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Crash Modification Factors for Super 2 Highways

Technical Report 0-7183-R1

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16. Abstract

Super 2 highways have been used across Texas for over 20 years, providing operational and safety benefits to rural two-lane highways at a lower cost than widening such facilities to four lanes. More Super 2 highways are planned as the demand increases on the state highway system. Previous research has provided insights on Super 2 safety, but updated crash modification factors (CMFs) based on a rigorous analysis of recent crash data would provide additional support for installing Super 2 corridors throughout the state and complement existing guidance. This project investigated the crash history of existing Super 2 highway corridors in Texas before and after installation, compared to similar two-lane highways without Super 2 passing lanes. Researchers developed a comprehensive dataset of 67 Super 2 corridors for analysis and identification of relationships between crashes and other characteristics. Researchers also developed a database of 165 reference segments (i.e., two-lane highway segments without passing lanes) for crash history comparisons with Super 2 corridors. An empirical Bayes before-after analysis of available crash data revealed that Super 2 highways reduced total crashes by 21 percent compared to traditional rural two-lane highways. This result was statistically significant, with a 95 percent confidence interval between 16 and 26 percent for total crashes. Similar results were observed for fatal-injury (KABC) and property damage only (PDO) crashes; Super 2 highways reduced KABC and PDO crashes by 16 and 23 percent, respectively. This research provides guidance on the selection and use of appropriate CMFs for construction projects that include Super 2 passing lanes, along with suggestions for updates to the TxDOT *Roadway Design Manual* and recommendations for future research.

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DISCLAIMER

This research was sponsored by the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA). The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of FHWA or TxDOT. This report does not constitute a standard, specification, or regulation.

This report is not intended for construction, bidding, or permit purposes. The engineer in charge of the project was Marcus A. Brewer, P.E. #92997.

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VALUE OF RESEARCH ASSESSMENT

The research team completed a value of research (VoR) assessment as part of the project. The VoR assessment was based on the benefit areas selected at the beginning of the project (Table 1).

Table 1. Selected Benefit Areas for VoR Assessment.

Selection	Benefit Area	Qualitative	Economic	Both	TxDOT	State	Both	Definition in Context to the Project Statement
X	Traffic and congestion reduction		X			X		Improved capacity, reduced delay and percent time following, and higher average speeds for through traffic on rural two-lane highways.
X	Engineering design improvement			X			X	Provision for passing on rural two-lane highways; left-turn accommodation on rural two-lane highways, particularly in the vicinity of Super 2 passing lanes; and length and spacing of passing lanes appropriate for conditions on each highway.
X	Safety			X			X	Reduction in crashes and associated injuries and fatalities associated with the improved passing and turning accommodation.

The VoR assessment was based on the assumption that one existing Super 2 corridor will be extended, or one new Super 2 corridor will be built, annually based on this research project's findings. Additional assumptions included the following:

- Average length of the corridor = 10 mi.
- Average annual daily traffic in the corridor = 9,000 vpd.
- Percent heavy vehicles in the corridor = 20 percent.
- Number of passing lane sections in each direction of travel = 2 passing lane sections.
- Length of each passing lane in the corridor = 3 mi.
- Previous cross-section of the corridor = 2-lane undivided.

These assumptions typically represent the lower to middle part of the range of values studied in the research. Increasing any of the assumption values would increase the VoR. The benefit-cost analysis spreadsheet tool developed in the Texas Department of Transportation (TxDOT)

Research Project 0-6997 (*1*) calculated monetary values for each of the following of the required variables:

- Variable 1: Vehicle operating cost savings.
- Variable 2: Business and personal time cost savings.
- Variable 3: Safety benefits.
- Variable 4: Environmental benefits.
- Variable 5: Capital costs.

Table 2 lists the assigned variable values for each applicable economic benefit area in the VoR assessment for one construction project that builds or extends a Super 2 corridor with the assumed characteristics. The total value represents the Net Present Value (NPV) for a single construction project over its 20-year expected life.

Table 2. Value of Variables for VoR Assessment of one Super 2 Construction Project.

Economic Benefit Area	Variable 1	Variable 2	Variable 3	Variable 4	Variable 5	Total
Traffic and congestion reduction	$45,900,000	$0	$0	$400,000	$0	$46,300,000
Engineering design improvement	$0	$77,500,000	$0	$0	–$9,700,000	$67,800,000
Safety	$0	$0	$47,000,000	$0	$0	$47,000,000
					Total	$161,100,000

The research team used the total value from Table 2 to calculate an annual NPV for the sample construction project, estimated to be $8,055,000. Based on that value, researchers then calculated the NPV for each year in the 20-year period covered by the VoR assessment; using a 3 percent discount rate and adding another construction project each year of the assessment period per the previously stated assumptions, the research team calculated an overall annual NPV of $81,909,806.30 for the final VoR calculation for this research project. The research team entered that value into the TxDOT VoR assessment spreadsheet to calculate the formal VoR measures. Figure 1 shows a screenshot of these results. Based on the assumptions provided previously, this research project was found to have an estimated benefit-cost ratio of approximately 2,883:1 over a 20-year expected value duration, with over $736 million in savings.

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1. Brewer, M. A., T. D. Barrette, D. Florence, B. Glover, K. Fitzpatrick, and S. P. Venglar. *Capacity and Cost Benefits of Super 2 Corridors*. Publication FHWA/TX-21/0-6997-R1. Texas A&M Transportation Institute, College Station, 2021.

	Project #	0-7183		
	Project Name:	Develop Crash Modification Factors for Super 2 Highways		
	Agency:	TTI	**Project Budget**	$ 236,030
	Project Duration (Yrs)	1.3	**Exp. Value (per Yr)**	$ 81,909,806
Expected Value Duration (Yrs)		20	**Discount Rate**	3%
Economic Value				
Total Savings:	$ 736,952,227	**Net Present Value (NPV):**		$ 680,443,646
Payback Period (Yrs):	0.002882	**Cost Benefit Ratio (CBR, $1 : $___):**		$ 2,883

Figure 1. Results of VoR Assessment for Project 0-7183.

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CHAPTER 1: INTRODUCTION

BACKGROUND

Super 2 highways have been used across Texas for over 20 years to provide operational and safety benefits to rural two-lane highways at a lower cost than widening such facilities to four lanes. More Super 2 highways are planned as the demand increases on the state highway system. Previous Super 2 research has provided insights on safety improvements, but the Texas Department of Transportation (TxDOT) would benefit from updated crash modification factors (CMFs) based on a rigorous review and analysis of recent crash data from the state's many Super 2 highways. These updated CMFs would provide additional support for installing Super 2 corridors throughout the state and complement existing guidance.

PURPOSE OF THE PROJECT

This project investigated the crash history of existing Super 2 highway corridors in Texas before and after installation, compared to similar two-lane highways without Super 2 passing lanes. In this project, the research team used their existing database of Super 2 highways and site characteristics, combined with additional data from TxDOT databases on traffic volumes, crashes, and other relevant factors, to develop a comprehensive Super 2 dataset for analysis and identification of relationships between crashes and other characteristics. Researchers also developed a database of reference segments (i.e., two-lane highway segments without passing lanes) for crash history comparisons with Super 2 corridors. Using the results of this analysis, the research team defined CMFs that TxDOT and other practitioners can use to make decisions regarding the installation of future Super 2 corridors.

ORGANIZATION OF THIS REPORT

This report consists of seven chapters. In addition to this introductory chapter, the report contains the following information:

- Chapter 2 summarizes findings from a review of existing guidance and research on Super 2 highways.
- Chapter 3 describes the process of identifying and selecting field study sites, as well as the data collection at those sites.
- Chapter 4 describes the steps and considerations when developing the model used to analyze the data.
- Chapter 5 summarizes the results from the modeling and analysis.
- Chapter 6 lists the CMFs produced in the analysis, along with guidance on their use in the selection of alternatives and suggested updates to relevant existing guidance documents.

- Chapter 7 provides the findings and conclusions identified and developed as part of this project, as well as suggestions for implementation and recommendations for future research.

CHAPTER 2: REVIEW OF EXISTING INFORMATION

BACKGROUND

Two-lane, two-way highways with passing lanes, often referred to as *Super 2 highways* in Texas and *2+1 highways* in other regions of the United States, serve as a practical solution for addressing congestion problems within the two-lane road network. They offer a cost-effective alternative to four-lane road sections. The need for auxiliary passing lanes arises when certain road characteristics, such as high oncoming vehicle traffic, road terrain, and alignment, make it challenging to pass using the oncoming lane (*1*). This chapter outlines existing Super 2 highway policies and shares insights from pertinent research regarding operational factors that influence the performance of Super 2 highways.

CURRENT STATE AND INTERNATIONAL POLICIES

The design principles for Super 2 highways in Texas can be found in Chapter 4, Section 6 of the TxDOT *Roadway Design Manual* (*2*). Section 6 provides an overview of Super 2 highways, highlighting key considerations for the design phase. These key design considerations are followed by an explanation of the core design standards, with the essential elements summarized in Table 1.

Table 1. TxDOT Super 2 Design Criteria (Table 4-6 in *2*).

Design Criteria	Minimum	Desirable
Design speed	See Table 4-2	See Table 4-2
Horizontal clearance	See Table 4-2	See Table 4-2
Lane width	11 ft	12 ft
Shoulder width	3 ft [a]	8–10 ft
Passing lane length	1 mi	1.5–2 mi [b]

[a] Where right-of-way is limited.
[b] Longer passing lanes are acceptable, but not recommended more than 4 miles. Consider switching the direction if more than 4 miles.

The discussion of basic design criteria also describes the taper length for beginning and ending a passing lane as $L = WS/2$ and $L = WS$, respectively, where:

- L is the length of taper (ft).
- W is the lane width (ft).
- S is the posted speed (mph).

The TxDOT *Roadway Design Manual* includes illustrations that depict different passing lane configurations and taper rate applications, including the appropriate separation distances when closing passing lanes in opposing directions.

Internationally, road agencies in various countries, as well as transportation departments in different U.S. states, have established geometric standards for their respective versions of Super 2 highways. Table 2 provides a comprehensive overview of these international geometric standards, sourced from a synthesis by Romana et al. (*3*). The corresponding design criteria for Texas are included for the purpose of comparison. This synthesis summarizes findings from multiple countries; some countries have more than one entry in the table.

Table 2. Comparison of International 2+1 Roadway Cross Sections (*3*).

Country	Median Barrier	Lane Width for Direction of Travel with Single Lane (m)	Lane Width for Direction of Travel with Two Lanes (m)	Width of Paved Shoulder (m)	Width of Median (m)	Total Paved Width (m)
Sweden (MML)	Yes	3.75	3.25	0.50	1.75	13.00
Sweden (MLV)	Yes	3.25	3.25	0.75	1.00	12.25
Germany (EKL1)	No	3.50	3.25–3.50	0.50–0.75	1.00	15.50
Germany (EKL2)	No	3.50	3.25–3.50	0.50–0.75	0.50	15.00
Finland	Yes	3.75	3.25–3.50	0.90–1.25	1.70	14.35
Finland	No	3.75	3.25–3.50	1.25	0	13.00
Denmark	No	3.75	3.50–3.75	0.50	1.00	13.00
Norway	Yes	3.50	3.25	0.75–1.50	2.50	14.75
Ireland	Yes	3.50	3.50	0.50–1.00	1.00	13.00
Ireland	Yes	3.25–3.50	3.50	0.50	1.00	12.25
United Kingdom	No	3.50	3.50	1.00	1.00	13.50
United Kingdom	No	3.50	3.50	1.00	0.75	13.00
South Korea	No	3.50	3.25	1.50	1.50	14.50
South Korea	No	3.50	3.25	1.50	0.50	13.50
France	Yes	3.00	3.00	1.50	1.50	13.50
France	No	3.25	3.25	0.50	1.00	12.50
Poland	Yes	3.50	3.50	1.00	0.50	13.00
Poland	No	3.50	3.50	1.00	0.50–1.00	13.00–13.50
Spain	Yes	3.50	3.20	1.00–1.50	1.60	14.00
Spain	Yes	3.50	3.25–3.50	1.50	1.00	14.25
Japan	Yes	3.25	3.25	1.00	1.25	13.00
Texas, United States	No	3.35–3.65	3.35–3.65	0.90–3.00	0	11.85–16.95

Note: 1 m = 3.28 ft.

Table 2 reveals that international 2+1 roads typically include a median area, and supporting information from (*3*) suggests that they often feature a median barrier. Sweden is a prime example, initiating the conversion of existing 13-m (42.6-ft), two-lane, two-way cross-section roads to 2+1 roads back in 1998. More recently, the Swedish approach involves implementing intermittent 2+1 roads on existing 9- and 10-m (29.5- and 32.8-ft) two-lane, two-way cross-section roads. This implementation is achieved by widening specific areas within the corridor to accommodate passing lanes (*4*, *5*). Additionally, Table 2 highlights that lane widths on international 2+1 roads typically fall within the range of 3.00 to 3.75 m (9.8 to 12.3 ft), although shoulders are often less than 1 m (3.3 ft) in width.

Turning to the United States, numerous states have guidelines in their respective roadway design manuals outlining the construction requirements for Super 2-type roadways or single passing lanes on two-lane, two-way highways. In states where this information may not be readily accessible, there is often discussion in the media about the use of Super 2 highways, or there is evidence of state department of transportation-sponsored research on such highway types. Table 3 lists the states in which some evidence of Super 2 highway (or similarly designed two-lane road) usage has been identified, along with specific geometric data when available.

Table 3. State Policies and Other Documentation for Super 2 or 2+1 Roadways.

State	Term(s)	Summary
Arizona (*6*)	• Passing lanes • Climbing lanes	• Intervals of 3 to 5 mi are used, alternately in the opposite directions of travel. • Passing lane length should allow several vehicles in line behind a slow-moving vehicle to pass before reaching the transition to the normal section. • Passing lanes should not be longer than 2 mi or shorter than 1,300 ft. • Climbing lanes are used under certain circumstances.
Arkansas (*7*)	• Alternating passing lanes	• None found.
California (*8*)	• Passing lanes • Climbing lanes	• Lanes should not normally be constructed on tangent sections where the length of tangent equals or exceeds the passing sight distance. • Where the average daily traffic exceeds 5,000 vpd, four-lane passing sections may be considered.
Colorado (*9*)	• Passing lanes	• Minimum recommended sight distance is 1,000 ft on the approach to the lane-add and lane-drop tapers. • Presence of intersections and high-volume driveways, as well as bridges and culverts, should be considered when selecting locations. • Minimum lane length, excluding tapers, is 1,000 ft.
Connecticut (*10*)	• Climbing lanes	• No design criteria exist for passing lanes. • Climbing lanes should have lane width of 11 ft and a shoulder width of 4 ft.
Florida (*11*)	• Passing lanes • Climbing lanes	• Passing lanes follow the same criteria as normal lanes. • Climbing lanes follow the same criteria for normal lanes, and the lane should not terminate until well after the crest of the hill.

State	Term(s)	Summary
Idaho (*12*)	N/A	• Passing lanes should be considered if volumes exceed average daily traffic values in the *Design Manual*. • • If separate passing lanes are used, the lanes should be separated by at least 1,500 ft. • Minimum lane length should be 0.25 mi.
Illinois (*13*)	• Passing lanes	• Passing lanes may be warranted on two-lane facilities where passing opportunities are not adequate. • Typical spacing for passing lanes may range from 3 to 10 mi. • Optimal length of passing lanes is between 0.5 and 1 mi.
Iowa (*14*)	• Super 2	• Lane width is 12 ft. • Shoulder width is 10 ft., partially paved. • Passing lane spacing is ~5 mi. • Climbing lanes are provided on long/steep grades. • Turn lanes are provided where needed. • Access is limited to the extent practicable.
Kansas (*15*)	• Passing lanes	• Passing lanes, where provided, should be at regular intervals of ~5 mi. • Passing lane widths should be 12 ft. • Preferred configuration is side-by-side passing lanes with one lane in each direction, thus creating a short four-lane section. • Lengths are taken from TxDOT Report 0-4064-R1 (*16*).
Kentucky (*17*)	• 2+1	• Passing lane length is determined by the one-way flow rate and ranges from 0.5 to 2 mi.
Louisiana (*18*)	• Passing lanes	• No directives exist for passing lanes; passing lanes may be considered if the two-lane road has inadequate safe passing zones.
Michigan (*19*)	• Passing relief lanes • Super 2 (*20*)	• Design hourly volumes are used to identify candidate locations. • Lane width should be 12 ft. • Desirable minimum length is 1 mi, with an upper limit of ~1.5 mi. • Spacing is 5 mi.
Minnesota (*21*)	• Passing lanes	• Passing lanes should be constructed systematically at regular intervals. • Optimal length of a passing lane to reduce platooning is 0.5 to 1.0 mi.
Missouri (*22*)	• •Passing lanes	• The need for passing lanes on a two-lane road arises when the demand for passing opportunities exceeds their supply. A capacity analysis, which measures the level of service of a facility, is used to determine if a passing lane is needed. The level of service for a two-lane highway is defined in terms of two primary service measures: o Percent time spent following: The average percentage of travel time that vehicles spend in platoons behind slow vehicles due to the inability to pass. o Average travel speed: The length of a highway segment divided by the average travel time of all vehicles traversing the segment during a designated interval of time.
Montana (*23*)	• Passing lanes	• Passing lane need may be determined based on an engineering study.

State	Term(s)	Summary
Nebraska (*24*)	• Super 2	• Some level of access control limiting driveways and intersections is required.
Nevada (*25*)	• Passing lanes	• Empirical sight distance data are required.
New Hampshire (*26*)	• Passing lanes	• Passing sections should be provided as frequently as possible based on terrain.
Ohio (*27*)	• Passing lanes	• If capacity is restricted below the design level of service due to a lack of sight distance, passing lane sections should be considered.
Oregon (*28*)	• Passing lanes	• Passing lanes should be considered on two-lane arterials with inadequate passing sight distances. • Passing lanes should be considered only in areas where the roadway can be widened on both sides.
Texas (*2*)	• Super 2	• Minimum lane widths are 11 ft; 12-ft lanes are desirable. • Minimum shoulder widths are 3 ft; 8- to 10-ft shoulders are desirable. • Minimum passing lane length is 1 mi; 1.5- to 2-mi lengths are desirable. Longer passing lanes are acceptable, but should not exceed 4 mi.
Utah (*29*)	• Passing lanes	• Passing lanes offer localized improvements that optimize existing capacity for minimal cost.
Washington (*30*)	• Passing lanes	• Passing lanes are desirable where sufficient safe passing zones do not exist and the warrant for a climbing lane is not satisfied.
Wisconsin (*31*)	• Passing lanes	• If 20-year traffic projections of annual average daily traffic exceed 12,000 vpd or if two-way design hourly volumes exceed 1,400 vpd, it may be appropriate to consider expanding the facility to four lanes.

Note: N/A = not available.

PREVIOUS RESEARCH IN TEXAS

Extensive research on Super 2 highways has been undertaken in Texas, with some of these studies referenced in the preceding sections. Due to the significance of these endeavors in shaping the current practices related to Super 2 highways and passing lanes in Texas, the following sections provide a comprehensive summary of these research initiatives.

Design Guidelines for Passing Lanes on Two-Lane Roadways (Super 2)

The first statewide research initiative—TxDOT Project 0-4064 completed in 2001—focused on establishing guidelines for passing lanes on two-lane roadways (*16*). To achieve this, the project collected on-site data in Kansas and Minnesota. The research team actively traversed Super 2 corridors to gain firsthand experience and collected traffic flow data using traffic counters.

As a complementary task within the project, the research team conducted a survey of drivers at various locations throughout the state of Texas. This survey served as a valuable source of information on several critical aspects related to passing lanes. Key survey findings included the following:

- Drivers expressed their willingness to wait for a passing lane section for a distance of 3 mi or less.
- Many drivers were uncertain about whether they could use a passing lane in the opposing direction.
- As the width of the shoulder decreased, the percentage of drivers comfortable with stopping on the shoulder also decreased.
- Drivers indicated a preference for more frequent passing lanes.
- Suggestions from drivers pointed to the necessity for enhanced signing and road marking.

The research team also simulated roadway corridors using the TWOPAS microsimulation program. This effort enabled the team to offer data-driven suggestions for passing lane length and spacing that account for terrain and traffic volume. Table 4 details these recommended lengths and spacings based on average daily traffic (ADT) and terrain.

Regarding lane width, the research team recommended a width of 12 ft and/or then-current values contained in TxDOT's *Roadway Design Manual* (*2*). Special attention was devoted to shoulder width considering that common practice in Texas is for slower drivers to use the shoulder to facilitate being overtaken by faster vehicles. As a result, the researchers proposed 4-ft shoulders when rumble strips were not used and 6-ft shoulders when they were used.

Table 4. Recommended Length and Spacing Values Based on ADT and Terrain (*16*).

Level Terrain ADT (vpd)	Rolling Terrain ADT (vpd)	Recommended Passing Lane Length (mi)	Recommended Distance Between Passing Lanes (mi)
≤ 1,950	≤ 1,650	0.8–1.1	9.0–11.0
2,800	2,350	0.8–1.1	4.0–5.0
3,150	2,650	1.2–1.5	3.8–4.5
3,550	3,000	1.5–2.0	3.5–4.0

Operations and Safety of Super 2 Corridors with Higher Volumes

In 2011, a second research project (TxDOT Project 0-6135) was completed, with its primary focus on examining the operations and safety aspects of Super 2 corridors (*32*). This comprehensive research initiative encompassed various facets, including a thorough review of existing literature, a synthesis of practices in different states, surveys conducted with TxDOT to understand the state-of-the-practice in Texas, an analysis of crash data, comparisons of computer simulation models, the utilization of field data for calibration and simulation, and the subsequent formulation of recommendations based on these extensive efforts. Key findings included the following:

- Super 2 highways are best suited for use in areas with level terrain and rolling terrain with constraints on sight distance.

- Intersections and driveways should be avoided within the passing section.
- Passing lane locations and configurations may be influenced by the need to address an operational problem, appear logical to the driver, provide adequate sight distance on approach and departure tapers, and avoid low-speed curves.
- The theoretical capacity of a two-lane highway is 1,700 passenger cars per hour in one direction or 3,200 passenger cars per hour in both directions.
- Minimum passing lane lengths typically ranged from 1,000 ft to ~0.25 mi.
- Common passing lane spacings ranged from 3 to 10 mi.
- The minimum sight distance at the lane removal site can be calculated on a site-specific basis using *length* = *lane width* × *posted speed*.

The research team conducted a crash data analysis to investigate the safety impacts of Super 2 corridors. In this analysis, they compiled a dataset using crash data from five Super 2 corridors in Texas, covering a total of 53 centerline-mi. The analysis revealed a statistically significant 35 percent reduction in fatal and injury crashes within the corridor segments compared to locations without passing lanes.

As part of this research, a field study was conducted to demonstrate that Super 2 corridors enhanced the operation of rural, two-lane highways. The research team observed a significant number of vehicles initiating overtaking maneuvers at the beginning of the passing lane. While not all left-lane drivers adhered to Texas law for passing, large trucks commonly shifted to the right lane at the start of passing sections to allow faster-moving traffic to overtake. Field data indicated that compliance with left-lane use was higher at the beginning of the passing lane than at the end, suggesting that drivers might not move to the right lane after completing their overtaking maneuvers.

A total of 648 traffic microsimulations were conducted to assess the operational characteristics of Super 2 corridors using a hypothetical 10-mi corridor. The outcomes from these simulation efforts included the following:

- Calibration of the simulation model confirmed that the Traffic Analysis Module within the Interactive Highway Safety Design Model was suitable for modeling passing lanes.
- Univariate analysis revealed that the percentage of time spent following other vehicles ranged from 40 to 85 percent when no passing lanes were available, resulting in delays of 0.6 to 1.6 minutes. With the addition of six passing lanes, these percentages and delays decreased to 13 to 75 percent and 0.25 to 1.5 minutes, respectively. Notably, the additional benefit of introducing passing lanes diminished as more lanes were added.
- Multivariate analysis indicated that both adding new passing lanes and extending their lengths had a positive impact on operational performance. The improvements resulting from adding passing lanes surpassed the improvements resulting from extending the existing lanes. Passing lane length was also observed to exhibit diminishing returns; as

more passing lanes were introduced, the corridor began to function as a four-lane cross section.

- Traffic volume emerged as the primary factor influencing performance, while the percentage of trucks and the type of terrain had more limited effects.

Based on the totality of the aforementioned efforts, the research team recommended that ADT be removed as an upper limit for the installation of passing lanes; however, they stated that ADT should still be considered in terms of prioritizing installation locations. The researchers stated that adding passing lanes was a more desirable alternative to lengthening additional passing lanes. Four areas were identified as playing a major role when considering adding passing lanes: (1) right-of-way, terrain, and roadway structure constraints; (2) location of traffic generators; (3) restrictive existing geometry; and (4) sufficient sight distance at passing lane termination points. The findings from this study were used to make recommendation for revisions to the TxDOT *Highway Safety Improvement Program Manual* (*33*) and the TxDOT *Roadway Design Manual* (*2*). The recommendations from this project form the basis of the guidance found in TxDOT's current *Roadway Design Manual* (*2*).

Develop Capacity and Cost Benefits of Super 2 Corridors

In 2020, TxDOT Project 0-6997 (*34*) compared the operational and economic benefits of Super 2 corridors to traditional two-lane and four-lane cross-sections. The operational analysis found that the simulated Super 2 corridor with the most and longest passing lanes performed best, and all of the Super 2 options outperformed the other two-lane and four-lane corridors analyzed. The analysis also confirmed that more passing lanes provided more operational benefits than longer passing lanes. In economic terms, Super 2 scenarios had the highest benefit-cost ratios across all ADT and truck percentage configurations evaluated. At higher volumes (i.e., ADTs of 15,000–19,000 vpd), corridors with two-way left-turn lanes (TWLTLs) provided additional benefits because those volumes tended to be associated with more driveways and intersections. These results indicated that, when considering treatment options for high-volume conditions, the design process should reflect the presence of turning vehicles rather than emphasize only through vehicles traveling the length of the corridor.

SAFETY ANALYSIS

Review of Analysis Methods

The *Highway Safety Manual* (HSM), Volume 3, Part D (*35*), as well as several other studies (*36*, *37*), have employed advanced techniques to conduct before-after safety analyses of countermeasures. These methods include observational before-after (BA) studies and cross-sectional approaches.

In the observational BA method, a variety of approaches, such as the comparison group, empirical Bayes (EB), and full Bayes (FB), are utilized. Alternatively, the cross-sectional method involves developing a comprehensive regression model, known as a *crash prediction model* or *safety performance function* (SPF), which was introduced and is recommended for predicting crashes (*38*). Subsequently, this approach was deemed a logical choice in the HSM and Safety Analyst software due to its capacity to yield satisfactory results with less data compared to observational BA studies. Note that the EB before-after analysis method also requires SPF development.

Safety Performance Function

An SPF is an equation used to predict the average number of crashes per year at a location as a function of exposure and, in some cases, roadway or intersection characteristics (e.g., number of lanes, traffic control, or median type) (*35*). For highway segments, exposure is represented by the segment length and annual average daily traffic (AADT) associated with the study section. Equation (1) defines a sample SPF as follows:

$$\text{Predicted Crashes} = \exp[\alpha + \beta * \ln(\text{AADT}) + \ln(\text{Segment Length})] \tag{1}$$

where α is the intercept and β is the coefficient of AADT.

For intersections, exposure is represented by the AADT on the major and minor intersecting roads. Equation (2) defines a sample SPF for intersections as follows:

$$\text{Predicted Crashes} = \exp\left[\alpha + \beta_1 * \ln\left(AADT_{major}\right) + \beta_2 * \ln(AADT_{minor})\right] \tag{2}$$

where α is the intercept, β_1 is the coefficient of the major-road AADT, and β_2 is coefficient of the minor-road AADT.

An SPF supports the forecasting of crash frequency based on specific site conditions. These forecasts, derived from SPFs, can be employed independently or alongside the site's historical crash data using methods such as EB. The EB method helps estimate the expected long-term crash experience by incorporating both observed crashes at the site in question and projected crashes derived from an SPF (*39*).

The use of SPFs to predict the number of crashes plays a crucial role in various aspects of project development, including network screening, countermeasure comparison, and project evaluation as follows:

- **Network screening:** SPFs are valuable in the network screening process because they help determine whether the safety performance at a specific location is better or worse than the average performance of other sites with similar road characteristics and exposure. This information aids in identifying sites with the potential for safety improvements, which is a vital step in the safety management process.

- **Countermeasure comparison:** SPFs are employed to forecast the baseline crash frequency for given site conditions when evaluating potential countermeasures. They are used independently or in conjunction with crash history to estimate the long-term crash frequency for baseline conditions (without treatment). To assess the effectiveness of various safety interventions, CMFs are applied to estimate the expected crashes with treatment. This is particularly useful when there are multiple safety improvement options, and it is necessary to quantify and compare the potential benefits of each treatment.
- **Project evaluation:** Evaluating the safety impact of roadway improvements is crucial for informing future planning, policy, and programming decisions. The common practice involves using the EB method in an observational BA study to create CMFs. SPFs play a crucial role within the EB method by integrating the historical crash data for a specific site with the crash predictions derived from an SPF. Notably, the SPF aids in accommodating variations in traffic volume over time, making it an essential component of the process.

To assess the safety impact of introducing passing lanes, two commonly used methods are cross-sectional studies, which compare safety between different locations, and BA studies, which analyze safety changes over time (*40*, *41*). Each method has its strengths and weaknesses. The cross-sectional approach offers the advantage of using regression models for assessing alternative improvements in various highway sections. However, one disadvantage is that a typical regression model might not account for all potential factors—some may not be significant in the model, and others might not be measurable within the study's parameters. Factors not considered could affect the model's accuracy. In contrast, the BA approach is advantageous because it is a controlled experiment that examines differences in samples with similar characteristics except for the treatment, reducing the potential influence of other factors. Nevertheless, this approach has two disadvantages: (1) it often demands significant resources and time, which some state departments of transportation may not possess, and (2) the results may not be universally applicable to other locations (*41*).

Early research studies (*42*, *43*) initially assessed safety effectiveness through cross-sectional studies using basic statistical techniques focused on crash rates derived from traffic volumes and crash counts. For example, one study (*43*) analyzed 15 sites in 12 states and found that introducing passing lanes on two-lane highways led to reduced accident rates in both directions, without any notable safety issues linked to lane-addition or lane-drop transition zones. Similarly, another cross-sectional study (*44*) conducted in Michigan reported a 12 percent reduction in the accident rate following the installation of passing lanes. Furthermore, a safety evaluation of existing passing lanes in Missouri (*42*) revealed that the overall accident rates on National Highway System segments with passing lanes were approximately 12–24 percent lower than the accident rates on traditional two-lane highways. Nevertheless, cross-sectional assessments based solely on traffic volumes and crash counts could produce biased research findings, resulting in generalized conclusions that may not be applicable or replicable in other locations (*45*).

Conversely, the BA approach using the EB method with a comparison group is considered more robust than the cross-sectional or basic BA approach. It combines the strengths of both the simple BA approach (addressing temporal effects) and the cross-sectional approach (addressing spatial effects) (*40*). To obtain statistically reliable and accurate results, multiple factors beyond the treatment effect must be considered. One of these factors is the regression-to-the-mean bias, a statistical phenomenon that arises when a nonrandom sample is selected from a population (*40*, *41*). It has long been recognized that this issue in simple BA studies can often lead to an overestimation of treatment benefits. This issue occurs because locations with high crash rates selected for such studies tend to exhibit reductions in the after period even without treatment because their values tend to regress or return to long-term mean values (*41*). Furthermore, the phenomenon of crash migration can influence the results of safety-effectiveness evaluations of passing lane treatments. Crash migration implies that a reduction in crash risk at a treated location could result in increased risks in nearby areas (*46*, *47*). Long-term trends should also be considered in statistical analyses (*48*). For instance, a reduction in crash risk could be attributed to the treatment, but it might also be due to a national trend resulting from factors like increased driver safety awareness. Among the issues associated with the basic BA method, the regression-to-the-mean bias is generally regarded as the most significant (*41*). Therefore, the EB method (*39*) was developed to mitigate this problem and is based on the following three assumptions (*41*):

- The number of crashes at any given site conforms to a Poisson distribution.
- The means for a population of systems can be estimated using a gamma distribution.
- Year-to-year changes resulting from various factors are consistent for all reference sites.

The goal of the EB method is to predict the number of crashes that would have occurred during the after period if the treatment had not been implemented. This method typically involves two main steps: (1) establishing a basis for prediction by estimating the frequency of the studied crashes during the before period, and (2) predicting how the expected number of crashes would have changed from the before period to the after period based on this foundation (*39*). The EB method enhances the accuracy of assessing the treatment's impact by utilizing two critical pieces of information: (1) the crash history of the treated site and (2) data from reference sites with similar geometric characteristics (*40*, *41*).

In the EB method in a BA study, the change in safety (crashes) reflects the difference between the expected number of crashes that would have occurred in the after period without the treatment (B) and the number of observed crashes in the after period (A). Equation (3) formulates this relationship as follows:

$$Change\ in\ Crashes = B - A \tag{3}$$

Because changes in safety may result from the regression-to-the-mean phenomenon (caused by the bias selection) or from external factors, such as traffic volume changes or time trends in crashes and other factors, the count of crashes before the treatment is not a good estimate of *B* (*39*). Instead, *B* is estimated using the EB procedure in which an SPF is used to estimate the number of crashes predicted at the treated sites based on reference sites with similar traffic and physical characteristics (*36*). Equation (4) provides the following formulation for B_i:

$$B_i = (1 - w).(Obs) + w(Pred) \tag{4}$$

where B_i is the EB estimate of the expected number of crashes before treatment at site *i*, *Pred* is the sum of annual SPF estimates, and *Obs* is the observed crash count in the before period. The weight, *w*, is evaluated using the overdispersion parameter of the negative binomial estimate by using a maximum likelihood calibration methodology for SPFs and the predicted number of crashes in the before period.

A multiplicative factor is then applied to B_i to account for possible differences in time frame, weather, traffic volume, and other external factors between the before and after periods (*39*). The unbiased overall expected number of crashes without treatment, B_{sum}, is estimated by summing all values of B_i in the before period of the treatment group. The sums of the observed crashes in the after period of the treated sites are the unbiased value of the crash frequency in the after period. Finally, the CMF can be estimated using Equation (5) as follows:

$$CMF = \frac{A_{sum}/B_{sum}}{1+[\frac{var(B_{sum})}{B_{sum}^2}]} \tag{5}$$

where A_{sum} is the sum of observed crashes in the treated sites in the after period. Equation (6) can be used to calculate the standard deviation of the CMF as follows:

$$Stddev\ (CMF) = [\frac{CMF^2\{\left[\frac{Var(A_{sum})}{A_{sum}^2}\right]+Var(B_{sum})/B_{sum}^2}{\left[1+\frac{Var(B_{sum})}{B_{sum}^2}\right]^2}]^{0.5} \tag{6}$$

The negative binomial dispersion parameter (*k*) was estimated from the calibration of the SPF using the maximum likelihood methodology (*49*, *50*, *51*) and varied from site to site, depending on the length of a roadway segment (*52*, *53*). Equation (7) can be used to estimate *k* as follows:

$$k = a.L^b \tag{7}$$

where *a* and *b* are regression terms calibrated to maximize the log-likelihood function with a modified negative binomial regression technique. The results of the model calibrations for the total and target crashes were shown previously in Table 4, referring to the following SPF model form shown in Equation (8):

$$E(Y) = y_i \times e^{\alpha} \times L \times AADT^{\gamma} \times e^{\beta.DD} \tag{8}$$

where $E(Y)$ is the expected annual (fatal plus injury) crash frequency of the random variable Y; L is the length of the road segment (m); AADT is the average annual daily traffic (vpd); α, γ, and β are regression terms; y_i is the yearly coefficient in year i; and DD is driveway density (driveways/km).

The EB methodology examines the estimation of the expected number of crashes that would have occurred without treatment and compares them with the crashes observed at the treated sites (*54*). Even with no significant regression-to-the-mean bias, the EB technique requires a reliable and large dataset with sufficient years of observation and numbers of treated sites, adequate for estimating the safety effects of a treatment with acceptable standard errors. In this framework, an FB approach can mitigate the problem of using small datasets by providing more detailed causal inferences and more flexibility in selecting crash count distributions, acknowledging that a more complex methodology must be applied. With the aim of estimating the safety improvements of new, short 2+1 road sections in Poland limited by the existing road network, EB and FB estimations were compared and different SPF model forms were used to evaluate the performance of the two methodologies (*54*). Results indicated that, even if CMFs resulted in similar average values, the EB method tended to underestimate CMFs compared with the more complex methodology, while the FB approach provided a lower standard deviation overall. The differences were more pronounced between the EB and FB approaches when a simple SPF model form was used for the analyzed dataset. Moreover, for this specific dataset, the difference between the FB and EB methods using a refined regression model with more variables was negligible.

Compared to the EB and FB linear approaches, the FB nonlinear approach facilitates the use of noncontinuous and noncategorical variables like the number of driveways, *ND*, through the introduction of a second nonlinear factor in Equations (9), (10), (11) and (12) as follows:

$$ln(\theta_{it}) = ln(\gamma_{it}) + \varepsilon_i \tag{9}$$

$$\gamma_{it} = \mu_{it} + exp(\beta_2 \cdot ND) \tag{10}$$

$$ln(\mu_{it}) = \alpha_0 + \alpha_1 T_i + \alpha_6 T_i I_i + \beta_1 AADT_{it} + ln(L) \tag{11}$$

$$\varepsilon_i \tilde{} N(0, \sigma_\varepsilon^2) \tag{12}$$

where λ is the comprehensive crash count estimation, and μ in this case represents the crash count estimation without the contribution of *ND*. Non-informative priors for all the regression coefficients were used in the FB analysis.

The deviance information criterion, *DIC*—a Bayesian generalization of Akaike's information criterion—was used to measure the complexity and fit of the linear versus the nonlinear model. Equation (13) can be used to calculate the DIC as follows:

$$DIC = \bar{D} + pD \tag{13}$$

where $\bar{D}$ is the measure of model fitting, and pD is the effective number of parameters. A smaller *DIC* indicates a better model fit. When comparing the two models, a difference of more than 10 in the value of the *DIC* might rule out the model with the higher *DIC* (*55*). When the difference is less than 10, the models are competitive.

Crash Risks Associated with Super 2 Corridors

To conduct a comprehensive evaluation of the safety effectiveness of Super 2 corridors, numerous researchers and agencies have undertaken observational BA studies using the statistically robust EB method on Super 2 highway sections (*1*, *40*, *45*, *56*). These studies consistently demonstrated a positive correlation between Super 2 treatments and improved safety in terms of crash risk reduction.

As part of TxDOT Project 0-6135, researchers investigated the safety effectiveness of four Super 2 corridor sites in Texas using the EB method (*40*). To ensure the method's reliability, four reference groups of Super 2 sites were categorized based on their spatial and operational characteristics. The observed data were statistically calibrated using an SPF developed using negative binomial regression, employing data from these reference groups spanning 1997 to 2009. As discussed previously regarding operational benefits, this study revealed that the installation of passing lanes resulted in a statistically significant reduction in crashes, with a 35 percent reduction in segment-only crashes and a 42 percent reduction in segment-and-intersection crashes, both at a 95 percent confidence level. The research concluded that passing lanes offered substantial benefits, particularly at higher traffic volumes, by reducing crashes, delays, and the percentage of time spent following.

The HSM initially provided a baseline CMF of 0.75 for the implementation of passing lanes on two-lane highways (*35*). However, Persaud et al. (*57*) argued that this CMF value was based on overly simplistic research studies that could not reliably validate the results. Consequently, additional analyses were required to establish a statistically sound measure of the safety effectiveness of passing lanes. To obtain a more dependable set of CMF values for passing lanes, Persaud et al. (*57*) evaluated safety effectiveness using traffic volume and crash history data from 100 reference sites (without passing lanes) and 231 passing lane sites in Michigan. Given the limited number of sites suitable for developing CMFs for passing lanes through a BA study, two complementary analyses were performed: (1) an EB-based observational BA study of sites with passing lanes and (2) a cross-sectional analysis using generalized linear modeling to estimate the difference in safety performance between sections with and without passing lanes. An additional companion study examined adjacent untreated sites within 1 mi using the same procedures to investigate potential spillover effects (crash migration). In this analysis, only crashes not involving animals, intersections, or interchanges were considered, focusing on the crash types most associated with the impact of passing lanes. Equation (14) shows the model formulation used to estimate the CMFs as follows:

$$crashes\ per\ year = exp^{(\alpha+\beta_3)}(AADT)^{\beta_1}(segment\ length)^{\beta_2} \quad (14)$$

Table 5 lists the resulting set of CMFs for various types of crashes estimated from the model. All coefficients were found to be above the 95 percent confidence level (*57*). In cases where a CMF was estimated with a statistical significance of less than 10 percent, a note was added.

Table 5. Implied CMFs from Cross-Sectional Models for Non-animal/Non-intersection Crashes (*57*).

Crash Type	CMF[a]	CMF with 1-mi Adjacency
Total	0.67	0.63
Injury	0.71[b]	0.65
Target[c]	0.53	0.46
Day	0.60	0.58
Night	0.91[b]	0.81[b]
Wet	0.81[b]	0.71[b]
Dry	0.53	0.57
Peak months (June–August)	0.54	0.54
Nonpeak	0.72[b]	0.68

[a]CMFs were estimated from models formulated using Equation (14).
[b]Estimated CMFs were not statistically significant.
[c]Target crashes included run-off-road, head-on, rear-end straight, sideswipe same direction, and sideswipe opposite direction crashes.

Variables that were not significant were excluded in the final models. For this study, every effort was made to eliminate the confounding effects of other factors by selecting reference sites that were as similar as possible to the passing lane sites (*57*). The results of this study supported the effectiveness of passing lanes in improving safety. Additionally, the results suggested the benefit extends as far as 1 mi upstream and downstream of the passing lane (*58*).

The effects of installing a passing lane will depend on the actual length of that lane (*59*). By extension, it is also reasonable to expect that the safety effects of lengthening an existing lane will depend not only on the amount of the lengthening but also on the original length. Yet, knowledge that can be applied to estimate these two sets of effects in a design process is lacking. The CMFs in the HSM and in the CMF Clearinghouse for installing a passing lane are all single-valued (on the order of 0.75), and neither source provides CMFs for lengthening an existing passing lane. Persaud et al. (*59*) sought to address these voids by developing continuous CMFs for both sets of design decisions using crash, geometric, and traffic data for passing lane and reference sections in Michigan and Ontario, Canada. Generalized linear modeling (GLM) and FB

Markov Chain Monte Carlo (MCMC) simulation were used to develop cross-section regression models from which CMFs were derived and compared. The results were consistent with those from credible BA studies, and thus are recommended for implementation in practice, particularly for HSM applications.

As noted, conventional GLMs were also estimated for comparison purposes. These models do allow the CMF for installing a passing lane to depend on its length, but the implied CMFs for extending a passing lane by a given amount do not depend on the original length. Equation (15) supported development of the estimated models based on the combined Michigan and Ontario data for total and fatal-injury crashes as follows:

$$\frac{Crashes}{Year*km} = e^{(\alpha+\beta_3)} AADT^{\beta_1} e^{\beta_2(LngPL)} \tag{15}$$

where *LngPL* is the passing lane length (km) (*LngPL* = 0 for segments without passing lanes); and α, β_1, β_2, and β_3 are model parameters to be estimated (β_3 is a Michigan-specific constant).

Table 6 lists the parameter estimates obtained using the SAS software package. All estimates were statistically significant at the 5 percent level ($p < 0.05$) except for the Michigan-specific constant. That constant was still included in the final models because it improved the precision of the β_2 estimate—the parameter that is essential for the estimation of the CMFs—as will be seen later. Figure 1 and Figure 2, which show cumulative residual (CURE) plots for total and fatal-injury crashes, respectively, indicate a reasonable model fit over the range of passing lane lengths.

Table 6. GLM Parameter Estimates for Combined Michigan and Ontario Data (*59*).

Coefficient	Estimate for Total Crashes	*p*-value for Total Crashes	Estimate for Fatal-Injury Crashes	*p*-value for Fatal-Injury Crashes
α	–7.0024	< 0.0001	–8.1530	< 0.0001
β_1	0.7830	< 0.0001	0.7539	< 0.0001
β_2	–0.1762	< 0.0001	–0.1863	0.0001
β_3	0.0031	0.9761	0.1752	0.1708

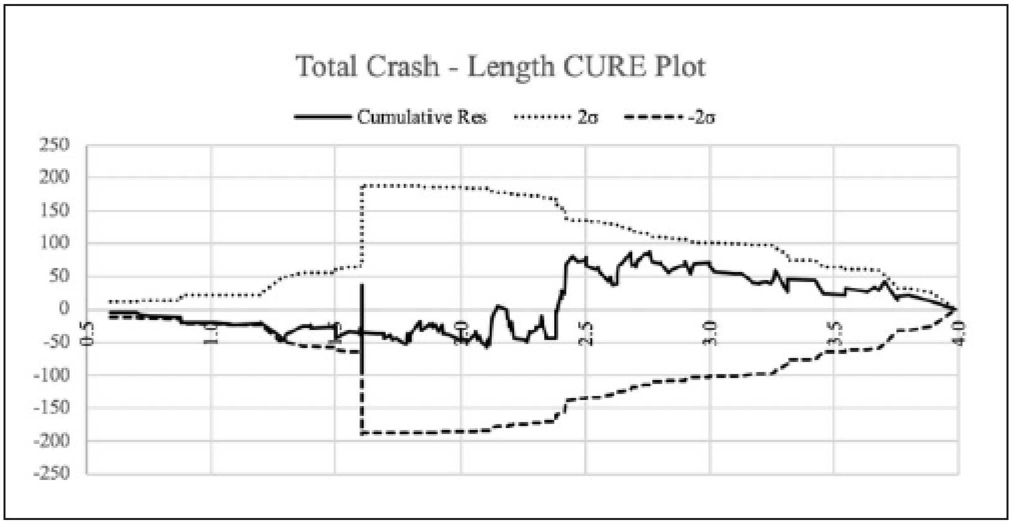


Figure 1. Total Crash-Length (km) CURE Plot for GLM (*59*).

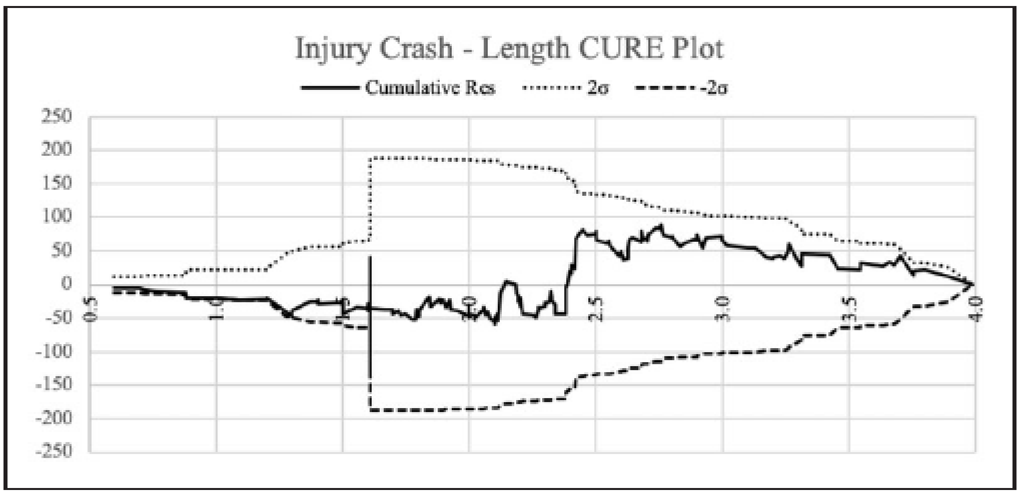


Figure 2. Fatal-injury Crash-Length (km) CURE Plot for GLM (*59*).

Equation (16) supported development of the estimated models for total and fatal-injury crashes as follows :

$$\frac{Crashes}{Year*km} = \exp(a) * \exp\big(b * (if\ in\ Michgan)\big) * AADT^{c} \exp\left(\exp(e^{LngPL})\right) \tag{16}$$

where *LngPL* is the passing lane length (km) (*LngPL* = 1 at Michigan sites and *LngPL* = 0 at Ontario sites); and *a*, *b*, *c*, and *e* are model parameters to be estimated.

The MCMC sampling procedure was run in WinBUGS software for 100,000 iterations to calculate the posterior estimates of parameters. The first 10,000 iterations were not considered for the parameter estimation and were discarded as burn-in. Table 7 and Table 8 present the parameter estimates and their associated statistics at 95 percent credible intervals for the total crash and fatal-injury crash models, respectively. The results showed that all parameters used were effectively significant at the 95 percent credible interval in both total and fatal-injury crash models. Note that the Michigan-specific constant was now significant, in contrast to the GLM estimate. This conflicting outcome is likely a result of the use of a more appropriate model form.

Table 7. Parameter Estimates and MCMC Statistics for the Total Crash Model (*59*).

Parameter	Mean	Standard Deviation	MC Error	2.50%	Median	97.50%
a	–8.458	0.794	0.043	–10.080	–8.422	–7.015
b	0.216	0.111	0.002	–0.005	0.217	0.434
c	0.836	0.095	0.005	0.666	0.833	1.031
e	–0.193	0.076	0.001	–0.363	–0.185	–0.063
Dispersion parameter	1.103	0.092	0.001	0.933	1.100	1.294

Table 8. Parameter Estimates and MCMC Statistics for the Fatal-injury Crash Model (*59*).

Parameter	Mean	Standard Deviation	MC Error	2.50%	Median	97.50%
a	–9.765	0.804	0.043	–11.320	–9.775	–8.176
b	0.399	0.133	0.002	0.135	0.400	0.655
c	0.824	0.095	0.005	0.636	0.825	1.008
e	–0.185	0.084	0.001	–0.373	–0.176	–0.048
Dispersion parameter	1.022	0.113	0.001	0.819	1.016	1.264

For the GLM approach, based on Equation (15), the CMF for installing a passing lane of length *LngPL* can be calculated using Equation (17) as follows:

$$CMF = e^{(\beta_2 \times LngPL)} \quad (17)$$

For the FB MCMC modeling approach, based on Equation (16), the CMF for increasing a passing lane from $LngPL_1$ to $LngPL_2$ can be calculated using Equation (18) as follows:

$$CMF = \frac{\exp\left(\exp(e * LngPL_2)\right)}{\exp\left(\exp(e * LngPL_1)\right)} \quad (18)$$

When using this equation for installing a passing lane of length PL_2, a value of 0 is substituted for PL_1. Both Equations (17) and (18) can be used directly with Imperial units if desired. Figure 3 and Figure 4 depict the CMFs for total and fatal-injury crashes, respectively, estimated from Equations (17) and (18) for installing a passing lane of various lengths and compared to the single-value CMF from the HSM. The recommended ranges for passing lane length in Ontario (ON) and Michigan (Mich.) are also shown for reference.

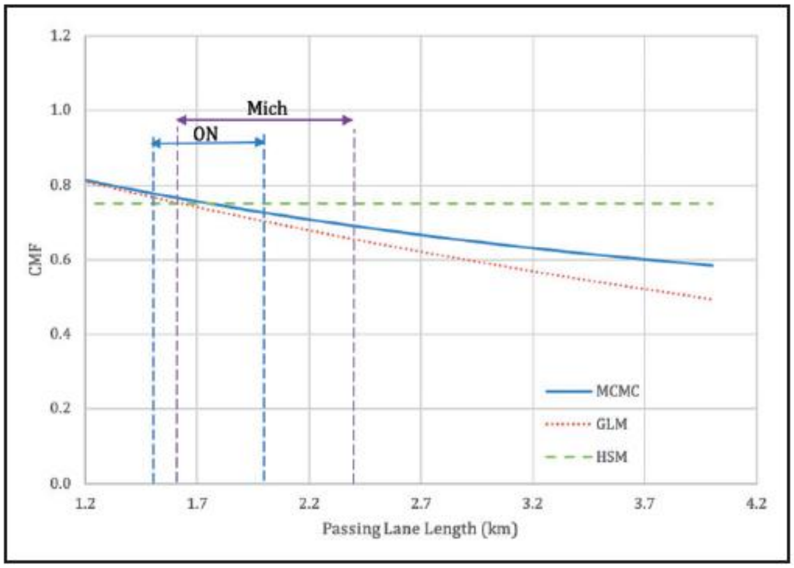


Figure 3. Total CMFs for Installing a Passing Lane (*59*).

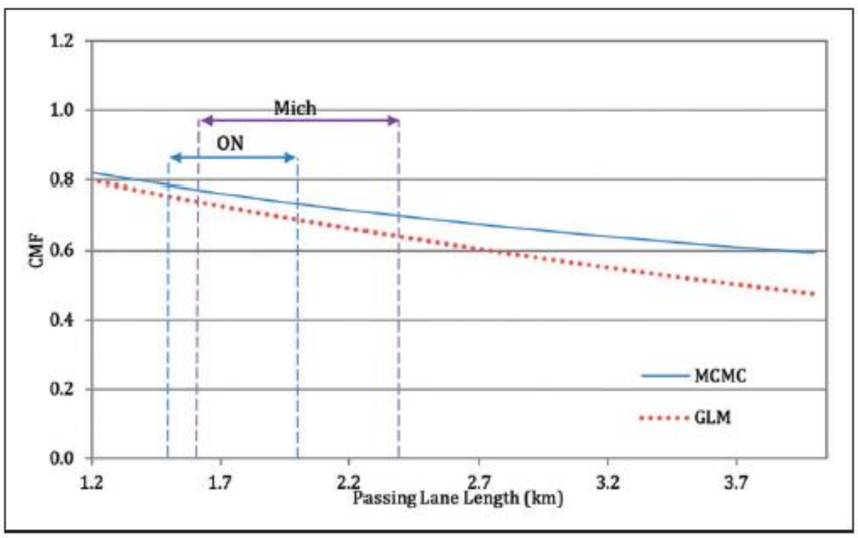


Figure 4. Fatal-injury CMFs for Installing a Passing Lane (*59*).

Table 9 provides illustrative CMF estimates for total crashes when a passing lane is extended by 500 m (0.31 mi) and 1 km (0.62 mi), based on Equations (17) and (18). The first conclusion is that the safety benefit increased as the length of the extension increased, as might be expected. It can also be seen that, for a given increase in length, the CMFs based on the FB MCMC models was dependent upon the original passing lane length and logically increased as the original length increased. In contrast, the CMFs based on the GLMs were independent of the original length, which seems illogical. To illustrate, extending a passing lane by 500 m from 1.5 km to 2 km resulted in an expected 6.7 percent decrease in crashes, while the same amount of change from 3.5 km to 4 km decreased crashes by 4.6 percent. These reductions increased to 12.3 percent and 7.8 percent, respectively, for a 1-km extension.

Table 9. CMFs for Extending an Existing Passing Lane by 500 m (0.31 mi) and 1 km (0.62 mi) (*59*).

Original Passing Lane Length	CMF from FB MCMC for 500-m (0.31-mi) Extension	CMF from GLM for 500-m (0.31-mi) Extension	CMF from FB MCMC for 1-km (0.62-mi) Extension	CMF from GLM for 1-km (0.62-mi) Extension
1.5 km (0.93 mi)	0.933	0.916	0.877	0.839
2.0 km (1.24 mi)	0.939	0.916	0.888	0.839
2.5 km (1.55 mi)	0.945	0.916	0.897	0.839
3.0 km (1.86 mi)	0.950	0.916	0.906	0.839
3.5 km (2.17 mi)	0.954	0.916	0.915	0.839
4.0 km (2.49 mi)	0.958	0.916	0.922	0.839

However, representing CMFs as point estimates may not adequately explain how a safety countermeasure affects collision frequency over time and evaluate the presence of novelty effects associated with the treatment (*60*). Therefore, Sacchi et al. (*60*) performed a study in which the main goal was to overcome this drawback through the development of CMFs that incorporated changes over time for the treatment effectiveness, rather than CMFs that used a single value. Within an FB context, linear and nonlinear intervention models, which acknowledge that the safety treatment effects do not always occur instantaneously but are spread over future time periods, provided a promising methodological framework for estimating CMFs with time trends. A case study was presented where the linear and nonlinear intervention models were applied to estimate the effectiveness of the Signal Head Upgrade Program recently implemented in the city of Surrey in British Columbia, Canada. The results of the case study highlighted the advantages of estimating CMFs with time trends and the impacts on the economic evaluation of safety countermeasures. Given the way future benefits are discounted to present values, the results of using a CMF can affect the cost-effectiveness of safety countermeasures significantly. The results also showed that the nonlinear intervention models provided a more realistic trend where the treatment effect in the long run converges to an everlasting treatment impact.

The different exponents of AADT in the SPFs are more sensitive to AADT than total crashes, as expected. Figure 5 shows the CURE plots for goodness to fit measures for two calibrated models describing total and target crashes (56). Both models had an acceptable fit to the observed data with only a small drift outside the -2σ boundaries for an AADT of about 14,000 vpd in the model calibrated on the target crashes.

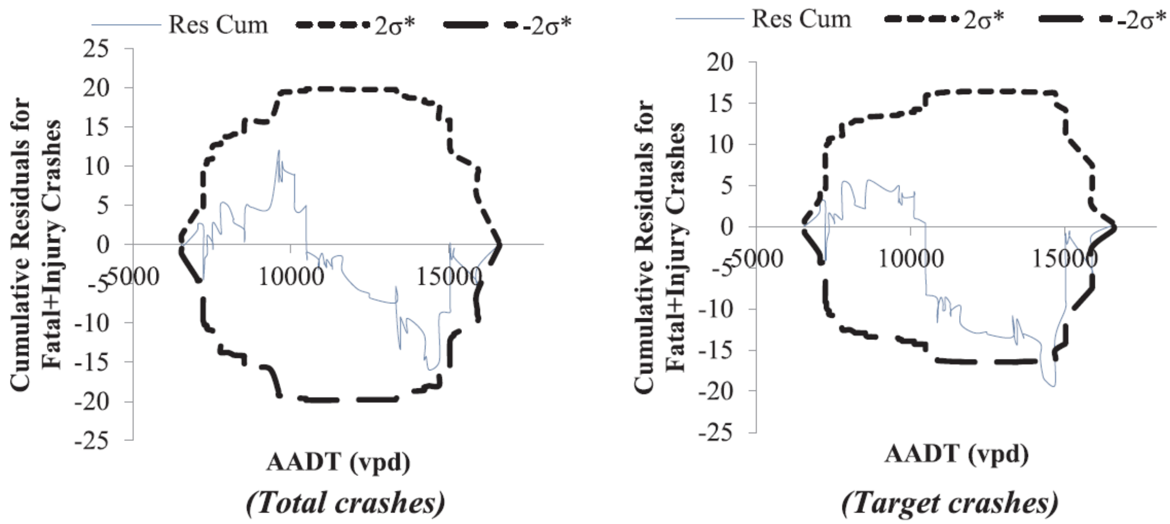


Figure 5. Model Fit CURE Plots with ± 2σ for Total and Target Crashes (*56*).

Following the SPFs calibration, CMFs were calculated using the EB method in a BA analysis. Table 10 summarizes the CMF estimation results. The results indicated a strong safety benefit attributable to a 2+1 treatment on fatal and injury crashes; CMFs of 0.53 and 0.51 were estimated for total and target crashes, respectively. Based on these averaged results, a 47 percent reduction in total crashes and a 49 percent reduction in target crashes at the treated sites was expected. Moreover, the upper limit at the 95 percent confidence level below 1.0 showed a statistically significant safety benefit. When the upstream and downstream segments were included in the sample, the CMF estimates were higher (indicating a lower crash reduction) and more uncertain because the confidence interval including the value of 1.0.

Table 10. CMF Estimation Results for Fatal-Injury Crashes (*56*).

Parameter	Total for Short 2+1	Target for Short 2+1	Total for Short (2+1) with Up/Downstream Segments	Target for Short (2+1) with Up/Downstream Segments
Comparison sites crash ratio	0.78	0.87	0.72	0.80
B_{sum}	37.27	31.01	28.03	24.05
Var(B_{sum})	9.93	8.08	6.53	6.61
A_{sum}	20	16	27	18
CMF	0.53	0.51	0.96	0.74
Standard deviation	0.127	0.136	0.20	0.19
95% confidence interval	0.28–0.78	0.24–0.78	0.56–1.35	0.36–1.12

Note: The 95% confidence interval reports the lower and upper interval combinations of the CMF and standard deviation.

The overall results exhibited decreased crash frequencies in the 2+1 sections, which should be balanced by an increase in the up/downstream sections due to a “migration” phenomenon after the implementation of the treatment. However, the effects on the upstream and downstream segments show uncertain results due to the large standard deviation, with confidence intervals that included the value of 1.0. Although reported here, these results need to be investigated carefully and in more detail using an ad hoc dataset of information; the high variability could be related to lack of data.

Not only can conflicts be surrogate measures of safety, but the effects of geometric parameters, such as the length of the additional lane, can play a fundamental role in the safety performance of the 2 +1 passing lane (*61*). An assessment was conducted using VISSIM and Surrogate Safety Assessment Model simulations, and comparisons were made based on different simulated passing lane lengths, merging areas, and AADT values. Particular attention was paid to the calibration and validation of the microsimulation models using real traffic characteristic and crash history data.

Table 11 compares the results of two BA evaluations of passing lane installations. These studies included all types of crashes in both directions of travel within the portion of the two-lane highway where passing lanes were installed. A California study by Rinde (*62*), which considered 23 sites in level, rolling, and mountainous terrain, found crash rate reductions due to passing lane installations of 11 to 27 percent, depending on the road width. The crash rate reduction effectiveness at the 13 sites in level or rolling terrain was 42 percent. Using data from 22 sites in four states, Harwood and St. John (*63*) found the crash rate reduction effectiveness of passing lanes to be 9 percent for all crashes and 17 percent for fatal-injury crashes. The combined data from both studies indicated that passing lane installations reduced the crash rate by 25 percent. No difference was found between the crash rates of passing lanes in level and rolling terrain.

Table 11. Crash Reduction Effectiveness of Passing Lanes (*43*).

Source	Type of Terrain[a]	Total Roadway Width (ft)	Number of Passing Lane Sites	Percent Reduction in All Crashes	Percent Reduction in Fatal-Injury Crashes
Rinde (*62*)	L, R, M	36	4	11	N/A[b]
Rinde (*62*)	L, R, M	40	14	25	N/A
Rinde (*62*)	L, R, M	42–44	5	27	N/A
Rinde (*62*)	L, R	36–44	13	42	N/A
Harwood and St. John (*63*)	L, R	40–48	22	9	17
Combined totals for level and rolling terrain	L, R	All	35	25	N/A

[a]L = level, R = rolling, and M = mountainous.
[b]N/A = not available.

Harwood and St. John (*63*) found no indication in the crash data of any marked safety problem in either the lane addition or lane drop transition areas of passing lanes. In field studies of traffic conflicts and erratic maneuvers at the lane drop transition areas of 10 passing lanes, lane drop transition areas were found to operate smoothly. Overall, 1.3 percent of the vehicles passing through the lane drop transition area created a traffic conflict, while erratic maneuver rates of 0.4 and 0.3 percent were observed for centerline and shoulder encroachments, respectively. The traffic conflict and encroachment rates observed at lane drop transition areas in passing lanes were much smaller than the rates found in lane drop transition areas at other locations on the highway system, such as in work zones. Table 12 presents crash rate ratios for passing lane sections and short four-lane sections relative to the expected crash rate on a conventional two-lane highway.

Table 12. Relative Crash Rates for Improvement Alternatives (*43*).

Alternative	All Crashes	Fatal-Injury Crashes
Conventional two-lane highway	1.00	1.00
Passing lane section	0.75	0.70
Four-lane section	0.65	0.60

Gattis et al. (*7*) compared crash rates from 19 passing lane sites with the statewide average crash rates for rural two-lane highways in Arkansas. Figure 6 shows a plot of crash rates for each passing lane site versus the corresponding traffic volume for that site; the graph also includes notations for the statewide AADT and average crash rates. Figure 7 shows crashes per mile-year versus volume. The following observations were made:

- Even though most of the passing lane sites had volumes higher than the statewide average for two-lane rural roads (ADT of 5,293 vpd versus 1,857 vpd) the crash rates of passing lane sites were usually less than the statewide average crash rate for two-lane rural roads.
- The crash rates of the passing lane sites exhibited a weak trend of increasing with volume. If the trend line were extended, the projected crash rate trend line would cross the statewide crash rate at a volume of 22,740 vpd.
- The crash rates of three passing lane sites (1-13, 65-1, and 82-1e) exceeded the state average. The researchers did not identify any obvious reasons for this.

Crash severity attributes were similar for the crashes that took place in passing lanes and the statewide crash averages. The passing lane severe injury rate was slightly higher than the statewide average, and the property damage only (PDO) rate was slightly lower.

The percentages by crash type (e.g., head-on, right-angle, etc.) for the passing lane sites were also similar to the rural two-lane percentages. The greatest difference was for the single-vehicle crashes, which comprised 51.6 percent of passing lane crashes versus 54.1 percent for rural two-lane roads.

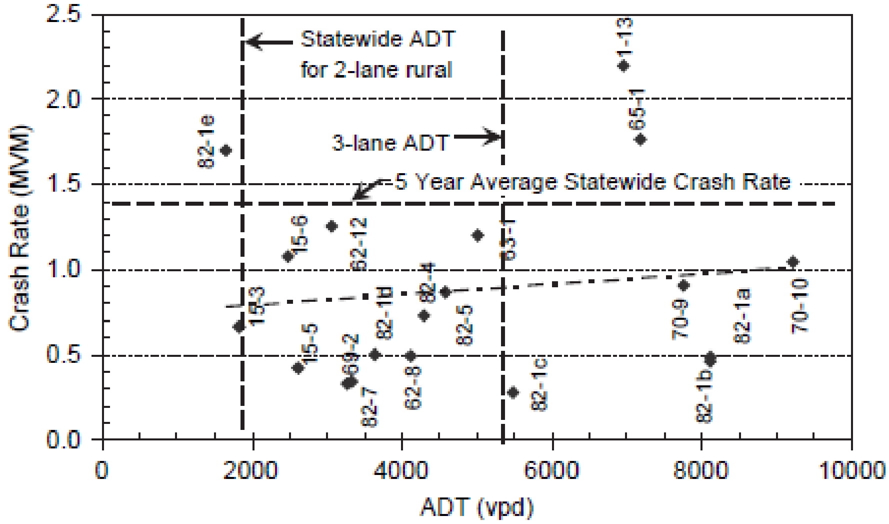


Figure 6. Crash Rates for Passing Lane Segments Versus Rural Two-Lane Roads (*7*).

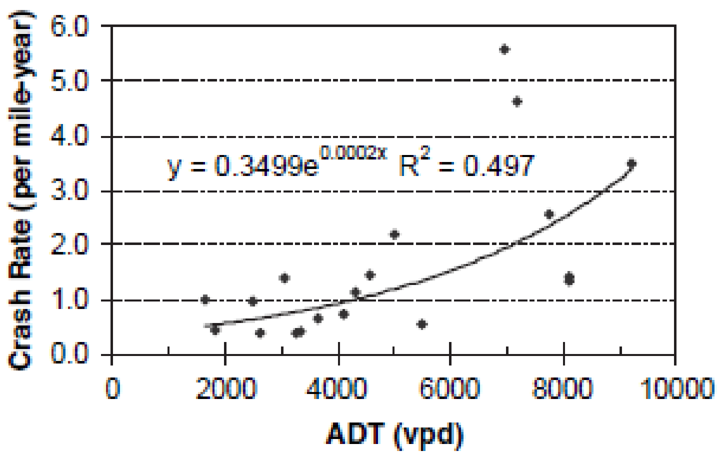


Figure 7. Crash Rates per Mile-Year for Passing Lane Segments (*7*).

Schrock et al. (*64*) considered how historical data and CMFs were combined to estimate the safety benefits of widening shoulders and adding passing lanes on rural two-lane roads. The primary objectives were to determine specific CMFs for the state of Kansas and states with similar rural environments and to estimate the number of crashes avoided. Safety and traffic volume data for rural two-lane roads from 2000 to 2008 were obtained from the State of Kansas. This dataset was combined with existing models from earlier studies to create state-specific CMFs, which allowed for more accurate estimation of safety benefits. The study determined that CMFs for shoulder widening and the addition of passing lanes for low-volume roads were 0.95 and 0.65, respectively. These CMFs were based on shoulder widening from 2 ft (0.61 m) to 8 ft (2.44 m) or the construction of a short four-lane passing section. Based on these results, 20-year projections were developed to estimate the benefits in terms of crash reductions that could be achieved through the implementation of these safety improvements.

Wang et al. (*37*) estimated the effects of signalization at intersections in Florida. The research approach applied the EB method to develop CMFs for different types of crashes, specifically KABCO (total), KABC (fatal-injury), and rear-end crashes, based on several SPFs from various jurisdictions, adjusted by calibration factors to suit the local context. (In the KABCO/KABC scale, K represents fatal (killed), A denotes incapacitating injury, B indicates non-incapacitating injury, C refers to possible injury, and O signifies property damage only.) Data from both Florida and Ohio were used to develop the SPFs. The SPFs suggested in the HSM were also considered when calculating the CMFs. The study involved the development of these SPFs and a comparison of SPFs from different states. It concluded that it might not be appropriate to directly apply SPFs from one state to another without thorough examination and adjustments. These researchers also presented findings related to CMFs, particularly for KABCO crashes. When using the SPF developed in Florida, the CMF was 0.785, significantly smaller than 1, indicating that signalization at intersections resulted in fewer total crashes. However, when the SPFs from Ohio and the HSM were applied, higher CMFs of 1.06 and 1.07 were obtained, respectively. These values were significantly larger than 1, suggesting that signalization resulted in more total crashes. This study also discussed CMFs for KABC and rear-end crashes. The major finding of this study was that CMF values can vary significantly when SPFs developed from data in other states are applied. Therefore, CMFs may be biased if SPFs from other states are used without proper adjustments.

The Wyoming Department of Transportation constructed and evaluated 9 passing lane segments on a 26-mi rural two-lane highway between 2005 and 2006. In an initial analysis using simplistic BA statistics, no significant safety improvements from these Super 2 treatments were detected (*45*). However, a more thorough investigation utilizing the EB method with the same dataset subsequently conducted by Schumaker et al. (*45*) revealed a statistically significant safety enhancement resulting from the Super 2 corridor treatment. The CMFs in this research were estimated based on crash rates calculated using the mean AADT and the length of the treated and untreated segments. A Poisson test of significance was performed to ensure the statistical rigor of the findings. The outcome of this study indicated a CMF for the passing lane segments of 0.58 with a 95 percent confidence interval. In practical terms, this CMF value corresponds to a 42 percent reduction in crashes, decreasing the crash rate from 0.86 to 0.48 crashes per million vehicle miles. A comparison of these two studies highlights the significance of employing a robust safety impact assessment (*65*). Initially, with the basic BA analysis results, the assessed economic benefits stemming from avoided crashes were minor, putting the project at risk of being canceled in favor of a traditional four-lane alignment option. However, when the agency conducted an EB-based analysis one year later, the economic benefits of improved safety became substantial: $9.20 million for reductions in fatal (K) crashes, $0.42 million for reductions in injury (A) crashes, $0.13 million for reductions in evident injury (B) crashes, $0.13 million for reductions in possible injury (C) crashes, and $3,200 for reductions in property damage-only (O) crashes (*66*).

A follow-up study (*67*) developed CMFs for passing lanes by investigating *oil counties* and *non-oil counties* separately. The CMFs were calibrated using the initial naïve BA model and the EB method in a BA study. The models included potential effects of other geometric and weather characteristics. They found that passing lanes had significant effects (at the 95 percent confidence level) in both categories of counties. In non-oil counties, the safety effectiveness of passing lanes was 38 and 59 percent for total and fatal-injury crashes, respectively. For oil counties, the safety effectiveness was 31 and 58 percent for total and fatal-injury crashes, respectively. The findings from these studies conducted in Wyoming reinforce the notion within the profession that the simplistic BA approach has often yielded imprecise results. In light of these findings, the study by Schumaker et al. (*45*) recommended that transportation agencies consistently employ a reliable statistical analysis method, such as the EB method, instead of relying solely on simple (naïve) BA comparisons. In summary, it is crucial to utilize a rigorous and dependable method for assessing the economic impact of passing lanes, particularly in the context of safety risk assessment.

Calvi et al. conducted a driving simulator study to see whether different types of median separation on 2+1 roads affect driving behavior and to provide new insights for designing more effective and safer 2+1 roads (*68*). The scenario exactly reproduced an existing two-lane rural road in Poland where 2+1 sections were implemented. Four different median separation types were tested: (1) double-line markings only, (2) reflective elements, (3) flexible guideposts, and (4) cable barriers. The effects of the different types of median separation on driving behavior were statistically analyzed using data from 184 simulation tests. The results of the study suggested that the type of median separation significantly affected driving behavior on 2+1 roads. While the driving speeds on the passing lane did not differ significantly between the four configurations of median separation, the lateral position of the passing vehicle on the additional lane was found to be significantly influenced by the type of separation, with a greater distance from the median recorded when the cable barriers were implemented.

Left-turn traffic at unsignalized T-intersection on undivided rural two-lane high-speed highways poses both operational and safety challenges. More complexities are faced by through drivers in the same direction as the stopped or slowed down left-turn vehicle, who must choose to either slow down and wait or bypass the left-turn vehicle. Therefore, a recent study (*69*) examined the operational characteristics of these facilities. The focus was on the reaction of the drivers behind the left-turn vehicle in terms of the types of maneuvers taken to avoid collision and the distance upstream for the evasive maneuvers using field observations. Further, the impact of the drivers' reaction on the intersection delay was assessed using simulation analysis of 17 generic 10.5-mi two-lane corridors with varying configurations of passing lanes at or near the intersection with and without a left-turn lane. The field observation findings from five sites revealed that drivers moved to the shoulder to avoid slowing and stopping or colliding with the left-turn vehicle. The distance at which drivers moved to the shoulder differed for the sites studied. The simulation results showed that a relatively similar magnitude of reduction in intersection delay was achieved through the addition of either a passing lane or a left-turn lane. Such an addition is beneficial for intersections with volumes of at least 17,000 vpd and where the passing lane does not end within 1,500 ft downstream of the intersection.

The retrofitting of some road sections by adding a passing relief lane can improve traffic performance by decreasing platoons and driver delays and increasing speeds. Nevertheless, the effects of this measure on safety may be controversial. With higher traffic speeds, the diverging and merging conflicts may escalate and deteriorate the safety conditions of the treated sites. Cafiso et al. (56) aimed to address this dilemma by presenting an operations and safety study based on experimental data from two-lane rural roads. Serious (fatal-injury) crashes were considered in the estimation of the CMFs. The EB method in BA study was performed using data from 2005 to 2013, with the exclusion of 2009 when road segments were retrofitted by adding a passing relief lane. Certain improvements in safety were observed for both total and target crashes. The research also encompassed traffic performance by investigating the changes in speed and platoon size at the beginning and end of the treated sections. The results showed that the platoon reduction value depended on the length of the passing section and the share of heavy vehicles. The operational results proved less beneficial than expected.

Serious crashes on two-lane highways are frequently linked to passing maneuvers across the centerline. The introduction of passing lanes can mitigate the risk of crashes by offering safe passing opportunities, eliminating the necessity for passing drivers to enter the lane designated for oncoming traffic. Passing lanes help disperse traffic groups, reducing the need for subsequent passing maneuvers (*42*). The construction of passing lanes is regarded as one of the most common and cost-efficient measures for enhancing traffic safety on rural two-lane roads (*16*) and has been widely adopted by numerous states (*42*).

Dissanayake and Shams (*70*) conducted a cross-sectional study approach to compare intersections with bypass lanes to intersections with no bypass lanes for more than 1,100 intersections in Kansas. Three-legged and four-legged intersections were considered separately by looking at intersection-related crashes and crashes within the intersection box. According to the results, the number of crashes and crash severities were lower at three-legged intersections with bypass lanes compared with three-legged intersections without bypass lanes, even though these reductions were not statistically significant at the 95 percent confidence level. When considering a 300-ft intersection box, statistically significant crash reductions were observed at four-legged intersections for all considered crash and crash rate categories. At the 90 percent confidence level, crash reductions at three-legged intersections were also statistically significant when considering a 300-ft intersection box. The CMFs calculated to evaluate safety effectiveness of bypass lanes at unsignalized rural intersections in Kansas showed values less than 1.0 for almost all cases, indicating safety benefits of bypass lanes. According to this study, it was beneficial to continue with the practice of adding shoulder bypass lanes at rural unsignalized intersections on two-lane roads where the traffic volumes are relatively low.

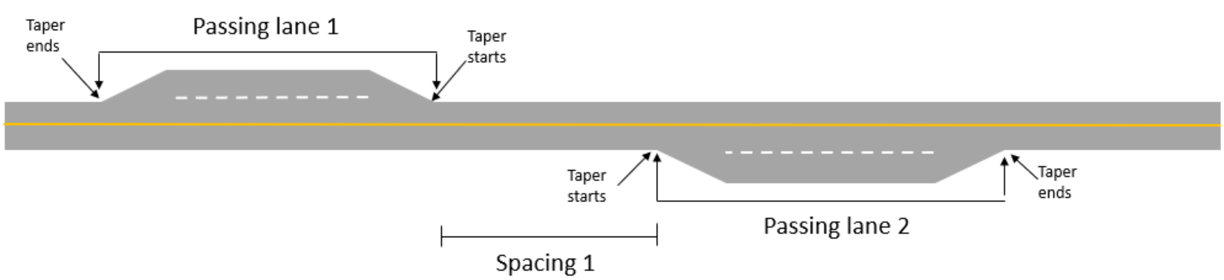
A study by Srinivasan et al. (*71*) used data from rural two-lane roads in Arizona to address two important issues. The first issue concerned the significance of selecting an appropriate sample size for calibrating the HSM predictive models based on the desired accuracy of the calibration factor, rather than relying on the guidance from the HSM. Notably, the HSM recommends using 30 to 50 sites with at least 100 crashes per year for calibration, but it does not provide a statistical basis for this recommendation. The second issue pertained to the usefulness of estimating calibration functions when individual calibration factors did not properly fit the local data. Based on the findings related to these two issues, this study recommended a simple calibration function for predicting total crashes on rural two-lane roads in Arizona. Additionally, the researchers provided a brief overview of a procedure in Microsoft Excel that can be employed by practitioners (after receiving appropriate training) to estimate simple calibration functions.

CHAPTER 3: DATA COLLECTION

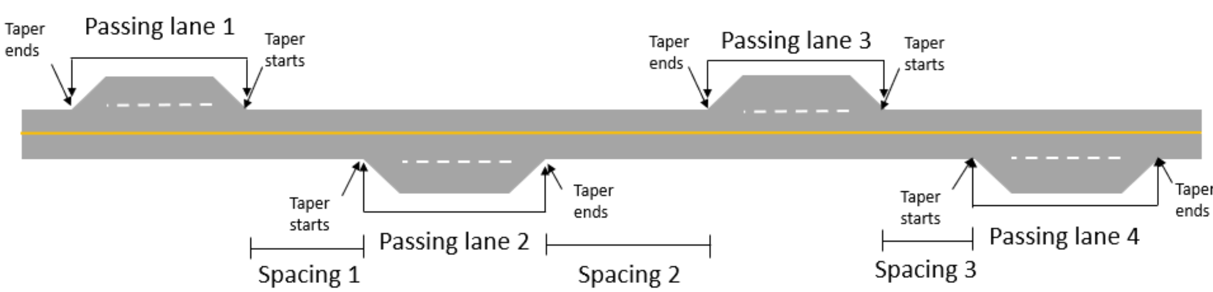
This chapter summarizes the research team's efforts to identify study sites and collect the necessary data at those sites for analysis.

SITE IDENTIFICATION

The first step in the data collection process involved the identification of the Super 2 corridors. The research team used the data from previous projects and TxDOT's Roadway Highway Inventory Network Offload (RHINO) database to identify the corridors. From the investigation, researchers found that the TxDOT Project 0-7035 data (*72*) included the most comprehensive database of Super 2 sections. The research team used that database as the starting point and developed a complete list of Super 2 corridors in the state. Researchers identified 84 corridors through this initial effort; among these corridors, 9 had only one passing lane in one direction. These 9 corridors were excluded from consideration because they perform as an isolated passing lane rather than as a Super 2 corridor. The remaining 73 corridors were categorized into three types: (1) alternating passing lanes separated by a spacing (Figure 8), (2) side-by-side passing lanes (Figure 9), and (3) passing lanes that are a combination of the previous two types (Figure 10).

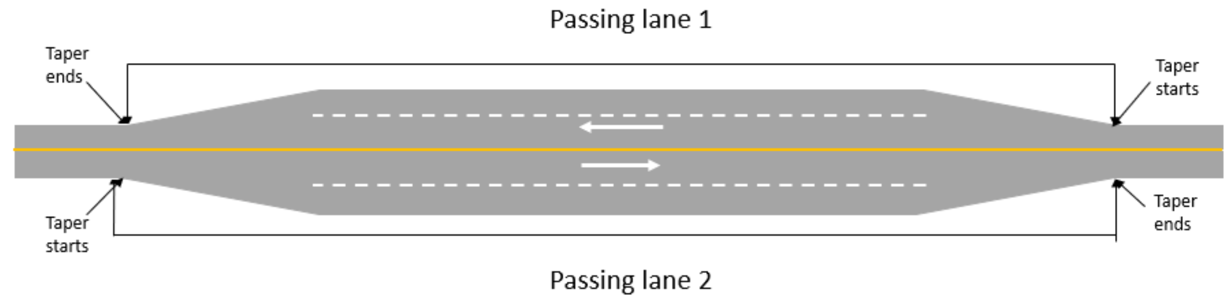


(a) Type A1: One alternating passing lane pair

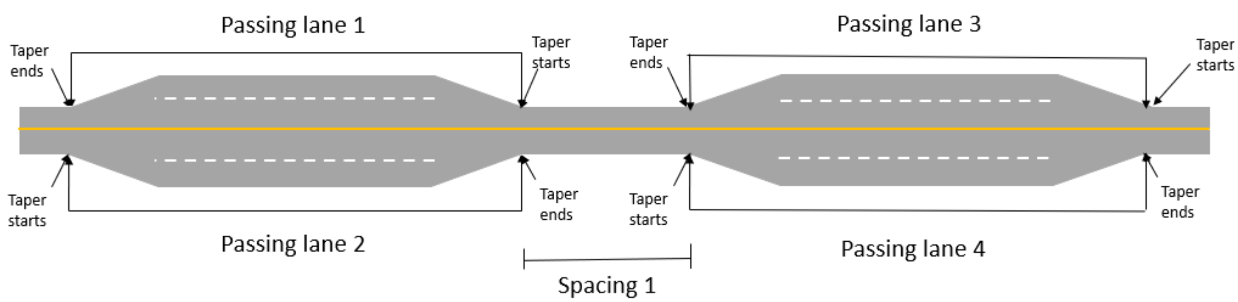


(b) Type A2: Two alternating passing lane pairs

Figure 8. Super 2 Corridors with Alternating Passing Lanes.

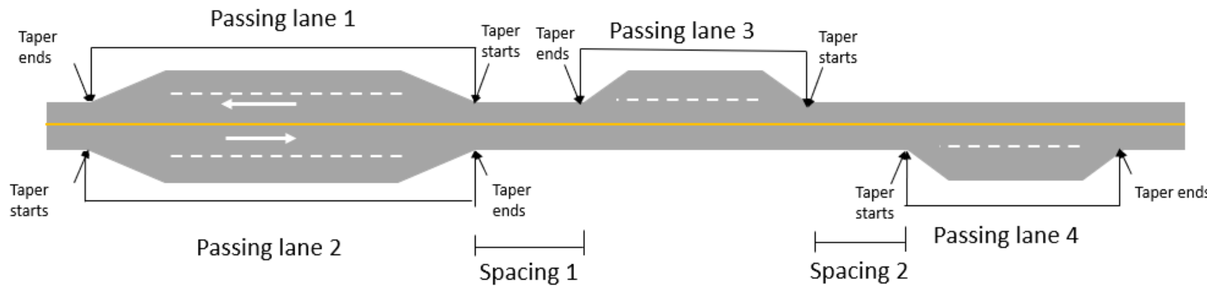


(a) Type S1: One side-by-side passing lane pair



(b) Type S2: Two side-by-side passing lane pairs

Figure 9. Super 2 Corridors with Side-by-Side Passing Lanes.



Type S1-A1: One side-by-side and one alternating passing lane pair

Figure 10. Super 2 Corridors with a Combination of Two Passing Lane Types.

These passing lane types were used as a framework for identifying and categorizing potential study sites during the data collection process. Researchers reviewed individual corridors to confirm Super 2 presence and obtain additional site characteristics. Based on this review, researchers removed or redefined several corridors, resulting in the combination of some corridors and the subdivision of other corridors. For the final analysis, a total of 67 corridors were selected. Table 13 summarizes the number of corridors and their mileage by passing lane type after this review.

Table 13. Number of Corridors and Mileage Used in the Analysis by Passing Lane Type.

Passing Lane Type	Number of Corridors	Total Length (mi)
Alternating	59	587.61
Side-by-side	6	25.27
Combination	2	43.98
Total	67	656.86

After identifying the final corridors, the next step involved confirming or remeasuring the lengths of the passing lanes and the distances between them. Table 14 presents summary statistics for the 67 study site corridors, including the minimum, maximum, and average lengths and spacings of the passing lanes.

Table 14. Study Site Corridor Lengths and Spacing by Passing Lane Type.

PL[a] Type	PL Pairs	Corridors	Segments	Min. PL Length (mi)	Max. PL Length (mi)	Avg. PL Length (mi)	Min. Spacing (mi)	Max. Spacing (mi)	Avg. Spacing (mi)
Alternating	1	12	24	0.38	2.24	1.78	0	3.74	1.38
Alternating	1.5	10	30	0.54	3.44	1.81	0	2.87	0.69
Alternating	2	13	52	0.16	3.23	1.48	0	7.02	1.19
Alternating	> 2	19	148	0.27	2.86	1.61	0	2.92	0.78
Side-by-side	1	4	8	0.93	1.39	1.24	N/A[b]	N/A	N/A
Side-by-side	> 1	2	10	1.64	2.26	1.91	N/A	N/A	N/A
Combination	Any	2	16	0.42	2.87	1.72	0	3.63	1.26

[a]PL = passing lane.
[b]N/A = not applicable.

Data Collection for Segments

After finalizing the corridors, the research team collected roadway geometry, traffic volume, and other relevant information using roadway inventory data and Google Earth. In Google Earth, each corridor was initially divided into segments (Figure 11). In Figure 11, the “Dir1” text and red line indicates one direction of the passing lane, the “Dir2” text and green line indicates the opposite direction of the passing lane, and the “Sp” text and yellow line indicates the spacing between the two passing lanes.

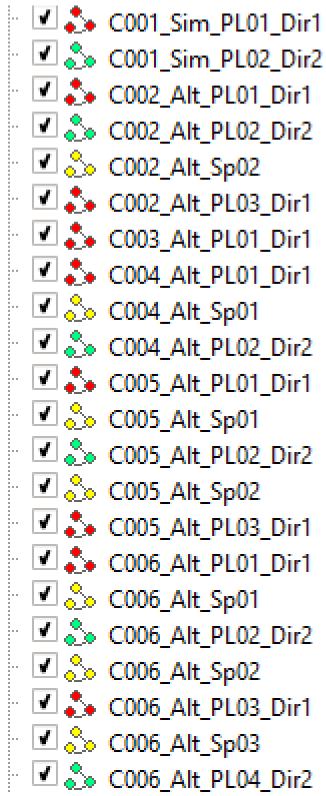


Figure 11. Segmentation of Super 2 Corridors.

Figure 12 shows the street view of a typical Super 2 corridor, and Figure 13 shows a map of the selected corridors for analysis. Figure 14 shows an example segmented corridor, where red indicates one direction of the passing lane, green indicates the opposite direction of the passing lane, and yellow indicates the spacing between the two passing lanes.

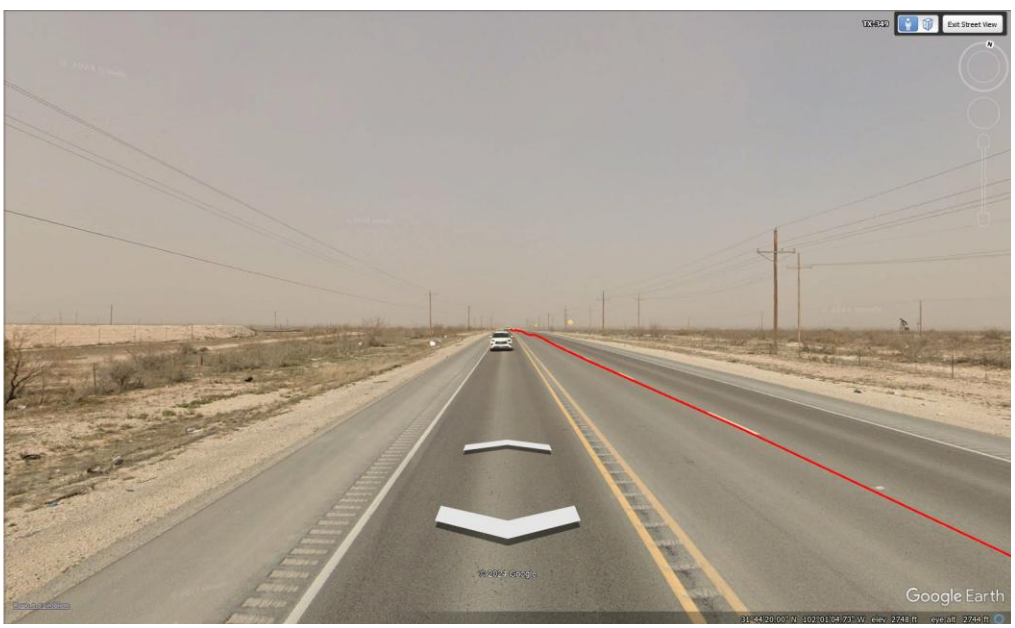


Figure 12. Street View of a Typical Super 2 Corridor.

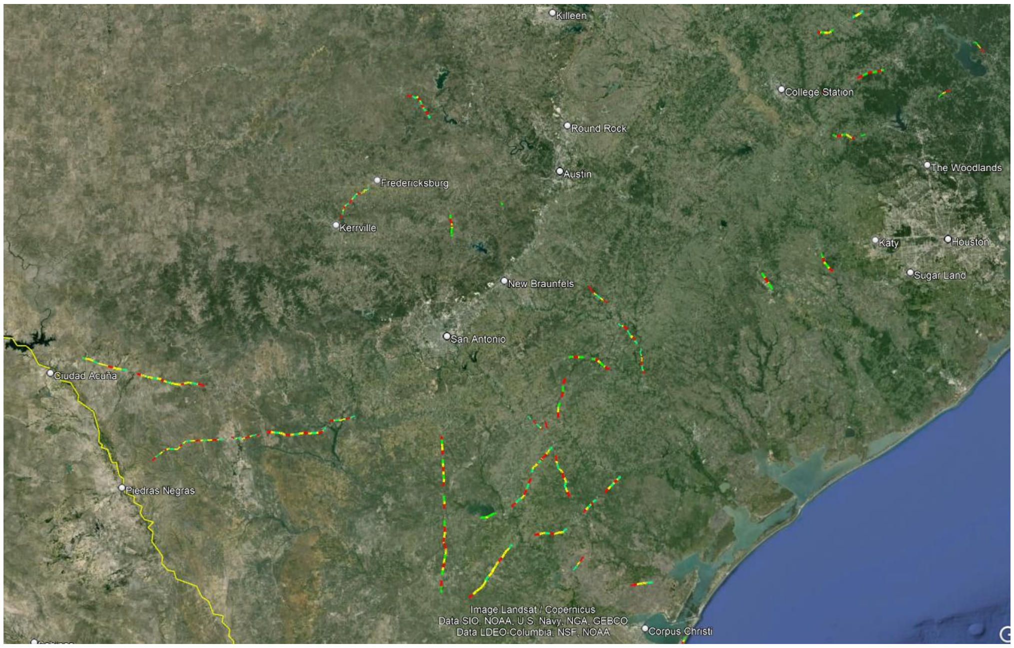


Figure 13. Map View of Super 2 Corridors Selected for Analysis.

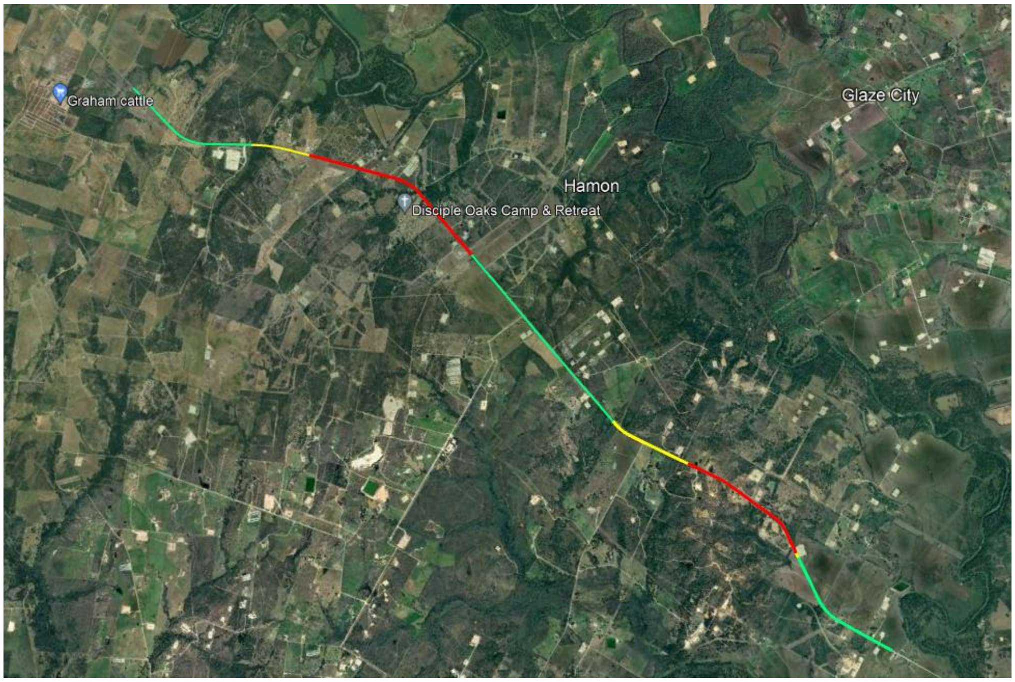


Figure 14. Map View of an Example Segmented Super 2 Corridor.

Researchers collected 10 specific site characteristics from each of these sites, for every passing lane and for the spacing between each passing lane. These characteristics included the following:

- **Problem_flag:** Notes whether the Google Earth photo quality was poor, or Street View was not available.

- **Lane_width (ft):** Average lane width, determined by measuring the surface width excluding shoulders and then dividing by the number of lanes.
- **Shld_width (ft):** Average of two shoulder widths, measured from edge of the travel lanes to the edge of pavement, excluding gravel.
- **3L_minor_int:** Number of three-legged intersections with minor roads along the segment.
- **4L_minor_int:** Number of four-legged intersections with minor roads along the segment.
- **Driveway counts:** Number of driveways in each of the following categories:

    - **Dway_res_undev_full:** Full residential/undeveloped driveways.
    - **Dway_ind_full:** Full industrial driveways.
    - **Dway_com_bus_full:** Full commercial/business driveways.
    - **Dway_office_full:** Full office driveways.
    - **Dway_other_full:** Full other type driveways.
    - **Dway_res_undev_part:** Partial residential/undeveloped driveways.
    - **Dway_ind_part:** Partial industrial driveways.
    - **Dway_com_bus_part:** Partial commercial/business driveways.
    - **Dway_office_part:** Partial office driveways.
    - **Dway_other_part:** Partial other type driveways.

    Full driveways allow both left and right turns, while partial driveways allow only right turns.

- **Adjacent land use:** Research team's estimate of adjacent land use along the corridor, using the land use categories described in Table 15.
- **Shld_rumble:** Presence of shoulder rumble strips. Figure 15 shows a typical road segment with both shoulder and centerline rumble strips.
- **Center_rumble:** Presence of centerline rumble strips.
- **Speed_limit:** Posted speed limit along the corridor.
- **AADT**: Traffic volumes as AADT obtained from the TxDOT Statewide Planning Map.

To properly define the study period for each corridor, researchers identified the installation dates for each Super 2 project; Table 16 summarizes these installation dates. Because the Super 2 corridors were installed over many years, the research team collected AADT data for each segment since 2003 to provide traffic volume information for similar periods before and after installation for each corridor. This data collection was performed using ArcGIS Pro by spatially joining different years of roadway inventory data with the selected Super 2 segments.

Table 15. Adjacent Land Use Characteristics.

Land Use	Characteristics	Examples
Residential or undeveloped	• Small buildings • Small percentage of paved land • No (or very low volume) driveways • High ratio of land-use acreage to parking	• Single-family home • Undeveloped or farmland • Cemetery • Park or green-space area
Industrial	• Large and production-oriented buildings • Driveways and parking possibly designed to accommodate large trucks • Moderate driveway volume at shift-change times, otherwise low throughout the day • Moderate ratio of land-use acreage to parking	• Factory • Warehouse • Storage tanks • Farmyard with barns and machinery
Commercial business	• Larger buildings separated by convenient parking between building and roadway • Moderate driveway volume from mid-morning to early evening • Small ratio of land-use acreage to parking	• Strip commercial, shopping mall • Apartment complex, trailer park • Airport • Gas station • Restaurant
Office	• Possible multi-story buildings • Parking may be distant from the building or behind it • High driveway volume at morning and evening peak traffic hours, otherwise low • Small ratio of land-use acreage to parking	• Office tower • Public building, school • Church • Clubhouse, buildings at a park • Parking lot for 8-to-5 workers

Figure 15. Typical Super 2 Segment with Centerline and Shoulder Rumble Strips.

Table 16. Super 2 Installation Periods

PL Type	Number of PL Pairs	Installed Before 2010	Installed 2010–2020
Alternating	1	0	35
Alternating	1.5	13	32
Alternating	2	34	54
Alternating	> 2	110	137
Side-by-side	1	6	2
Side-by-side	> 1	0	14
Combination	Any	6	21

Note: PL = passing lane.

Table 17 provides summary statistics of the data collected for study sites with alternating passing lanes. Table 18 and Table 19 provide similar statistics for study sites with side-by-side passing lanes and with combinations of alternating and side-by-side passing lanes, respectively. These sites and their corresponding characteristics and crash data were used in the analysis.

Table 17. Updated Summary Statistics of Variables for Super 2 Corridors with Alternating Passing Lanes.

Segment	Variable	Minimum	Maximum	Average
Passing	Length (mi)	0.38	3.44	1.69
Passing	AADT (vpd)	774	15,061	5,134
Passing	Truck AADT (vpd)	213	2,592	1,079
Passing	Posted speed limit (mph)	50	75	71
Passing	Minor intersections: Three-legged	0	3	0.05
Passing	Minor intersections: Four-legged	0	1	0.02
Passing	Full driveways: Undeveloped	0	9	0.76
Passing	Full driveways: Industrial	0	2	0.07
Passing	Full driveways: Business	0	2	0.03
Passing	Full driveways: Office	0	1	0.01
Passing	Full driveways: Other	0	0	0
Passing	Partial driveways: Undeveloped	0	53	6.90
Passing	Partial driveways: Industrial	0	9	0.64
Passing	Partial driveways: Business	0	10	0.24
Passing	Partial driveways: Office	0	5	0.17
Passing	Partial driveways: Other	0	0	0
Passing	Rumble strips: Shoulder	0	1	0.69
Passing	Rumble strips: Center	0	1	0.58

Segment	Variable	Minimum	Maximum	Average
Spacing	Length (mi)	0	7.02	1.08
Spacing	AADT (vpd)	774	15,061	4,994
Spacing	Truck AADT (vpd)	213	2,592	1,054
Spacing	Posted speed limit (mph)	35	75	71
Spacing	Minor intersections: Three-legged	0	3	0.12
Spacing	Minor intersections: Four-legged	0	1	0.04
Spacing	Full driveways: Undeveloped	0	9	0.91
Spacing	Full driveways: Industrial	0	3	0.07
Spacing	Full driveways: Business	0	1	0.006
Spacing	Full driveways: Office	0	1	0.018
Spacing	Full driveways: Other	0	0	0
Spacing	Partial driveways: Undeveloped	0	24	2.96
Spacing	Partial driveways: Industrial	0	3	0.20
Spacing	Partial driveways: Business	0	5	0.13
Spacing	Partial driveways: Office	0	1	0.04
Spacing	Partial driveways: Other	0	0	0
Spacing	Rumble strips: Shoulder	0	1	0.68
Spacing	Rumble strips: Center	0	1	0.57

Table 18. Updated Summary Statistics of Variables for Super 2 Corridors with Side-by-Side Passing Lanes.

Segment	Variable	Minimum	Maximum	Average
Passing	Length (mi)	0.93	2.26	1.72
Passing	AADT (vpd)	614	7,547	3,019
Passing	Truck AADT (vpd)	51	1,580	411
Passing	Posted speed limit (mph)	50	75	69
Passing	Minor intersections: Three-legged	0	0	0
Passing	Minor intersections: Four-legged	0	1	0.11
Passing	Full driveways: Undeveloped	0	2	0.22
Passing	Full driveways: Industrial	0	0	0
Passing	Full driveways: Business	0	0	0
Passing	Full driveways: Office	0	0	0
Passing	Full driveways: Other	0	0	0
Passing	Partial driveways: Undeveloped	1	15	3.89
Passing	Partial driveways: Industrial	0	1	0.11

Segment	Variable	Minimum	Maximum	Average
Passing	Partial driveways: Business	0	1	0.11
Passing	Partial driveways: Office	0	0	0
Passing	Partial driveways: Other	0	0	0
Passing	Rumble strips: Shoulder	0	1	0.78
Passing	Rumble strips: Center	0	1	0.11
Spacing	Length (mi)	1.75	3.04	2.71
Spacing	AADT (vpd)	892	3,333	2,723
Spacing	Truck AADT (vpd)	247	435	388
Spacing	Posted speed limit (mph)	70	75	74
Spacing	Minor intersections: Three-legged	0	0	0
Spacing	Minor intersections: Four-legged	0	0	0
Spacing	Full driveways: Undeveloped	2	5	3.5
Spacing	Full driveways: Industrial	0	0	0
Spacing	Full driveways: Business	0	0	0
Spacing	Full driveways: Office	0	0	0
Spacing	Full driveways: Other	0	0	0
Spacing	Partial driveways: Undeveloped	0	0	0
Spacing	Partial driveways: Industrial	0	0	0
Spacing	Partial driveways: Business	0	0	0
Spacing	Partial driveways: Office	0	0	0
Spacing	Partial driveways: Other	0	0	0
Spacing	Rumble strips: Shoulder	1	1	1
Spacing	Rumble strips: Center	0	1	0.5

Table 19. Updated Summary Statistics of Variables for Super 2 Corridors with Combinations of Passing Lane Types.

Segment	Variable	Minimum	Maximum	Average
Passing	Length (mi)	0.42	2.87	1.72
Passing	AADT (vpd)	3,530	7,169	4,836
Passing	Truck AADT (vpd)	776	1,178	920
Passing	Posted speed limit (mph)	60	75	68
Passing	Minor intersections: Three-legged	0	1	0.06
Passing	Minor intersections: Four-legged	0	0	0
Passing	Full driveways: Undeveloped	0	1	0.125
Passing	Full driveways: Industrial	0	0	0

Segment	Variable	Minimum	Maximum	Average
Passing	Full driveways: Business	0	0	0
Passing	Full driveways: Office	0	0	0
Passing	Full driveways: Other	0	0	0
Passing	Partial driveways: Undeveloped	0	17	3.75
Passing	Partial driveways: Industrial	0	1	0.063
Passing	Partial driveways: Business	0	0	0
Passing	Partial driveways: Office	0	1	0.063
Passing	Partial driveways: Other	0	0	0
Passing	Rumble strips: Shoulder	0	1	0.69
Passing	Rumble strips: Center	0	1	0.81
Spacing	Length (mi)	0	3.63	1.26
Spacing	AADT (vpd)	3,723	7,036	4,662
Spacing	Truck AADT (vpd)	794	1,168	902
Spacing	Posted speed limit (mph)	60	75	68
Spacing	Minor intersections: Three-legged	0	0	0
Spacing	Minor intersections: Four-legged	0	0	0
Spacing	Full driveways: Undeveloped	0	6	1.63
Spacing	Full driveways: Industrial	0	1	0.09
Spacing	Full driveways: Business	0	0	0
Spacing	Full driveways: Office	0	0	0
Spacing	Full driveways: Other	0	0	0
Spacing	Partial driveways: Undeveloped	0	6	2.45
Spacing	Partial driveways: Industrial	0	1	0.09
Spacing	Partial driveways: Business	0	2	0.18
Spacing	Partial driveways: Office	0	0	0
Spacing	Partial driveways: Other	0	0	0
Spacing	Rumble strips: Shoulder	0	1	0.73
Spacing	Rumble strips: Center	0	1	0.73

REFERENCE SEGMENT INFORMATION

For the statistical analysis, reference sites were compared to the Super 2 sites. Reference segments should be located on roadways with similar characteristics to the study corridors before they were converted to Super 2 roadways. Ideally, reference segments should be under consideration for conversion to Super 2 highways when traffic conditions and volumes are warranted.

Selection Process for Reference Segments

Step 1: Select Candidate Segments Warranting Conversion to Super 2 Configurations.

For this project, the research team identified over 220 projects on two-lane corridors that TxDOT has scheduled to be converted to Super 2 roadways within the current 10-year planning cycle. The team downloaded project data from the TxDOT Transportation Planning and Programming Division's planning data repository, Project Tracker.

Step 2: Augment Reference Segment Candidate Data with Roadway Inventory Information.

Data from Project Tracker does not include several fields important for the study such as traffic data, speed limits, and pavement widths. Therefore, the team sorted project records by control and section number, compared the control and section limits against TxDOT's current RHINO database of roadway information, and merged additional necessary data (e.g., traffic volumes, speed limits, pavement types, etc.) with the reference segment data.

Step 3: Redefine Limits of Reference Segment Candidates.

The team redefined the limits of each project to create roadway segments where speed limits were constant, where only two lanes existed, and where overall roadway geometry did not vary.

Step 4: Remove Incompatible Reference Segment Candidates.

The team compared the redefined reference segment candidates against the study segments for compatibility. Candidate segments were removed from consideration when they could not be redefined to create a sufficient length of constant roadway conditions (the targeted minimum length for this project was about 2 mi), or when the segment limits could not be aligned to limits used to define roadway data in RHINO or crash data from TxDOT's Crash Records Information System (CRIS). Steps 2 and 3 resulted in the removal of over 50 segments, leaving a total of 169 initial reference segments.

Step 5: Remove Outlier Reference Segments.

As the team assessed crash data and traffic characteristics further, they removed additional reference segments where general boundary conditions were not consistent with study segment boundary conditions. Ultimately, researchers identified 165 corridors, with a total length of 1,466.50 mi, for use as reference segments.

Researchers documented the final selection of reference segments into a GIS format, using the revised project limits to create KMZ Google Map files for each reference segment. These files were automatically generated using the *Event* function on the TxDOT Transportation Planning and Programming Division's Statewide Planning site, ensuring compatibility with TxDOT systems.

Summary of Reference Segments

The reference segments were combined with TxDOT's RHINO database using the spatial attributes to obtain the traffic data. For the access point and geometric data, the research team used Google Earth aerial and Street View images and data. These data included lane and shoulder widths, number of minor intersections, driveway frequency by type, rumble strip presence, and posted speed limit, as collected for the Super 2 corridors. Table 20 provides the summary statistics of variables for these reference sites.

Table 20. Summary Statistics of Variables for Reference Segments.

Variable	Minimum	Maximum	Average
Length (mi)	1.02	29.13	9.05
AADT (vpd)	471	8,411	2,944.1
Truck AADT (vpd)	108	3,309	752.45
Posted speed limit (mph)	55	75	72
Minor intersections: Three-legged	0	23	5.19
Minor intersections: Four-legged	0	14	2.13
Full driveways: Undeveloped	1	136	24.6
Full driveways: Industrial	0	69	11.31
Full driveways: Business	0	11	0.733
Full driveways: Office	0	3	0.219
Full driveways: Other	0	2	0.071
Partial driveways: Undeveloped	0	2	0.017
Partial driveways: Industrial	0	0	0
Partial driveways: Business	0	0	0
Partial driveways: Office	0	0	0
Partial driveways: Other	0	1	0.023
Rumble strips: Shoulder	0	1	0.609
Rumble strips: Center	0	1	0.571

CRASH DATA

The research team obtained crash data from TxDOT's CRIS from January 2003 to December 2023. The earliest year of crash data that the team had access to was 2003. Because it is recommended to consider three years of crash data before the construction of Super 2 highways in the BA analysis, the team considered only the highways that were constructed in 2006 or later. The CRIS data elements are divided into three major groups: (1) crash event and roadway characteristics, (2) primary person characteristics, and (3) vehicle (unit) characteristics. CRIS includes over 150 fields that contain data about spatial and temporal characteristics (e.g., time, date, and geodesic coordinates); roadway and traffic characteristics (e.g., intersection-related, ADT); crash contributing factors (e.g., distracted driving, weather, lighting, pavement conditions); manner of collision (e.g., head-on, rear-end, sideswipe); crash severity; vehicle type; driver characteristics; passenger characteristics; and other information.

The research team created shapefiles of all Super 2 highways and reference sites, overlaid the crashes within the site boundaries, and spatially joined them using PostgreSQL and QGIS software.

CHAPTER 4: MODEL DEVELOPMENT

Based on the insights gained from the literature review described in Chapter 2, the research team developed a model suited for analyzing the site data from the Super 2 sites. This chapter describes those efforts.

BEFORE-AFTER ANALYSIS

The HSM, Volume 3, Part D (*35*), as well as several other studies (*36*, *37*) have employed advanced techniques to conduct BA safety analyses of countermeasures. These methods include observational BA studies and cross-sectional approaches. Each method has its strengths and weaknesses. The cross-sectional approach offers the advantage of using regression models for assessing alternative improvements in various highway sections. However, one disadvantage is that a typical regression model might not account for all potential factors—some may not be significant in the model, and others might not be measurable within the study's parameters. Factors not considered could affect the model's accuracy. In contrast, the BA approach is advantageous because it is a controlled experiment that examines differences in samples with similar characteristics, except for the treatment, reducing the potential influence of other factors.

In observational BA studies, a variety of approaches are utilized, such as the naïve, comparison group, EB, and FB methods. Independent of the method used, BA studies are usually accomplished through two tasks (*39*):

- Task 1 is to predict what the safety of a site would have been in the after period had the treatment not been implemented.
- Task 2 is to estimate the safety of the treatment at the site after implementation.

In accomplishing these two tasks, the following terms need to be explained:

- The variable π is defined as the expected number of crashes at a specific site in the after period if the treatment had not been implemented. This variable only applies for the targeted (i.e., single-vehicle run-off-road, opposite-direction, rear-end, etc.) crashes and/or their severity (i.e., fatal, incapacitating injury, PDO, etc.). The variable π is referred to as the *predicted value*.
- The variable λ is used to define the expected number of crashes in the after period (after the implementation of the treatment). The variable λ is referred to as the *estimated value*.

The effects of a treatment are estimated by comparing both variables above in the following manner:

- The decrease (or increase) in the expected number of crashes is given as $\delta = \pi - \lambda$. A positive number indicates a decrease in the expected number of crashes.

- The ratio or *index of safety effectiveness* is defined as $\theta = \lambda/\pi$. If the number of crashes analyzed is below 500 for the before period, θ needs to be adjusted using the following factor: $1 + \text{Var}\{\pi\}/\pi^2$. This adjustment is used to minimize the bias caused by a small sample size. The index of safety effectiveness can therefore be formulated as follows:

$$\theta = \frac{\lambda/\pi}{\left[1+Var\{\pi\}/\pi^2\right]} \tag{19}$$

  A value below 1.0 indicates a reduction in the number of crashes.

The next section describes the EB method in a BA analysis, which was used for this study.

BEFORE-AFTER STUDY WITH EMPIRICAL BAYES METHOD

This approach incorporates the EB method in the BA analysis to minimize the regression-to-the-mean bias (*39*, *73*). This approach supports the estimation of safety benefits at treated sites using information from reference sites. The expected crash frequency ($E[k/K]$) at a treated site is a result of the combination of the predicted crash count($E[k]$) based on the reference sites with similar traits and the crash history (K) of that site. It should be noted that the terms k and $E[k]$ are technically the same, but the latter is usually used for statistical models. Hence, for the EB method, we used $E[k]$ rather than k.

The parameter $E[k]$ was estimated from the SPFs developed using a negative binomial regression (also known as Poisson-gamma) model under the assumption that the covariates in the SPFs represent the main safety traits of the reference sites (*74*). The seven-step procedure for using the EB method in a BA analysis is described next.

Step 1: Estimate the Predicted Number of Crashes in the Before Period.

The first step in this method involves developing a new SPF or using an existing and reliable SPF. In this study, the research team developed new SPFs for total, KABC, and KABC non-intersection crashes using rural two-lane highways that were planned for Super 2 construction but were not yet built.

Using the SPFs, the number of crashes for the before period at each treatment site i ($E[k_i]$) is estimated. Next, an EB estimate of the expected number of crashes ($E[\hat{k}_i|K_i]$) before implementation of the countermeasure at each treatment site is obtained and an associated estimate of variance is calculated. The ^ symbol indicates an estimate of a variable.

The expected number of crashes ($E[\hat{k}_i|K_i]$) is estimated by combining the SPF predictions for the before period ($E[k_i]$) with the total count of crashes during the before period (K_i) as follows:

$$E[\hat{k}_i|K_i] = \hat{w}_i \cdot E[\hat{k}_i] + (1 - \hat{w}_i) \cdot K_i \tag{20}$$

The weight $\widehat{w}_i$ can be calculated as follows:

$$\widehat{w}_i = \frac{1}{1+\frac{E[\hat{k}_i]}{\phi}} \tag{21}$$

where ϕ is the inverse dispersion parameter of a negative binomial regression model (i.e., $Var[Y_i] = E[k_i] + E[k_i]^2/\phi$).

The variance of the estimate can be calculated as follows:

$$Var\left[E[\hat{k}_i|K_i]\right] = (1 - \widehat{w}_i) \cdot E\left[\hat{k}_i\middle|K_i\right] \tag{22}$$

Step 2: Calculate the After-Period to Before-Period Crash Estimate Proportion.

Using the SPFs developed in Step 1, the number of crashes in the after period at each treatment site i ($E[z_i]$) is estimated. The proportion (P_i) of the after-period crash estimate to the before period estimate was calculated as follows:

$$P_i = \frac{E[\hat{z}]_i}{E[\hat{k}]_i} \tag{23}$$

Step 3: Calculate the Expected Crashes and Estimated Variance.

Next, the expected crashes ($\hat{\pi}_i$) during the after period that would have occurred without implementing the countermeasure is calculated. The expected crashes ($\hat{\pi}_i$) can be calculated as follows:

$$\hat{\pi}_i = P_i \times E[\hat{k}_i|K_i] \tag{24}$$

The estimated variance of $\hat{\pi}_i$ can be calculated as follows:

$$Var[\hat{\pi}_i] = {P_i}^2 Var\left[E[\hat{k}_i|K_i]\right] = {P_i}^2(1 - \widehat{w}_i) \cdot E\left[\hat{k}_i\middle|K_i\right] \tag{25}$$

Step 4: Compute the Sum of Predicted Crashes over All Treated Sites.

The sum of predicted crashes for each of the treated sites can then be computed. The after-period crashes and their variances for a group of sites had the treatment not been implemented at the treated sites can be calculated as follows:

$$\hat{\pi} = \sum_{i=1}^{j} \hat{\pi}_i \tag{26}$$

Where j is the total number of sites in the treatment group, and $\hat{\pi}$ is the expected after-period crashes at all treated sites had there been no treatment, as described above.

Step 5: Compute the Sum of Observed Crashes over All Treated Sites.

For a treated site, crashes in the after period are influenced by the implementation of the treatment. The safety effectiveness of a treatment is known by comparing the observed crashes with the treatment to the expected crashes without the treatment. The actual number of after-period crashes for a group of treated sites can be calculated as follows:

$$\hat{\lambda} = \sum_{i=1}^{j} C_i \tag{27}$$

where C_i is the crash frequency during the after period at site i. The parameter $\hat{\lambda}$ is equal to the sum of the observed number of crashes at all treated sites during the after-study period.

Step 6: Calculate the Estimated Variances for the Predicted and Observed Crash Values.

The estimated variances of $\hat{\lambda}$ and $\hat{\pi}$ can be calculated as follows:

$$Var[\hat{\lambda}_i] = C_i \tag{28}$$

$$Var[\hat{\lambda}] = \sum_{i=1}^{j} Var[\hat{\lambda}_i] \tag{29}$$

$$Var[\hat{\pi}_i] = (1 - \widehat{w}_i) \cdot E[\hat{k}_i | K_i] = (1 - \widehat{w}_i) \cdot \hat{\pi}_i \tag{30}$$

$$Var[\hat{\pi}] = \sum_{i=1}^{j} Var[\hat{\pi}_i] \tag{31}$$

Based on the assumption of a Poisson distribution, the estimated variance of $\hat{\lambda}$ is assumed to be equal to C_i.

Step 7. Compute the Safety Effectiveness of the Treatment.

The index of effectiveness ($\hat{\theta}$), also referred to as the CMF, is defined as the ratio of the number of crashes with the treatment to what would have been the number of crashes without the treatment. The parameter $\hat{\theta}$ gives the overall safety effect of the treatment and can be calculated as follows:

$$\hat{\theta} = \frac{\left(\frac{\lambda}{\pi}\right)}{\left(1 + \frac{Var(\hat{\pi})}{\hat{\pi}^2}\right)} \tag{32}$$

The percentage change in the number of target crashes due to the treatment is calculated as $100(1-\hat{\theta})$. If $\hat{\theta}$ is less than 1, then the treatment has a positive safety effect. The estimated variance and standard error of the estimated safety effectiveness can be calculated as follows:

$$Var(\hat{\theta}) = \hat{\theta}^2 \frac{\left(1/L + Var(\hat{\pi})/\hat{\pi}^2\right)}{\left(1 + Var(\hat{\pi})/\hat{\pi}^2\right)^2} \quad (33)$$

$$s.e.(\hat{\theta}) = \sqrt{Var(\hat{\theta})} \quad (34)$$

The approximate 95 percent confidence interval for $\hat{\theta}$ is given by adding and subtracting $1.96 \times s.e.(\hat{\theta})$ from $\hat{\theta}$. If the confidence interval contains the value 1, then no significant effect has been observed. The lower limit of the confidence interval for the safety index is θ_L and the upper limit is θ_U. The corresponding confidence interval for the reduction in crashes is $100(1-\theta_L)$ and $100(1-\theta_U)$.

Previous studies have indicated that a BA analysis with the EB method is the most robust approach for developing CMFs and identifying hotspots (*36*, *75*, *76*, *77*). A large portion of CMFs in the CMF Clearinghouse (*78*) was developed with this approach. The safety effectiveness of many treatments has been estimated using this method; examples include Ma et al. (*79*), Persaud et al. (*80*), and Yang and Loo (*81*).

However, the EB method is not free of limitations—notably mixed safety effects and possible effects from low sample sizes. Recently, Lord and Kuo (*82*) documented the limitations of the EB method. One of those limitations is the presence of site selection bias, which is similar to regression-to-the-mean but with different effects because the sites are selected based on a known or unknown entry criterion (e.g., five crashes per year). Even with these limitations, however, the EB method is the typical method of analysis used in CMF development and the method researchers used in this study.

CHAPTER 5: RESULTS FROM ANALYSIS

In Task 3, researchers developed SPFs and conducted an EB analysis. That analysis was expanded in Task 4 to produce a series of CMFs for Super 2 passing lanes. This chapter documents these findings.

SAFETY PERFORMANCE FUNCTIONS

Separate SPFs were developed for total, KABC, and KABC non-intersection crashes using the reference sites. The database assembled for calibration included crash frequency in each year as a dependent variable and the geometric and traffic variables of each site as independent variables. Each site was repeated in the database to represent every year from 2003 to 2023. As a result, the calibration database had 21 times as many observations as the reference database. The research team considered various functional forms and interactions before finalizing the model form. All variables were included in the first model, and the variables that were significant and intuitive were retained in the final models. Indicator variables for different years were introduced in the models to capture the safety trends over time. For driveways, the research team collected data by driveway type (residential, industrial, and commercial). The research team developed an equation to convert industrial and commercial driveways into equivalent residential driveways based on traffic volumes in their previous research in TxDOT Project 0-7035 (*72*). This previous equation established the following equivalencies: 1 industrial driveway = 3 residential driveways and 1 commercial driveway = 12 residential driveways. This equation was used in this study to estimate the equivalent driveways at each site. The predicted annual crash frequency was calculated as follows:

$$N_j = N_{base} \times CMF_{sw} \times CMF_{int3} \times CMF_{int4} \times CMF_{dw} \quad (35)$$

with:

$$N_{base} = L^{b_l} \times \ e^{b_0 + b_{adt} \ln(AADT) + b_{04} I_{04} + b_{05} I_{05} + \cdots + b_{23} I_{23}} \quad (36)$$

$$CMF_{sw} = e^{b_{sw}(SW-8)} \quad (37)$$

$$CMF_{int3} = e^{b_{int3}\left(N_{int3}/L\right)} \quad (38)$$

$$CMF_{int4} = e^{b_{int4}\left(N_{int4}/L\right)} \quad (39)$$

$$CMF_{dw} = e^{b_{dw}\left(N_{dw}/L\right)} \quad (40)$$

where:

N_{base} = base predicted annual crash frequency (crashes/yr).
CMF_{sw} = shoulder width crash modification factor.

CMF_{int3} = three-legged intersection density crash modification factor.
CMF_{int4} = four-legged intersection density crash modification factor.
CMF_{dw} = driveway density crash modification factor.
L = segment length (mi).
$AADT$ = annual average daily traffic on the segment (vpd).
I_{yy} = indicator variable for year yy (I_{yy} = 1 if it is in year yy; I_{yy} = 0 otherwise).
SW = average shoulder width (ft).
N_{int3} = number of three-legged intersections.
N_{int4} = number of four-legged intersections.
N_{dw} = number of equivalent driveways.
b_i = calibration coefficient for variable i.

Modeling Results

Table 21 contains the calibrated coefficients for total crashes. The researchers considered a significance level of 5 percent for retaining the variables in the models. Selected variables that were not significant at a 5 percent level were also retained if those variables were important to the model (even if the specific value was not known with a great deal of certainty).

Table 21. Calibrated Coefficients for Total Crashes.

Coefficient	Variable	Value	Std. Dev.	*t*-statistic	*p*-value
b_0	Intercept	–6.731	0.175	–38.38	< 0.0001
b_{len}	Segment length	0.860	0.020	42.43	< 0.0001
b_{adt}	AADT	0.813	0.021	38.89	< 0.0001
b_{sw}	Average shoulder width	–0.030	0.006	–5.06	< 0.0001
b_{int3}	Three-legged intersections	0.184	0.021	8.70	< 0.0001
b_{int4}	Four-legged intersections	0.069	0.044	1.56	0.1194
b_{dw}	Equivalent driveways	0.121	0.021	5.87	< 0.0001
b_{03}	Adjustment for year 2003 (reference)	0.000	N/A	N/A	N/A
b_{04}	Adjustment for year 2004	0.000	N/A	N/A	N/A
b_{05}	Adjustment for year 2005	0.000	N/A	N/A	N/A
b_{06}	Adjustment for year 2006	0.000	N/A	N/A	N/A
b_{07}	Adjustment for year 2007	0.086	0.052	1.65	0.0991
b_{08}	Adjustment for year 2008	0.000	N/A	N/A	N/A
b_{09}	Adjustment for year 2009	–0.124	0.056	–2.21	0.0274
b_{10}	Adjustment for year 2010	0.000	N/A	N/A	N/A
b_{11}	Adjustment for year 2011	0.000	N/A	N/A	N/A
b_{12}	Adjustment for year 2012	0.143	0.052	2.76	0.0059

Coefficient	Variable	Value	Std. Dev.	*t*-statistic	*p*-value
b_{13}	Adjustment for year 2013	0.105	0.051	2.07	0.0386
b_{14}	Adjustment for year 2014	0.241	0.049	4.87	< 0.0001
b_{15}	Adjustment for year 2015	0.327	0.049	6.63	< 0.0001
b_{16}	Adjustment for year 2016	0.152	0.051	3.00	0.0027
b_{17}	Adjustment for year 2017	0.223	0.050	4.50	< 0.0001
b_{18}	Adjustment for year 2018	0.233	0.049	4.77	< 0.0001
b_{19}	Adjustment for year 2019	0.000	N/A	N/A	N/A
b_{20}	Adjustment for year 2020	0.000	N/A	N/A	N/A
b_{21}	Adjustment for year 2021	0.152	0.050	3.04	0.0024
b_{22}	Adjustment for year 2022	0.159	0.050	3.19	0.0014
b_{23}	Adjustment for year 2023	0.173	0.049	3.51	0.0004
ϕ	Inverse dispersion parameter	5.458	0.276	19.78	< 0.0001

Note: N/A = not applicable.

Figure 16 shows the safety change over time relative to 2003. The safety change was calculated for both observed crashes (dotted line) and model-predicted crashes (solid line). Note that the model also captured the change in traffic volumes over time. Both lines followed a similar trend, suggesting that the model accurately predicted the safety change over time.

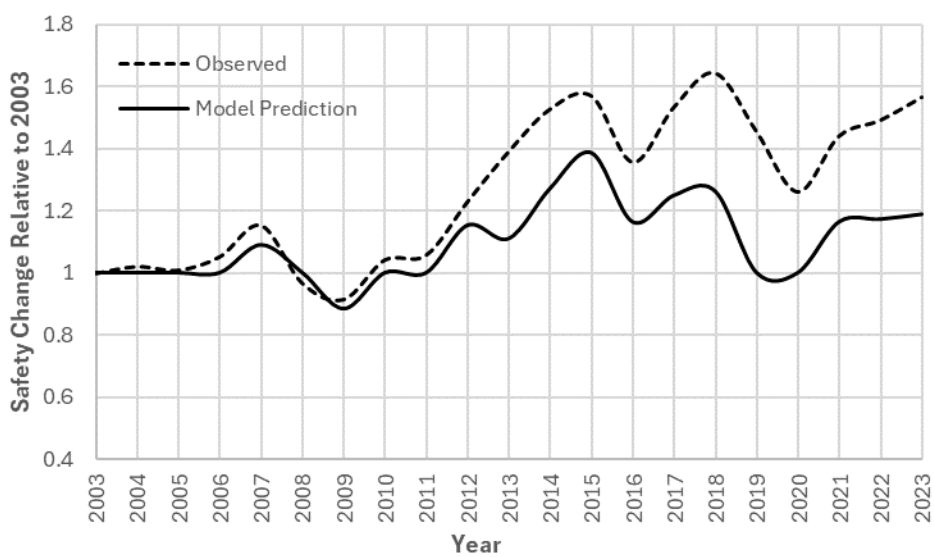


Figure 16. Safety Change Over Time.

Similar to Table 21 for total crashes, Table 22 through Table 25 contain the calibrated coefficients for all KABC crashes (Table 22), all PDO crashes (Table 23), all KABC intersection crashes (Table 24), and all KABC non-intersection crashes (Table 25).

Table 22. Calibrated Coefficients for KABC Crashes.

Coefficient	Variable	Value	Std. Dev.	*t*-statistic	*p*-value
b_0	Intercept	–7.947	0.238	–33.44	< 0.0001
b_{len}	Segment length	0.915	0.028	33.20	< 0.0001
b_{adt}	AADT	0.844	0.028	30.10	< 0.0001
b_{sw}	Average shoulder width	–0.030	0.008	–3.79	0.0002
b_{int3}	Three-legged intersections	0.171	0.028	6.20	< 0.0001
b_{int4}	Four-legged intersections	0.098	0.059	1.67	0.0956
b_{dw}	Equivalent driveways	0.157	0.026	5.95	< 0.0001
b_{03}	Adjustment for year 2003 (reference)	0.000	N/A	N/A	N/A
b_{04}	Adjustment for year 2004	0.000	N/A	N/A	N/A
b_{05}	Adjustment for year 2005	0.000	N/A	N/A	N/A
b_{06}	Adjustment for year 2006	0.000	N/A	N/A	N/A
b_{07}	Adjustment for year 2007	0.000	N/A	N/A	N/A
b_{08}	Adjustment for year 2008	–0.172	0.072	–2.38	0.0173
b_{09}	Adjustment for year 2009	–0.113	0.044	–2.58	0.0098
b_{10}	Adjustment for year 2010	–0.113	0.044	–2.58	0.0098
b_{11}	Adjustment for year 2011	–0.113	0.044	–2.58	0.0098
b_{12}	Adjustment for year 2012	0.000	N/A	N/A	N/A
b_{13}	Adjustment for year 2013	–0.118	0.066	–1.78	0.0755
b_{14}	Adjustment for year 2014	0.000	N/A	N/A	N/A
b_{15}	Adjustment for year 2015	0.000	N/A	N/A	N/A
b_{16}	Adjustment for year 2016	0.000	N/A	N/A	N/A
b_{17}	Adjustment for year 2017	0.000	N/A	N/A	N/A
b_{18}	Adjustment for year 2018	–0.173	0.042	–4.15	< 0.0001
b_{19}	Adjustment for year 2019	–0.173	0.042	–4.15	< 0.0001
b_{20}	Adjustment for year 2020	–0.173	0.042	–4.15	< 0.0001
b_{21}	Adjustment for year 2021	–0.115	0.067	–1.72	0.0847
b_{22}	Adjustment for year 2022	0.000	N/A	N/A	N/A
b_{23}	Adjustment for year 2023	0.000	N/A	N/A	N/A
ϕ	Inverse dispersion parameter	5.624	0.501	11.22	< 0.0001

Note: N/A = not applicable.

Table 23. Calibrated Coefficients for PDO Crashes.

Coefficient	Variable	Value	Std. Dev.	*t*-statistic	*p*-value
b_0	Intercept	–7.089	0.211	–33.63	< 0.0001
b_{len}	Segment length	0.840	0.024	34.69	< 0.0001
b_{adt}	AADT	0.800	0.025	31.90	< 0.0001
b_{sw}	Average shoulder width	–0.031	0.007	–4.41	< 0.0001
b_{int3}	Three-legged intersections	0.193	0.025	7.71	< 0.0001
b_{int4}	Four-legged intersections	0.064	0.053	1.21	0.2278
b_{dw}	Equivalent driveways	0.108	0.024	4.42	< 0.0001
b_{03}	Adjustment for year 2003 (reference)	0.000	N/A	N/A	N/A
b_{04}	Adjustment for year 2004	–0.081	0.068	–1.19	0.2347
b_{05}	Adjustment for year 2005	–0.119	0.068	–1.75	0.0795
b_{06}	Adjustment for year 2006	0.000	N/A	N/A	N/A
b_{07}	Adjustment for year 2007	0.106	0.064	1.65	0.0997
b_{08}	Adjustment for year 2008	0.000	N/A	N/A	N/A
b_{09}	Adjustment for year 2009	–0.170	0.070	–2.41	0.0158
b_{10}	Adjustment for year 2010	0.000	N/A	N/A	N/A
b_{11}	Adjustment for year 2011	0.000	N/A	N/A	N/A
b_{12}	Adjustment for year 2012	0.159	0.064	2.49	0.0127
b_{13}	Adjustment for year 2013	0.000	N/A	N/A	N/A
b_{14}	Adjustment for year 2014	0.304	0.060	5.07	< 0.0001
b_{15}	Adjustment for year 2015	0.416	0.060	6.98	< 0.0001
b_{16}	Adjustment for year 2016	0.188	0.062	3.05	0.0023
b_{17}	Adjustment for year 2017	0.314	0.060	5.24	< 0.0001
b_{18}	Adjustment for year 2018	0.380	0.058	6.51	< 0.0001
b_{19}	Adjustment for year 2019	0.145	0.061	2.37	0.0177
b_{20}	Adjustment for year 2020	0.137	0.062	2.20	0.0277
b_{21}	Adjustment for year 2021	0.224	0.061	3.70	0.0002
b_{22}	Adjustment for year 2022	0.261	0.060	4.34	< 0.0001
b_{23}	Adjustment for year 2023	0.270	0.060	4.54	< 0.0001
ϕ	Inverse dispersion parameter	4.409	0.249	17.70	< 0.0001

Note: N/A = not applicable.

Table 24. Calibrated Coefficients for KABC Intersection Crashes.

Coefficient	Variable	Value	Std. Dev.	*t*-statistic	*p*-value
b_0	Intercept	–12.264	0.506	–24.23	< 0.0001
b_{len}	Segment length	0.931	0.053	17.51	< 0.0001
b_{adt}	AADT	1.176	0.059	20.02	< 0.0001
b_{sw}	Average shoulder width	–0.098	0.015	–6.39	< 0.0001
b_{int3}	Three-legged intersections	0.296	0.051	5.82	< 0.0001
b_{int4}	Four-legged intersections	0.880	0.099	8.90	< 0.0001
b_{dw}	Equivalent driveways	0.245	0.047	5.20	< 0.0001
b_{03}	Adjustment for year 2003 (reference)	0.000	N/A	N/A	N/A
b_{04}	Adjustment for year 2004	0.000	N/A	N/A	N/A
b_{05}	Adjustment for year 2005	0.000	N/A	N/A	N/A
b_{06}	Adjustment for year 2006	–0.179	0.140	–1.28	0.201
b_{07}	Adjustment for year 2007	–0.143	0.140	–1.02	0.307
b_{08}	Adjustment for year 2008	–0.181	0.146	–1.24	0.214
b_{09}	Adjustment for year 2009	–0.218	0.147	–1.49	0.137
b_{10}	Adjustment for year 2010	–0.282	0.147	–1.92	0.055
b_{11}	Adjustment for year 2011	–0.153	0.139	–1.10	0.272
b_{12}	Adjustment for year 2012	0.000	N/A	N/A	N/A
b_{13}	Adjustment for year 2013	–0.184	0.130	–1.42	0.156
b_{14}	Adjustment for year 2014	0.000	N/A	N/A	N/A
b_{15}	Adjustment for year 2015	0.000	N/A	N/A	N/A
b_{16}	Adjustment for year 2016	0.000	N/A	N/A	N/A
b_{17}	Adjustment for year 2017	0.000	N/A	N/A	N/A
b_{18}	Adjustment for year 2018	–0.166	0.127	–1.30	0.192
b_{19}	Adjustment for year 2019	0.000	N/A	N/A	N/A
b_{20}	Adjustment for year 2020	–0.174	0.132	–1.32	0.188
b_{21}	Adjustment for year 2021	0.000	N/A	N/A	N/A
b_{22}	Adjustment for year 2022	0.000	N/A	N/A	N/A
b_{23}	Adjustment for year 2023	0.000	N/A	N/A	N/A
ϕ	Inverse dispersion parameter	1.765	0.187	9.45	< 0.0001

Note: N/A = not applicable.

Table 25. Calibrated Coefficients for KABC Non-intersection Crashes.

Coefficient	Variable	Value	Std. Dev.	*t*-statistic	*p*-value
b_0	Intercept	–7.381	0.263	–28.09	< 0.0001
b_{len}	Segment length	0.916	0.030	30.18	< 0.0001
b_{adt}	AADT	0.736	0.031	23.80	< 0.0001
b_{sw}	Average shoulder width	0.000	N/A	N/A	N/A
b_{int3}	Three-legged intersections	0.128	0.031	4.11	< 0.0001
b_{int4}	Four-legged intersections	0.000	N/A	N/A	N/A
b_{dw}	Equivalent driveways	0.110	0.029	3.76	0.0002
b_{03}	Adjustment for year 2003 (reference)	0.000	N/A	N/A	N/A
b_{04}	Adjustment for year 2004	0.000	N/A	N/A	N/A
b_{05}	Adjustment for year 2005	0.000	N/A	N/A	N/A
b_{06}	Adjustment for year 2006	0.000	N/A	N/A	N/A
b_{07}	Adjustment for year 2007	0.000	N/A	N/A	N/A
b_{08}	Adjustment for year 2008	0.000	N/A	N/A	N/A
b_{09}	Adjustment for year 2009	0.000	N/A	N/A	N/A
b_{10}	Adjustment for year 2010	0.000	N/A	N/A	N/A
b_{11}	Adjustment for year 2011	0.000	N/A	N/A	N/A
b_{12}	Adjustment for year 2012	0.000	N/A	N/A	N/A
b_{13}	Adjustment for year 2013	0.000	N/A	N/A	N/A
b_{14}	Adjustment for year 2014	0.000	N/A	N/A	N/A
b_{15}	Adjustment for year 2015	0.000	N/A	N/A	N/A
b_{16}	Adjustment for year 2016	0.000	N/A	N/A	N/A
b_{17}	Adjustment for year 2017	0.000	N/A	N/A	N/A
b_{18}	Adjustment for year 2018	0.000	N/A	N/A	N/A
b_{19}	Adjustment for year 2019	–0.172	0.075	–2.29	0.0222
b_{20}	Adjustment for year 2020	–0.184	0.078	–2.35	0.0190
b_{21}	Adjustment for year 2021	0.000	N/A	N/A	N/A
b_{22}	Adjustment for year 2022	0.000	N/A	N/A	N/A
b_{23}	Adjustment for year 2023	0.000	N/A	N/A	N/A
ϕ	Inverse dispersion parameter	5.073	0.521	9.74	< 0.0001

Note: N/A = not applicable.

EB Analysis

The research team used the EB method in the BA analysis to determine the safety effectiveness of Super 2 treatments. A preliminary analysis was conducted in Task 3, then researchers refined the analysis to provide more detail on crash severity, collision type, passing lane type, AADT range, and driveway density; this refined analysis added SPFs for PDO crashes and produced some modest changes from the preliminary analysis conducted in Task 3. Results from the refined analysis are provided in this section. Table 26 presents the average safety effect of Super 2 highways based on the EB method by crash severity. There were 1,575 total crashes reported during the three-year period after the construction of the Super 2 highways. The analysis results showed that if the treatment had not been installed, the expected number of the crashes would have been 1,991 crashes during the three-year after-study period. In other words, it was estimated that Super 2 highways reduced crashes by 21 percent. This result was statistically significant, and the 95 percent confidence interval was estimated to be 16 to 26 percent for total crashes. A similar result was observed separately for KABC and PDO crashes, with reductions of 16 and 23 percent, respectively.

Table 26. Results of the EB Analysis by Crash Severity.

Variables	Total Crashes	KABC Crashes	PDO Crashes
Number of segments	374	374	374
Predicted crashes ($\hat{\pi}$)	1,990.9 (39.7)	728.3 (16.3)	1,252.6 (27.9)
Observed crashes ($\hat{\lambda}$)	1,575 (40.0)	610 (24.7)	965 (31.1)
Reduction in crashes ($\hat{\delta}$)	415.9 (58.1)	118.3 (31.5)	287.6 (45.1)
Safety index ($\hat{\theta}$)	0.79 (0.02)	0.84 (0.04)	0.77 (0.03)
95% confidence interval for reduction in crashes	16–26%	9–24%	17–29%

Note: Value in parentheses is the standard error of the estimate.

Table 27 presents the results of the EB method for KABC crashes by collision location. The analysis results showed that the number of KABC intersection crashes at treated corridors decreased by 77 crashes (37 percent) compared to predicted crashes if the treatment had not been installed. Similarly, non-intersection crashes decreased by 45 crashes (9 percent). Because Super 2 passing lanes are a corridor treatment that provide an operational benefit to non-intersection locations, it would be expected that non-intersection crashes would see a greater reduction than intersection crashes; however, these results showed a greater reduction in intersection crashes. A more thorough investigation is needed to determine why the reduction is lower for non-intersection crashes.

Table 27. Results of the EB Analysis for KABC Crashes by Collision Location.

Variables	KABC Intersection Crashes	KABC Non-intersection Crashes
Number of segments	374	374
Predicted crashes ($\hat{\pi}$)	207.4 (9.8)	525.1 (12.3)
Observed crashes ($\hat{\lambda}$)	130 (11.4)	480 (21.9)
Reduction in crashes ($\hat{\delta}$)	77.4 (17.4)	45.1 (26.0)
Safety index ($\hat{\theta}$)	0.63 (0.06)	0.91 (0.05)
95% confidence interval for reduction in crashes	25–50%	–1–18%

Note: Value in the parentheses is the standard error of the estimate.

Table 28 presents the results of the EB method for KABC crashes by passing lane type. The analysis indicated that adding alternating passing lanes decreased the number of KABC crashes at those sites by 116 crashes (16 percent). Side-by-side passing lanes were also analyzed and showed only a modest reduction (1 crash or 6 percent) based on a much smaller sample size than for alternating passing lanes. The results indicated a significant reduction in crashes for alternating passing lanes, which are more commonly found in Texas. The results for side-by-side passing lanes also suggested the potential for crash reduction; however, the results were not statistically significant, probably due to smaller sample sizes.

Table 28. Results of the EB Analysis for KABC Crashes by Passing Lane Type.

Variables	Alternating Passing Lanes	Side-by-Side Passing Lanes
Number of segments	363	14
Predicted crashes ($\hat{\pi}$)	710.2 (16.1)	21.1 (2.5)
Observed crashes ($\hat{\lambda}$)	594 (24.4)	20 (4.5)
Reduction in crashes ($\hat{\delta}$)	116.2 (31.1)	1.1 (5.2)
Safety index ($\hat{\theta}$)	0.84 (0.04)	0.94 (0.23)
95% confidence interval for reduction in crashes	9–24%	–40–52%

Note: Value in the parentheses is the standard error of the estimate. Three segments contain both alternating and side-by-side passing lanes and are included in the number of segments for both types; the total number of segments is 374.

Table 29 presents the results of the EB method for KABC crashes by AADT range. Results indicated that the addition of passing lanes produced consistent crash reductions between 15 and

18 percent for all ranges of daily volumes. The sample size was insufficient to determine a lower or upper bound to the positive reduction in crashes, but these findings are consistent with previous research (*34*, *72*) that indicates the operational benefits of Super 2 passing lanes found at lower AADTs remain as volumes increase to between 10,000 and 15,000 vpd. The results for each AADT range showed no significant difference as volumes changed, and the results are also not significantly different from the CMF of 0.84 for total KABC crashes, so no formal CMF based on AADT was produced from these results.

Table 29. Results of the EB Analysis for KABC Crashes by AADT Range.

Variables	AADT (vpd)		
	< 4,000	**4,000–7,000**	**> 7,000**
Number of segments	147	153	74
Average AADT	3059	5211	9816
Predicted crashes ($\hat{\pi}$)	192.7 (7.1)	291.6 (9.8)	244.0 (10.9)
Observed crashes ($\hat{\lambda}$)	163 (12.8)	247 (15.7)	200 (14.1)
Reduction in crashes ($\hat{\delta}$)	29.7 (15.6)	44.6 (19.7)	44.0 (19.1)
Safety index ($\hat{\theta}$)	0.84 (0.07)	0.85 (0.06)	0.82 (0.07)
95% confidence interval for reduction in crashes	1–30%	3–27%	5–32%

Note: Value in the parentheses is the standard error of the estimate.

Table 30 presents the results of the EB method for KABC crashes by driveway density. Results indicated that crash reduction varied inversely with driveway density—as driveway density increased, crashes decreased, with a reduction of 37 percent for sites with no driveways and 10 percent for sites with an average driveway density of greater than 19 driveways/mi. Note that the results are not statistically significant for the categories of 10-19 and greater than 19 driveways/mi, mainly due to small sample size.

Table 30. Results of the EB Analysis for KABC Crashes by Driveway Density.

Variables	Equivalent Residential Driveway Density Group (driveways/mi)					
	1	**2**	**3**	**4**	**5**	**6**
Density lower bound	0	> 0	> 2	> 5	> 10	> 19
Density upper bound	0	≤ 2	≤ 5	≤ 10	≤ 19	N/A
Number of segments	51	88	86	68	42	39
Average driveway density	0	1.5	4.0	7.5	13.8	35.3
Predicted crashes ($\hat{\pi}$)	37.6 (2.9)	126.9 (5.8)	170.0 (7.5)	149.1 (7.3)	96.6 (5.9)	145.2 (8.9)

Observed crashes ($\hat{\lambda}$)	24 (4.9)	93 (9.6)	141 (11.9)	128 (11.3)	85 (9.2)	131 (11.4)
Reduction in crashes ($\hat{\delta}$)	13.6 (6.8)	34.0 (12.7)	29.0 (15.0)	21.1 (14.2)	11.6 (11.4)	14.2 (14.9)
Safety index ($\hat{\theta}$)	0.63 (0.14)	0.73 (0.08)	0.83 (0.08)	0.86 (0.09)	0.88 (0.11)	0.90 (0.09)
95% confidence interval for reduction in crashes	10–64%	11–43%	2–33%	-3–31%	-9–34%	-9–29%

Note: Value in the parentheses is the standard error of the estimate.
N/A = Not applicable.

CHAPTER 6: CRASH MODIFICATION FACTORS FOR SUPER 2 CORRIDORS

This chapter lists the CMFs defined in this analysis, offers guidance to users regarding their appropriate applications, and provides recommendations for updates to existing documents.

LISTING OF CRASH MODIFICATION FACTORS

The EB analyses in Tasks 3 and 4 investigated various aspects of crashes in Super 2 passing lane corridors. Results showed that the installation of passing lanes produced reductions in crashes based on a variety of metrics. Table 31 lists the CMFs generated from the EB analyses, along with their corresponding crash reduction factors (CRFs). Table 26 through Table 30 provide additional details for these CMFs.

Table 31. CMFs and CRFs for Super 2 Passing Lane Corridors.

Description	CMF	CRF (%)
Total crashes	0.79	21
KABC crashes	0.84	16
PDO crashes	0.77	23
KABC intersection crashes	0.63	37
KABC non-intersection crashes	0.91	9
KABC crashes for alternating passing lanes	0.84	16
KABC crashes for side-by-side passing lanes	0.94[a]	6[a]
KABC crashes for driveway density = 0 driveways/mi[b]	0.63	37
KABC crashes for 0 < driveway density ≤ 2 driveways/mi	0.73	27
KABC crashes for 2 < driveway density ≤ 5 driveways/mi	0.83	17
KABC crashes for 5 < driveway density ≤ 10 driveways/mi	0.86	14
KABC crashes for 10 < driveway density ≤ 19 driveways/mi	0.88	12
KABC crashes for driveway density > 19 driveways/mi	0.90	10

[a]Results are not statistically significant.
[b]Driveway density is based on the equivalent number of residential driveways, where one industrial driveway equals 3 residential driveways, and one commercial driveway equals 12 residential driveways.

USE OF CRASH MODIFICATION FACTORS

The CMFs produced in this research and taken from other sources provide practitioners with a resource for estimating the benefit of a particular treatment, such as Super 2 passing lanes. Practitioners can consider a particular treatment individually or in comparison with other treatment alternatives, but must importantly use CMFs that are applicable for the treatment and conditions being treated in any evaluation. The CMF Clearinghouse (*78*) provides guidance on effectively applying CMFs in practice, and the *Quick Start Guide to Using CMFs* (*83*) provides the following six-step process for comparing multiple treatment alternatives:

1. **Develop a list of prospective countermeasures**. Confirm that the prospective countermeasures address the types of crashes that are occurring.
2. **Identify all relevant CMFs for each prospective countermeasure**. Review resources such as state-specific lists of CMFs and the CMF Clearinghouse (*78*) to develop a list of relevant CMFs for each prospective countermeasure.
3. **Confirm CMF applicability**. Prioritize the list of CMFs based on their relevance to the characteristics of the project site.
4. **Select a single CMF for each countermeasure**. Choose the most appropriate CMF from the list.
5. **Apply the CMF**. Use the CMF to predict the expected number of crashes after the countermeasure is implemented.
6. **Select countermeasure**. Compare crash reductions for all countermeasures considered, and use this comparison to select a countermeasure for the location.

Potential CMFs in step 2 can be found in both the CMF Clearinghouse (*78*) and in research such as this project's findings. Multiple CMFs may have applicability for a given potential treatment or set of alternatives, so it is important for practitioners to consider which CMFs are most appropriate. For example, Table 31 provides 23 CMFs related to Super 2 passing lanes. If the details of a potential treatment corridor are not well known, the *total crashes* CMF could be considered as a general guide, but if the corridor has a particular issue with intersection crashes, or if driveway density is low, then those respective CMFs could also be considered. Thus, identifying the relevant CMFs requires compiling the appropriate details (related to crash history, operational performance, and design considerations, among other factors) of the corridor in question, so that practitioners can better understand which CMFs are most applicable.

Chapter 5.4 of the TxDOT *Traffic and Safety Analysis Procedures* (TSAP) *Manual* (*84*) provides additional guidance on the identification and selection of appropriate CMFs for TxDOT construction projects. In Section 5.4.4, the TSAP Manual notes that the primary goal is to select a CMF that was developed under the same (or very similar) conditions as the location for which it is being applied. When selecting CMFs, the TSAP Manual advises to consider the following characteristics:

- Countermeasure type.
- Crash type.
- Crash severity.
- Roadway or intersection type.
- Area type (rural vs. urban).
- AADT ranges.

- Prior conditions.
- Similarity to locality where data is used.

If multiple CMFs are available and appear to be similar or equally applicable, the TSAP Manual (*84*) suggests that the following items related to the equality of the CMFs be reviewed to determine the best possible match:

- Star rating.
- Score details.
- Age of data or study.

The TSAP Manual (*84*) also notes that it is critical that sound engineering or professional judgment be used when selecting CMFs and refers readers to the CMF Clearinghouse. The manual further advises that, when selecting CMFs for use in analysis from the CMF Clearinghouse, the CMF IDs are recommended to be documented, and the CMF detail summary page can be provided. This information can be saved as a PDF and included as an appendix or attachment to the analysis documentation.

SELECTION OF ALTERNATIVES

A practitioner considering whether to install a Super 2 corridor on a rural two-lane highway should first consider available alternatives and determine which of those alternatives is best for that location. The *Quick Start Guide to Using CMFs* (*83*) provides additional details and scenarios for selecting and applying CMFs when selecting a treatment for a given situation. The CMF Clearinghouse (*78*) also provides additional resources for applying CMFs in practice and conducting roadway safety analyses. Even in their examples, however, CMFs are just one component of a decision-making process for selecting an appropriate treatment. From a safety standpoint, Super 2 passing lanes provide crash reductions at existing installations, as found in this research. However, the expected crash reduction is not the only consideration. Practitioners must also consider operations, design guidelines, and economic considerations, among other factors. For example, TxDOT Project 0-7035 (*72*) produced a set of guidelines for selecting appropriate Super 2 cross-sections based on both safety and operational performance. For example, the preferred cross-section for a highway with an AADT ≤ 15,000 vpd and a driveway density > 30 driveways/mi is a Super 2 + TWLTL. In this example, the higher driveway density results in a higher number of turning movements to and from the highway, increasing the benefit of a cross-section with a TWLTL. As AADT increases above 15,000, the favorability of a four-lane cross-section (with either a TWLTL or a median buffer) also increases, because those higher volumes approach or exceed the capacity of a Super 2 corridor. Thus, it is important to consider crashes in connection with operational needs and design factors when considering design treatments for implementation.

The resource document *Guidelines for Implementing Super 2 Corridors in Texas* (*85*) describes how to consider operational, safety, and economic measures when selecting a particular alternative, with an emphasis on the features of Super 2 passing lanes. The TxDOT *Roadway Design Manual* (*2*) provides design guidelines for Super 2 highways in Chapter 4, Section 6. These guidelines contain a list of issues to consider when designing a Super 2 project. These issues should be considered in addition to crash reduction when determining what treatment to implement on a particular corridor. Consideration of these supplemental operations, safety, design, and economics measures will provide a better basis for the decision-making process.

UPDATES TO EXISTING DOCUMENTS

The results from this research are consistent with the results from previous research that led to the development of existing guidance documents; however, the findings from this research can be used to update certain details in the existing guidance documents to provide more comprehensive and well-informed guidance. This section provides suggestions for updates to existing guidance documents that reflect the findings from this project.

TxDOT Roadway Design Manual

The 2022 edition of the TxDOT *Roadway Design Manual* (*2*) was in effect at the beginning of this research project and provided the basis for guidance on the design of Super 2 corridors. Findings from this and previous projects generated content suitable for recommending updates to the 2022 *Roadway Design Manual* to provide more comprehensive guidance to practitioners. TxDOT had an ongoing process to revise the 2022 *Roadway Design Manual* during this research project, and the research team coordinated with the revision team to suggest and incorporate appropriate updates into the Super 2 design guidance found in the manual. Just prior to the completion of this project, the November 2024 edition of the TxDOT *Roadway Design Manual* (*86*) was released, and Section 10.6 of the new manual contains updated guidance on the design of Super 2 corridors. Members of the research team reviewed the content of Section 10.6 and confirmed that the updated guidance in that section includes recommended updates from previous projects as well as updates that would be suggested from this project, including additional references to guidance from previous research, such as *Guidelines for Implementing Super 2 Corridors in Texas* (*85*). As a result, the relevant findings and corresponding updates from this project have already been incorporated into the current manual and no additional updates to the TxDOT *Roadway Design Manual* are suggested from this research project.

Guidelines for Implementing Super 2 Corridors in Texas

Chapter 2 of the *Guidelines for Implementing Super 2 Corridors in Texas* (*85*) provides guidance on the selection of alternatives, including a section on Safety Measures. Based on the findings from this project and previous research, the research team recommends the following updates to that section of the document, with underlined text indicating additions to existing text and ~~strikethrough text~~ representing suggested deletions:

Safety Measures

While the fundamental benefits of a Super 2 emphasize operational measures of effectiveness, safety benefits exist as well, because drivers are less likely to execute a passing maneuver in a two-lane section of the corridor. Depending on the traffic characteristics of the site in question, a Super 2 can also provide safety benefits that should be considered when determining what specific design alternative to select in an improvement project.

Previous research in Texas (*32*) showed that that the installation of passing lanes on the corridors that were studied led to a statistically significant crash reduction of 35 percent for KABC segment-only crashes and 42 percent for KABC segment and intersection crashes. This finding is consistent with findings of previous safety-related studies of Super 2 corridors, which show improvements in safety from the installation of passing lanes, even at traffic volumes higher than those considered under previous guidance in Texas.

More recent research in Texas (*87*) included a more comprehensive investigation into the crash reduction potential of Super 2 passing lanes. Results showed that the installation of passing lanes produced reductions in crashes between 6 and 37 percent based on a variety of metrics. Table 32 summarizes the crash modification factors (CMFs) generated from the empirical Bayes analyses, along with their corresponding crash reduction factors (CRFs).

Table 32. CMFs and CRFs for Super 2 Passing Lane Corridors (*87*).

Description	**CMF**	**CRF (%)**
Total crashes	0.79	21
KABC crashes	0.84	16
PDO crashes	0.77	23
KABC intersection crashes	0.63	37
KABC non-intersection crashes	0.91	9
KABC crashes for alternating passing lanes	0.84	16
KABC crashes for side-by-side passing lanes	0.94[a]	6[a]
KABC crashes for driveway density = 0 driveways/mi[b]	0.63	37
KABC crashes for 0 < driveway density ≤ 2 driveways/mi	0.73	27
KABC crashes for 2 < driveway density ≤ 5 driveways/mi	0.83	17
KABC crashes for 5 < driveway density ≤ 10 driveways/mi	0.86	14
KABC crashes for 10 < driveway density ≤ 19 driveways/mi	0.88	12
KABC crashes for driveway density > 19 driveways/mi	0.90	10

[a]Results are not statistically significant.
[b]Driveway density is based on the equivalent number of residential driveways, where one industrial driveway equals 3 residential driveways, and one commercial driveway equals 12 residential driveways.

A combination of data from studies in other states (*62, 63*) produced a ~~crash modification factor (~~CMF~~)~~ for a conventional passing or climbing lane added in one direction of travel on a two-lane highway of 0.75 (i.e., a 25 percent reduction) for total crashes in both directions of travel over the length of the passing lane from the upstream end of the lane addition taper to the downstream end of the lane drop taper. This CMF assumed that the passing lane is operationally warranted and that the length of the passing lane is appropriate for the operational conditions on the roadway.

In addition to the crash reductions documented in the Texas research (*32, 87*), the CMF Clearinghouse (*78*) provides results from other studies with similar reductions in crashes. A search on "passing lane" in the CMF Clearinghouse produces results ranging from 7 to 42 percent reductions in crashes, depending on the type of crash (e.g., roadway departure, head-on, etc.) or location (e.g., at intersection, not at intersection, etc.) being studied. Crash reduction benefits on a specific corridor will vary, but a practitioner installing new Super 2 passing lanes on a rural two-lane highway should expect some crash reduction along the improved corridor. Thus, while a Super 2 is primarily an operational treatment, the treatment typically comes with safety benefits as well. Chapter 5.4 of the TxDOT *Traffic and Safety Analysis Procedures* (TSAP) *Manual* (*84*) provides additional guidance on the identification and selection of appropriate CMFs for TxDOT construction projects. Guidance on the use of CMFs to select appropriate treatments can also be found in the CMF Clearinghouse (*78*) and in the research (*87*).

CHAPTER 7: FINDINGS AND RECOMMENDATIONS

This chapter summarizes the work completed throughout this project, as documented in the previous chapters of this report, and provides the researcher team's key findings. This chapter also includes the research team's recommendations for future actions based on these conclusions.

FINDINGS FROM THE LITERATURE/POLICY REVIEW

During the course of this project, researchers reviewed relevant literature and research findings, as well as current policies in other states. Observations from these efforts led to the following findings regarding the design and performance of Super 2 roadways:

- Texas policy on Super 2 design is discussed in Chapter 4, Section 6 of the TxDOT *Roadway Design Manual* (*2*), which provides guidance on lane width, shoulder width, passing lane length, taper dimensions, and other notes for practitioners.
- Other states have provisions in their respective roadway design guides describing the construction requirements for Super 2-type roadways or single passing lanes on two-lane, two-way highways, but many of them do not provide the level of detail on geometric design guidance found in the TxDOT *Roadway Design Manual* (*2*).
- Internationally, similar roads are found in a number of countries, but they are usually built as a constant three-lane cross-section, where the passing lane alternates from one direction of travel to the other. These cross-sections are generally labeled as 2+1 roads and often have a median area and/or median barrier that restricts passing outside of the provided passing lanes. Design speeds and speed limits for international 2+1 roads are similar to those found in Texas, although they are often used at higher volumes than are typically found on two-lane roads in this state.
- Previous Super 2 research in Texas (*16, 32, 34, 72*) provided the basis for existing guidance on geometric design, signs and markings, suitable locations and traffic conditions for installation, and economic benefits. That research also included a crash data analysis from five Super 2 corridors in Texas, covering a total of 53 centerline-mi. The analysis revealed a statistically significant 35 percent reduction in fatal and injury crashes within the corridor segments compared to locations without passing lanes.
- Other crash analyses of Super 2-type roadways and passing lanes on rural two-lane highways have produced a variety of CMFs with values between 0.51 and 0.96 (as described in Chapter 2), depending on the details of the crashes and roadway characteristics. These analyses have also produced SPFs in some cases, providing equations that are sensitive to one or more variables rather than a single value to apply to many locations.
- An investigation of different analysis methods revealed that the EB method in BA analysis with reference sites was the most robust analysis method for determining CMFs

and SPFs for Super 2 corridors and passing lanes and was thus appropriate for analyzing the data in this project.

SUMMARY OF DATA COLLECTION EFFORTS

Researchers used information already collected in TxDOT Project 0-7035 (*72*) along with details from other sources to identify existing Super 2 corridors. This effort produced a database with 73 Super 2 corridors with at least one passing lane in each direction. These corridors were categorized into three types—alternating, side-by-side, and combination—based on the configuration of passing lanes in either direction of travel. These passing lane types were used as a framework for identifying and categorizing potential study sites during the data collection process. Researchers reviewed individual corridors to confirm Super 2 presence and obtain additional site characteristics. Based on this review, researchers removed or redefined several corridors, resulting in the combination of some corridors and the subdivision of other corridors. For the final analysis, a total of 67 corridors were selected, with a total length of nearly 660 mi. Researchers also identified 165 corridors, with a total length of nearly 1,470 mi, for use as reference segments.

Researchers also identified the installation dates for each Super 2 project and collected data on AADT, lane and shoulder width, access (i.e., driveways and intersections), adjacent land use, presence of rumble strips, and posted speed limit. The longest passing lane length was 3.44 mi. AADTs ranged between 600 and 15,100 vpd, and posted speed limits were between 50 and 75 mph. For reference sites, AADTs ranged between 471 and 8,411 vpd, and posted speed limits were between 50 and 75 mph.

The research team obtained crash data from TxDOT's CRIS from January 2003 to December 2023, identifying 1,575 observed crashes on 374 segments across the 67 Super 2 corridors.

RESULTS FROM THE CRASH ANALYSIS

Results of the EB analysis of crashes on the Super 2 corridors yielded the following results:

- Super 2 highways were estimated to reduce total crashes by 21 percent. If the treatment had not been installed, the expected number of crashes would have been 1,991 crashes during the three-year after-study period, as compared to the 1575 crashes that did occur in that study period. This result was statistically significant. The 95 percent confidence interval for crash reduction was estimated to be 16 to 26 percent for total crashes. A similar result was observed for KABC and PDO crashes, with reductions of 16 and 23 percent, respectively.
- The addition of passing lanes produced consistent crash reductions between 15 and 18 percent for all ranges of daily volumes. The sample size was insufficient to determine an upper bound to the positive reduction in crashes, but these findings are consistent with

previous research (*34*, *72*) that indicates the operational benefits of Super 2 passing lanes found at lower AADTs remain as volumes increase to between 10,000 and 15,000 vpd.

- As driveway density increased, crashes decreased, with a reduction of 37 percent for sites with no driveways and 10 percent for sites with an average driveway density of greater than 19 driveways/mi.
- Table 31 provides a full list of the CMFs produced in this research, along with their corresponding CRFs. Table 26 through Table 30 provide additional details for these CMFs.

RECOMMENDATIONS FOR IMPLEMENTATION

The research team developed the following recommendations for implementation based on the aforementioned results:

- Practitioners should follow the guidance provided in the CMF Clearinghouse (*78*), the TSAP Manual (*84*), and other resources when selecting and applying CMFs in the process for comparing multiple treatment alternatives.
- Practitioners considering whether to install a Super 2 corridor on a rural two-lane highway should first consider the available alternatives and determine which of those alternatives is best for that location. This consideration should include operations, design, economic, and other factors in addition to safety performance. *Guidelines for Implementing Super 2 Corridors in Texas* (*85*) describes how to consider operational, safety, and economic measures in the decision to select a particular alternative, with an emphasis on the features of Super 2 passing lanes.
- The results from this research are consistent with the results from previous research that led to the development of the existing guidance documents mentioned previously; however, the findings from this research can be used to update certain details in those existing documents to provide more comprehensive and well-informed guidance.
    - In particular, the research team recommends updating the Safety Measures section in Chapter 2 of the *Guidelines for Implementing Super 2 Corridors in Texas* (*85*), which provides guidance on the selection of alternatives. These updates include the addition of the table listing the CMFs developed in this project and references to the TSAP Manual (*84*) and the CMF Clearinghouse (*78*) for more guidance on the use of CMFs when evaluating and selecting potential alternatives.
    - In addition, the research team investigated potential updates to the TxDOT *Roadway Design Manual* and coordinated with the team revising that manual to suggest and incorporate appropriate updates into existing Super 2 design guidance. Just prior to the completion of this project, the November 2024 edition of the TxDOT *Roadway Design Manual* (*86*) was released, and Section 10.6 of the new manual contains updated guidance on the design of Super 2 corridors. Members of the research team reviewed the content of Section 10.6 and confirmed that the updated guidance in that

section includes recommended updates from previous projects as well as updates that would be suggested from this project, including additional references to guidance from previous research, such as *Guidelines for Implementing Super 2 Corridors in Texas* (*85*). As a result, the relevant findings and corresponding updates from this project have already been incorporated into the current manual and no additional updates to the TxDOT *Roadway Design Manual* are suggested from this research project.

SUGGESTIONS FOR FUTURE RESEARCH

This project considered various potential effects of Super 2 corridors on safety and crashes. The findings from this project reinforced the conclusions and recommendations from previous research related to the benefits of Super 2 corridors compared to traditional two-lane highways. During this project, the research team identified items that would be beneficial in further describing the usefulness of Super 2 highways, provide more support and justification for their use across the state, and aid practitioners in their decisions to install them. These items—presented as suggestions for future research—include the following:

- **Speed as a variable in a crash analysis:** The emergence of big datasets provides an opportunity to more directly compare speed data to crash data on Super 2 corridors. Speed from the INRIX dataset could be used as an additional variable in the analyses conducted in this project.
- **Passing lane length versus spacing length:** Guidelines recommend the lengths of passing lanes on Super 2 corridors based on operational performance, but less is known about the spacing between passing lanes. Some initial analyses on the relationships between crashes and passing lane lengths in this project produced inconclusive results. A more detailed look at different combinations of passing lane lengths and spacing lengths could provide insights into safety performance.
- **Intersection versus non-intersection analysis:** This analysis was primarily dependent on how law enforcement officers coded the crashes. A preliminary investigation revealed that many crashes at intersections and driveways were coded as non-intersection related. Future research can explore excluding crashes within intersection influence areas and include additional analyses related to non-intersection crashes.

SUGGESTIONS FOR IMPLEMENTATION

This research project developed a series of CMFs for practitioners to use when making decisions on whether to include a Super 2 corridor as part of a future construction project. These CMFs are provided within this report, along with discussion and instructions on their use. In addition, this project includes suggestions for updates to the *Guidelines for Implementing Super 2 Corridors in Texas* (*85*) to more widely share these research results with practitioners and facilitate implementation. In addition to these tangible project products and outcomes, a series of virtual or in-person workshops to share the information from this project and previous TxDOT-sponsored Super 2-related research may be useful in encouraging more widespread use.

The research team also suggests sharing the CMFs from this research with the CMF Clearinghouse (*78*); adding these current, Texas-based CMFs to the inventory in the CMF Clearinghouse (*78*) would facilitate their use by practitioners on future projects. The research team will submit these CMFs to the CMF Clearinghouse (*78*) for consideration.

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