conversion.

Using a simple before-and-after crash analysis, average number of crashes per year was found to have reduced by between 7 to 82 % after the two-lane to four-lane conversion for all 12 projects. Five out of the 12 project sites reviewed reduced in all types of injury crashes and property damage crashes after the conversion. Three additional projects reduced in all types of injury crashes and property damage crashes after the conversion except in fatal injury in which there were no recorded crashes before or after the conversion. The remaining four projects showed an increase in fatal, incapacitating, non-incapacitating, or possible injury crashes per year. All projects, however, reduced in property damage only crashes per year. In all manners of collision considered, three projects showed a reduction in average crash volume per year. One additional project reduced in all types of injury crashes and property damage crashes after the conversion except in head on collision and sideswipe in the same direction crashes in which the recorded crashes before and after the conversion remained unchanged. The remaining eight projects showed an increase in angle, head on collision, no collision with another vehicle, rear end, or sideswipe in the same direction crashes per year. However, all projects reduced in sideswipe in the opposite direction crashes per year. The increase in crashes usually occurred at specific intersections within a project within a particular year and therefore was not thought to reflect the effect of the two-lane to four-lane conversion on the safety of the entire roadway section. Also, as four out the 12 projects had less than five years of comparable after conversion crash data, the percent increase or

decrease of average crash volume per year may be different for those four projects if five years of after conversion crash data is later used.

EB analysis was performed using the Highway Safety Manual (HSM) Rural 2-Lane Road spreadsheet to compare the observed average number of crashes per year to the expected average number of crashes per year if the two-lane to four-lane conversion had not been implemented. The expected average crash volume per year accounted for changes in traffic volumes between the before and after conversion periods. The HSM spreadsheet was populated with geometric and crash data before the conversion, and traffic volume data after the conversion. Without the conversion, expected average number of crashes per year increased as traffic volume increased. Therefore, when compared to the observed average number of crashes per year after the conversion, all the projects showed that expected average number of crashes per year was higher. The EB analysis proved that the two-lane to four-lane conversion resulted in a reduction in average number of crashes per year of between 10 and 85 %. One out of the 12 projects showed that expected average number of crashes per year still reduced without the conversion but a further reduction of approximately 10 to 11 % was obtained after the conversion. Effects of geometric considerations such as lane width, shoulder width and type, percent grade, intersection type, and number of turn lanes on safety from the EB analysis proved to be difficult to accurately determine as lane width, shoulder width and shoulder type remained unchanged within each project and so there was no data within the project to make the comparison while percent grade, intersection type, and number of turn lanes changed constantly with traffic volume within each project and so it was difficult to isolate the effect of the individual geometric characteristic on safety. However, percent grade was observed to have minimal effect on the expected average number of crashes per year while intersections with higher number of turn lanes, and signalized or four-leg stop control types had higher average number of crashes per year.

Overall, conclusion from both analyses was that two-lane to four-lane conversions are a safety benefit and result in average number of crashes per year reduction of as much as 85 %.

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CHAPTER 1 INTRODUCTION

As the U.S. and global populations continue to soar, an enormous strain is put on the natural and built environments. As traffic increases on a two-lane rural roadway, highway agencies are tasked with fixing the congested roadway by increasing capacity. Several options are considered including adding short sections of passing lanes to reduce traffic queues at intersections, restriping the travelway with narrower lanes and converting all or part of the shoulder to a travel lane, converting to a higher-order roadway such as a four-lane roadway, or simply leaving the roadway as a two-lane roadway (*1*, *3*). Leaving the roadway as a two-lane roadway may seem the best solution if there is a lack of funds; however, this solution does not fix the ever-increasing congestion problem. Adding short sections of passing lanes at intersections or restriping the travelway with narrower lanes and converting all or part of the shoulder to a travel lane, addresses the congestion issue but may not be adequate in some heavy congestion situations. A full-scale conversion to a higher-order roadway congestion solution is usually more costly as it involves acquiring additional rights-of-way, widening the existing roadbed, and/or regrading roadside areas. However, a full-scale conversion addresses the congestion issue more adequately by increasing capacity throughout the stretch of roadway. Given the constant increase in traffic volume on existing roadways and the lack of funds for new highway construction, higher-order conversions are becoming increasingly important to highway agencies such as state and local transportation departments, and the Federal Highway Administration (FHWA), as well as the general public. Currently, there is a lack of research on estimating safety benefits of actual two-lane to four-lane conversions. This lack is due in large part to the difficulty of conducting such research in that each two-lane to four-lane conversion will have different roadway geometry and traffic characteristics in the before-and-after configurations.

Previous research includes: a developed model that would take a set of existing (before) two-lane conditions and predict the benefit of conversion to a second set (after) of four-lane conditions, performed by Council and Stewart (*1*); a developed guidance in the decision to upgrade rural two-lane highways from a two-lane highway with wide shoulders to a four-lane highway with minimal shoulder, performed by Fitzpatrick et. al. (*2*); the safety benefit of converting shoulders and narrowing existing lanes to add high-occupancy-vehicle lanes to urban freeways, performed by Bauer et. al. (*3*); the safety benefit of converting the paved shoulders of a two-lane rural roadway to a four-lane roadway without paved shoulders (i.e., converting a roadway with the same right of way without adding any additional land), performed by Rogness et. al. (*4*); and a safety analysis of the addition of short four-lane passing sections to two-lane segments, performed by Harwood and St. John (*5*).

However, no research was found on the safety benefits of actual two-lane to four-lane conversions. Therefore, a simple before-and-after analysis and an Empirical Bayes (EB) analysis using HSM Rural 2-Lane Road spreadsheet to determine the safety benefits of 12 two-lane to four-lane conversions constructed in Wisconsin within the last decade was performed.

Other factors affecting road safety such as lane width, presence of shoulder and shoulder width, presence of median and median width, speed, traffic volume, and human factors are also addressed in this thesis. The factors affecting road safety are discussed in further detail in the literature review chapter.

This thesis is organized into six chapters. Chapter 1 presents the problem and goal of this study. The second chapter presents the literature review detailing previous lane conversion studies in other states as well as general factors that affect road safety. Chapter 3 describes the methodology for the simple before-and-after analysis, as well as the Empirical Bayes before-and-after analysis used for this study. The fourth chapter explains in detail the data collection and processing while the fifth chapter presents the final results and analyses. The sixth and final chapter presents the research conclusions and future study recommendations.

CHAPTER 2 LITERATURE REVIEW

The literature review chapter addresses factors affecting road safety. Road conversions are the main factor reviewed in this chapter. Five research papers on different road conversions are reviewed. However, none of the research papers reviewed is of actual two-lane to four-lane conversions. Council and Stewart's research is on a developed model that takes a set of existing (before) two-lane conditions and predicts the benefit of conversion to a second set (after) of four-lane conditions. Fitzpatrick et. al.'s research is on a developed guidance in the decision to upgrade rural two-lane highways from a two-lane highway with wide shoulders to a four-lane highway with minimal shoulders. Bauer et. al. looked at converting shoulders and narrowing existing lanes to add high-occupancy-vehicle lanes to urban freeways. Rogness et. al. looked at converting the paved shoulders of a two-lane rural roadway to a four-lane roadway without paved shoulders (i.e., converting a roadway with the same right of way without adding any additional land). Harwood and St. John performed a safety analysis of the addition of short four-lane passing sections to two-lane segments. Other factors affecting road safety are addressed as well. These are lane width, presence of shoulder and shoulder width, presence of median and median width, speed, traffic volume, and human factors.

Factors Affecting Road Safety

Road Conversions

Council and Stewart (*1*) used the methodology of developing cross-sectional models to predict crash rate per kilometer or mile as a measure of safety for typical two-lane, four-lane undivided and four-lane divided rural roadway sections in Minnesota, North Carolina, Washington, and California. Crash data in the research was obtained from the FHWA's Highway Safety Information System (HSIS), which is a research database containing crash, traffic, roadway inventory, and other related data from California, Illinois, Maine, Michigan, Minnesota, North Carolina, Utah, Washington, and Ohio. Roadway layouts before and after conversion were assumed to be based on American Association of State Highway and

Transportation Officials (AASHTO) typical sections. "Typical sections were defined by a careful review of guidelines in AASHTO's *A Policy on Geometric Design of Highways and Streets*, a review of state highway design standards from two different states, and cross-tabulations of surface width, shoulder width, and median width for state system mileage in four HSIS states" (*1*). The analysis database was also restricted to ADT between the 5th and 95th percentiles within the roadway classes and only included homogeneous roadway sections that were at least 0.16 km (0.10 mi) in length. The variables used in the analysis were segment length, surface, shoulder and median widths, and ADT, as shown in Table 2.1. The surface width represented the width of the paved travelway.

TABLE 2.1 Samples From Variables Used In Modeling (*1*)

	Two-lane Roads				Four-lane, Divided Roads				Four-Lane, Undiv.
	CA	MN	NC	WA	CA	MN	NC	WA	CA
					Model Restrictions				
Segment Length	≥.16 km (.1 mi)				≥ .16 km (.1 mi)				≥.16 km (.1 mi)
Surface Width	6.7-7.3 m (22-24 ft)				14.6 m (48 ft)				14.6 m (48 ft)
Shoulder Width	1.2-3.1 m (4-10 ft)				2.4-4.3 m (8-14 ft)				1.2-3.1 m (4-10 ft)
Median Width					9.2-30.2 m (30-99 ft) ("Variable" for MN)				
					Samples Used in Models				
Surface Width (Mean/ Std. Dev/ Range[a])	7.2 m (23.7 ft) 0.2 m (0.69 ft)	7.3 m (23.9 ft) 0.1 m (0.45 ft)	7.1 m (23.1 ft) 0.3 m (1.0 ft)	7.1 m (23.1 ft) 0.3 m (1.0 ft)	14.6 m (48 ft)	14.6 m (48 ft)	14.6 m (48 ft)	14.6 m (48 ft)	14.6 m (48 ft)
Shoulder Width (Mean/ Std. Dev/ Range)	1.9 m (6.4 ft) 0.6 m (1.9 ft)	2.4 m (7.8 ft) 0.6 m (1.9 ft)	2.1 m (6.9 ft) 0.6 m (1.9 ft)	2.0 m (6.4 ft) 0.6 m (1.8 ft)	2.9 m (9.6 ft) 0.5 m (1.5 ft) 2.4-4.0 m (8-13 ft)	2.9 m (9.4 ft) 0.3 m (0.9 ft) 2.4-3.4 m (8-11 ft)	3.5 m (11.6 ft) 0.5 m (1.6 ft) 2.4-4.3 m (8-14 ft)	3.0 m (9.9 ft) 0.1 m (0.5 ft) 2.4-3.7 m (8-12 ft)	1.9 m (6.1 ft) 0.6 m (2.1 ft) 1.2-3.1 m (4-10 ft)
Median Width (Mean/ Std. Dev/ Range)					19.7 m (64.6 ft) 6.7 m (21.9 ft) 9.2-30.2m (30-99 ft)	"Variable" (not used in model)	13.9 m (45.5 ft) 5.1 m (16.6 ft) 9.2-30.2m (30-99 ft)	17.2 m (56.4 ft) 5.5 m (18.0 ft) 10.7-25.0m (35-82 ft)	

[a]Range only shown when it differs from model restriction

Overdispersed Poisson models were fit to the data using the statistical analysis software SAS PROC GENMOD. A log link function was used in the actual modeling first. Predicted crashes per kilometer on a two-lane road were estimated as *A*. Crash data for North Carolina, Washington, and California was from 1993 to 1995 while crash data for Minnesota was from 1991 to 1993. Median width was included in the four-lane divided roadway models. Median width for Minnesota, however, was not included because a sizeable proportion of the segments had medians of inconsistent widths. Statistically insignificant variables (i.e., variables with $P > 0.05$) were deleted and the model re-estimated. To account for over dispersion in the modeling, standard errors and variances were inflated using a scale factor estimated as the square root of the chi-squared statistic divided by its degrees of freedom. Also, intersections and intersection-related crashes, as well as driveways, were omitted from the analysis database due to the potential bias between the models. Shown below is the analysis process.

$$L(x) = b_0 + \ln(\text{segment length}) + b_1[\ln(\text{ADT})] + b_2(\text{shoulder width}) + b_3(\text{surface width})$$

$$A = \exp\,[L(x)]$$

$$A = (\text{segment length}) * e^{b0} * (\text{ADT})^{\text{b1}} * e^{b2(\text{shoulder width})} * e^{b3(\text{surface width})}$$

Safety effect as a percentage of reduction in crashes per kilometer was then estimated by comparing the output of the two-lane model to that of the four-lane model at the same ADT using the two-lane output as the base.

$$R(v) = [\hat{a}_{21}(v) - \hat{a}_{41}(v)] \,/\, [\, \hat{a}_{21}(v)] \text{ x } 100$$

Where $\hat{a}_{21}$ and $\hat{a}_{41}$ represent the predicted crashes per kilometer for two-lane and four-lane roadway sections respectively, and v represents the ADT. Displayed in Table 2.2 are the estimated model parameters, standard errors, P-values, sample sizes and ADT ranges for each of the models.

TABLE 2.2 Crash Predictions (*1*)

North Carolina

	2-Lane Model			4-Lane Divided Model			4-Lane Undivided Model		
Parameter	Estimate	s.e.	P-value	Estimate	s.e.	P-value			
Intercept	-2.9915	.1772	.0001	-4.6914	.7393	.0004	Insufficient Data		
Log (ADT)	.6725	.0185	.0001	.7615	.1142	.0001			
Shoulder width	-.1230	.0194	.0001	-.2877	-.0945	.0023			
Surface width	-.1506	.0397	.0002	--	--	--			
Scale	1.8775			2.1973					
N	3405			252					
Kilometers	7886 (4900 mi)			523 (325 mi)					
ADT Range	500-10,500			2750-22,500					

Washington

	2-Lane Model			4-Lane Divided Model			4-Lane Undivided Model		
Parameter	Estimate	s.e.	P-value	Estimate	s.e.	P-value			
Intercept	-6.2152	.1484	.0001	-4.5387	.9918	.0101	Insufficient Data		
Log (ADT)	.9669	.0289	.0001	.6355	.1663	.0001			
Shoulder width	-.4541	.0331	.0001	--	--	--			
Scale	1.4531			1.6120					
N	3097			151					
Kilometers	2890 (1796 mi)			108 (67 mi)					
ADT Range	800-13,500			4500-35,000					

Minnesota

	2-Lane Model			4-Lane Divided Model			4-Lane Undivided Model		
Parameter	Estimate	s.e.	P-value	Estimate	s.e.	P-value			
Intercept	-8.1823	.1352	.0001	-7.2548	.4877	.0001	Insufficient Data		
Log (ADT)	1.1758	.0290	.0001	1.0644	.0712	.0001			
Shoulder width	-.2949	.0272	.0001	-.2339	.1056	.0265			
Scale	1.4650								
N	4425			948					
Kilometers	7032 (4370 mi)			666 (414 mi)					
ADT Range	670-7140			3000-17,000					

California

	2-Lane Model			4-Lane Divided Model			4-Lane Undivided Model		
Parameter	Estimate	s.e.	P-value	Estimate	s.e.	P-value			
Intercept	-3.0188	.2942	.0001	-8.9871	.4080	.0001	-8.7176	.7796	.0001
Log (ADT)	.9048	.0218	.0001	1.0707	.0668	.0001	1.1213	.1379	.0001
Shoulder width	-.3419	.0246	.0001	--			--	--	--
Surface width	-.4167	.0623	.0001	--			--	--	--
Scale	1.7195			1.4600			1.8088		
N	4823			501			251		
Kilometers	6030 (3747 mi)			449 (279 mi)			177 (110 mi)		
ADT Range	700-15,000			4180-37,000			2780-18,100		

Cross-sectional model results showed that conversion of a typical two-lane to four-lane roadway section in Minnesota and Washington, produced greater reduction in crashes as average daily traffic volume (ADT) increased while in North Carolina and California there was a slight decrease in crash reduction as ADT increased. Also, conversion from a typical two-lane to a typical four-lane divided roadway section produced crash per kilometer reduction values of 40 to 60 %. However, for ADTs between 8,000 and 15,000 vehicles per day (vpd), when the best typical two-lane section with the widest lanes and shoulders was compared to the most restricted four-lane divided section with the narrowest shoulders, and the worst two-lane section was compared to the best typical four-lane divided section, crashes per kilometer ranged from 10 to 70 %. California was the only state that had enough data to carry out the two-lane to four-lane undivided typical roadway conversion analysis. Crash rates in California ranged from about a 20 % reduction in crashes when ADT was less than 12,000 vpd to a slight increase in crashes when ADT was greater than 12,000 vpd. Results from Council and Stewart analysis are evident in Figures 2.1 and 2.2.

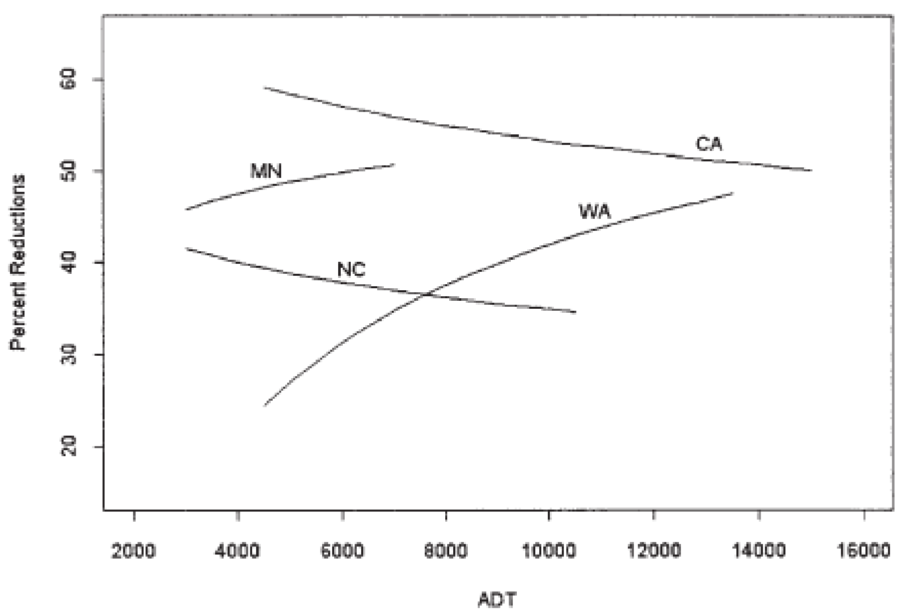

FIGURE 2.1 Predicted crash reductions for most typical sections on two- to four-lane divided road conversions (*1*)

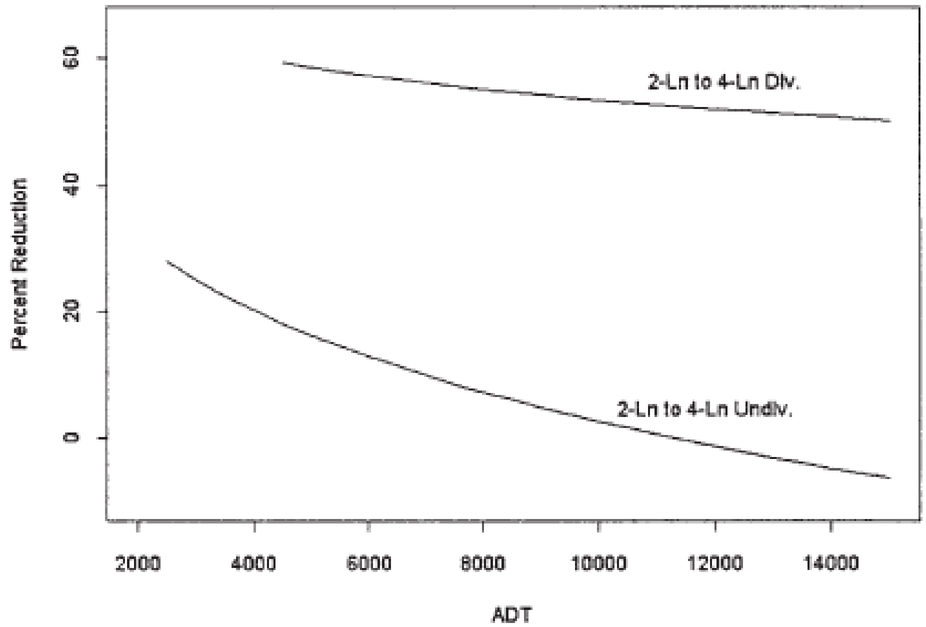

FIGURE 2.2 Predicted crash reductions on two- to four-lane divided and undivided road conversions in California (*1*)

In the Fitzpatrick et al. (*2*) two-lane rural highway with wide shoulders to a four-lane rural highway with minimal shoulders conversion analysis, crash data from 1999 to 2001 in Texas was used to examine the safety performance on highways with a surface width (sum of lane width and shoulder width) of 44 to 54 ft. Surface width included either two lanes with wide shoulders or four lanes with narrow shoulders. For example, two-lane highways with a surface width of 44 ft would generally consist of two 11 or 12 ft lanes with 11 or 10 ft shoulders while four-lane highways with the same surface width would generally consist of four 11 ft lanes with no shoulders. In the same light, two-lane highways with a surface width of 54 ft would generally consist of two 12 ft lanes (13 or 14 ft in some cases) with remaining width distributed to shoulders or a wider centerline while four-lane highways with the same surface width would generally consist of four 12 ft lanes with 3 ft shoulders. Displayed in Table 2.3 are the range and average values for rural two-lane and four-lane highways datasets. Highway variables considered in the prediction equations were median width, number of lanes (LN), segment length (Seg

Len), lane width (RT Lane), ADT, and shoulder width (RT Shou). Median width was removed from the independent variables because most of the highway segments in the database had a median width of zero. KAB crashes were crash severity levels 1, 2, and 4 (incapacitating, non-incapacitating, and fatal respectively) merged to form the number of fatal/injury crashes. Crash rates (crashes per mile) were used to present crash data because each segment length varied.

TABLE 2.3 Range And Average Values For Rural Two-Lane And Four-Lane Highways With Surface Widths Of 44 – 54 ft (*2*)

Variable	Average (Range) or Total for Data Set	
	Two Lane	Four Lane
ADT	3176 (170–14,033)	6546 (253–18,166)
Seg Len (mi)	1.39 (0.20–10.80)	1.88 (0.20–9.80)
RT Lane (ft)	11.97 (10–15)	11.42 (11–12)
RT Shou (ft)	10.34 (2–13)	1.68 (0–5)
SWICs	754	1135
Total Crashes	991	1745
Total Crashes KAB	378	661
SWIC KABs	286	441
Total Miles	514	290

Collection of data was performed by driving by the segments at near or highway speed and pulling geometric information by videotaping the segments. Information was also obtained from straight-line diagrams supplied by the districts, as well as data such as ADT values from the Texas Reference Marker database. Surface width influence crashes (SWIC), a subset of the Texas on-system crashes, for the years 1999 to 2001 were used, as well as other crash types such as total crashes and fatal/injury crashes. SWICs represented the non-intersection crashes (intersection related code = 4) with a collision code, and vehicle movement/manner of either single vehicles, two motor vehicles moving in the opposite direction, or two motor vehicles moving in the same direction. SWICs were crashes recorded between beginning mile point and end mile point. The roadway segment dataset included 328 segments (237 two-lane segments and 91 four-lane segments) totaling 804 mi of data. Rural two-lane roadway segment dataset comprised of 514 mi with 991 total crashes occurring but only 754 crashes met the SWIC criteria.

Rural four-lane roadway segment dataset also comprised of 290 mi with 1,745 total crashes occurring but only 1,135 crashes met the SWIC criteria. Prediction models to find the mean crash value over the roadway segments with similar conditions were also used. Effects of independent variables on SWICs were determined using the negative binomial regression model and an analysis of covariance model. Negative binomial regression model is used to model the count data when there is overdispersion of data (i.e., the variance is much larger than the mean). Mean function of the negative binomial regression is used to predict crash frequencies. Three separate models were developed for the negative binomial regression model for both two-lane and four-lane highways:

Model 1A) Segment Length (Length) and the log of ADT (LogADT) are included as the independent variables in addition to other roadway characteristic variables (RT Lane, and RT Shou).

Model 1B) The log of Segment Length (LogLength) and the log of ADT (LogADT) are included as the independent variables in addition to other roadway characteristic variables (RT Lane, and RT Shou).

Model 1C) Exposure is defined as a function of Segment Length and ADT. It is included as an offset variable in addition to RT Lane, and RT Shou.

An analysis of covariance model is used to stabilize the variance and make the distribution of the transformed variable close to the normal distribution. Based on the transformed count (transformed count = $(\text{count} + 3/8)^{0.5}$), an analysis of covariance model with a normal error distribution can be used to develop a prediction equation. By back transforming the transformed crash frequency, prediction can be made for the original untransformed crash frequency once the coefficients of the equation are estimated. The negative binomial model was determined to be the preferred model over the analysis of covariance model as the results for the negative binomial regression model were more practical for the segment length ranges (1 to 10 mi) reviewed. Also Model 1B was preferred over 1A and 1C because for longer segment lengths of 7 mi and more Model 1A and Model 1C showed debatable results. The regression prediction equations selected were:

Two-Lane Highway:

$$E(SWIC) = [\exp(-6.8674 + 0.9691\ LogLen + 0.9139\ LogADT)] / 3$$

Four-Lane Highway:

$$E(SWIC) = [\exp(-4.4688 - 0.1338\ RT\ Shou + 1.0009\ LogLen + 0.6895\ LogADT)] / 3$$

Figure 2.3 illustrates the relationship between various ADT levels, surface widths, and SWICs per million vehicle miles (MVM) for two-lane and four-lane highways. In general, SWIC/MVM on the four-lane highway was lower than on the two-lane highway at higher ADT levels and larger surface widths. Fitzpatrick et. al. concluded that conversions should only be considered when ADTs were at least 10,000 and surface widths were at least 53 ft based on safety.

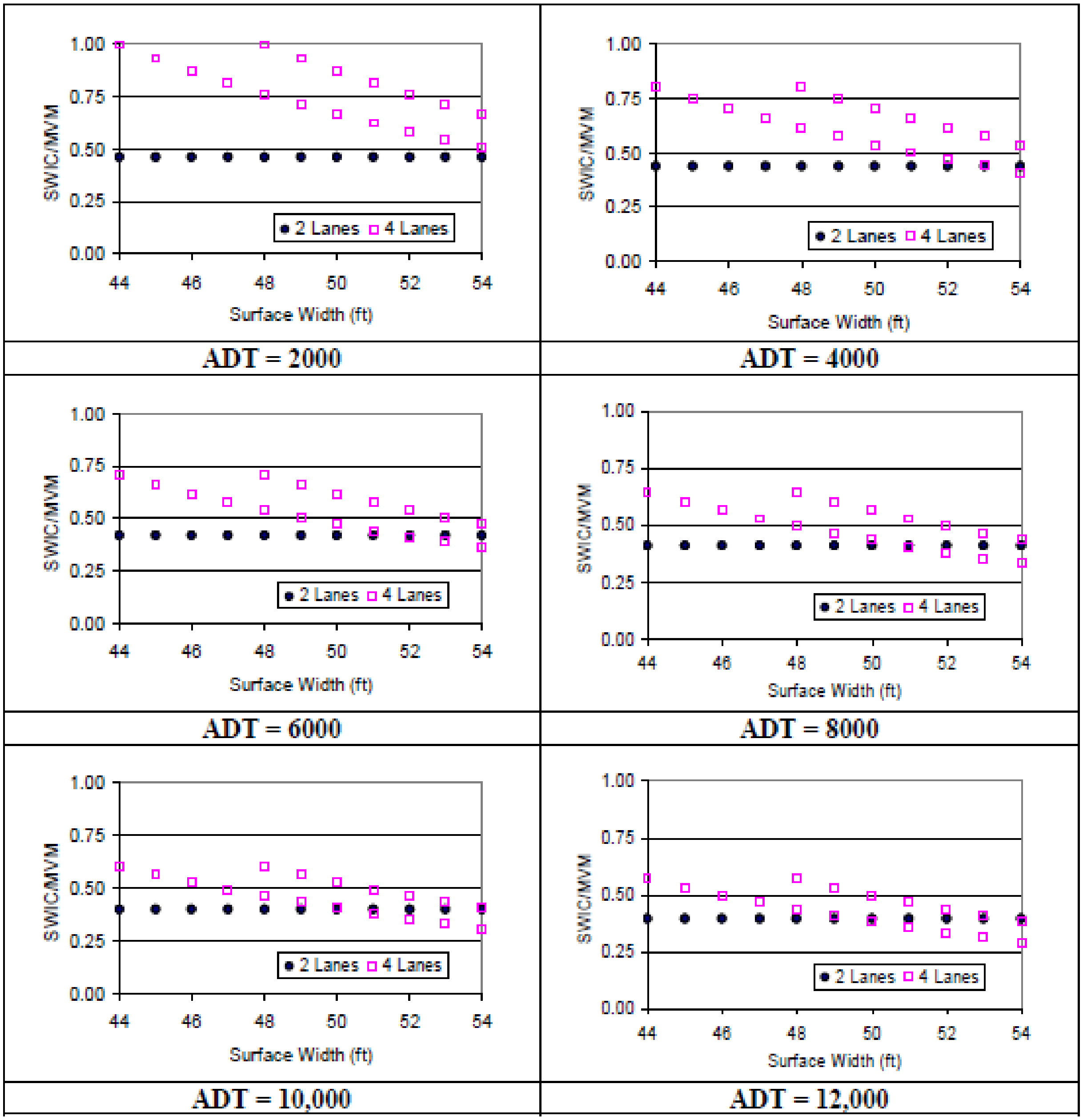

FIGURE 2.3 Examples of predicted number of annual SWICs by surface width for different ADT levels (*2*)

Bauer et al. (*3*) looked at converting shoulders and narrowing existing lanes to add high-occupancy-vehicle lanes to urban freeways in California. Three types of sites (treatment, downstream, and reference) were evaluated. Descriptive statistics including the number of lanes, sites and ramps, total length of sites, and Annual Average Daily Traffic volume (AADT), for the three site types are displayed in Table 2.4.

TABLE 2.4 Descriptive Statistics Of Evaluation Sites (*3*)

Type of Site	Number of Lanes	Number of Sites	Total Length of Sites (mi)	AADT (veh/day)[a] (1994)			Number of Ramps		
				Minimum	Mean	Maximum	On-Ramps	Off-Ramps	Total
Treatment	4 to 5[b]	79	36.4	79,000	104,081	128,000	60	51	111
	5 to 6[b]	45	12.5	77,000	107,497	126,500	14	15	29
	Total	124	48.9	77,000	104,951	128,000	74	66	140
Downstream	4 to 5[c]	45	11.4	62,500	103,267	128,000	28	23	51
	5 to 6[c]	33	7.9	77,000	114,121	126,500	14	19	33
	Total	78	19.3	62,500	107,859	128,000	42	42	84
Reference	3[d]	92	45.7	5,600	63,958	142,500	205	222	427
	4[d]	270	138.6	14,250	79,965	164,000	559	534	1,093
	5[d]	128	63.4	48,500	109,245	164,000	154	149	303
	Total	490	247.6	5,600	81,227	164,000	918	905	1,823

[a]Annual average daily traffic volume (veh/day) for one direction of travel.
[b]Number of lanes before and after the project (i.e., conversion type).
[c]Number of lanes before and after the project on the adjacent treated site.
[d]Number of lanes on the reference site.

Treatment sites were roadway segments for one direction of travelway on an urban freeway where another freeway lane was added. Treatment sites were homogeneous for conversion type and traffic volume with an average length of 0.39 mi. Treatment sites included four-to-five-lane conversions and five-to-six-lane conversions and were improved in 1993. Downstream sites, which were used to study the possibility of crash migration from the treatment sites, were directional freeway segments located immediately downstream of the treatment site that were not improved during the study period. Downstream sites had a maximum length of 1.1 mi. Reference sites, which were used to develop safety performance functions (SPFs), were urban freeway sites that were not improved during the study period. SPFs, which were negative binomial models, were used to estimate how crash frequencies on the treatment sites would have changed if there had not been any treatments. FHWA HSIS was used as the primary source of data, which included traffic volumes, records of roadway geometrics, and traffic crashes during periods before (1991 to 1992) and after (1994 to 2000) evaluated treatments. Crashes on ramps were excluded and only mainline freeway crashes were evaluated due to the expectation that lane addition projects would not have a direct effect on ramp crashes. There was a total of 2,441 crashes on the treatment sites during the before study period and 10,244 crashes on the same treatment sites during the after study period. To test whether the treatments significantly affected the change in before to after crash frequencies, an overall assessment of crash frequencies was made using the Empirical Bayes (EB) method as the primary safety evaluation. Both the four-to-five-lane and five-to-six-lane conversion types were considered in the primary safety evaluation as a whole and not as the individual bundles of geometric changes that made up the conversion types. The EB evaluation approach was described in the Bauer et al. (*3*) analysis as follows:

> Obtain data for the observed crash frequency on each treatment site during the before and after periods.
>
> By using the reference group data for the entire period during which data are available, develop SPFs that model crash frequencies as a function of site parameters (e.g., traffic volumes and site geometrics). This is generally done by means of negative binomial regression analysis.
>
> Estimate the predicted crash frequency at each treated site during the before period by using the SPF developed for that type of site.
>
> Compute a weighted average of the predicted and observed crash frequencies at each treated site during the before period. This crash frequency is known as the EB-adjusted expected crash frequency.
>
> By using the EB-adjusted expected crash frequency at each site during the before period, make an estimate of the expected crash frequency at each treated site during the after period had no change been

made. This step of the analysis accounts for changes in traffic volumes between the before and after periods.

Compare the observed after crash frequencies at the treated sites to the expected after crash frequencies at the treated sites had the change not been made. The difference between these observed and expected crash frequencies is an estimate of the safety effectiveness of the treatment.

The three measures of effectiveness used were total crashes [fatal, injury, and property-damage-only (PDO) crashes, including both tow-away and non-tow-away crashes]; fatal, injury, and PDO tow-away crashes (excluding PDO non-tow-away crashes); and fatal and injury crashes (excluding all PDO crashes). AADT, segment length, and EXPO = (AADT x segment length)/10^6 in million vehicle miles, were the primary independent variables considered in the SPFs. The reference site data was used to develop several model forms containing the independent variables with the most suitable model obtained by using the form:

expected number of crashes per year = $\exp(\beta_1)$ x $AADT^{\beta 2}$ x (segment length)

Maximum likelihood method was used to estimate the regression coefficients β_1 (intercept) and β_2 (exponent of AADT), the overdispersion parameter of the negative binomial distribution, the ordinary multiple correlation coefficient, R^2, and the Freeman-Tukey coefficient, R_{FT}^2. Statistical analysis software (SAS) and the PROC GENMOD methodology were used to estimate the regression coefficients and other model parameters. Table 2.5 shows the estimates of the regression parameters by the number of lanes for each measure of effectiveness. Calibration factors for years 1991 to 2000 (c_{91},..,c_{00}), shown in Table 2.5, are used to ensure that the SPF-predicted and observed crashes at each treated site during the before period are the same by adjusting the predicted crashes for each specific year whenever a negative binomial regression mode is applied.

TABLE 2.5 Negative Binomial Regression Models Used As SPFs (*3*)

Measure of Effectiveness/ Dependent Variable	Number of Lanes[a]	Number of Site-Years	Regression Coefficients[b] (standand error) Intercept	logAADT	Dispersion Parameter[c]	Measures of Fit R^2 (%)	R_{FT}^2 (%)	Yearly Calibration Factors c_{91}	c_{92}	c_{93}	c_{94}	c_{95}	c_{96}	c_{97}	c_{98}	c_{99}	c_{00}
Total crashes	6	828	-12.529	1.378	0.323	63	71	0.94	0.86	0.96	0.96	0.97	1.00	1.04	1.03	1.08	1.16
			(0.776)	(0.0625)	(0.024)												
	8	2,430	-18.130	1.826	0.276	66	68	0.87	0.89	0.83	0.95	0.91	0.91	0.90	0.94	0.97	1.01
			(0.505)	(0.042)	(0.014)												
	10	1,152	-20.717	2.031	0.271	52	67	0.91	0.85	0.84	0.87	0.83	0.91	0.94	0.94	0.91	0.94
			(1.025)	(0.083)	(0.015)												
Fatal, injury, and PDO tow-away crashes	6	828	-10.891	1.188	0.281	61	67	0.69	0.75	0.94	0.92	0.99	1.04	1.10	1.08	1.11	1.24
			(0.838)	(0.070)	(0.025)												
	8	2,430	-14.929	1.511	0.218	69	68	0.76	0.78	0.83	0.97	0.97	1.01	1.00	1.05	1.05	1.11
			(0.482)	(0.040)	(0.011)												
	10	1,152	-16.889	1.667	0.220	60	68	0.78	0.76	0.83	0.91	0.88	0.98	1.03	1.05	1.00	1.02
			(0.962)	(0.078)	(0.014)												
Fatal and injury crashes	6	828	-11.024	1.149	0.226	62	65	1.02	1.00	0.99	0.89	0.96	1.02	1.01	0.97	0.94	1.06
			(0.901)	(0.075)	(0.025)												
	8	2,430	-13.629	1.355	0.190	66	64	1.08	1.06	0.88	0.99	0.93	0.93	0.88	0.91	0.94	0.98
			(0.516)	(0.043)	(0.012)												
	10	1,152	-15.426	1.501	0.191	62	67	1.12	1.03	0.88	0.97	0.87	0.91	0.92	0.89	0.91	0.93
			(1.019)	(0.083)	(0.015)												

[a]Both directions of travel combined.
[b]On log scale.
[c]This is the dispersion parameter as defined by SAS/STAT User's Guide, Version 8. Some authors prefer to report the inverse of this parameter.

Table 2.6 shows the EB analysis results which includes the mean treatment effectiveness across all sites expressed as a percentage change in crash frequency, the measure of precision of the treatment effectiveness expressed in terms of its standard error, the ratio of the mean treatment effectiveness divided by its standard error, and the statistical significance of the treatment effectiveness. Treatment effectiveness with a ratio of 2.0 or more was considered to be significant.

TABLE 2.6 EB Analysis Results For Primary Evaluation Of Specific Conversion Types (*3*)

Conversion Type	Measure of Effectiveness/ Dependent Variable	Number of Sites	Percent Change in Crash Frequency		Ratio[b]	Significant?[c]
			Mean[a]	Standard Error		
4 to 5 lanes	Total crashes	79	10.96	2.88	3.8	Yes
	Fatal, injury, and PDO tow-away crashes	78	9.67	3.89	2.5	Yes
	Fatal and injury crashes	78	10.59	4.56	2.3	Yes
5 to 6 lanes	Total crashes	43	3.02	4.56	0.7	No
	Fatal, injury, and PDO tow-away crashes	45	3.71	6.08	0.6	No
	Fatal and injury crashes	45	7.08	7.22	1.0	No

[a]A positive mean percent change indicates an increase in crash frequency while a negative mean indicates a decrease.
[b]Ratio of mean percent change in crash frequency to standard error of percent change in crash frequency.
[c]Significant if ratio ≥ 2; not significant if ratio < 2.

Bauer et al. (*3*) analysis showed that there was a statistically significant 10 to 11 % increase in crash frequency for the four- to five-lane conversion and statistically insignificant 3 to 7 % increase in crash frequency for the five- to six-lane conversion. The five- to six-lane conversion, however, had about half the sample size of the four- to five-lane conversion, which would affect how statistically significant its results were compared to the latter.

Other notable research included Rogness et al. (*4*) who also looked at converting the paved shoulders of a two-lane rural roadway to a four-lane roadway without paved shoulders (i.e., converting a roadway with the same right of way without adding any additional land). Over a four year period, data from 60 sites in Texas were divided into three categories: less than 3,000 vpd, between 3,000 and 5,000 vpd, and between 5,000 and 7,000 vpd. Rogness et al. discovered that overall crashes decreased by 9.1 % in the 3,000 to 5,000 vpd category and increased by 12.6 % for non-intersection crashes in the less than 3,000 vpd category. Also, for non-intersection crashes in the 3,000 to 5,000 vpd and 5,000 to 7,000 vpd categories, there was a decrease of 19 % and 28 % respectively.

Harwood and St. John (*5*), on the other hand, looked at improvements in 13 states. This included a safety analysis of the addition of short four-lane passing sections to two-lane segments at 10 locations using a comparative evaluation of case sites, which are the short four-lane undivided roadway segments, and control sites, which are the two-lane roadway segments directly before and after the treated segment. Harwood and St. John discovered from the crash rate analysis that there was a decrease in total crash rate by 34 % and a decrease in cross-centerline crashes by 50 % as well as decreases in all other types of crashes.

A summary of the data collection and analysis as well as crash rate results from the different conversions discussed by the various authors is shown in Table 2.7 and Table 2.8 respectively below.

TABLE 2.7 Summary Of Data Collection And Analysis From Road Conversions

	Council and Stewart	**Fitzpatrick et al.**	**Bauer et al.**
Type of Data	Cross-sectional model of typical two- and four-lane roadway sections including segment length, surface, shoulder, and median width; Homogeneous sections; At least 0.1 mi segment length; 17,853 segments totaling 16,178 mi of data; 5^{th} < ADT < 95^{th} percentiles within roadway classes; Data collected from 1993 to 1995 for California, North Carolina, and Washington, and from 1991 to 1993 for Minnesota.	Two- and four-lane segments; Range of lane and shoulder widths (surface width of 44 to 54 ft); Sample of median types and widths; At least 0.2 mi segment length; 328 segments totaling 804 mi of data from 1999 to 2001 in the roadway segment data set; SWIC crash type data from 1999 to 2001 used;	Treatment, Downstream, and Reference sites; Homogeneous treatment sites for conversion type and traffic volume; 692 treatment, downstream, and reference sites totaling 315.8 mi; Traffic volumes, records of roadway geometrics, and traffic crashes during periods before (19991 to 1992) and after (1994 to 2000) evaluated treatments obtained from FHWA HSIS
Analysis	Log link function used; Overdispersed Poisson models fit to data using statistical analysis software SAS, and the PROC GENMOD methodology Output of two-lane compared to output of four-lane at equal ADT; Safety effect measured as percent reduction in crashes per kilometer with two-lane as base.	Negative binomial regression model and an analysis of covariance model used to determine effects of independent variables on SWIC; Negative binomial regression model determined to be the best model to fit the data provided.	Primary safety effectiveness analysis; Negative binomial SPFs used; Before-and-after Empirical Bayes method used;

TABLE 2.8 Summary Of Crash Rate Results From Road Conversions

Author	Type of Conversion	Location	Results
Council and Stewart	A developed cross-sectional model conversion of a typical two-lane to four-lane divided or undivided roadway	Minnesota, North Carolina, Washington, and California	Conversion of a typical two-lane to four-lane roadway section in Minnesota and Washington, produced greater reduction in crashes as ADT increased while in North Carolina and California there was a slight decrease in crash reduction as ADT increased. Conversion from a typical two-lane to a typical four-lane divided roadway section produced crash per kilometer reduction values of 40 to 60 %. Conversion from a typical two-lane to four-lane undivided roadway section in California produced crash rates ranging between about a 20 % reduction to a slight increase when ADT was greater than 12,000 vpd.
Fitzpatrick et al.	Conversion from a rural two-lane highway with wide shoulders to a rural four-lane highway with minimal shoulders	Texas	Conversions should only be considered when ADTs were at least 10,000 and surface widths were at least 53 ft based on safety.
Bauer et al.	Conversion of shoulders and narrowing of existing lanes to add high-occupancy-vehicle lanes to urban freeways	California	Statistically significant 10 to 11 % increase in crash frequency for the four- to five-lane conversion and statistically insignificant 3 to 7 % increase in crash frequency for the five- to six-lane conversion.
Rogness et al.	Conversion of paved shoulders of a two-lane rural roadway to a four-lane roadway without paved shoulders	Texas	Crashes decreased by 9.1 % in the 3,000 to 5,000 vpd category and increased by 12.6 % for non-intersection crashes in the less than 3,000 vpd category. Also, for non-intersection crashes in the 3,000 to 5,000 vpd and 5,000 to 7,000 vpd categories there was a decrease of 19 % and 28 % respectively.
Harwood and St. John	Addition of short four-lane passing sections to two-lane segments	10 locations within 13 states	Decrease in total crash rate by 34 % and a decrease in cross-centerline crashes by 50 %.

Other factors affecting road safety are discussed in the upcoming sections in order to obtain a full view of what may cause crashes to increase or decrease on a roadway.

Lane Width and Presence of Shoulder and Shoulder Width

The practice of reducing lanes widths and shoulders widths is sometimes used in road conversions to facilitate the addition of more lanes to increase capacity. As this study focuses on two- to four-lane road conversions, it is important to find out the effects of lane width and shoulder width on road safety exclusively. Fitzpatrick et al. (*2*) discovered that for rural two-lane and four-lane highways in Texas, crash rates were predicted to increase with decrease in lane width and shoulder width to as much as 51 to 52 % when comparing a 9 to 12 ft lane to a 12 ft lane and as much as 62 to 64 % when comparing a 0 to 10 ft shoulder width to an 8 ft shoulder width. Based on the ratio of total crash results displayed in Table 2.9, Fitzpatrick et al. showed that the narrower the lane and shoulder widths from the standard 12 ft lane and 8 ft shoulder, the more adverse the effect on road safety..

TABLE 2.9 Ratio Of Total Crashes Between Adjusted Lane And Shoulder Widths To 12 ft Lane And 8 ft Shoulder, Respectively (*2*)

Rural Two-Lane Highways		Rural Four-Lane Highways	
Lane Width (ft)	Ratio to 12 ft Lane	Lane Width (ft)	Ratio to 12 ft Lane
12	1.00	12	1.00
11	1.15	11	1.15
10	1.32	10	1.32
9	1.51	9	1.52
Shoulder Width (ft)	Ratio to 8 ft Shoulder	Shoulder Width (ft)	Ratio to 8 ft Shoulder
10	0.89	10	0.88
9	0.94	9	0.94
8	1.00	8	1.00
7	1.06	7	1.06
6	1.13	6	1.13
5	1.20	5	1.20
4	1.27	4	1.28
3	1.35	3	1.36
2	1.43	2	1.45
1	1.52	1	1.54
0	1.62	0	1.64

In the American Association of State Highway and Transportation Officials' (AASHTO) *A Policy on Geometric Design of Highways and Streets, Sixth Edition (6),* popularly referred to as the Greenbook, there is flexibility in the selection of lane and shoulder widths, which suggests a low traffic safety risk in doing so, as AASHTO considers the needs of the vehicle, pedestrian and bicycle traffic when providing its guidelines for appropriate lane and shoulder width selections. Lane width is, however, noted to influence comfort in driving, operational characteristics, and in some cases, crash probability. Wider lane widths (12 ft) are noted to be the preferred lane widths for high-volume, high-speed highways and also provide desirable clearances between large commercial vehicles traveling in opposite directions on two-lane rural highways when high traffic volumes are expected. Roadway sections with narrow travelway and shoulders, and high traffic volume were also noted to have a relatively higher crash rate.

Shoulders improve safety by providing a space for drivers to maneuver to avoid crashes and recover safely (*7*). Crash modification factors shown in Figure 2.4 are provided to offset the selection of less than ideal shoulder widths for a given traffic volume on two-lane rural highways (*7*). These crash modification factors relate to multiple-vehicle-opposite-direction crashes, single-vehicle-run-off-road crashes, and multiple-vehicle-same-direction-sideswipe crashes. The provision of crash modification factors shows that as shoulder width decreases the probability of crash increases, hence the increase in the crash modification factor. The crash modification factors also increase with increase in ADT for shoulder widths less than 6 ft.

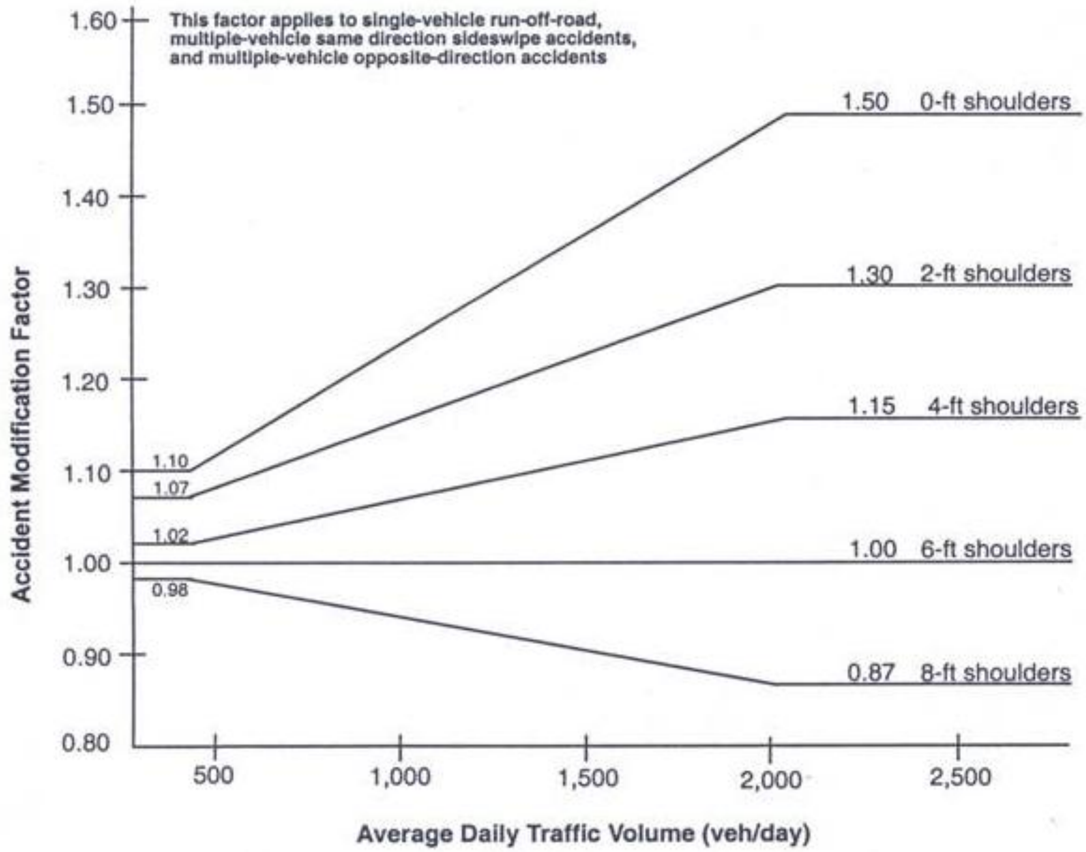

FIGURE 2.4 Shoulder width crash modification factors on two-lane rural highways (*7*)

On the other hand, after evaluating lane widths for urban and suburban arterial roadway segments in Minnesota and Michigan using cross-sectional safety analysis approach for both midblock segments and intersection approaches, Potts et al. (*8*) found no significant statistical relationship that indicated that narrower lanes (i.e., lane widths less than 3.6 m or 12 ft) increased crash frequency. Potts et al. noted that on arterial intersection approaches, the use of narrower lanes could enhance safety by providing space for medians or turn lanes. However, for one state, crash frequency was higher for 10 ft lanes than for 11 ft and 12 ft lanes on four-lane undivided arterials while in the other state crash frequency was higher for 9 ft lanes than for 10 ft lanes on four-lane divided arterials. Also, for the arterial intersection approaches safety evaluation, data for one state showed higher crash frequencies for approaches to four-leg STOPcontrolled intersection, for approaches with 10 ft lanes than those with 12 ft lanes but the opposite was found in the data from the other state.

Presence of Median and Median Width

Medians provide width for future lanes and a stopping area for emergencies, as well as storage area for left- or U-turning vehicles and crossing pedestrians (*6*). Medians are also highly advantageous on arterials with at least four lanes and help reduce headlight glare. Flush medians on urban roadways that are converted to two-way left-turn lanes reduce travel time and crash frequency (especially that of the rear-end kind), improve capacity, and offer greater flexibility due to their use as travel lanes during a closure of a through lane compared to a no median roadway (*6*). Medians widths that are 40 ft or more provide the driver with ease and freedom of vehicle operation from the sense of separation from oncoming traffic, along with the reduction of headlight glare and noise and air pressure from the oncoming traffic (*6*). Median barriers reduce cross-median collisions but increase total crash frequency, due to the reduction in the space for drivers to maneuver to avoid crashes and return to the road, as a result of the placement of the barrier.

Speed

It is the perception that with an increase in number of lanes on a roadway segment comes an increase in speed. Generally, with increased speed comes increased severity in crashes. “Speed reduces the visual field, restricts peripheral vision, and limits the time available for drivers to receive and process information” (*6*). Research has shown that lowering speed limits decreases frequency and severity of crashes. Also, crash risk increases with an increasing speed differential. Speed differential could be the result of a transition between adjoining highway sections, with a reduction from the 85th percentile speed due to a roadway geometry change or the result of varying vehicle speeds between multiple vehicles such as trucks and passenger vehicles within the same traffic stream, as shown in Table 2.10 (*7*).

TABLE 2.10 Speed Differential Safety Risk Due To Changes In Roadway Geometry (*7*)

Speed Differential (ΔV)	Safety Risk
ΔV < 5 mi/hr	Low
5 mi/hr < ΔV < 15 mi/hr	Medium
ΔV > 15 mi/hr	High

Traffic Volume

A conversion to a higher-order roadway is performed to facilitate an increase in road capacity. Literature review analysis shows that crash frequency increases with a traffic volume increase, while crash rate, which is specified as "the number of crashes per motor vehicle kilometre", decreases (*9*). Duivenvoorden noted, however, that the road safety to traffic volume relationship had not been examined extensively and thus the conclusion was a preliminary one, and not a solid fact. Duivenvoorden also noted that one study showed that the number of multiple vehicle crashes increased with increase in traffic volume in contrast to single vehicle crashes.

Human Factors

Human reaction to the features of a roadway is an essential evaluation during the roadway design process to ensure safety, as driver error accounts for a large proportion of crashes. According to AASHTO's Greenbook (*6*), "The selection of speeds and paths is dependent on drivers being able to see the road ahead" because drivers need to perceive the alignment of the roadway including the environment immediately adjacent to the roadway and the necessary information to perform driving tasks optimally and avoid crashes in time. The reaction times of drivers increase as the complexity and volume of the information to be processed increases which results in a greater chance of error due to the decrease in time available to focus on other necessary driving tasks. Roadway design features need to be consistent and uniform with other roadway designs, clear to understand, and simple to follow to support driver expectancy and aid in driver performance.

SUMMARY

From the road conversions study, there appeared to be generally a decrease in crash frequency. Although studies discussed here give an overview of safety effects of lane additions either through shoulder conversions, short passing lane implementations or a model of two-lane to four-lane conversion, no research on before-after safety evaluations of actual conversions from two-lane to four-lane were discovered. Also, depending on the type of road revision and the variables considered, variation in decrease in crash frequency ranged from about 9 to 70 %.

Noted from this literature review was that aside from road conversions, there are other factors that affect road safety, including lane width, presence of shoulder and shoulder width, presence of median and median width, speed, traffic volume, and human factors. Decrease in lane width and shoulder width adversely affected roadway safety however, one research found no significant statistical relationship that indicated that narrower lanes (i.e., lane widths less than 3.6 m or 12 ft) increased crash frequency. Also with increase in traffic volume and speed produced increase in crash frequency. Another observation was that human reaction to the features of a roadway is an essential evaluation during the roadway design process to ensure safety as driver error accounts for a large proportion of crashes. Median barriers were discovered to reduce cross-median collisions but increase total crash frequency due to the reduction in the space for drivers to maneuver to avoid crashes and return to the road as a result of the placement of the barrier.

CHAPTER 3 METHODOLOGY

A simple before-and-after study, as well as an Empirical Bayes before-and-after study was performed for this research. Simple before-and-after study was performed to see the effect on safety when conditions such as number of lanes, traffic volume, lane and shoulder width, and presence of median/ median width were not assumed to remain unchanged while EB before-and-after study was performed to see the effect on safety when the before mentioned conditions were assumed to remain unchanged before and after the conversion except for the traffic volume. The step by step process of both studies is detailed in the upcoming section.

Simple Before-and-After Study

Simple before-and-after study used in evaluating the safety benefit of a two-lane to four-lane conversion was calculated as the difference between the average annual crash frequency before and after the conversion. Since some projects had less than five years of comparable after conversion crash data, the average crashes per year was used in the before-and-after study. Given by:

Change in safety: $\Delta = B - A$ or

Ratio (also called the index of effectiveness): $\varepsilon = B/A$

Where: B = average number of crashes per year occurring in the period before the conversion

A = average number of crashes per year occurring in the period after the conversion

A positive value for the change in safety, or a ratio greater than one indicates a desirable safety outcome.

Empirical Bayes (EB) Before-and-After Study

In the EB analysis, simple before-and-after comparison assumes that conditions remain unchanged before and after the conversion, even though this may necessarily not always be the case. As such, a traffic volume adjustment is frequently used to normalize for differences in traffic volume between before and after volumes. Moreover, the difference or ratio computed directly from the observed crash counts or rates between before and after periods may be biased as a result of regression-to-the-mean (RTM). RTM effect, or bias-by-selection, is a phenomenon that repeated measures of the data drifts towards the mean value in the long run. Due to this natural fluctuation, an extreme observation will usually be followed by a less extreme observation without any intervention. Locations slated for safety treatments usually have high crash counts, rates, or severities. A simple before-and-after analysis may inflate the countermeasure effectiveness by including the difference caused by RTM. Hauer (*10*) suggested using the expected number of crashes that would have occurred in the after period had the countermeasure not been implemented as "B", which is the expected mean of a conditional (gamma) distribution of the long-term crash average of a location, given the observed short-term crash history. The expected mean can be formulated as the weighted average of a predicted number of crashes and site-specific crash history as follows:

$E = W \times \mu + (1-W)N$

Where:

$W = 1/(1+\mu YK)$ = Weight of Prediction

E = Expected Crash Count (Estimate of Long Term Mean over Y years)

N = Observed Crashes (over Y years)

μ = Predicted Number of Crashes (SPF Calculated Value for Y years)

Y = Number of Years in Study

k = Overdispersion Parameter

Estimating the expected number of crashes is called EB analysis. When the expected number of crashes that would have occurred in the after period without safety improvements, denoted as B, is compared with the actual number of crashes after safety improvements are implemented, the procedure is called EB before-and-after analysis. Note that in the actual calculation, B is the expected average number of crashes in the after period. Any change in the traffic volume (AADT) or analysis time period needs to be factored into the comparison. An adjustment factor can account for these changes as shown in the equation below.

$$r_i = (SPF_{After}/SPF_{Before})\ (Years_{After}/Years_{Before})$$

Multiplying the ‘r’ factor by the EB expected number of crashes offers a correct estimate of the number of crashes that would have happened during the after time period had the treatment not been implemented.

The procedure is listed as follows:

1) Estimate EB expected average crashes in the before period for the intersection;

2) Estimate EB expected average crashes in the after period for the intersection through a traffic exposure adjustment factor r_i (B);

3) Observe average crashes in the after period for the two-lane to four-lane conversion (A);

4) Calculate the change in safety by (B-A) or the safety effectiveness index (B/A); and

5) Estimate the confidence interval of the change in safety or the safety effectiveness based on all the sites evaluated.

Safety performance can be computed for individual two-lane to four-lane conversion segments. When each segment shows varying performance, the difference in or the ratio of the total number of crashes before and after the conversion can provide a quantifiable mean (average) safety performance measure, as well as the variance of the measurement from an overall perspective.

Safety Performance Function

From HSM (*11*), a safety performance function (SPF) is a regression model used to estimate the predicted average crash frequency of individual roadway segments or intersections. SPFs describe the relationship between the predicted number of crashes (dependent variable) and a set of crash contributing factors (independent variables). The state-of-the-practice distribution considered for modeling crashes is Poisson-gamma (or negative binomial (NB)). Poisson-gamma models can account for over dispersion of the crash data, which, if not properly considered, may lead to estimation inefficiency and inference errors. In safety applications, the number of crashes (N_i) at a site 'i' is assumed to follow a Poisson distribution.

$$N_i | \mu_i \sim \text{Poisson}(\mu_i) \qquad i=1,2,\ldots,n$$

The log function used to link the mean number of crash counts with all possible covariates and unstructured errors is defined as:

$$\mu_i = (\textit{traffic exposure})^{\alpha} \exp(X_i\beta) \exp(e_i) \quad i = 1,2,\ldots, n$$

SPFs are subdivided into different types of intersections and road segments. SPFs are used to estimate the predicted number of crashes, which can then be used in the EB analysis methodology by combining it with observed crashes to calculate the expected average crash number. The SPFs used in this thesis were selected from HSM (*11*). Appropriate SPFs found for a variety of highway facilities and intersection types in HSM were identified using the before conversion intersection geometric characteristics (number of legs, number of lanes) and area setting (urban, rural), as well as traffic control types (Yield, TWSC, AWSC, Signalized) where TWSC stands for two-way stop control and AWSC stands for all-way stop control. SPFs for three types of intersections (3ST, 4ST, and 4SG) were used where 3ST stands for three-leg intersection with minor road stop control, 4ST stands for four-leg intersection with minor road stop control and 4SG stands for four-leg signalized intersection. All project sites considered were two-lane and two-way. All projects sites were assumed to be rural in order to use the HSM Rural 2-Lane Roads spreadsheet which only had SPFs for two-lane rural roads. Furthermore, HSM also provides a fixed value for fatal/injury crashes as a proportion of the total number of crashes in cases where specific fatal/injury crash SPFs are missing. Detailed lists of SPFs used in this study are presented in Appendix A.

CHAPTER 4 DATA COLLECTION AND PROCESSING

Data on two-lane to four-lane conversions within the past decade was requested from all the Wisconsin Department of Transportation (WisDOT) regions. The WISDOT regions represent the four quadrants of Wisconsin. Data on nine projects was received. Two projects were in the NE region, two in the SE region, four in the SW region, and one in NW region. One project (USH 12) was split into four sections and constructed in different time frames so the four sections were evaluated independently from each other. Therefore a complete total of 12 projects were evaluated. Table 4.1 represents the project data summary, including, region and county, project limits, segment lengths, construction dates, and before and after years of crash data collected. Figure 4.1 shows the locations of the projects on the Wisconsin State Trunk Network map.

TABLE 4.1 Project Data Summary

Project	Region	County	Project Limits	Segment Length (mi)	Construction Dates	Before Crash Years	After Crash Years
USH 141	NE	Marinette, Oconto	6th Rd – CTH 22	16.92	04/2005 – 11/2006	2000 – 2004	2007 – 2011
STH 57	NE	Brown, Door, Kewaunee	Stone Rd– CTH K / Fischer Rd	25.92	04/2005 – 12/2008	2000 – 2004	2009 – 2012
STH 164	SE	Waukesha	Swan Rd – Prospect Ct	3.82	01/2004 – 12/2005	1999 – 2003	2006 – 2010
STH 83	SE	Waukesha	Frog Alley Rd – STH 59	5.62	03/2011 – 12/2011	2006 – 2010	2012
STH 26	SW	Jefferson	Whitetail Ln – STH 89	6.12	05/2010 – 11/2011	2005 – 2009	2012
USH 12(77)	SW	Dane	STH 78 – CTH KP	4.47	11/2003 – 05/2005	1998 – 2002	2006 – 2010
USH 12(71)	SW	Dane	CTH KP – STH 19	4.76	11/2002 – 11/2003	1997 – 2001	2004 – 2008
USH 12(74)	SW	Dane	STH 19 – CTH K	6.05	05/2003 – 11/2005	1998 – 2002	2006 – 2010
USH 12(76)	SW	Dane	CTH K – Graber Rd	1.70	07/2003 – 11/2005	1998 – 2002	2006 – 2010
USH 14	SW	Vernon	CTH BB / Airport Ln – Locust St	5.03	2011	2006 – 2010	2012
USH 21	SW	Monroe	Emerson Rd – Ensign Rd	2.17	02/2005 – 11/2005	2000 – 2004	2006 – 2010
USH 10	NW	Trempealeau	East St – Industrial Rd	1.49	03/2003 – 11/2003	1998 – 2002	2004 – 2008

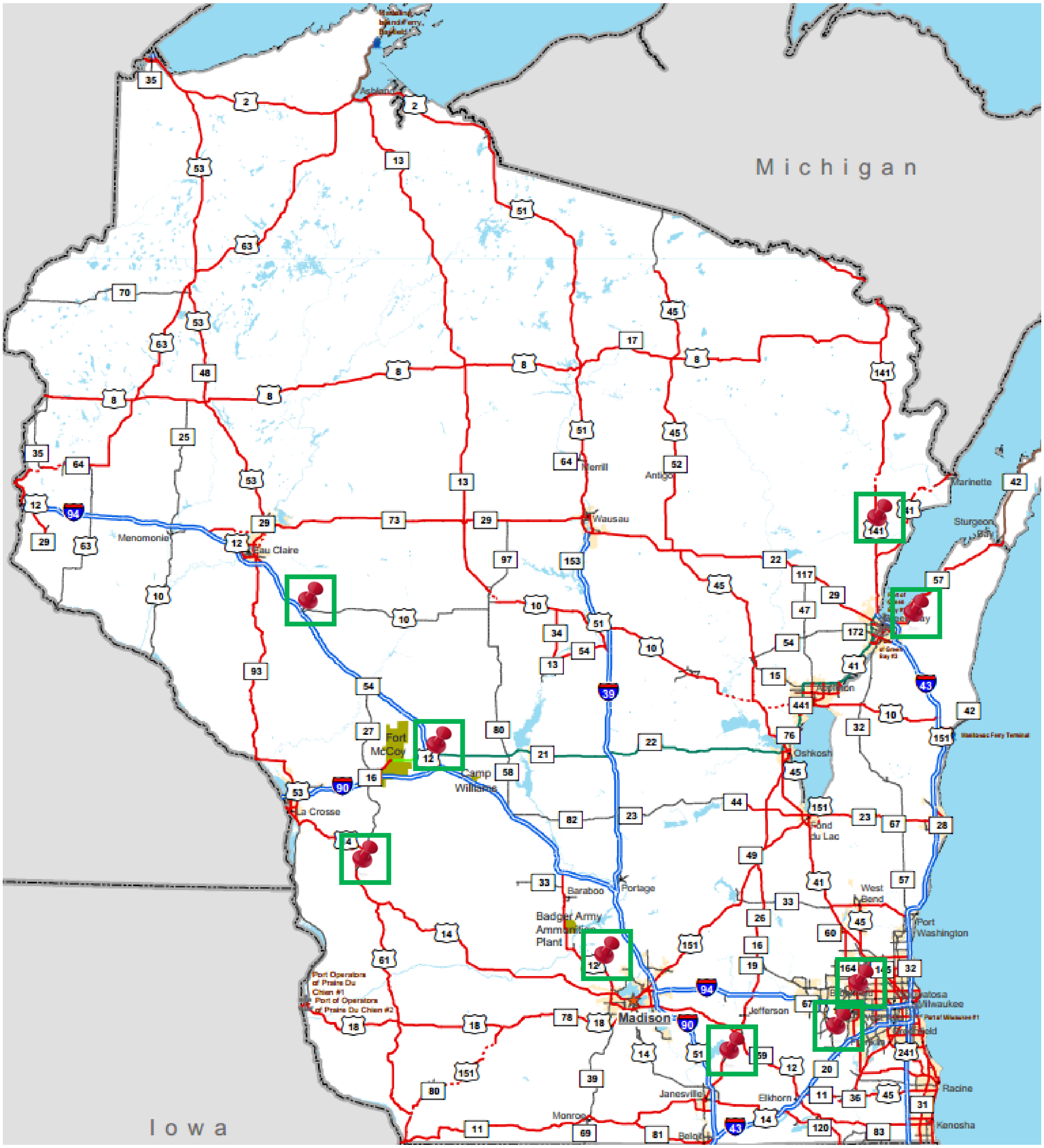

FIGURE 4.1 Two-Lane to Four-Lane Conversion Project Sites

(Image obtained from http://www.fhwa.dot.gov/planning/national_highway_system/nhs_maps/wisconsin/)

Crash Data

Crash data (five years before and up to five years of after data, when available) was collected on the mainline as well as on the cross streets for all the projects from the WisTransPortal (*12*). Crash location for the cross streets was defined not only by the address, but also by the police definition as "intersection located", as evidenced in the crash data as "I" in the crash location section. Since some of these conversions occurred as recently as 2011, that created some challenges in obtaining adequate after conversion crash data as 2012 is the latest year with complete crash data on the WisTransPortal. Out of the 12 projects, eight had five years of after crash data. STH 57 had four years of after crash data while STH 83, STH 26, and USH 14 each had one year of after crash data. Crashes that occurred during the construction year were not included to minimize the effects of construction activities and other complications such as being partially open to traffic during the construction. A detailed manual review of about 2,468 crash reports (across the 12 projects) was also conducted for all retrieved crash data using RP and WISLR maps to determine whether or not crashes were truly within the project limits. As some crashes were not mapped on the RP and WISLR maps at the time retrieved, it was difficult to determine whether these crashes were within the project limits based on their description alone. Nonetheless, these crashes were included in the total crashes reviewed in the simple before-and-after study. However, in the EB before-and-after study, a total of 2,323 crashes were used which only included the mapped crashes within the project limits.

Geometric and Traffic Data

The as-builts and other plans for obtaining the prior roadway geometry information were also obtained from WisDOT. Important design features obtained from the as-built files and other plans included number of lanes, lane width, shoulder width, shoulder type, percent grade, traffic control type, intersection type, and number of left and right turn lanes. Google Earth Time Slider was also used to examine road segments before conversion to determine or confirm the types of intersections on the road as well as the number of left and right turn lanes. Since some of these conversions occurred as far back as 2002, obtaining the before conversion geometric data was challenging for some projects. Default values in the HSM Rural 2-Lane Roads spreadsheet were used when geometric data was not provided or could not be retrieved. Traffic volume was obtained from the WisDOT traffic count website (*14*). Traffic volume on the mainline ranged from 4,700 to 24,200. An adjustment of 3% increase per year rounded up to the nearest hundredth (or tenth for traffic volumes on cross streets less than 1,000) was used for traffic volume that was missing for the year being evaluated but had had a previous year's traffic volume collected. For cross streets missing traffic volume data, default maximum AADT for the minor streets was used in the HSM Rural 2-Lane Roads spreadsheet. Area type categorized as urban if the municipality where the two-lane to four-lane conversion was located had a population greater than 5000 and rural if otherwise was also collected. STH 164, STH 83, and STH 21 were characterized as urban areas while USH 141, STH 57, STH 26, USH 12, and USH 10 were characterized as rural areas. USH 14 had a unique situation where it was partially characterized as urban and partially characterized as rural. Table 4.2 shows the project area type summary per municipality. Even though STH 164, STH 83, STH 21, and part of USH 14 were characterized as urban areas, the HSM Rural 2-Lane Roads spreadsheet was still used to analyze these project sites as well as there was no developed HSM Urban 2-Lane Roads spreadsheet available.

TABLE 4.2 Project Area Type Summary

Project	Municipality	Area Type
USH 141	Beaver	Rural
	Pound	Rural
	Coleman	Rural
	Lena	Rural
	Stiles	Rural
STH 57	Nasewaupee	Rural
	Gardner	Rural
	Brussels	Rural
	Union	Rural
	Red River	Rural
	Scott	Rural
STH 164	Lisbon	Urban
	Sussex	Urban
	Pewaukee	Urban
STH 83	Genesee	Urban
	Mukwonago	Urban
STH 26	Koshkonong	Rural
USH 12(77)	Roxbury	Rural
USH 12(71)	Roxbury	Rural
	Springfield	Rural
USH 12(74)	Springfield	Rural
USH 12(76)	Springfield	Rural
USH 14	Viroqua	Urban
	Westby	Rural
STH 21	Tomah	Urban
USH 10	Osseo	Rural

In order to perform the analysis as recommended by HSM, each roadway needed to be divided into segments and intersections and analysis done for each segment and intersection separately for each traffic count year evaluated. For seven of the 12 projects there were two years of traffic volumes available for the time frame evaluated. A separate analysis was done for each year. A total of 137 intersections and 133 segments across all 84.07 mi of the 12 projects were determined with a total of 2,323 crashes mapped. Extensive information on each segment and intersection needed to be input to the HSM Rural 2-Lane Roads spreadsheet used for this analysis. This was a manual process and required

WISLR maps to identify crossing non-STN roads within the project limits. Furthermore, some projects had involved relocation of intersections and/or changes in roadway alignment, therefore the data collection process required looking at blueprints, older maps and using WISLR/STN maps from as far back as 1998.

For each segment the following data had to be input in to the spreadsheet:

1. Length of segment, L (mi),
2. AADT (veh/day),
3. Lane width (ft),
4. Shoulder width (ft),
5. Shoulder type,
6. Grade (%),
7. Average Annual Crash History (5-yr average) Segment crashes (KABC and PDO),
8. Length of horizontal curve (mi),
9. Radius of curvature (ft),
10. Spiral transition curve (present/not present),
11. Superelevation variance (ft/ft),
12. Driveway density (driveways/mile),
13. Centerline rumble strips (present/not present),
14. Passing lanes [present (1 lane) /present (2 lane) / not present)],
15. Two-way left-turn lane (present/not present),
16. Roadside hazard rating (1-7 scale),
17. Segment lighting (present/not present),
18. Auto speed enforcement (present/not present), and
19. Calibration Factor, Cr.

The first seven parameters were retrieved and input in the segment sections of the HSM Rural 2-Lane Roads spreadsheet leaving the remaining 12 parameters at default value (base condition) as data for these were not readily available for all projects. Lane width used for all projects was 12 ft. Shoulder width used varied from 6 to 8 ft. Shoulder type used was either paved or composite. Base conditions used for the remaining 12 segment input data parameters are shown in Table 4.3 below:

TABLE 4.3 Base Conditions Used for Segment Input Data Parameters

Input Data Parameter	Base Condition
Length of horizontal curve (mi)	0.0
Radius of curvature (ft)	0
Spiral transition curve (present/not present)	Not Present
Superelevation variance (ft/ft)	0.01
Driveway density (driveways/mile)	5
Centerline rumble strips (present/not present)	Not Present
Passing lanes [present (1 lane) /present (2 lane) / not present)]	Not Present
Two-way left-turn lane (present/not present)	Not Present
Roadside hazard rating (1-7 scale)	3
Segment lighting (present/not present)	Not Present
Auto speed enforcement (present/not present)	Not Present
Calibration Factor, Cr	1.00

For each segment the following data had to be input in to the spreadsheet:

1. Intersection type (3ST, 4ST, 4SG),
2. $AADT_{major}$ (veh/day),
3. $AADT_{minor}$ (veh/day),
4. Intersection skew angles (degrees) [If 4ST, does skew differ for minor legs? Else, No.],
5. Number of signalized or uncontrolled approaches with a left turn lane (0, 1, 2, 3, 4),
6. Number of signalized or uncontrolled approaches with a right turn lane (0, 1, 2, 3, 4),
7. Intersection lighting (present/not present),
8. Average Annual Crash History (5-yr average) Intersection crashes (KABC and PDO), and
9. Calibration Factor, Cr.

The first eight parameters were also retrieved and input in the intersection portions of the HSM Rural 2-Lane Roads spreadsheet leaving the calibration factor at default value 1.00. Intersection lighting was assumed present for all four-leg signalized intersections (4SG) and not present for all other intersection types, i.e., three-leg intersections with minor road stop control (3ST) and four-leg intersections with minor road stop control (4ST). A detailed list of input data for all intersections and segments for all 12 projects is presented in Appendix B.

The HSM Rural 2-Lane Road spreadsheet works using the predictive method shown in the flow chart (Figure 4.2) below from the Highway Safety Manual (*11*). The initial steps include defining the roadway limits and period of study, as well as AADT and crash and geometric data for the period of interest. The roadway is then divided into distinct intersections and segments with observed before crashes assigned to each. An evaluation year is selected next and then SPFs and crash modification factors (CMFs), along with the calibration factor are applied for each intersection and segment. Results from all sites are then summed and evaluated. If there is another evaluation year, the process is repeated. This process was performed a total of 19 times as seven of the 12 projects had two traffic volume years evaluated.

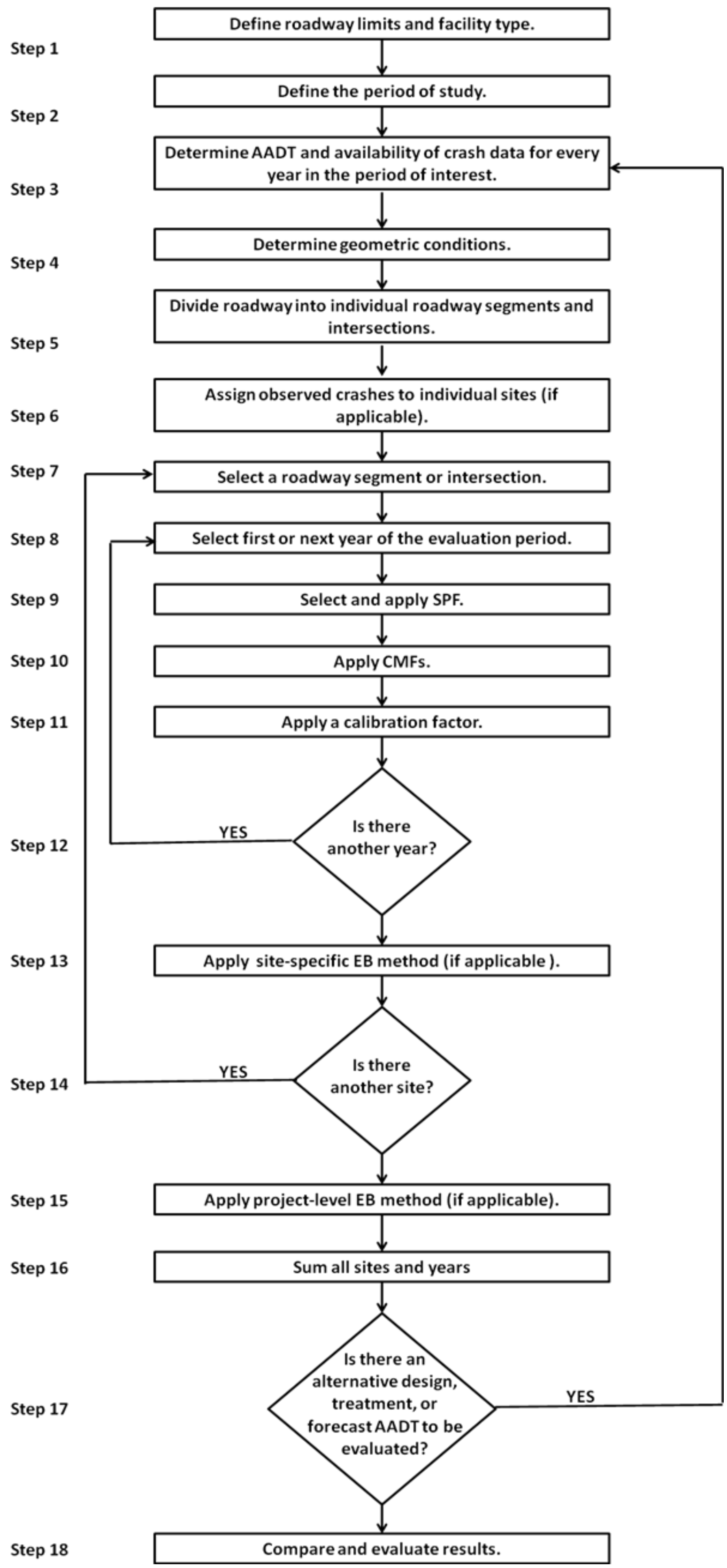

FIGURE 4.2 HSM Predictive Method for Rural Two-Lane Two-Way Roads (*11*)

Figure 4.3 and Figure 4.4 show an example of input data populated into the HSM Rural 2-Lane Roads spreadsheet for a specific segment and intersection of a project respectively.

Input Data		Site Conditions				Base Conditions
Length of segment, L (mi)		0.263779528				--
AADT (veh/day)	$AADT_{MAX}$ = 17,800 (veh/day)	8,100				--
Lane width (ft)		12				12
Shoulder width (ft)		Right Shld:	8	Left Shld:	8	6
Shoulder type		Right Shld:	Composite	Left Shld:	Composite	Paved
Length of horizontal curve (mi)		0.0				0
Radius of curvature (ft)		0				0
Spiral transition curve (present/not present)		Not Present				Not Present
Superelevation variance (ft/ft)		0.01				< 0.01
Grade (%)		3.02				0
Driveway density (driveways/mile)		5				5
Centerline rumble strips (present/not present)		Not Present				Not Present
Passing lanes [present (1 lane) /present (2 lane) / not present)]		Not Present				Not Present
Two-way left-turn lane (present/not present)		Not Present				Not Present
Roadside hazard rating (1-7 scale)		3				3
Segment lighting (present/not present)		Not Present				Not Present
Auto speed enforcement (present/not present)		Not Present				Not Present
Calibration Factor, Cr		1.00				1
Average Annual Crash History (3 or 5-yr average)						
Segment crashes	KABC	Fatal and Injury Only		0.0		Next Element
	PDO	Property Damage Only		0.2		
NOTES: * AADT: It is important to remember that the AADT(major) = AADT(major approach1) + AADT(minor approach2) (refer to p.10-6 in Part C of the HSM)						

FIGURE 4.3 Segment Data Input Example for HSM Rural 2-Lane Road Spreadsheet

Input Data							Site Conditions				Base Conditions
Intersection type (3ST, 4ST, 4SG)							4ST				--
$AADT_{major}$ (veh/day)		$AADT_{MAX}$ =	14,700	(veh/day)			8,100				--
$AADT_{minor}$ (veh/day)		$AADT_{MAX}$ =	3,500	(veh/day)			960				--
Intersection skew angle (degrees)	[If 4ST, does skew differ for minor legs? Else, No.]				No		Skew for Leg 1 (All):	0	Skew for Leg 2 (4ST only):	0	0
Number of signalized or uncontrolled approaches with a left-turn lane (0, 1, 2, 3, 4)							0				0
Number of signalized or uncontrolled approaches with a right-turn lane (0, 1, 2, 3, 4)							0				0
Intersection lighting (present/not present)							Not Present				Not Present
Calibration Factor, C_i							1.00				1.00
Average Annual Crash History (3 or 5-yr average)											
Intersection crashes							KABC	Fatal and Injury Only		0.2	
							PDO	Property Damage Only		1.4	
NOTES: * AADT: It is important to remember that the AADT(major) = AADT(major approach1) + AADT(minor approach2) (refer to p.10-6 in Part C of the HSM)											

Next Element

FIGURE 4.4 Intersection Data Input Example for HSM Rural 2-Lane Road Spreadsheet

Figure 4.5 through Figure 4.16 show the mapped before and after 2,323 crashes for all 12 projects used in the EB before-and-after study.

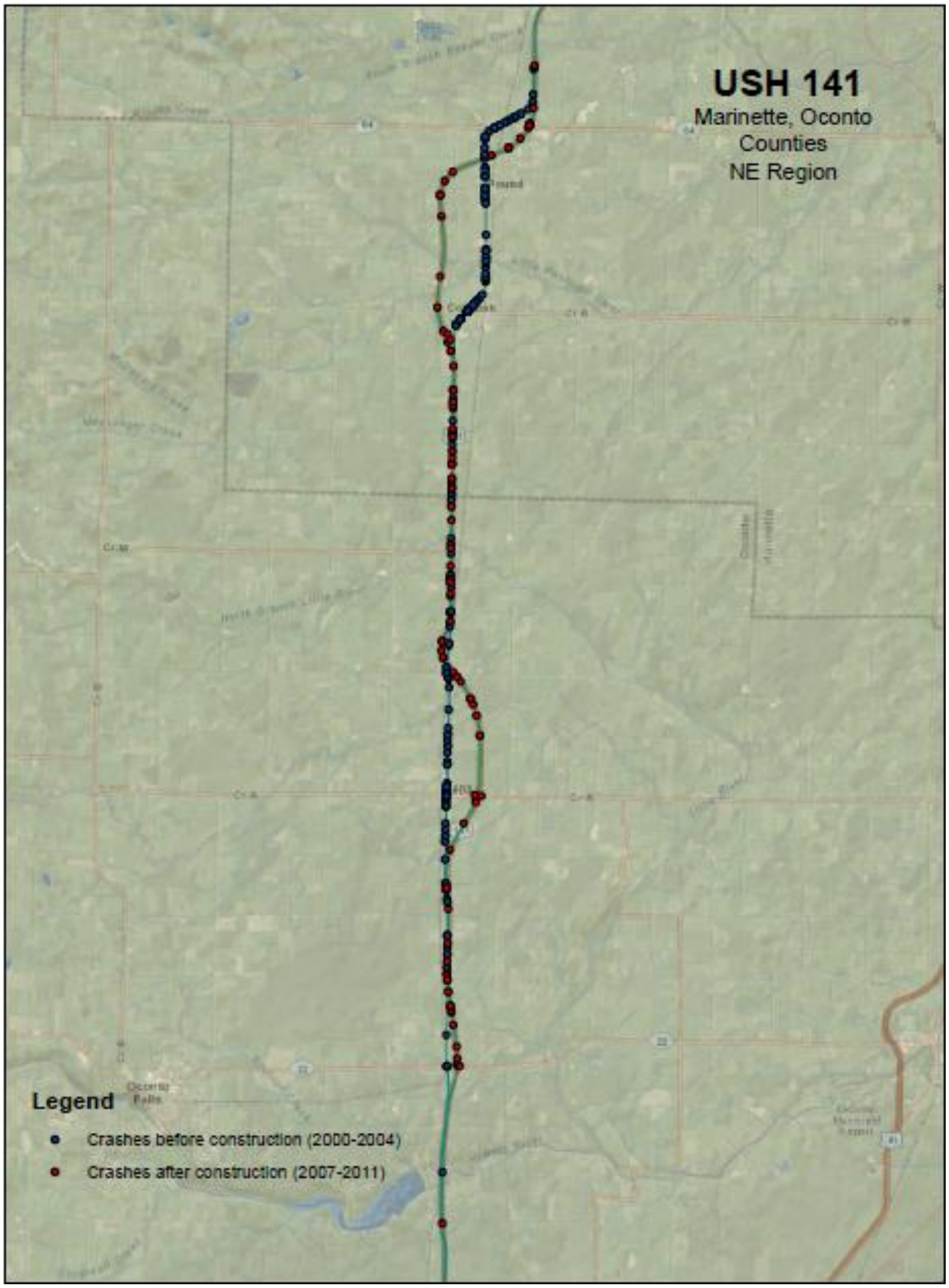

FIGURE 4.5 USH 141 Before and After Crash Map

FIGURE 4.6 STH 57 Before and After Crash Map

FIGURE 4.7 STH 164 Before and After Crash Map

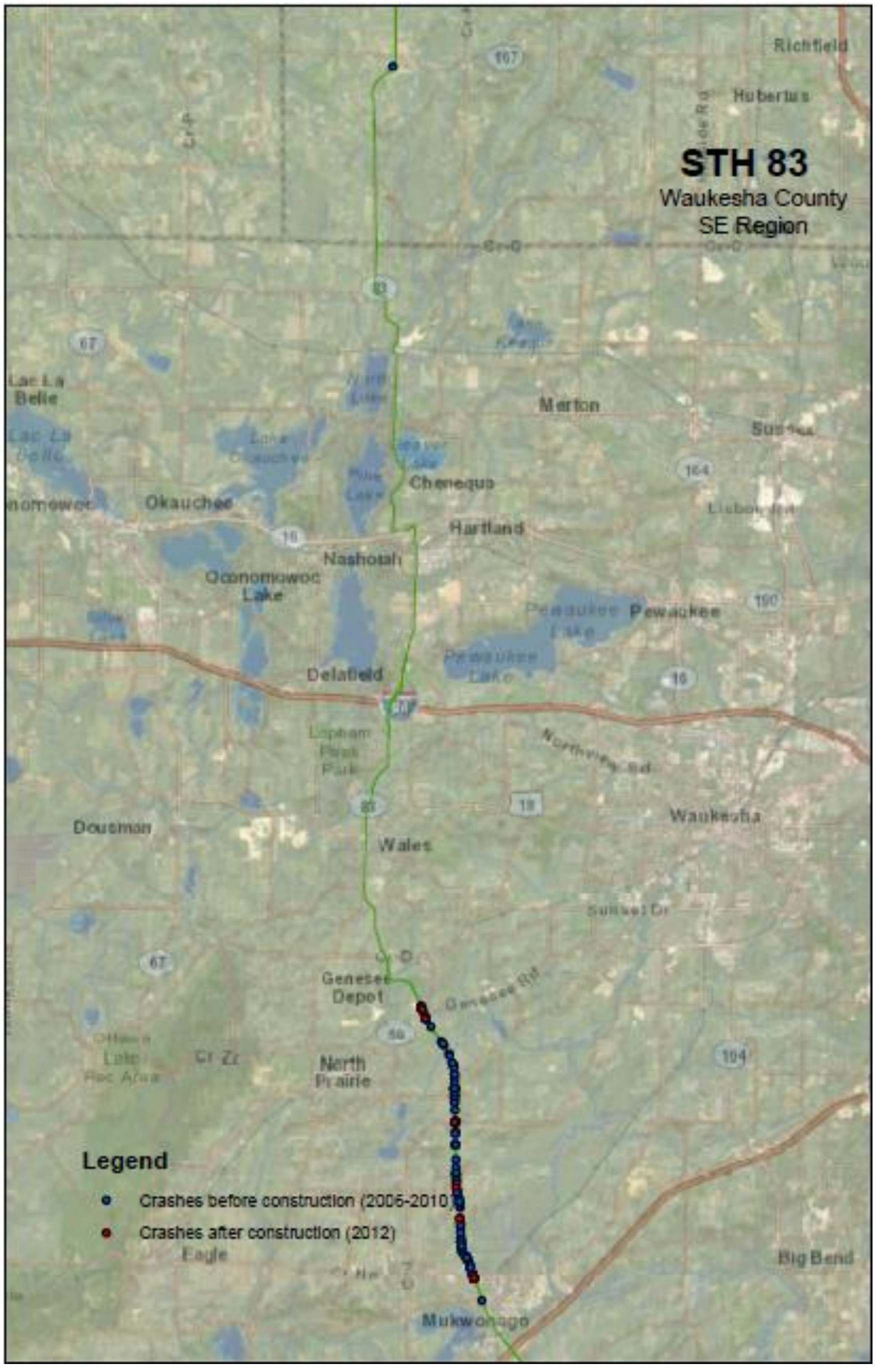

FIGURE 4.8 STH 83 Before and After Crash Map

FIGURE 4.9 STH 26 Before and After Crash Map

FIGURE 4.10 USH 12(77) Before and After Crash Map

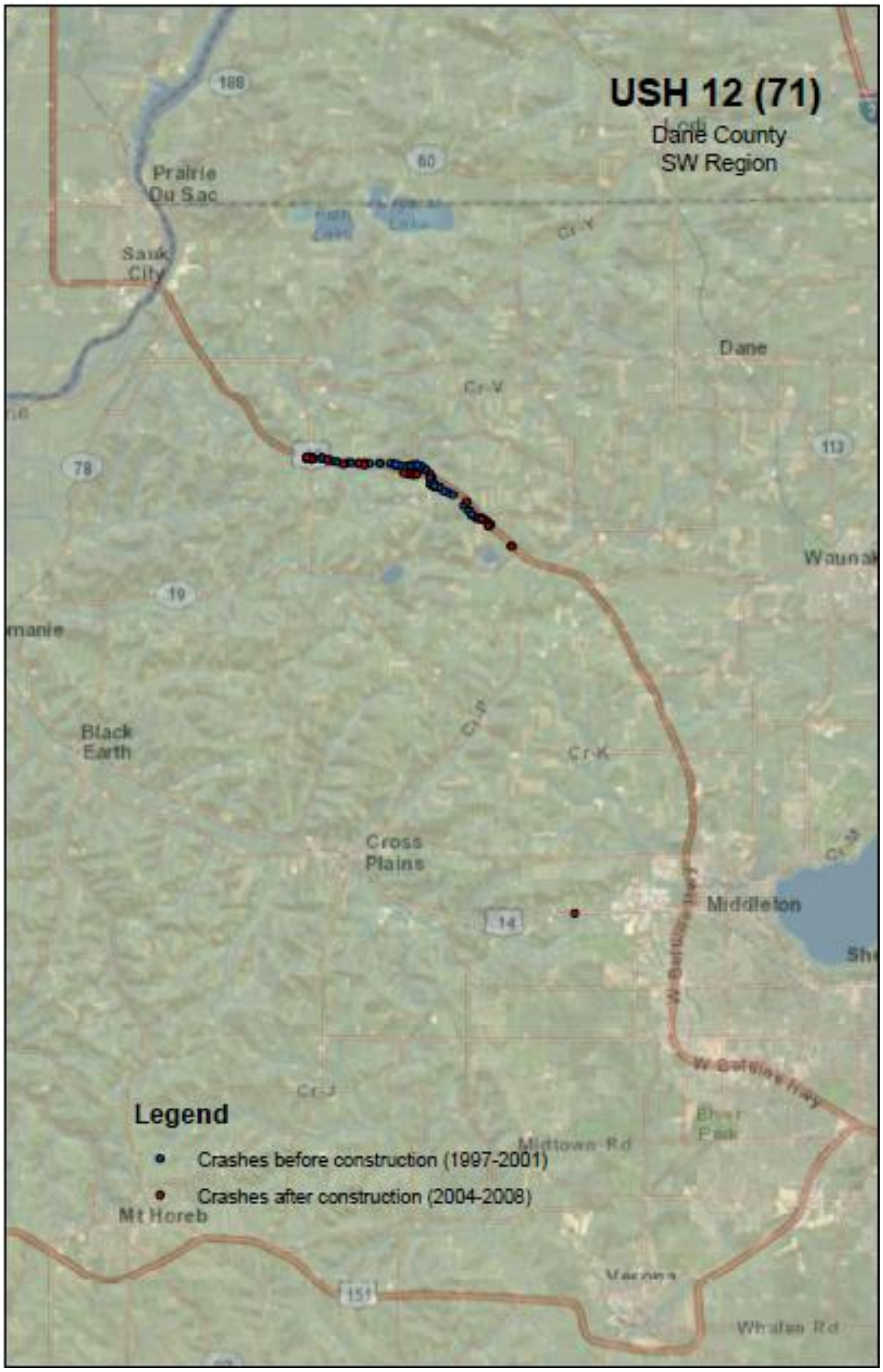

FIGURE 4.11 USH 12(71) Before and After Crash Map

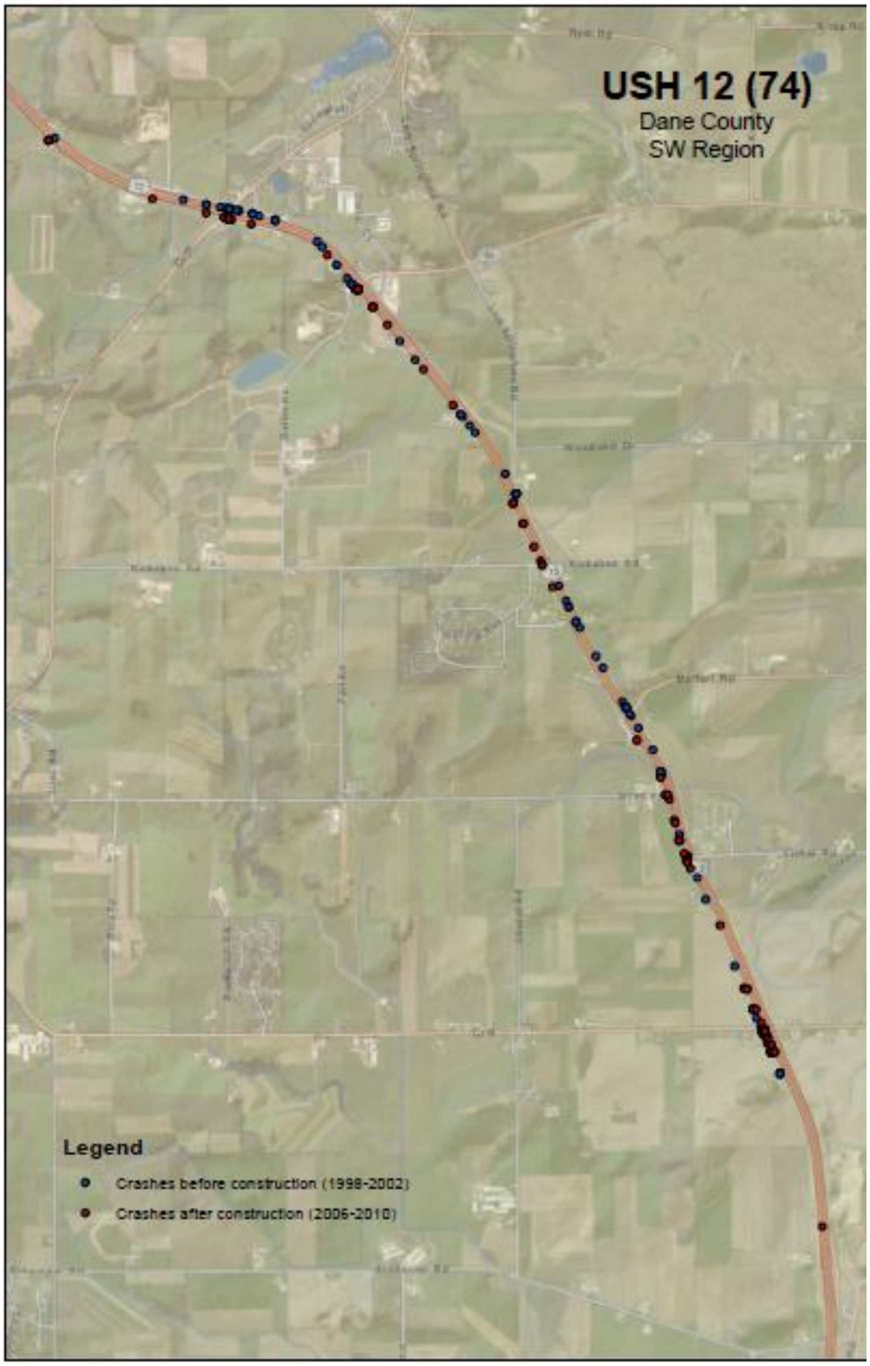

FIGURE 4.12 USH 12(74) Before and After Crash Map

FIGURE 4.13 USH 12(76) Before and After Crash Map

FIGURE 4.14 USH 14 Before and After Crash Map

FIGURE 4.15 STH 21 Before and After Crash Map

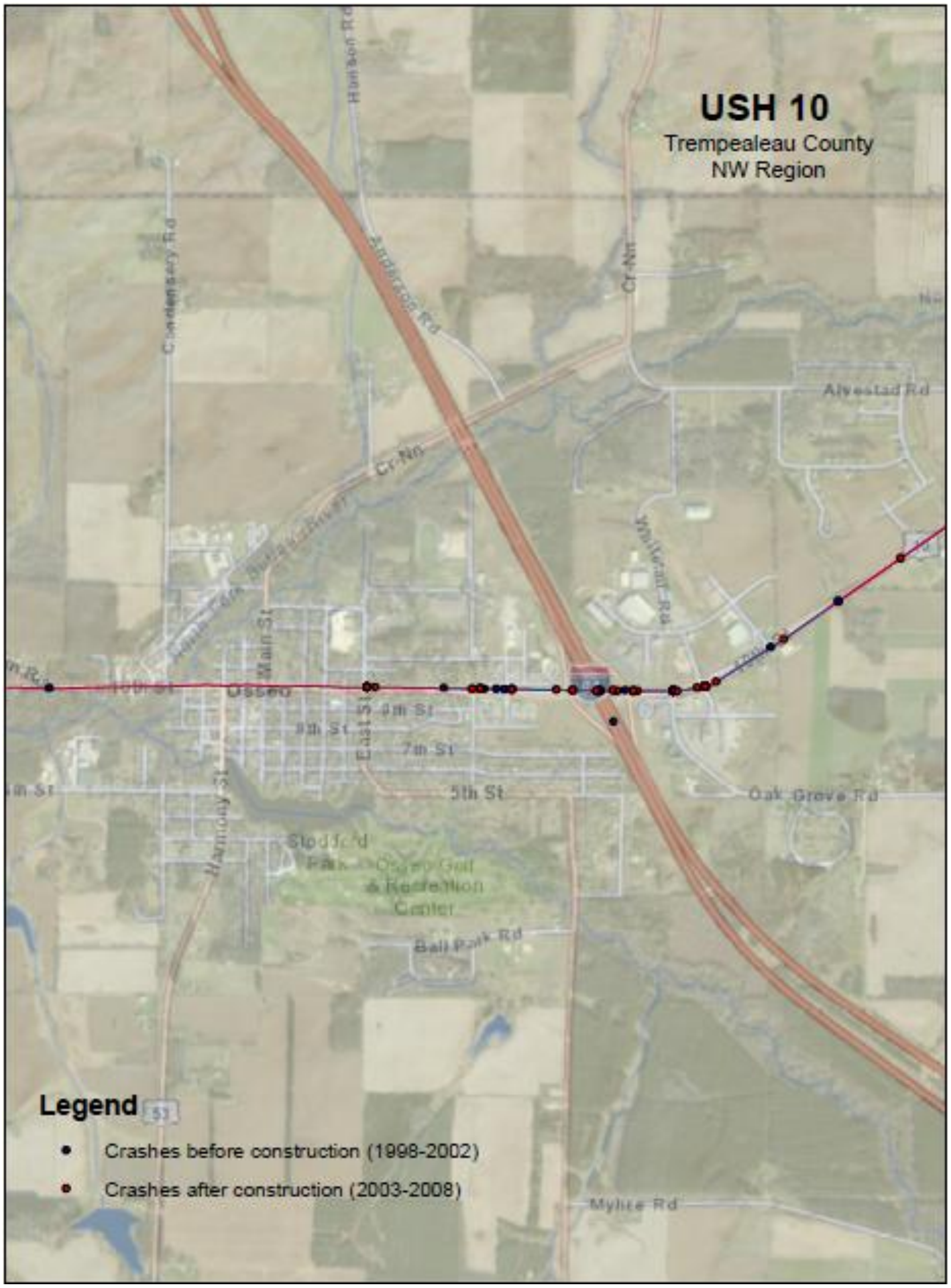

FIGURE 4.16 USH 10 Before and After Crash Map

CHAPTER 5 RESULTS

Simple Before-and-After Study

As stated in Chapter 3, a simple before-after analysis was completed for a total of 12 two-lane to four-lane conversion project locations in Wisconsin. It should be noted that the simple before-and-after analysis does not take into consideration the RTM effects. Also, STH 57 had four years of after crash data while STH 83, STH 26, and USH 14 each had one year of after crash data, compared to the five years of before crash data collected for all projects. Crash statistics are classified by crash/injury severity, i.e., worst level of crash severity to life and property taken for all persons involved in a crash by K, A, B, C, and O. Crash statistics are also classified by manner of collision, i.e., first harmful event in which participants collided in the crash by ANGL, BLNK, HEAD, NO, REAR, SSO, SSS, and UNKN. Defined by the Crash Data User Guide (*15*) as:

K = Fatal Injury

A = Incapacitating Injury

B = Non-incapacitating Injury

C = Possible Injury

O = Property Damage Only

ANGL = Angle

BLNK = Blank

HEAD = Head On Collision

NO = No collision with another vehicle

REAR = Rear End

SSO = Sideswipe/Opposite Direction

SSS = Sideswipe/Same Direction

UNKN = Unknown

Results from the simple before-and-after study show total average crashes per year reduced for all the 12 projects after the two-lane to four-lane conversion ranging from 7 to 82 %. Five out of the total 12 project sites reviewed, i.e., STH 164, STH 83, USH 12(77), USH 12(71), and USH 14 had reductions in all types of injury crashes and property damage crashes after the conversion. USH 12(76), along with STH 21 and USH 10 also had reductions in all types of injury crashes and property damage crashes after the conversion except in fatal injury in which there were no recorded crashes before or after the conversion. For all fatal and injury (K, A, B, and C) crashes, the magnitude of decrease in injury crashes was higher than the magnitude of increase. All projects had decreases in property damage only crashes. The following summarizes the trends for fatal/injury and property damage only crashes:

- Fatal (K) crashes: 14 fatal crashes were recorded in the before period throughout the 12 projects and six fatal crashes recorded in the after period. Percent average crash per year reduction ranged from 50 to 100 %. USH 164, STH 83, STH 26, USH 12(77), and USH 14 showed the largest percent decrease in fatal crashes of 100 %. Two project sites (USH 141 and USH 12(74)) each showed an increase in fatal injury crashes by a single crash.

- Incapacitating (A) crashes: 112 incapacitating crashes were recorded in the before period throughout the 12 projects and 56 incapacitating crashes recorded in the after period. Percent average crash per year reduction ranged from 23 to 100 %. STH 83, USH 12(76), and USH 14 showed the largest percent decrease in incapacitating crashes of 100 %. USH 141 showed the highest reduction of 19 crashes. STH 57 and STH 26 both showed an increase in crashes after the conversion. STH 57 showed an increase of 11 crashes which was mostly observed in year 2010. STH 26 showed three crashes recorded in the before period and two crashes recorded in the after period. However, STH 26 compared five years of before crash data to one year of after crash data and hence the average showed an increase in the after period.

- Non-incapacitating (B) crashes: 225 non-incapacitating crashes were recorded in the before period throughout the 12 projects and 99 non-incapacitating crashes recorded in the after period. Percent average crash per year reduction ranged from 17 to 100 %. USH 12(76) showed the largest percent decrease in non-incapacitating crashes of 100 %. USH 12(71) showed the highest reduction of 20 crashes. USH 12(74) and STH 57 both showed an increase in crashes after the conversion. USH 12(74) showed an increase of two crashes. STH 57 showed 27 crashes recorded in the before period and 24 crashes recorded in the after period. However, STH 57 compared five years of before crash data to four years of after crash data and hence the average showed an increase in the after period.

- Possible Injury (C) crashes: 314 possible injury crashes were recorded in the before period throughout the 12 projects and 118 possible injury crashes recorded in the after period. Percent average crash per year reduction ranged from 14 to 100 %. USH 14 showed the largest percent decrease in possible injury crashes of 100 %. USH 12(77) showed the highest reduction of 33 crashes. USH 12(74) showed an increase of two crashes (about 8 %).

- Property Damage Only Crashes: Property damage only crashes were the highest recorded crashes in both the before and after period. There were 1019 property damage only crashes recorded in the before period throughout the 12 projects and 505 property damage only crashes recorded in the after period. Percent average crash per year reduction ranged from 4 to 83 %. USH 12(71) showed the largest percent decrease in property crashes of 83 %. USH 141 showed the highest reduction of 93 crashes. There was no increase in after crashes.

Table 5.1 shows whether the observed crash statistics for the 12 projects reduced after the conversion in crash severity and Table 5.2 shows by what percentage.

TABLE 5.1 Simple Before-And-After Analysis Crash Severity Results Summary

Project	Average Before Crashes/Average After Crashes					
	Average Number of Crashes (per year)	Injury Crashes (per year)				Property Damage Only Crashes (per year)
		K	A	B	C	O
USH 141	53.60/22.40	0.40/0.60	5.00/1.20	6.00/2.80	9.20/3.40	33.00/14.40
STH 57*	47.80/35.50	0.60/0.25	1.80/5.00	5.40/6.00	10.00/4.25	30.00/20.00
STH 164	19.40/16.20	0.20/0.00	1.40/1.00	3.60/2.00	4.20/3.60	10.00/9.60
STH 83**	39.40/19.00	0.40/0.00	2.40/0.00	6.40/3.00	8.20/2.00	22.00/14.00
STH 26**	13.60/11.00	0.40/0.00	0.60/2.00	2.20/1.00	2.00/1.00	8.40/7.00
USH 12(77)	25.20/9.80	0.20/0.00	1.80/0.80	3.60/1.20	7.20/0.60	12.40/7.20
USH 12(71)	28.40/5.20	0.40/0.20	2.20/0.40	4.80/0.80	3.60/0.80	17.40/3.00
USH 12(74)	36.40/33.80	0.00/0.20	2.60/2.00	5.00/5.40	5.60/6.00	23.20/20.20
USH 12(76)	13.00/2.80	0.00/0.00	0.40/0.00	2.20/0.00	2.40/0.40	8.00/2.40
USH 14**	12.80/5.00	0.20/0.00	1.20/0.00	1.20/1.00	1.60/0.00	8.60/4.00
STH 21	30.80/20.20	0.00/0.00	2.00/1.00	3.20/1.20	7.00/3.80	18.60/14.20
USH 10	16.40/11.00	0.00/0.00	1.00/0.40	1.40/0.60	1.80/1.00	12.20/9.00
Projects with Reduction	12	7	10	10	11	12
Projects with Increase	0	2	2	2	1	0
Projects with No Change	0	3	0	0	0	0

* Four years of crash data after construction compared to five years of crash data before construction
** One year of crash data after construction compared to five years of crash data before construction

[green] average crashes decreased
[red] average crashes increased
[yellow] average crashes did not change

TABLE 5.2 Simple Before-And-After Analysis Crash Severity Percent Increase or Decrease

Project	Percent Increase or Decrease After Crashes					
	Total Crashes	Injury Crashes				Property Damage Only Crashes
		K	A	B	C	O
USH 141	58%	-50%	76%	53%	63%	56%
STH 57*	26%	58%	-178%	-11%	58%	33%
STH 164	16%	100%	29%	44%	14%	4%
STH 83**	52%	100%	100%	53%	76%	36%
STH 26**	19%	100%	-233%	55%	50%	17%
USH 12(77)	61%	100%	56%	67%	92%	42%
USH 12(71)	82%	50%	82%	83%	78%	83%
USH 12(74)	7%	0.00/0.20	23%	-8%	-7%	13%
USH 12(76)	78%	0%	100%	100%	83%	70%
USH 14**	61%	100%	100%	17%	100%	53%
STH 21	34%	0%	50%	63%	46%	24%
USH 10	33%	0%	60%	57%	44%	26%

* Four years of crash data after construction compared to five years of crash data before construction
** One year of crash data after construction compared to five years of crash data before construction

average crashes decreased
average crashes increased
average crashes did not change

Three project sites (USH 12(77), USH 12(71) and USH 14) showed a reduction in crashes in all manner of collisions considered. All projects reduced in sideswipe in the opposite direction crashes. The following summarizes the trends for manner of collision crashes:

- ANGL crashes: 329 angle crashes were recorded in the before period throughout the 12 projects and 223 angle crashes recorded in the after period. Percent average crash per year reduction ranged from 3 to 100 %. Angle crash per year reduction is attributed to the projects being converted to four-lane divided roadways and therefore having less conflict points for angle crashes. USH 14 showed the largest percent decrease in angle crashes of 100 %. USH 141 showed the highest reduction of 35 crashes. STH 26 showed no change between before and after crashes recorded. However, STH 26 compared five years of before crash data to one year of after crash data. Two project sites (STH 57 and USH 12(74)) showed an increase angle crashes. STH

57 showed an increase of 17 crashes which was mostly observed in year 2010 at intersection CTH C. USH 12(74) showed an increase of 25 crashes which was mostly observed in years 2006, 2007, and 2009 at intersection STH 19.

- BLNK crashes: Blank crashes are those in which a collision type is not stated in the crash report. The crashes attributed here could therefore be placed in any of the other collision categories but were however kept separate in order not to taint the accuracy of the other collision type category results. There were 28 blank crashes recorded in the before period throughout the 12 projects and three blank crashes recorded in the after period.

- HEAD crashes: 51 head on collision crashes were recorded in the before period throughout the 12 projects and 10 recorded in the after period. Percent average crash per year reduction ranged from 50 to 100 %. Head on collision crash per year reduction is attributed to the projects being converted to four-lane divided roadways and therefore having less conflict points for head on crashes. USH 141, STH 83, STH 26, USH 12(77), USH 14, and USH 10 showed the largest percent decrease in head on crashes of 100%. USH 141 showed the highest reduction of 14 crashes. STH 164 showed an increase of two crashes after the conversion at the Lisbon Rd. intersection.

- NO crashes: 567 no collision with another vehicle crashes were recorded in the before period throughout the 12 projects and 277 recorded in the after period. Percent average crash per year reduction ranged from 20 to 85 %. USH 12(71) showed the largest percent decrease of 85 %. USH 12(71) also showed the highest reduction of 78 crashes. USH 141, STH 164, STH 21 and USH 10 all showed increase in crashes after the conversion. USH 141 showed an increase of six crashes. The slight increase in crashes on USH 141 occurred in year 2009. STH 164 showed an increase of 17 crashes. The increase in crashes on STH 164 occurred between years 2007 and 2009 and between Prospect Ct. and Clover Dr., Seven Stones Dr. and Lake Park Dr., and

Chesterwood Ln. and Swan Rd. STH 21 showed an increase of five crashes by an average of one crash increase per year. These crashes on STH 21 were mostly traffic sign collisions at the USH 12 and I-94 intersections. USH 10 showed an increase of seven crashes. These crashes on USH 10 were mostly traffic sign collisions at the Hilltop Rd. intersection.

- REAR crashes: 511 rear end crashes were recorded in the before period throughout the 12 projects and 186 recorded in the after period. Percent average crash per year reduction ranged from 29 to 89 %. Rear end crash per year reduction is attributed partly to the projects being converted to four-lane roadways and therefore having less conflict points for rear end crashes as now safe passing lanes are available to avoid hard breaks. USH 12(76) showed the largest percent decrease in rear end crashes of 89 %. USH 141 showed the highest reduction of 81 crashes. STH 26 and USH 12(74) both showed an increase in crashes after the conversion. STH 26 showed 10 crashes recorded in the before period and three crashes recorded in the after period. However, STH 26 compared five years of before crash data to one year of after crash data and hence the average showed an increase in the after period. USH 12(74) showed an increase of six crashes after the conversion mostly in years 2006, 2007, 2009, and 2010 at the intersections CTH K and STH 19.

- SSOP crashes: There was a reduction in sideswipe in the opposite direction crashes across all 12 projects. There were 75 sideswipe opposite direction crashes recorded in the before period throughout the 12 projects and eight recorded in the after period. Percent average crash per year reduction ranged from 50 to 100 %. Sideswipe in the opposite direction crash per year reduction is attributed to the projects being converted to four-lane divided roadways and therefore having less conflict points for sideswipe opposite direction crashes. STH 57, STH 164, STH 83, STH 26, USH 12(77), USH 14, and USH 10 showed the largest percent decrease in sideswipe opposite direction crashes of 100%. USH 141 showed the highest reduction of 14 crashes.

- SSS crashes: 121 sideswipe in the same direction crashes were recorded in the before period throughout the 12 projects and 75 recorded in the after period. Percent average crash per year reduction ranged from 31 to 100 %. Sideswipe in the same direction crash per year reduction is attributed partly to the projects being converted to four-lane roadways and therefore having less conflict points for sideswipe opposite direction crashes as now safe passing lanes are available. USH 14 showed the largest percent decrease in sideswipe same direction crashes of 100%. USH 141 showed the highest reduction of 13 crashes. USH 12(74) and USH 12(76) showed no change between before and after crashes recorded. STH 83, STH 26 and STH 21 all showed an increase in crashes after the conversion. STH 83 showed 10 crashes recorded in the before period and four crashes recorded in the after period. However, STH 83 compared five years of before crash data to one year of after crash data and hence the average showed an increase in the after period. In the same light, STH 26 showed three crashes recorded in the before period and three crashes recorded in the after period. However, STH 26 as well compared five years of before crash data to one year of after crash data and hence the average showed an increase in the after period. STH 21 showed an increase of eight crashes after the conversion mostly in year 2006 at the Sam Walton Dr. intersection.

- UNKN crashes: Unknown crashes are those in which a collision type is unknown when recorded in the crash report. The crashes attributed here could therefore be placed in any of the other collision categories but were however kept separate in order not to taint the accuracy of the other collision type results. There was one unknown crash recorded in the before period throughout the 12 projects and three unknown crashes recorded in the after period both within the same project (STH 83).

Table 5.3 shows whether the observed crash statistics for the 12 projects reduced after the conversion in manner of collision and Table 5.4 shows by what percentage.

TABLE 5.3 Simple Before-And-After Analysis Manner of Collision Results Summary

Project	Average Before Crashes/Average After Crashes							
	Manner of Collision (per year)							
	ANGL	BLNK	HEAD	NO	REAR	SSOP	SSS	UNKN
USH 141	11.00/4.00	1.00/0.00	2.80/0.00	12.40/13.60	18.60/2.40	3.00/0.20	4.80/2.20	0.00/0.00
STH 57*	5.80/11.50	1.00/0.00	1.20/0.25	23.20/18.50	10.60/3.50	2.60/0.00	3.40/1.75	0.00/0.00
STH 164	5.80/5.60	0.60/0.20	0.80/1.20	1.40/4.80	8.20/3.80	1.40/0.00	1.20/0.60	0.00/0.00
STH 83**	9.00/2.00	0.60/0.00	1.00/0.00	13.60/8.00	11.40/2.00	1.60/0.00	2.00/4.00	0.20/3.00
STH 26**	2.00/2.00	0.00/0.00	0.40/0.00	8.00/4.00	2.00/3.00	0.60/0.00	0.60/2.00	0.00/0.00
USH 12(77)	4.40/2.80	0.60/0.00	0.80/0.00	9.80/5.00	7.60/1.20	0.60/0.00	1.40/0.80	0.00/0.00
USH 12(71)	3.20/0.80	0.20/0.00	0.60/0.20	18.40/2.80	2.20/0.80	1.80/0.20	2.00/0.40	0.00/0.00
USH 12(74)	4.20/9.20	0.80/0.20	1.60/0.20	11.40/5.00	15.80/17.00	0.40/0.20	2.00/2.00	0.00/0.00
USH 12(76)	1.20/0.20	0.60/0.00	0.00/0.00	6.80/1.60	3.60/0.40	0.40/0.20	0.40/0.40	0.00/0.00
USH 14**	1.40/0.00	0.00/0.00	0.40/0.00	6.00/3.00	2.80/2.00	0.80/0.00	1.40/0.00	0.00/0.00
STH 21	12.60/7.80	0.00/0.20	0.40/0.20	1.00/2.00	12.80/5.20	1.60/0.80	2.40/4.00	0.00/0.00
USH 10	5.20/4.20	0.20/0.00	0.20/0.00	1.40/2.80	6.60/2.20	0.20/0.00	2.60/1.80	0.00/0.00
Projects with Reduction	9	9	11	8	10	12	7	0
Projects with Increase	2	1	1	4	2	0	3	1
Projects with No Change	1	2	1	0	0	0	2	11

* Four years of crash data after construction compared to five years of crash data before construction
** One year of crash data after construction compared to five years of crash data before construction

average crashes decreased

average crashes increased

average crashes did not change

TABLE 5.4 Simple Before-And-After Analysis Manner of Collision Percent Increase or Decrease

Project	Percent Increase or Decrease After Crashes							
	Manner of Collision							
	ANGL	BLNK	HEAD	NO	REAR	SSOP	SSS	UNKN
USH 141	64%	100%	100%	-10%	87%	93%	54%	0%
STH 57*	-98%	100%	79%	20%	67%	100%	49%	0%
STH 164	3%	67%	-50%	-243%	54%	100%	50%	0%
STH 83**	78%	100%	100%	41%	82%	100%	-100%	-1400%
STH 26**	0%	0%	100%	50%	-50%	100%	-233%	0%
USH 12(77)	36%	100%	100%	49%	84%	100%	43%	0%
USH 12(71)	75%	100%	67%	85%	64%	89%	80%	0%
USH 12(74)	-119%	75%	88%	56%	-8%	50%	0%	0%
USH 12(76)	83%	100%	0%	76%	89%	50%	0%	0%
USH 14**	100%	0%	100%	50%	29%	100%	100%	0%
STH 21	38%	0.00/0.20	50%	-100%	59%	50%	-67%	0%
USH 10	19%	100%	100%	-100%	67%	100%	31%	0%

* Four years of crash data after construction compared to five years of crash data before construction
**One year of crash data after construction compared to five years of crash data before construction

average crashes decreased (green)
average crashes increased (red)
average crashes did not change (yellow)

Overall, there was a total reduction of crashes of 900 which is about a 53 % decrease. Again, four out of the 12 projects had less than five years of comparable after conversion crash data so the percent increase or decrease of average number of crashes per year may be different if five years of after conversion crash data is later used.

Empirical Bayes Before-and-After Study

As stated in Chapter 3, an Empirical Bayes before-after analysis was also completed for a total of 12 two-lane to four-lane conversion project locations in Wisconsin. In the EB analysis, simple before-and-after comparison assumes that conditions remain unchanged before and after the conversion, even though this may necessarily not always be the case. As such, a traffic volume adjustment is frequently used to normalize for differences in traffic volume between before and after volumes. The EB analysis results were compared to up to five years (when available) of crash data after the conversion. Summarized below are the results from the EB analysis:

- All the projects showed an increase in crashes when compared to the observed average number of crashes per year after the conversion. Results from the EB analysis show that without the conversion, average number of crashes per year increased as traffic volume increased. Therefore, when compared to the observed average number of crashes per year after the conversion, all the projects showed that expected average number of crashes per year was higher.

- The EB analysis proved that the two-lane to four-lane conversion resulted in a reduction in total average number of crashes per year ranging from about 10 to 85 % on an average of about 62 %.

- One project (STH 21) showed that expected average number of crashes per year decreased even without the conversion, which could be due to stricter traffic law enforcement in that area, or a rerouting in traffic pattern or a number of unspecified factors. However, a further reduction of about 10 to 11 % was obtained after the conversion.

- Across all the 12 projects property damage only crashes were higher than injury crashes.

- Segment crashes were lower than intersection crashes for nine out of the 12 projects. This is common as there are more conflict points at intersections. The exceptions were STH 26, USH 12 (77), and USH 12(74). When considering injury crashes and property damage only crashes separately, USH 12(71), USH 12 (77), and USH 12(74) had fewer injury crashes and more property damage only crashes at the intersections compared to within the segments.

- USH 12(76) showed the highest percent reduction in average number of crashes per year of about 85 %.

- STH 21 showed the lowest percent reduction in average number of crashes per year of 10 to 11 %.

- STH 57 and USH 141 showed the highest expected average number of crashes per year of 121.4 and 83.7 crashes respectively. It is suggested that this is due to the two projects having very large numbers of intersections and segments compared to the other projects. As each intersection and segment results in a crash statistic greater than zero from the EB analysis, the larger the number of intersections and segments within a project, the higher the expected average number of crashes per year result.

Isolating the effects of geometric considerations such as lane width, shoulder width and type, percent grade, intersection type, and number of turn lanes on safety from the EB analysis proved to be difficult. The reason for the difficulty was because lane width, shoulder width and shoulder type remained unchanged within each project, so there was no data within the project to make the comparison. On the other hand, percent grade, intersection type, and number of turn lanes changed constantly with traffic volume within each project, so it was difficult to isolate the effect of the individual geometric characteristic on safety. However, a few conclusions were drawn:

- Percent grade was observed to have minimal effect on the expected average number of crashes per year result when percent grade was put in the HSM spreadsheet.

- For projects with more than one intersection having the same traffic volume and intersection control type, higher turn lanes produced higher average number of crashes per year for that intersection.

- For projects with more than one intersection having the same traffic volume on the mainline and number of turn lanes, four-leg stop control on minor road with a lower traffic volume on the minor produced higher average number of crashes per year than three-leg stop control on minor road with a higher traffic volume on the minor road.

- Signalized intersections usually had higher traffic volume on both the mainline and cross-streets and also had higher number of turn lanes and therefore produced higher average number of crashes per year.

Table 5.5 represents the EB analysis results summary, including, before and after mainline and cross street crash data as well as the total expected average crashes per year and the number of intersections/segments evaluated for each project. Table 5.6 shows the average number of crashes per year percent decrease.

TABLE 5.5 Empirical Bayes Analysis Results Summary

Project	USH 141	STH 57	STH 164	STH 83	STH 26	USH 12	USH 14	STH 21	USH 10
Segment Length in miles (Number of Intersections / Number of Segments)	16.92 (20/19)	25.92 (43/43)	3.82 (11/10)	5.62 (9/9)	6.12 (7/7)	03-77: 4.47 (5/6) 03-71: 4.76 (7/6) 03-74: 6.05 (8/8) 03-76: 1.70 (4/3)	5.03 (8/7)	2.17 (6/6)	1.49 (9/9)
Total Crashes Before (Number of Years Evaluated)	253 (5)	216 (5)	97 (5)	189(5)	68 (5)	03-77: 125 (5) 03-71: 112 (5) 03-74: 152 (5) 03-76: 65 (5)	64 (5)	143 (5)	77 (5)
Average Before Crashes / Year	50.6	43.2	19.4	37.8	13.6	03-77: 25 03-71: 22.4 03-74: 30.4 03-76: 13	12.8	28.6	15.4
Total Crashes After (Number of Years Evaluated)	111 (5)	142 (4)	80 (5)	14(1)	11 (1)	03-77: 49 (5) 03-71: 25 (5) 03-74: 168 (5) 03-76: 14 (5)	5 (1)	99 (5)	44 (5)
Average After Crashes / Year	22.2	35.5	16	14	11	03-77: 9.8 03-71: 5 03-74: 33.6 03-76: 2.8	5	19.8	8.8
Expected Average Crashes / Year (Traffic Count Year)	83.7 (2009)	121.4 (2009)	34.6 (2006) 39.0 (2009)	38.8 (2012)	21.3 (2012)	03-77: 30.2 (2006) 31.7 (2009) 03-71: 31.9 (2005) 32.1 (2006) 03-74: 44.1 (2006) 47.2 (2009) 03-76: 19.9 (2006) 19.9 (2009)	27.1 (2012)	22.0 (2006) 22.2 (2008)	33.0 (2004) 31.2 (2006)

TABLE 5.6 Empirical Bayes Analysis Average Number of Crashes Per Year Percent Decrease

Project	Expected Average Number of Crashes Per Year	Observed After Conversion Average Number of Crashes Per Year	Percent Decrease
USH 141	83.7	22.2	73%
STH 57*	121.4	35.5	71%
STH 164	34.6	16.0	54%
	39.0		59%
STH 83**	38.8	14.0	64%
STH 26**	21.3	11.0	48%
USH 12(77)	30.2	9.8	68%
	31.7		69%
USH 12(71)	31.9	5.0	84%
	32.1		84%
USH 12(74)	44.1	33.6	24%
	47.2		29%
USH 12(76)	19.9	3.0	85%
	19.9		85%
USH 14**	27.1	5.0	82%
STH 21	22.0	19.8	10%
	22.2		11%
USH 10	33.0	8.8	73%
	31.2		72%

* Four years of crash data after construction compared to five years of crash data before construction
**One year of crash data after construction compared to five years of crash data before construction
average crashes decreased

Overall, the EB analysis proved that the two-lane to four-lane conversion resulted in a reduction in average number of crashes per year ranging from about 10 to 85 % on an average of about 62 %. However, four out of the 12 projects had less than five years of comparable after conversion crash data so the percent increase or decrease of average number of crashes per year may be different if five years of after conversion crash data is later used.

CHAPTER 6 CONCLUSIONS

For this study, a total of 12 two-lane to four-lane conversion projects built in Wisconsin from 2002 to 2011 were considered. All 12 projects were analyzed using simple before-and-after and EB analysis. Five years of before crash data and up to five years of after crash data (when available) were gathered, as well as geometric and volume data. A simple before-and-after crash analysis was completed to analyze specific types of injury crashes and manner of collision for each project. An EB analysis was used to examine the safety benefits for total and injury crashes. Conclusion from both analyses was that two-lane to four-lane conversions are a safety benefit and result in average number of crashes per year reduction of about 7 to 85 %.

Simple Before-and-After Study

Results from the simple before-and-after study show average number of crashes per year reduced by about 7 to 82 % after the two-lane to four-lane conversion in all the 12 projects. Five out of the total 12 project sites reviewed reduced in all types of injury crashes and property damage crashes after the conversion. Three additional projects also reduced in all types of injury crashes and property damage crashes after the conversion except in fatal injury in which there were no recorded crashes before or after the conversion. Two projects each showed an increase fatal injury by a single crash. One project showed an increase in incapacitating crashes by l1 crashes which mostly occurred in one particular year. One project showed an increase in non-incapacitating and possible injury crashes by two crashes each. All projects reduced in property damage only crashes ranging from 4 to 83 %. Three project sites showed a reduction in crashes in all manner of collisions considered. All projects reduced in sideswipe in the opposite direction crashes.

Empirical Bayes Before-and-After Study

Results from the EB analysis show that without the conversion, average number of crashes increased as traffic volume increased. Therefore, when compared to the observed average number of crashes per year after the conversion, all the projects showed that expected average number of crashes per year was higher. The EB analysis proved that the two-lane to four-lane conversion resulted in a reduction in average number of crashes per year ranging from about 10 to 85 % on an average of about 62 %. One project (STH 21) showed that expected average number of crashes per year decreased even without the conversion but a further reduction of about 10 to 11 % was obtained after the conversion. Also, segment crashes were noted to be lower than intersection crashes for nine out of the 12 projects which is a common trend as there are typically more conflict points at intersections than within segments. Effects of geometric considerations such as lane width, shoulder width and type, percent grade, intersection type, and number of turn lanes on safety from the EB analysis proved to be difficult to accurately determine as lane width, shoulder width and shoulder type remained unchanged within each project and so there was no data within the project to make the comparison while percent grade, intersection type, and number of turn lanes changed constantly with traffic volume within each project and so it was difficult to isolate the effect of the individual geometric characteristic on safety. However, percent grade was observed to have minimal effect on the expected average number of crashes per year while intersections with higher number of turn lanes, and signalized or four-leg stop control types had higher average number of crashes per year.

Future Study

This research study has shown that overall, the two-lane to four-lane conversions have led to improvements in traffic safety, especially in terms of property damage and injury crashes. Detailed review of the crashes at specific locations within the projects that show an increase in crashes could reveal further insight into the crash trends and safety issues at such locations. For purposes of this research only SPFs for rural areas were used. In future study SPFs should be replaced for the projects with urban area type. Another recommendation is that additional locations be included, to increase the sample size studied. Furthermore, projects with fewer than five years of comparable after conversion crash data should be reviewed when adequate crash data can be retrieved. Effects of individual geometric considerations can be isolated and evaluated by keeping all the other parameters in the HSM spreadsheet constant and only changing the geometric feature being evaluated. Finally, further research should be performed to determine the statistical significance of the conversion such as in the case of STH 21 from the EB analysis where expected total average crash still reduced without the conversion.

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Appendix A – List of Safety Performance Functions

Safety Performance Functions Used in EB Before-And-After Study

Intersection Type	Total Crashes				Source
	a	b	c	k	
2Rur3ST	-9.86	0.79	0.49	0.54	HSM 10-18
2Rur4ST	-8.56	0.60	0.61	0.24	HSM 10-19
2Rur4SG	-5.13	0.60	0.20	0.11	HSM 10-20

Appendix B – EB Analysis Data Input and Results Summary

*Default value used because no value obtained or value varied
**Default value used because value obtained exceeds maximum
***Volume adjusted by 3% increase per year and rounded up to nearest 100/10 from traffic volume obtained
****Default volume used because volume adjusted by 3% increase per year and rounded up to the nearest 100/10 from traffic volume obtained exceeds maximum
(Actual value obtained/retrieved)

USH 141 Segments

Segment	Segment Length (mi)	2009 AADT (veh/day)	Lane Width (ft)	Shoulder Width (ft)	Shoulder type	Grade (%)	KABC	2009 EB KABC	PDO	2009 EB PDO
STH 22-Olson Rd	0.50	10900	12	8** (10)	Composite	0*	0.2	0.3	0.4	0.8
Olson Rd-McCarthy Rd	0.50	10900	12	8** (10)	Composite	0*	0.0	0.3	0.4	0.6
McCarthy Rd-Lemere Rd	0.52	10900	12	8** (10)	Composite	0*	0.0	0.3	0.2	0.6
Lemere Rd-Vernosh Rd	0.72	10900	12	8** (10)	Composite	0*	1.2	0.5	0.4	1.3
Vernosh Rd-Guelig Rd	0.76	10900	12	8** (10)	Composite	0*	0.4	0.6	0.6	1.1
Guelig Rd-Main St/CTH A	1.44	10900	12	8** (10)	Composite	0*	0.6	1.1	2.2	2.4
Main St/CTH A-Goatsville Rd	2.00	9800	12	8** (10)	Composite	0*	1.8	1.5	2.6	3.3
Goatsville Rd-Starlite Rd	1.00	9800	12	8** (10)	Composite	0*	0.6	0.6	0.6	1.4
Starlite Rd-Sunny Brook Rd	0.51	9800	12	8** (10)	Composite	0*	0.0	0.3	0.2	0.6
Sunny Brook Rd-CTH M	0.50	9800	12	8** (10)	Composite	0*	0.4	0.3	0.2	0.7
CTH M-2nd Rd E	1.42	9000***	12	8** (10)	Composite	0.31	1.2	0.8	0.6	1.8
2nd Rd E-CTH M	0.50	9000***	12	8** (10)	Composite	0.31	0.2	0.3	0.2	0.6
CTH M-8th Rd W	0.50	8800	12	8** (10)	Composite	0.31	0.2	0.3	0.4	0.7
8th Rd W-Main St/CTH B	1.56	8800	12	8** (10)	Composite	0.36	1.0	1.0	1.4	2.1
Main St/CTH B-16th Rd E	1.09	7900	12	8** (10)	Composite	0.50	1.8	1.0	3.0	2.0
16th Rd E-CTH Q	1.01	7900	12	8** (10)	Composite	0.68	0.6	0.6	0.6	1.3
CTH Q-STH 64	1.09	7600	12	8** (10)	Composite	0.84	0.8	0.8	2.2	1.6
STH 64-STH 64	0.34	7600	12	8** (10)	Composite	1.48	0.2	0.2	0.8	0.5
STH 64-6th Rd E	0.97	7600	12	8** (10)	Composite	1.45	0.8	0.5	0.6	1.2
Total							12.0	11.3	17.6	24.6

USH 141 Intersections

Intersection	Intersection Type	2009 $AADT_{major}$ (veh/day)	2009 $AADT_{minor}$ (veh/day)	Intersection Skew Angle, Leg 1 / Leg 2	# Left / Right turn lanes	KABC	2009 EB KABC	PDO	2009 EB PDO
STH 22	4ST	10900	3500** (4700)	0/0	0/2	0.8	1.4	1.0	2.0
Olson Rd	4ST	10900	3500*	0/0	0/0	0.0	1.2	0.2	1.6
McCarthy Rd	3ST	10900	4300*	0/0	0/0	0.0	0.5	0.0	0.8
Lemere Rd	4ST	10900	3500*	0/0	0/0	0.0	1.3	0.4	1.6
Vernosh Rd	3ST	10900	4300*	0/0	0/0	0.0	0.5	0.0	0.8
Guelig Rd	3ST	10900	4300*	0/0	0/0	0.4	0.8	0.4	1.1
Main St/CTH A	4ST	9800	2200***	0/0	0/0	0.4	1.8	3.0	2.4
Goatsville Rd	3ST	9800	4300*	0/0	0/0	0.6	0.8	0.4	1.2
Starlite Rd	3ST	9800	4300*	0/0	0/0	0.8	0.9	0.4	1.3
Sunny Brook Rd	4ST	9800	3500*	0/0	0/0	0.4	1.3	0.2	1.7
CTH M	4ST	9000***	990	0/0	0/0	0.0	0.8	0.2	1.0
2nd Rd E	3ST	9000***	4300*	0/0	0/0	0.2	0.7	0.4	1.0
CTH M	3ST	8800	4300*	0/0	0/0	0.2	0.7	0.4	1.0
8th Rd W	3ST	8800	4300*	0/0	0/0	0.0	0.6	0.2	0.8
Main St/CTH B	4ST	7900	3000***	40/0	0/4	1.2	1.8	2.4	2.3
16th Rd E	4ST	7900	3500*	0/0	0/0	0.4	1.3	0.6	1.8
CTH Q	4ST	7600	1200***	0/0	0/0	0.6	1.2	2.0	1.7
STH 64	3ST	7600	4300*	0/0	0/2	0.4	0.7	0.6	1.0
STH 64	3ST	7600	4300*	0/0	1/1	0.8	0.6	0.4	0.8
6th Rd E	4ST	7600	3500*	0/0	0/0	0.2	1.2	0.4	1.6
Total						7.4	20.1	13.6	27.5

STH 57 Segments

Segment	Segment Length (mi)	2009 AADT (veh/day)	Lane Width (ft)	Shoulder Width (ft)	Shoulder type	Grade (%)	KABC	2009 EB KABC	PDO	2009 EB PDO
Before Stone Rd	0.97	10100	12	8** (10)	Composite	0.30	0.4	0.8	1.8	1.6
Stone Rd-Cloverleaf Rd	0.56	10100	12	8** (10)	Composite	0.30	0.6	0.4	0.4	0.9
Cloverleaf Rd-School Ln	0.65	10100	12	8** (10)	Composite	0.92	0.0	0.4	0.8	0.8
School Ln-CTH K	0.60	10100	12	8** (10)	Composite	0.92	0.4	0.4	0.4	0.9
CTH K-Stevenson Pier Rd	0.46	10100	12	8** (10)	Composite	0.92	0.2	0.3	0.4	0.7
Stevenson Pier Rd S-Short Cut Rd	0.40	10100	12	8** (10)	Composite	1.28	0.0	0.2	0.2	0.5
Short Cut Rd-Tornado Rd	0.28	10100	12	8** (10)	Composite	0.68	0.0	0.1	0.0	0.3
Tornado Rd-CTH XC	0.60	10100	12	8** (10)	Composite	0.52	0.2	0.4	0.2	0.8
CTH XC-Stub Rd	0.82	9100	12	8** (10)	Composite	0.66	0.2	0.4	0.0	0.9
Stub Rd-CTH H	0.17	9100	12	8** (10)	Composite	0.76	0.0	0.1	0.4	0.3
CTH H-Dump Rd	0.35	9100	12	8** (10)	Composite	2.06	0.0	0.2	0.2	0.5
Dump Rd-High Rd	0.50	9100	12	8** (10)	Composite	2.04	0.2	0.3	0.2	0.6
High Rd-Cemetery Rd (1)	0.27	9100	12	8** (10)	Composite	0.30	0.2	0.2	0.0	0.3
Cemetery Rd (1)-Cemetery Rd (2)	0.28	9100	12	8** (10)	Composite	0*	0.0	0.1	0.0	0.2
Cemetery Rd (2)-Dead End Rd	0.55	9100	12	8** (10)	Composite	0.55	0.0	0.2	0.0	0.6
Dead End Rd-Junction Rd	0.44	9100	12	8** (10)	Composite	0.46	0.2	0.2	0.0	0.5
Junction Rd-CTH C	0.18	9100	12	8** (10)	Composite	0.43	0.0	0.1	0.4	0.3
CTH C-School Rd	0.20	8800	12	8** (10)	Composite	0.49	0.6	0.2	0.2	0.5
School Rd-Brussels Rd	0.84	8800	12	8** (10)	Composite	0.35	0.2	0.5	0.6	1.0
Brussels Rd-Thru Way Rd	0.48	8800	12	8** (10)	Composite	0.32	0.4	0.2	0.0	0.5
Thru Way Rd-Pit Rd	0.48	8800	12	8** (10)	Composite	0.32	0.0	0.2	0.2	0.5
Pit Rd-CTH N	0.39	8800	12	8** (10)	Composite	0.32	0.2	0.3	0.4	0.5
CTH N-CTH N	0.12	8800	12	8** (10)	Composite	2.32	0.0	0.1	0.0	0.1
CTH N-Belgian Dr	0.40	8800	12	8** (10)	Composite	2.32	0.0	0.2	0.0	0.4
Belgian Dr-CTH Y	0.46	8800	12	8** (10)	Composite	2.32	0.2	0.2	0.2	0.6
CTH Y-Pleasant Ridge Rd	0.57	8200	12	8** (10)	Composite	1.09	0.0	0.3	0.0	0.5
Pleasant Ridge Rd-CTH D	0.46	8200	12	8** (10)	Composite	1.03	0.0	0.3	0.8	0.6
CTH D-Sand Hill Rd	0.10	8200	12	8** (10)	Composite	1.57	0.0	0.1	0.0	0.1
Sand Hill Rd-Cedar Rd	2.02	8200	12	8** (10)	Composite	0.30	0.4	1.2	2.0	2.5
Cedar Rd-CTH Y	0.32	8200	12	8** (10)	Composite	1.24	0.0	0.1	0.2	0.4
CTH Y-County Line Rd	0.88	8200	12	8** (10)	Composite	1.58	0.4	0.5	0.6	1.2
County Line Rd-CTH A	1.54	8200	12	8** (10)	Composite	0.30	0.4	0.8	0.6	1.7
CTH A-Macco Rd	0.95	9400	12	8** (10)	Composite	0*	0.6	0.5	0.2	1.1
Macco Rd-CTH S	0.32	9400	12	8** (10)	Composite	0*	0.2	0.2	0.4	0.4
CTH S-County Line Rd	0.09	10800	12	8** (10)	Composite	0*	0.0	0.1	0.0	0.1

County Line Rd-CTH P	1.19	10800	12	8** (10)	Composite	0*	0.2	0.8	0.8	1.6
CTH P-Gravel Pit Rd	1.03	10800	12	8** (10)	Composite	0*	0.8	0.8	1.4	1.8
Gravel Pit Rd-Stone Pillar Rd	1.13	10800	12	8** (10)	Composite	0*	0.2	0.7	0.8	1.6
Stone Pillar Rd-CTH T	0.17	10800	12	8** (10)	Composite	0*	0.4	0.2	0.8	0.5
CTH T-Bowers Rd	0.83	10800	12	8** (10)	Composite	0*	0.0	0.5	0.6	1.1
Bowers Rd-CTH A	0.40	10800	12	8** (10)	Composite	0*	0.2	0.3	0.0	0.4
CTH A-Wequiock	1.36	11300	12	8** (10)	Composite	0*	1.0	0.9	0.2	1.9
Wequiock Rd-CTH K/ Fischer Rd	1.11	11300	12	8** (10)	Composite	0*	0.6	0.8	0.6	1.7
Total							9.6	15.8	17.0	34.0

STH 57 Intersections

Intersection	Intersection Type	2009 $AADT_{major}$ (veh/day)	2009 $AADT_{minor}$ (veh/day)	Intersection Skew Angle, Leg 1 / Leg 2	# Left / Right turn lanes	KABC	2009 EB KABC	PDO	2009 EB PDO
Stone Rd	4ST	10100	3500*	10/0	0/2	0.6	1.5	1.2	1.9
Cloverleaf Rd	3ST	10100	4300*	0/0	0/1	0.0	0.5	0.0	0.8
School Ln	4ST	10100	3500*	30/0	0/2	0.2	1.3	0.6	1.7
CTH K	3ST	10100	4300*	0/0	1/1	0.2	0.5	0.0	0.7
Stevenson Pier Rd S	4ST	10100	3500*	45/0	0/2	0.4	1.4	0.4	1.8
Short Cut Rd	3ST	10100	4300*	0/0	0/1	0.0	0.5	0.0	0.8
Tornado Rd	3ST	10100	4300*	0/0	0/1	0.0	0.5	0.0	0.8
CTH XC	4ST	10100	270	33/0	0/2	0.0	0.5	0.2	0.5
Stub Rd	3ST	9100	4300*	0/0	0/1	0.2	0.6	0.0	0.8
CTH H	3ST	9100	4300*	0/0	0/1	0.2	0.6	0.0	0.8
Dump Rd	4ST	9100	3500*	0/0	0/2	0.0	0.9	0.0	1.3
High Rd	3ST	9100	4300*	0/0	0/0	0.0	0.6	0.0	0.8
Cemetery Rd (1)	3ST	9100	4300*	0/0	0/1	0.2	0.6	0.0	0.8
Cemetery Rd (2)	3ST	9100	4300*	0/0	0/1	0.0	0.6	0.2	0.8
Dead End Rd	3ST	9100	4300*	0/0	0/1	0.0	0.6	0.2	0.8
Junction Rd	3ST	9100	4300*	0/0	0/1	0.0	0.6	0.2	0.8
CTH C	4ST	9100	920	39/0	0/2	0.2	0.8	0.6	1.1
School Rd	3ST	8800	4300*	0/0	0/1	0.0	0.7	0.6	1.0
Brussels Rd	4ST	8800	3500*	0/0	0/2	0.0	1.0	0.0	1.2
Thru Way Rd	4ST	8800	3500*	0/0	0/2	0.0	1.0	0.2	1.3
Pit Rd	4ST	8800	3500*	0/0	0/2	0.0	1.0	0.2	1.3
CTH N	3ST	8800	4300*	0/0	0/1	0.2	0.7	0.4	1.0
CTH N	3ST	8800	4300*	0/0	0/1	0.0	0.6	0.2	0.8
Belgian Dr	3ST	8800	4300*	0/0	0/1	0.0	0.5	0.0	0.7
CTH Y	3ST	8800	4300*	0/0	0/1	0.0	0.5	0.0	0.7
Pleasant Ridge Rd	4ST	8200	3500*	27/0	0/2	0.0	1.1	0.4	1.5
CTH D	3ST	8200	220	0/0	0/1	0.2	0.2	0.0	0.4
Sand Hill Rd	4ST	8200	3500*	30/0	0/2	0.2	1.1	0.2	1.5
Cedar Rd	3ST	8200	4300*	0/0	0/1	0.4	0.7	0.2	1.0
CTH Y	3ST	8200	4300*	0/0	0/1	0.2	0.7	0.4	1.0
County Line Rd	3ST	8200	4300*	0/0	0/1	0.0	0.6	0.2	0.8
CTH A	4ST	8200	770	21/0	0/2	0.4	0.7	0.0	0.9
Macco Rd	3ST	9400	4300*	0/0	0/1	0.2	0.6	0.4	1.0
CTH S	4ST	9400	1000	0/0	0/2	0.2	0.7	0.4	1.0
County Line Rd	4ST	10800	3500*	34/0	0/2	0.0	1.2	0.2	1.5
CTH P	3ST	10800	1100***	0/0	1/1	0.4	0.5	0.6	0.6
Gravel Pit Rd	3ST	10800	4300*	0/0	0/1	0.4	0.8	0.6	1.1
Stone Pillar Rd	3ST	10800	4300*	0/0	0/1	0.4	0.7	0.2	1.0

CTH T	3ST	10800	450	0/0	1/1	0.6	0.4	1.0	0.6
Bowers Rd	3ST	10800	4300*	0/0	0/1	0.0	0.5	0.0	0.7
CTH A	3ST	10800	480	0/0	1/1	0.4	0.3	0.0	0.4
Wequiock Rd	4ST	11300	3500*	22/0	0/2	0.0	1.1	0.2	1.5
CTH K/Fischer Rd	3ST	10900	450	0/0	1/1	0.0	0.2	0.0	0.3
Total						6.4	30.7	10.2	41.8

STH 164 Segments

Segment	Segment Length (mi)	2006 AADT (veh/day)	2009 AADT (veh/day)	Lane Width (ft)	Shoulder Width (ft)	Shoulder type	Grade (%)	KABC	2006 EB KABC	2009 EB KABC	PDO	2006 EB PDO	2009 EB PDO
Prospect Ct-Clover Dr	0.79	13200	17800** (22900)	12	8	Composite	0.50	0.0	0.6	0.7	0.8	1.2	1.4
Clover Dr-N Corporate Cir	0.13	13200	17800** (19000)	12	8	Composite	0.50	0.0	0.1	0.1	0.0	0.2	0.2
N Corporate Cir-Richmond Rd	0.03	13200	17800** (19000)	12	8	Composite	0.35	0.0	0.0	0.0	0.0	0.1	0.1
Richmond Rd-S Corporate Cir	0.40	13200	17800** (19000)	12	8	Composite	3.20	0.8	0.5	0.5	0.8	1.0	1.3
S Corporate Cir-Lisbon Rd	0.19	13200	17800** (22300)	12	8	Composite	0.50	0.0	0.2	0.2	0.4	0.3	0.4
Lisbon Rd-Seven Stones Dr	0.43	11800	15200	12	8	Composite	4.54	0.2	0.5	0.5	1.2	1.0	1.1
Seven Stones Dr-Lindsay Rd	0.39	11800	15200	12	8	Composite	2.09	1.2	0.4	0.5	0.4	0.9	1.0
Lindsay Rd-Lake Park Dr	0.44	11800	15200	12	8	Composite	1.18	0.2	0.3	0.4	0.4	0.7	0.8
Lake Park Dr-Chesterwood Ln	0.23	11800	15200	12	8	Composite	0.50	0.0	0.1	0.2	0.0	0.3	0.3
Chesterwood Ln-Swan Rd	0.79	13300	15900	12	8	Composite	0.51	0.2	0.7	0.7	1.0	1.3	1.5
Total								2.6	3.4	3.8	5.0	7.0	8.1

STH 164 Intersections

Intersection	Intersection Type	2006 $AADT_{major}$ (veh/day)	2009 $AADT_{major}$ (veh/day)	2006 $AADT_{minor}$ (veh/day)	2009 $AADT_{minor}$ (veh/day)	Intersection Skew Angle, Leg 1 / Leg 2	# Left / Right turn lanes	KABC	2006 EB KABC	2009 EB KABC	PDO	2006 EB PDO	2009 EB PDO
Prospect Ct	3ST	13200	19500** (22900)	4300*	4300*	0/0	0/1	0.2	0.7	0.8	0.4	1.1	1.1
Clover Dr	3ST	13200	19500** (22900)	4300*	4300*	0/0	0/1	0.2	0.7	0.8	0.2	0.9	1.1
N Corporate Cir	3ST	13200	19000	4300*	4300*	0/0	0/1	0.2	0.7	0.7	0.2	0.9	1.1
Richmond Rd	3ST	13200	19000	4300*	4300*	0/0	0/1	0.0	0.6	0.7	0.2	0.9	0.9
S Corporate Cir	3ST	13200	19000	4300*	4300*	0/0	0/1	0.4	0.7	0.7	0.0	0.9	1.1
Lisbon Rd	4SG	13200	22300	8400	8700	0/0	4/4	4.4	1.6	2.1	3.6	3.3	4.0
Seven Stones Dr	3ST	11800	15200	4300*	4300*	0/0	0/0	0.0	0.6	0.6	0.0	0.8	0.8
Lindsay Rd	4ST	11800	14700** (15200)	3500*	3500*	13/0	0/2	0.0	1.1	1.1	0.0	1.4	1.5
Lake Park Dr	4ST	11800	14700** (15200)	3500*	3500*	0/0	2/2	0.0	0.7	0.8	0.0	1.0	1.1
Chesterwood Ln	3ST	11800	15200	4300*	4300*	0/0	0/1	0.0	0.5	0.6	0.0	0.8	0.8
Swan Rd	4SG	13300	15900	12500*	12500*	10/0	2/2	1.4	1.5	1.6	0.4	3.0	3.2
Total								6.8	9.4	10.5	5.0	15.0	16.7

STH 83 Segments

Segment	Segment Length (mi)	2012 AADT (veh/day)	Lane Width (ft)	Shoulder Width (ft)	Shoulder type	Grade (%)	KABC	2012 EB KABC	PDO	2012 EB PDO
Before Frog Alley Rd	1.56	10400***	12	6* (2-10)	Paved	0.75	4.8	2.5	8.2	5.3
Frog Alley Rd-Crossgate Dr	0.21	9800***	12	6* (2-10)	Paved	1.05	0.0	0.1	0.0	0.2
Crossgate Dr-CTH I	1.27	9800***	12	6* (2-10)	Paved	0.74	1.2	1.2	2.6	2.4
CTH I-Road X	0.51	10800***	12	6* (2-10)	Paved	0.62	0.0	0.4	0.6	0.7
Road X-CTH X	0.29	10000***	12	6* (2-10)	Paved	2.30	1.8	0.6	1.4	1.1
CTH X-Mc Farlane	0.31	7400***	12	6* (2-10)	Paved	2.78	0.2	0.2	0.4	0.4
Mc Farlane Rd-Holiday Rd	0.56	7400***	12	6* (2-10)	Paved	3.58	0.6	0.4	0.4	0.7
Holiday Rd-Old Village Rd	0.66	7400***	12	6* (2-10)	Paved	2.31	0.2	0.3	0.2	0.7
Old Village Rd-STH 59	0.24	7400***	12	6* (2-10)	Paved	0.57	0.4	0.2	0.0	0.2
Total							9.2	5.9	13.8	11.7

STH 83 Intersections

Intersection	Intersection Type	2012 $AADT_{major}$ (veh/day)	2012 $AADT_{minor}$ (veh/day)	Intersection Skew Angle, Leg 1 / Leg 2	# Left / Right turn lanes	KABC	2012 EB KABC	PDO	2012 EB PDO
Frog Alley Rd	4ST	9800***	1100***	10/0	0/2	1.6	1.2	1.6	1.6
Crossgate Dr	3ST	9800***	4300*	0/0	1/1	0.6	0.5	0.0	0.8
CTH I	4ST	10800***	1800***	0/0	0/0	1.0	1.4	1.0	1.9
Road X	3ST	10000***	1300***	0/0	0/0	0.4	0.6	0.2	0.8
CTH X	3ST	7400***	4100***	0/0	1/1	0.2	0.4	0.0	0.6
Mc Farlane Rd	3ST	7400***	4300*	0/0	0/1	0.2	0.5	0.0	0.8
Holiday Rd	3ST	7400***	4300*	0/0	0/1	0.0	0.6	0.4	0.8
Old Village Rd	4ST	7400***	3500*	10/20	0/0	0.2	1.5	1.6	2.1
STH 59	4ST	7400***	3500**** (10200)	0/0	0/4	3.2	2.3	2.6	3.0
Total						7.4	9.0	7.4	12.4

STH 26 Segments

Segment	Segment Length (mi)	2012 AADT (veh/day)	Lane Width (ft)	Shoulder Width (ft)	Shoulder type	Grade (%)	KABC	2012 EB KABC	PDO	2012 EB PDO
Whitetail Ln-STH 26 NB Ramp	0.37	12700***	12	8** (11 Right / 10 Left)	Paved	0.54	0.2	0.3	0.4	0.6
STH 26 NB Ramp-STH 26 SB Ramp	0.63	8100***	12	8** (11 Right / 10 Left)	Paved	0.98	1.0	0.6	1.8	1.2
STH 26 SB Ramp-STH 106 WB Ramp	1.36	8100***	12	8** (11 Right / 10 Left)	Paved	1.04	0.8	0.9	0.8	1.9
STH 106 WB Ramp-STH 106 EB Ramp	0.53	7900***	12	8** (11 Right / 10 Left)	Paved	2.32	0.2	0.4	1.2	0.8
STH 106 EB Ramp-USH 12 WB Ramp	1.42	7900***	12	8** (11 Right / 10 Left)	Paved	1.00	1.0	0.7	0.0	1.5
USH 12 WB Ramp-USH 12 EB Ramp	0.54	7000***	12	8** (11 Right / 10 Left)	Paved	3.13	0.6	0.5	2.2	1.1
USH 12 EB Ramp-STH 89	1.27	7000***	12	8** (11 Right / 10 Left)	Paved	0.53	1.4	0.7	1.2	1.6
Total							5.2	4.1	7.6	8.7

STH 26 Intersections

Intersection	Intersection Type	2012 $AADT_{major}$ (veh/day)	2012 $AADT_{minor}$ (veh/day)	Intersection Skew Angle, Leg 1 / Leg 2	# Left / Right turn lanes	KABC	2012 EB KABC	PDO	2012 EB PDO
Whitetail Ln	4ST	12700***	3500*	0/0	0/0	0.0	1.2	0.0	1.5
STH 26 NB Ramp	3ST	8100***	1300***	0/0	0/1	0.0	0.4	0.0	0.6
STH 26 SB Ramp	3ST	8100***	2800***	0/0	0/1	0.0	0.5	0.2	0.7
STH 106 WB Ramp	3ST	7900***	630***	0/0	0/1	0.0	0.3	0.0	0.4
STH 106 EB Ramp	3ST	7900***	740***	0/0	0/1	0.0	0.4	0.2	0.5
USH 12 WB Ramp	3ST	7000***	1400***	0/0	0/1	0.0	0.5	0.4	0.6
USH 12 EB Ramp	3ST	7000***	1700***	0/0	0/1	0.0	0.4	0.0	0.5
Total						0.0	3.7	0.8	4.8

USH 12(77) Segments

Segment	Segment Length (mi)	2006 AADT (veh/day)	2009 AADT (veh/day)	Lane Width (ft)	Shoulder Width (ft)	Shoulder type	Grade (%)	KABC	2006 EB KABC	2009 EB KABC	PDO	2006 EB PDO	2009 EB PDO
Before STH 78	0.16	17700	16700	12*	6*	Paved*	3.00	1.0	0.4	0.4	1.0	0.9	1.0
STH 78-STH 188	0.29	13000	16700	12*	6*	Paved*	3.00	1.0	0.3	0.4	0.2	0.8	0.8
STH 188-CTH Y	1.06	13700	14800	12*	6*	Paved*	0.57	0.6	1.0	1.1	2.0	2.2	2.3
CTH Y-Dunlap Hollow Rd N	0.14	12500	15800	12*	6*	Paved*	2.38	0.0	0.1	0.1	0.0	0.2	0.2
Dunlap Hollow Rd N-Herbrand Rd	1.62	12500	15800	12*	6*	Paved*	0.67	1.8	1.6	1.8	2.8	3.5	3.9
After Herbrand Rd	1.20	12500	15800	12*	6*	Paved*	1.07	2.0	1.4	1.5	2.4	2.8	3.1
Total								6.4	4.8	5.3	8.4	10.4	11.3

USH 12(77) Intersections

Intersection	Intersection Type	2006 $AADT_{major}$ (veh/day)	2009 $AADT_{major}$ (veh/day)	2006 $AADT_{minor}$ (veh/day)	2009 $AADT_{minor}$ (veh/day)	Intersection Skew Angle, Leg 1 / Leg 2	# Left / Right turn lanes	KABC	2006 EB KABC	2009 EB KABC	PDO	2006 EB PDO	2009 EB PDO
STH 78	3ST	17700	16700	4300**** (5300)	4300** (5300)	0/0	1/1	3.0	1.9	1.9	2.0	2.6	2.5
STH 188	4ST	13000	14700** (16700)	2000***	2400	0/0	0/0	2.4	2.0	2.1	1.4	2.6	2.9
CTH Y	4ST	13700	14700** (14800)	1400***	990	0/0	0/2	0.6	1.1	1.0	0.2	1.5	1.3
Dunlap Hollow Rd N	3ST	12500	15800	4300*	4300*	0/0	0/0	0.2	0.8	0.8	0.4	1.1	1.1
Herbrand Rd	3ST	12500	15800	4300*	4300*	0/0	0/0	0.0	0.6	0.6	0.0	0.8	0.8
Total								6.2	6.4	6.4	4.0	8.6	8.6

USH 12(71) Segments

Segment	Segment Length (mi)	2005 AADT (veh/day)	2006 AADT (veh/day)	Lane Width (ft)	Shoulder Width (ft)	Shoulder type	Grade (%)	KABC	2005 EB KABC	2006 EB KABC	PDO	2005 EB PDO	2006 EB PDO
CTH KP-Breunig Rd	1.02	11900	12800	12*	6*	Paved*	1.40	0.6	0.9	0.9	1.8	2.0	2.1
Breunig Rd-Ballweg Rd	1.01	11900	12800	12*	6*	Paved*	2.34	1.4	0.9	0.9	1.0	2.0	2.0
Ballweg Rd-Matz Rd (2)	0.50	11900	11500	12*	6*	Paved*	4.03	0.8	0.9	0.7	2.8	1.7	1.6
Matz Rd (2)-Simpson Rd	0.91	11900	11500	12*	6*	Paved*	2.63	1.2	1.0	1.0	2.6	2.3	2.2
Simpson Rd-Rauls Rd	0.33	11900	11500	12*	6*	Paved*	2.63	0.2	0.3	0.4	1.2	0.8	0.8
Rauls Rd-STH 19	0.98	11900	11500	12*	6*	Paved*	3.98	0.8	0.7	0.7	0.0	1.5	1.5
Total								5.0	4.7	4.6	9.4	10.3	10.2

USH 12(71) Intersections

Intersection	Intersection Type	2005 $AADT_{major}$ (veh/day)	2006 $AADT_{major}$ (veh/day)	2005 $AADT_{minor}$ (veh/day)	2006 $AADT_{minor}$ (veh/day)	Intersection Skew Angle, Leg 1 / Leg 2	# Left / Right turn lanes	KABC	2005 EB KABC	2006 EB KABC	PDO	2005 EB PDO	2006 EB PDO
CTH KP	4ST	11900	12800	3500*	3500*	0/0	0/0	0.8	1.5	1.6	0.6	2.0	2.1
Breunig Rd	4ST	11900	12800	3500*	3500*	0/5	0/0	0.4	1.3	1.4	0.2	1.8	1.8
Ballweg Rd	3ST	11900	12800	4300*	4300*	0/0	1/1	0.2	0.6	0.6	0.4	0.8	0.8
Matz Rd (2)	3ST	11900	11500	4300*	4300*	0/0	0/1	1.4	1.7	1.7	2.6	2.4	2.4
Simpson Rd	3ST	11900	11500	4300*	4300*	0/0	0/0	0.2	0.7	0.7	0.2	1.0	1.0
Rauls Rd	3ST	11900	11500	4300*	4300*	0/0	0/0	0.2	0.9	0.9	0.8	1.3	1.2
STH 19	3ST	11900	11500	1500	1600***	0/0	1/1	0.0	0.3	0.3	0.0	0.5	0.5
Total								3.2	7.0	7.2	4.8	9.8	9.8

USH 12(74) Segments

Segment	Segment Length (mi)	2006 AADT (veh/day)	2009 AADT (veh/day)	Lane Width (ft)	Shoulder Width (ft)	Shoulder type	Grade (%)	KABC	2006 EB KABC	2009 EB KABC	PDO	2006 EB PDO	2009 EB PDO
STH 19-CTH P	0.86	13200	17800** (21300)	12*	6*	Paved*	0.49	0.4	0.7	0.7	0.4	1.4	1.6
CTH P-Baltes Rd	0.66	17200	17800** (20600)	12*	6*	Paved*	0.45	1.2	1.0	1.0	1.6	2.0	2.0
Baltes Rd-Kick-A-Boo Rd	1.43	15000	17800** (20600)	12*	6*	Paved*	1.53	1.2	1.6	1.8	3.4	3.6	3.8
Kick-A-Boo Rd-Meffert Rd	0.75	15000	17800** (20600)	12*	6*	Paved*	1.00	1.0	0.9	0.9	1.2	1.7	1.9
Meffert Rd-Riles Rd	0.39	15000	17800** (20600)	12*	6*	Paved*	1.37	0.4	0.5	0.5	1.0	1.0	1.1
Riles Rd-Fisher Rd	0.29	15000	17800** (20600)	12*	6*	Paved*	1.88	0.2	0.2	0.2	0.0	0.5	0.5
Fisher Rd-CTH K	0.81	15000	17800** (20600)	12*	6*	Paved*	1.55	0.6	0.8	0.9	1.4	1.8	2.0
After CTH K	0.87	15000	17800** (20600)	12*	6*	Paved*	2.55	1.6	1.3	1.4	3.0	2.7	3.0
Total								6.6	7.0	7.4	12.0	14.7	15.9

USH 12(74) Intersections

Intersection	Intersection Type	2006 $AADT_{major}$ (veh/day)	2009 $AADT_{major}$ (veh/day)	2006 $AADT_{minor}$ (veh/day)	2009 $AADT_{minor}$ (veh/day)	Intersection Skew Angle, Leg 1 / Leg 2	# Left / Right turn lanes	KABC	2006 EB KABC	2009 EB KABC	PDO	2006 EB PDO	2009 EB PDO
STH 19	3ST	13200	19500** (21300)	1600***	1600	0/0	1/1	0.4	0.5	0.6	0.2	0.7	0.8
CTH P	4ST	13200	14700** (21300)	3500*	3500*	0/0	0/1	0.8	1.9	2.0	2.0	2.5	2.5
Baltes Rd	3ST	17200	19500** (20600)	4300** (6500)	4300**** (7200)	0/0	0/1	1.0	1.1	1.1	0.6	1.5	1.6
Kick-A-Boo Rd	4ST	14700** (15000)	14700** (20600)	3500*	3500*	0/0	0/2	0.0	1.1	1.1	0.0	1.5	1.5
Meffert Rd	3ST	15000	19500** (20600)	4300*	4300*	0/0	0/0	1.0	1.2	1.2	0.8	1.6	1.7
Riles Rd	3ST	15000	19500** (20600)	4300*	4300*	0/0	0/0	0.2	0.7	0.7	0.2	1.0	1.1
Fisher Rd	3ST	15000	19500** (20600)	4300*	4300*	0/0	0/0	0.6	0.8	0.9	0.2	1.2	1.2
CTH K	4SG	15000	20600	12500*	12500*	0/0	2/4	0.6	1.9	2.0	3.2	3.5	3.9
Total								4.6	9.2	9.6	7.2	13.5	14.3

USH 12(76) Segments

Segment	Segment Length (mi)	2006 AADT (veh/day)	2009 AADT (veh/day)	Lane Width (ft)	Shoulder Width (ft)	Shoulder type	Grade (%)	KABC	2006 EB KABC	2009 EB KABC	PDO	2006 EB PDO	2009 EB PDO
CTH K-Greenbriar Rd	1.21	17800** (24100)	17800** (24200)	12*	6*	Paved*	0.65	2.6	2.0	2.0	3.8	4.1	4.1
Greenbriar Rd-Schneider Rd	0.20	17800** (24100)	17800** (24200)	12*	6*	Paved*	0.52	0.6	0.4	0.4	1.0	0.9	0.9
Schneider Rd-Graber Rd	0.28	17800** (24100)	17800** (24200)	12*	6*	Paved*	1.94	1.0	0.5	0.5	0.8	1.1	1.1
Total								4.2	2.9	2.9	5.6	6.1	6.1

USH 12(76) Intersections

Intersection	Intersection Type	2006 $AADT_{major}$ (veh/day)	2009 $AADT_{major}$ (veh/day)	2006 $AADT_{minor}$ (veh/day)	2009 $AADT_{minor}$ (veh/day)	Intersection Skew Angle, Leg 1 / Leg 2	# Left / Right turn lanes	KABC	2006 EB KABC	2009 EB KABC	PDO	2006 EB PDO	2009 EB PDO
CTH K	4SG	24100	24200	12500*	12500*	0/0	2/4	0.2	1.6	1.5	0.0	3.0	3.0
Greenbriar Rd	3ST	19500** (24100)	19500** (24200)	4300*	4300*	0/0	0/0	0.2	0.8	0.8	0.4	1.2	1.2
Schneider Rd	3ST	19500** (24100)	19500** (24200)	4300*	4300*	0/0	1/1	0.2	0.8	0.8	0.8	1.1	1.1
Graber Rd	3ST	19500** (24100)	19500** (24200)	4300*	4300*	0/0	0/1	0.2	1.1	1.1	1.2	1.5	1.5
Total								0.8	4.3	4.2	2.4	6.8	6.8

USH 14 Segments

Segment	Segment Length (mi)	2012 AADT (veh/day)	Lane Width (ft)	Shoulder Width (ft)	Shoulder type	Grade (%)	KABC	2012 EB KABC	PDO	2012 EB PDO
Airport Ln-Springville Rd	0.70	9700***	12	8** (10)	Composite	2.50	0.4	0.5	0.2	0.9
Springville Rd-CTH Y	0.80	9700***	12	8** (10)	Composite	1.35	0.4	0.5	1.2	1.5
CTH Y-Three Chimney Rd	1.01	8900***	12	8** (10)	Composite	1.00	1.2	0.7	1.2	1.6
Three Chimney Rd-Smith Rd	1.11	8900***	12	8** (10)	Composite	0.73	1.0	0.9	2.6	2.0
Smith Rd-Sherpe Rd	0.72	8900***	12	8** (10)	Composite	0.42	0.0	0.4	0.8	0.9
Sherpe Rd-Tri State Rd	0.34	8900***	12	8** (10)	Composite	1.73	0.2	0.2	0.6	0.5
Tri State Rd-Locust St	0.34	9000***	12	8** (10)	Composite	3.06	0.0	0.2	0.4	0.5
Total							3.2	3.4	7.0	7.9

USH 14 Intersections

Intersection	Intersection Type	2012 $AADT_{major}$ (veh/day)	2012 $AADT_{minor}$ (veh/day)	Intersection Skew Angle, Leg 1 / Leg 2	# Left / Right turn lanes	KABC	2012 EB KABC	PDO	2012 EB PDO
Airport Ln	4SG	9700***	3700***	0/0	4/2	0.0	0.7	0.0	1.4
Springville Rd	3ST	9700***	4300*	0/0	1/1	0.0	0.4	0.0	0.6
CTH Y	4ST	8900***	1100***	0/0	0/2	0.8	0.9	0.8	1.2
Three Chimney Rd	4ST	8900***	3500*	25/0	0/2	0.0	1.1	0.2	1.4
Smith Rd	4ST	8900***	3500*	25/0	0/2	0.2	1.1	0.2	1.5
Sherpe Rd	4ST	8900***	3500*	0/0	0/2	0.0	1.0	0.0	1.2
Tri State Rd	4ST	9000***	460***	25/0	0/2	0.0	0.6	0.4	0.7
Locust St	4ST	9000***	3500*	0/0	0/2	0.0	0.9	0.0	1.3
Total						1.0	6.7	1.6	9.3

STH 21 Segments

Segment	Segment Length (mi)	2006 AADT (veh/day)	2008 AADT (veh/day)	Lane Width (ft)	Shoulder Width (ft)	Shoulder type	Grade (%)	KABC	2006 EB KABC	2008 EB KABC	PDO	2006 EB PDO	2008 EB PDO
Emerson Rd-Buan St	0.26	4900	4700	12	7	Paved	0.15	0.0	0.1	0.1	0.0	0.2	0.1
Buan St-USH 12	0.23	4900	4700	12	6* (3-10)	Paved	0.75	1.0	0.2	0.2	1.0	0.5	0.5
USH 12-Wittig Rd	0.16	10600	12200	12	6* (3-10)	Paved	0.79	0.0	0.1	0.2	0.2	0.2	0.3
Wittig Rd-94E Ramp	0.14	10600	12200	12	6* (3-10)	Paved	2.32	0.0	0.1	0.1	0.0	0.2	0.2
94E Ramp-94W Ramp	0.19	10600	12200	12	6* (3-10)	Paved	2.32	1.0	0.6	0.5	2.0	1.2	1.1
94W Ramp-Ensign	1.19	6600	6600	12	8** (10)	Paved	2.15	0.8	0.5	0.5	0.4	1.2	1.2
Total								2.8	1.6	1.6	3.6	3.5	3.4

STH 21 Intersections

Intersection	Intersection Type	2006 $AADT_{major}$ (veh/day)	2008 $AADT_{major}$ (veh/day)	2006 $AADT_{minor}$ (veh/day)	2008 $AADT_{minor}$ (veh/day)	Intersection Skew Angle, Leg 1 / Leg 2	# Left / Right turn lanes	KABC	2006 EB KABC	2008 EB KABC	PDO	2006 EB PDO	2008 EB PDO
Emerson Rd	4ST	4900	4700	3500*	3500*	0/0	0/0	0.6	1.2	1.2	0.6	1.6	1.6
Buan St	4ST	4900	4700	3500*	3500*	0/0	0/0	0.0	1.0	0.9	0.0	1.2	1.2
USH 12	4SG	4900	4700	8900	9400	0/0	4/4	6.2	1.6	1.5	9.2	2.9	2.9
Wittig Rd	4ST	10600	12200	3500*	3500*	0/0	2/0	0.8	1.6	1.7	3.0	2.1	2.2
94E Ramp	3ST	10600	12200	4000	3600	0/0	1/1	0.2	0.5	0.6	0.4	0.8	0.8
94W Ramp	4ST	10600	12200	3500** (4700)	3500** (4600)	0/0	1/2	0.6	1.1	1.2	0.6	1.5	1.5
Total								8.4	7.0	7.1	13.8	10.1	10.2

USH 10 Segments

Segment	Segment Length (mi)	2004 AADT (veh/day)	2006 AADT (veh/day)	Lane Width (ft)	Shoulder Width (ft)	Shoulder type	Grade (%)	KABC	2004 EB KABC	2006 EB KABC	PDO	2004 EB PDO	2006 EB PDO
CTH B/East St-Rose Ln	0.26	8100	7100	12	8	Composite	3.02	0.0	0.2	0.2	0.2	0.3	0.3
Rose Ln-Cox Ln	0.04	8100	7100	12	8	Composite	1.10	0.0	0.0	0.0	0.0	0.1	0.0
Cox Ln-Radcliff St	0.05	11000	8400	12	8	Composite	1.10	0.0	0.0	0.0	0.0	0.1	0.1
Radcliff St-IH 94 EB Ramp	0.14	11000	8400	12	8	Composite	2.65	0.0	0.1	0.1	0.0	0.2	0.1
IH 94 EB Ramp-IH 94 WB Ramp	0.15	11000	8400	12	8	Composite	0.30	1.4	0.6	0.6	3.6	1.7	1.3
IH 94 WB Ramp-Oak Grove St	0.13	14300	12700	12	8	Composite	0.30	0.0	0.0	0.1	0.0	0.1	0.2
Oak Grove St-Hilltop Rd/Tru-Gas Rd	0.07	14300	12700	12	8	Composite	0.50	0.0	0.1	0.1	0.0	0.1	0.1
Hilltop Rd/Tru-Gas Rd-Industrial Rd	0.19	14300	12700	12	8	Composite	0.75	0.0	0.1	0.1	0.0	0.3	0.2
After Industrial Rd	0.46	14300	12700	12	8	Composite	0.34	0.0	0.3	0.3	0.4	0.7	0.7
Total								1.4	1.4	1.5	4.2	3.6	3.0

USH 10 Intersections

Intersection	Intersection Type	2004 $AADT_{major}$ (veh/day)	2006 $AADT_{major}$ (veh/day)	2004 $AADT_{minor}$ (veh/day)	2006 $AADT_{minor}$ (veh/day)	Intersection Skew Angle, Leg 1 / Leg 2	# Left / Right turn lanes	KABC	2004 EB KABC	2006 EB KABC	PDO	2004 EB PDO	2006 EB PDO
CTH B/East St	4ST	8100	7100	960	1100***	0/0	0/0	0.2	1.0	1.0	1.4	1.3	1.3
Rose Ln	4ST	8100	7100	3500*	3500*	0/0	0/0	0.4	1.4	1.3	0.6	1.8	1.7
Cox Ln	3ST	11000	8400	4300*	4300*	0/0	0/0	0.2	0.7	0.6	0.2	0.9	0.9
Radcliff St	4ST	11000	8400	3500*	3500*	0/0	0/0	0.6	1.7	1.6	1.4	2.2	2.1
IH 94 EB Ramp	4SG	11000	8400	3900	3800	0/0	1/1	0.0	1.2	1.1	0.0	2.3	2.1
IH 94 WB Ramp	4SG	11000	8400	2100	2500	0/0	1/1	0.0	1.1	1.0	0.2	2.2	2.1
Oak Grove St	3ST	14300	12700	4300*	4300*	0/0	0/0	0.6	1.5	1.5	2.4	2.2	2.1
Hilltop Rd/Tru-Gas Rd	4ST	14300	12700	3500*	3500*	0/0	0/0	0.4	1.6	1.6	1.0	2.1	2.1
Industrial Rd	4ST	14300	12700	3500*	3500*	0/0	0/0	0.2	1.2	1.3	0.0	1.6	1.6
Total								2.6	11.4	11	7.2	16.6	16.0