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Examine Trade-Offs between Center Separation and Shoulder Width Allotment for a Given Roadway Width

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16. Abstract

In Texas, four-lane undivided roadways constitute a significant amount of mileage in rural areas. These highways have poor safety performance compared to other cross sections. However, there is not always sufficient space within the available right of way to accommodate a traditional four-lane divided cross section. Thus, a framework can assist road design engineers in making decisions on cross sections for new and resurfaced rural roadway segments to optimize operational and safety performance.

In this research project, researchers evaluated the safety and operational benefits of converting four-lane undivided highways into other cross-section alternatives without changing the total roadway width. First, researchers selected a sample of rural highway segments with various cross sections, including four-lane undivided, four lane with 4-ft median buffer, four lane with two-way left-turn lane (TWLTL), Super 2, and Super 2 with TWLTL. They analyzed crash data on each cross section to determine the relationship between crash frequency, exposure, access-point density, horizontal curvature, cross-sectional widths, and operating speed variables. Second, researchers collected operating speed data in the field and developed VISSIM simulation models to evaluate the operational impacts of different cross sections considered in the project. Based on safety and operational effects, researchers developed a framework that allows the analyst to select a cross section depending on traffic volume, heavy vehicle mix, cross-sectional width, and access-point density.

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DISCLAIMER

This research was performed in cooperation with 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 published 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. It is not intended for construction, bidding, or permitting purposes. The engineer in charge of the project was Srinivas R. Geedipally, P.E. #109898.

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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- John Speed, district engineer, Odessa District.
- Khalid Jamil, section director, Traffic Simulation and Safety Analysis, Design Division.
- Sharlotte Teague, section director, Project Development Support, Design Division.

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CHAPTER 1: OVERVIEW

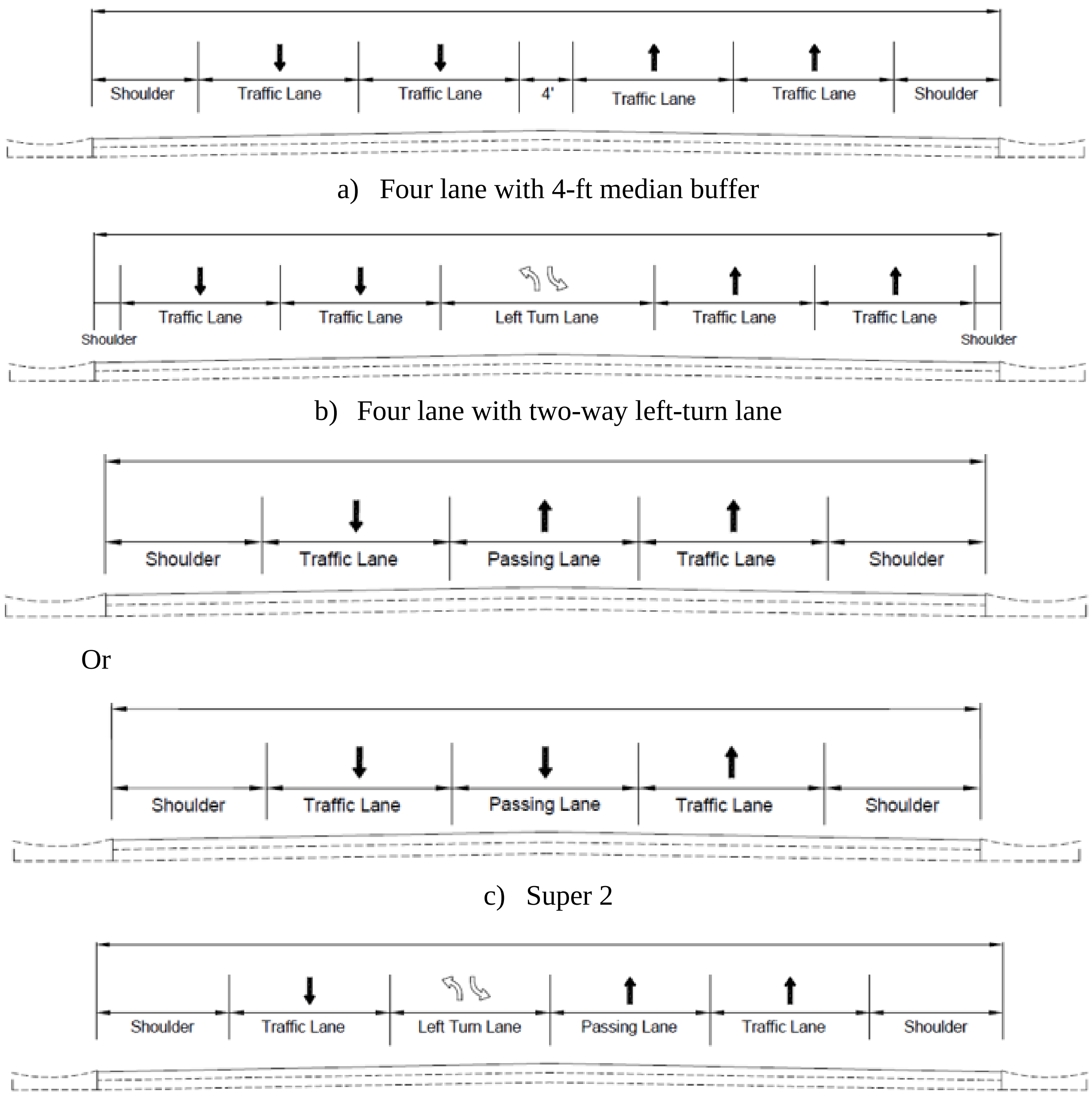
1.1. INTRODUCTION

Studies show that four-lane undivided roadways have poor safety performance compared to four-lane divided and two-lane cross sections. Four-lane undivided rural (4U) highways experience relatively high crash frequencies—especially as the traffic volume increases—resulting in conflicts with high-speed opposite-direction (OD) vehicles. However, there is not always sufficient space within the available right of way to accommodate a traditional four-lane divided cross section. Some states, including Texas, have started providing a narrow centerline buffer area that is separated by longitudinal pavement markings. This additional buffer area shifts the lateral placement of vehicles and introduces a greater physical separation between approaching vehicles. However, the provision of a centerline buffer comes at the cost of reduced lane or shoulder widths. Other cross sections, such as Super 2 (2S) with and without two-way-left-turn-lane (TWLTL) and four-lane with TWLTL (4T) highways, are also possible alternatives to four-lane undivided roadways. A better understanding of the benefits of center separation, as well as lane and shoulder combinations, would provide useful information to designers who make decisions on cross sections for new and resurfaced roadway segments.

The purpose of this project was to provide a practical framework that the Texas Department of Transportation (TxDOT) can use to choose between cross-sectional design alternatives to optimize operational and safety performance on rural highways. This framework incorporates variables such as traffic volume, heavy vehicle mix, cross-sectional width, and access-point density.

1.2. RESEARCH APPROACH

The research team collected and analyzed data for traditional four-lane undivided sites and compared their safety and operational performances with other alternative cross-sectional designs. Alternative cross sections considered were four-lane with 4-ft median buffer (4M), 4T, 2S, and Super 2 with TWLTL (2ST). Figure 1 shows these alternative cross sections.



a) Four lane with 4-ft median buffer

b) Four lane with two-way left-turn lane

c) Super 2

d) Super 2 with two-way left-turn lane

Figure 1. Cross-Sectional Alternatives.

1.3. Research Results

Horizontal curve presence, driveway density, shoulder width, and operating speed have been identified as key influential variables of rural highway safety. There is no one best cross section for all circumstances, although it is clear that the four-lane undivided cross section generally has the worst safety performance of all the cross sections considered. The 2S cross section has the best safety performance in all circumstances at volumes up to 15,000 vehicles per day (vpd). Shoulder width and driveway density have varying effects on different cross sections. Mainly, the effect of shoulder width on the safety performance of four-lane roadways with a 4-ft median buffer is substantial, with shoulders of less than 6 ft significantly increasing crashes. These cross sections are highly effective in reducing lane departure crashes. The cross section produces excellent safety performance at volumes above 15,000 vpd as long as it has at least 6-ft shoulders and driveway density is low. Four-lane highways with TWLTL sections provide better safety performance when the driveway density is higher.

With respect to operations, all cross sections experience significant delay if the total pavement width is less than 60 ft. Four-lane highway with TWLTL is the only cross section to have a much lower average delay for any heavy vehicle proportion and driveway density when the traffic volume is greater than 25,000 vpd.

It is necessary to account for both safety and operational effects when selecting a cross section. The research team developed a framework that can assist practitioners in making decisions on cross sections for new and resurfaced roadway segments, as shown in Table 1.

Table 1. Guidelines for Selecting Cross Sections.

Nominal Pavement Width (Range)	AADT	Driveway Activity Index[a] per Mile	Truck Percentage	Preferred Cross Section
50 ft (≤55 ft)	≤15,000	≤30	Any	Super 2
		>30	Any	Widen to Super 2 with TWLTL
	>15,000	≤30	Any	Widen to Four Lanes with 4-ft Median Buffer[b]
		>30	Any	Widen to Four Lanes with TWLTL
60 ft (56–65 ft)	≤15,000	≤30	Any	Super 2
		>30	≤15%	Super 2 with TWLTL
			15–25%	Super 2 with TWLTL
			>25%	Widen to Four Lanes with TWLTL
	15,000–20,000	≤30	≤15%	Four Lanes with 4-ft Median Buffer[b]
			15–25%	Four Lanes with 4-ft Median Buffer[b]
			>25%	Widen to Four Lanes with TWLTL
		>30	≤15%	Four Lanes with 4-ft Median Buffer[b]
			15–25%	Four Lanes with 4-ft Median Buffer[b]
			>25%	Widen to Four Lanes with TWLTL
	>20,000	Any	Any	Widen to Four Lanes with TWLTL
70 ft (≥66 ft)	≤15,000	≤30	Any	Four Lanes with 4-ft Median Buffer[b]
		>30	Any	Super 2 with TWLTL
	15,000–20,000	≤30	≤15%	Four Lanes with 4-ft Median Buffer[b]
			15–25%	Four Lanes with 4-ft Median Buffer[b]
			>25%	Four Lanes with TWLTL
		>30	≤15%	Four Lanes with 4-ft Median Buffer
			15–25%	Four Lanes with TWLTL
			>25%	Four Lanes with TWLTL
	>20,000	Any	Any	Four Lanes with TWLTL

Note: AADT = annual average daily traffic.

[a] Driveway activity index is the number of residential driveways. The index is equal to three times the number of industrial driveways, or 12 times the number of commercial driveways (measured per mile).

[b] 6-ft minimum shoulder width. Greater widths are desirable.

CHAPTER 2: LITERATURE REVIEW

This chapter presents a detailed review of the safety performance of different cross-sectional designs. The chapter is divided into four sections. The first section focuses on the distribution of collision types on 4U and rural two-lane undivided (2U) highways. The second section presents the safety performance of different roadway types. The third section documents the operational and safety effects of roadway conversions. The last section provides background on traffic simulations.

2.1. DISTRIBUTION OF COLLISION TYPES

Cross-section design affects both operational characteristics and safety performance of highways. To compare the safety performance of different cross sections, it is important to identify the predominant collision types on different roadways. This study mainly considered the following five cross sections: 4U, 4M, 4T, 2S and 2ST. Due to the relatively low sample size of 4M, 4T, and 2ST, no accurate distribution of collision types on these roadway segments is available. The 4M and 4T cross sections belong to the category of four-lane undivided highways, while the 2S and 2ST cross sections belong to the category of two-lane highways. It can be assumed that the distribution of crashes on these cross sections falls into two roadway categories: 4U and 2U. The first edition of the *Highway Safety Manual* (HSM) (American Association of State Highway and Transportation Officials [AASHTO], 2011) provides the distribution of collision types by crash severity level on 4U and 2U roadways, separately. Table 2 shows the distribution of crash types on rural roadways.

Table 2. Distribution of Crash Types on Rural Roadways.

Collision Type	Total		Fatal and Injury		PDO	
	4U[a]	**2U[b]**	**4U[a]**	**2U[b]**	**4U[a]**	**2U[b]**
Head-On	0.9%	1.6%	2.9%	3.4%	0.1%	0.3%
Sideswipe	9.8%	3.7%	4.8%	3.8%	12.0%	3.8%
Rear-End	24.6%	14.2%	30.5%	16.4%	22.0%	12.2%
Angle	35.6%	8.5%	35.2%	10.0%	35.8%	7.2%
Single	23.8%	69.3%	23.8%	63.8%	23.7%	73.5%
Other	5.3%	2.7%	2.8%	2.6%	6.4%	3.0%
Total	100%	100%	100%	100%	100%	100%

Note: Data reproduced from the HSM. 4U data are based on roadways from five states (i.e., California, Minnesota, New York, Texas, and Washington) (Lord et al., 2008); 2U data are based on roadways in two states (i.e., Minnesota and Washington) (Harwood et al., 2000, p. 99-207). PDO = property damage only.
[a] 4U = rural four-lane undivided highway.
[b] 2U = rural two-lane undivided highway.

The distribution of collision types differs substantially on the two types of highways. On 4U highways, the predominant collisions are angle, rear-end, and single-vehicle (SV) run-off-the-road (ROR), accounting for 35.6 percent, 24.6 percent, and 23.8 percent, respectively. On rural two-lane highways, the predominant collisions are SV ROR and rear-end, accounting for 69.3 percent and 14.2 percent, respectively. The proportions of multi-vehicle (MV) crashes (i.e., angle, rear-end, and sideswipe) on four-lane highways are higher than those on two-lane highways, except for head-on crashes. Generally, there are more interactions between vehicles (e.g., passing, lane change) on four-lane highways.

Although extensive efforts have been made to understand the safety effects of median treatments, such as Elvik (2009) and Abdel-Aty et al. (2016), very limited studies have focused on median buffers (also known as centerline buffer). So far, no crash modification factor (CMF) can be found for median buffer in the CMF Clearinghouse (FHWA, 2019). Recently, Dixon et al. (2018) conducted an analysis on the safety effectiveness of centerline buffer in reducing OD crashes. The researchers collected roadway, traffic, and crash data on 12 mi of two-lane highways and 30 mi of four-lane highways with centerline buffer in Texas. The centerline buffer showed a positive safety effect in reducing OD crashes (i.e., mainly head-on crashes) on two-lane roadways, with a CMF of 0.79. The result is statistically significant at the 85 percent confidence level. However, no significant effect of median buffer was found on four-lane roadways.

In the first version of the HSM, the TWLTL was presented as an effective treatment for reducing crashes related to driveways and left turns on rural two-lane highways. The number of driveway-related crashes was reduced by up to 40 percent with the use of TWLTL on rural two-lane highways. The actual reduction depends on the density of driveways on the segment. Lyon et al. (2008) also found that the TWLTL reduces crashes involving a vehicle desiring to make a turn. The crashes typically can be classified as rear-end or head-on crashes. However, the trend of crashes on rural four-lane highways after installing a TWLTL is unknown (AASHTO, 2011). No results have been reported regarding the effectiveness of TWLTL on rural four-lane highways. The research team assumes that the TWLTL has similar safety effects on four-lane highways as those on two-lane highways. It is expected that rear-end and head-on crashes will be reduced on both types of roadways after the installation of TWLTL.

Brewer et al. (2012) systemically analyzed the operational characteristics and safety effectiveness of 2S highways in Texas. Fatal and injury (FI) crashes reduced by 35 percent to 42 percent after the installation of passing lanes on rural two-lane highways (i.e., 2S). The results are consistent with previous studies (Harwood et al., 2000; Rinde, 1977).

In summary, the predominant crashes on 4U highways are angle, rear-end, and SV ROR collisions. The crashes are assumed to be similar on 4M and 4T. It is expected that the proportion of rear-end and head-on crashes on 4T highways will be smaller than that of 4U. On 2U highways, the predominant crashes are SV ROR and rear-end crashes. On rural 2S and 2ST highways, the proportion of rear-end crashes is expected to be smaller than that of 2U.

2.2. SAFETY PERFORMANCE

This section presents the safety performance of different cross-sectional designs.

2.2.1. Rural Four-Lane Undivided Highways

There are several safety performance functions (SPFs) for 4U highways. This report mainly focuses on the results documented in the HSM and studies conducted using Texas data.

The SPFs for 4U highways in the HSM are shown in Equations 1–2 (AASHTO, 2011).

$$Y_{4U\_HSM} = \mathrm{L} \times e^{-9.653+1.176\times\ln(AADT)} \tag{1}$$

$$Y_{4U\_FI\_HSM} = \mathrm{L} \times e^{-9.410+1.094\times\ln(AADT)} \tag{2}$$

where:

Y_{4U_HSM}	=	predicted number of total crashes on 4U highways in one year (HSM model).
$Y_{4U\_FI\_HSM}$	=	predicted number of FI crashes on 4U highways in one year (HSM model).
L	=	segment length (mi).
AADT	=	annual average daily traffic (vpd).

The base conditions are 12-ft lane, 6-ft paved shoulder, 1V:7H sideslope, without lighting, and without automated speed enforcement.

Bonneson and Pratt (2009) developed an FI crash SPF for 4U highways using Texas data. The SPF is shown in Equation 3.

$$Y_{4U\_FI\_TX} = \mathrm{L} \times [0.00749 \times \left(\frac{\mathrm{AADT}}{1000}\right)^{1.63} + 0.109 \times \left(\frac{\mathrm{AADT}}{1000}\right)^{0.631}] + n_e \times 0.0169 \times \left(\frac{\mathrm{AADT}}{15000}\right)^{0.738} \tag{3}$$

where:

$Y_{4U\_FI\_TX}$	=	predicted number of FI crashes on 4U highways in one year (Texas model).
n_e	=	number of equivalent residential driveways.

The number of equivalent residential driveways is a weighted average of residential driveway, industrial driveway, business driveway, and office driveway numbers. Their weight factors are 1.0, 2.68, 2.33, and 9.66, respectively. In Equation 3, the local factor has been taken as 1.0 (i.e., default value).

The base conditions are 12-ft lane, 8-ft shoulder, 1V:4H sideslope, straight and flat, without rigid or semi-rigid barrier, and 30-ft horizontal clearance.

As mentioned in the previous section, the safety effects of TWLTL and median buffer on rural four-lane highways are unknown, and their CMFs are unavailable. It is difficult to predict the accurate number of crashes on 4M or 4T roadways.

2.2.2. Super 2 Highways and Super 2 Highways with TWLTL

There is no SPF developed specifically for 2S highways. However, the CMFs for installing passing lanes on rural two-lane highways (i.e., 2S) are available. Thus, the SPF for 2S highways can be indirectly obtained.

The SPFs of base conditions in the HSM for all crashes on two-lane roads are given in Equation 4.

$$Y_{2U\_HSM} = \text{AADT} \times \text{L} \times 365 \times 10^{-6} \times e^{-0.312} \tag{4}$$

where:

Y_{2U_HSM} = predicted number of FI crashes on 4U highways in one year (Texas model).

The base conditions are 12-ft lane; 6-ft paved shoulder; five driveways per mile; roadside hazard rating of 3; straight and flat; and without lighting, centerline rumble strips, passing lane, TWLTL, or automated speed enforcement.

Based on the work conducted by Harwood and St John (1985) and Rinde (1977), the HSM determined the CMF for passing lane on rural two-lane highways as 0.75.

The SPF for a 2S highway can be obtained as shown in Equation 5.

$$\begin{aligned} Y_{2S\_HSM} &= 0.75 \times \text{AADT} \times \text{L} \times 365 \times 10^{-6} \times e^{-0.312} \\ &= 2 \times 10^{-4} \times \text{AADT} \times \text{L} \end{aligned} \tag{5}$$

where:

Y_{2S_HSM} = predicted number of total crashes on rural 2S highways in one year (HSM model).

In addition, the HSM also provides the CMF for TWLTL on rural two-lane highways. The CMF is a function of driveway-related crash proportion and driveway density, as shown in Equations 6 and 7.

$$CMF_{TWLTL\_HSM} = 1.0 - (0.7 \times p_{dwy} \times p_{LD/D}) \tag{6}$$

$$p_{dwy} = \frac{0.0047 \times DD + 0.0047 \times DD^2}{1.199 + 0.0047 \times DD + 0.0047 \times DD^2} \tag{7}$$

where:

CMF_{TWLTL_HSM} = CMF for TWLTL on rural two-lane highways (HSM).
p_{dwy} = driveway-related crashes as a proportion of total crashes.
$p_{LD/D}$ = left-turn crashes susceptible to correction by a TWLTL as a proportion of driveway-related crashes, typically taken as 0.5.
DD = driveway density considering driveways on both sides of the highway (driveways/mile).

Assuming that the safety effects of passing lane and TWLTL are independent on rural two-lane highways, the SPF for a 2ST is shown in Equation 8.

$$Y_{2ST\_HSM} = 2 \times 10^{-4} \times \text{AADT} \times \text{L} \times [1.0 - (0.35 \times \frac{0.0047 \times DD + 0.0047 \times DD^2}{1.199 + 0.0047 \times DD + 0.0047 \times DD^2})] \quad (8)$$

where:

Y_{2ST_HSM} = predicted number of total crashes on 2STs in one year (HSM model).

Bonneson and Pratt (2009) developed an FI crash SPF for 2U highways using Texas data. The SPF is shown in Equation 9.

$$Y_{2U\_FI\_TX} = 0.0537 \times \left(\frac{\text{AADT}}{1000}\right)^{1.20} \times \text{L} \quad (9)$$

where:

$Y_{2U\_FI\_TX}$ = predicted number of FI crashes on 2U highways in one year (Texas model).

The base conditions are defined as 12-ft lane, 8-ft shoulder, 1V:4H sideslope, straight and flat, without rigid or semi-rigid barrier, and 30-ft horizontal clearance.

Park et al. (2012) collected traffic and crash data on 83.73 mi of 2S highways in Texas and conducted an empirical Bayes (EB) before–after analysis. The results revealed that FI crashes decreased by 35 percent on roadway segments after the installation of 2S highways in Texas (i.e., CMF for 2S is 0.65). By combining the CMF with the SPF (i.e., Equation 5), the SPF for 2S highway can be obtained as shown in Equation 10.

$$Y_{2S\_FI\_TX} = 0.65 \times 0.0537 \times \left(\frac{\text{AADT}}{1000}\right)^{1.20} \times \text{L} = 0.0349 \times \left(\frac{\text{AADT}}{1000}\right)^{1.20} \times \text{L} \quad (10)$$

where:

$Y_{2S\_FI\_TX}$ = predicted number of FI crashes on 2S highways in one year (Texas model).

Bonneson and Pratt (2009) used the same CMF for TWLTL as that in the HSM; thus, the SPF for 2ST highways with Texas data can be derived as given in Equation 11.

$$Y_{2ST\_FI\_TX} = 0.0349 \times \left(\frac{\text{AADT}}{1000}\right)^{1.20} \times \text{L} \times [1.0 - (0.35 \times \frac{0.0047 \times DD + 0.0047 \times DD^2}{1.199 + 0.0047 \times DD + 0.0047 \times DD^2})] \quad (11)$$

where:

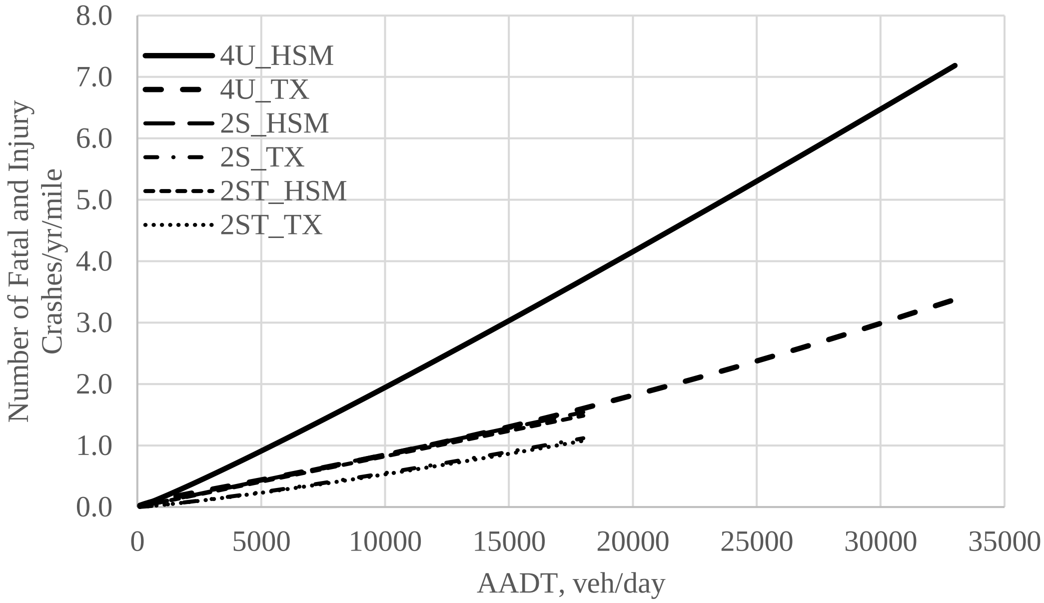
$Y_{2ST\_FI\_TX}$ = predicted number of FI crashes on 2ST highways in one year (Texas model).

These SPFs are summarized in Table 3, and the predicted number of FI crashes (i.e., SPF curves) is shown in Figure 2. When traffic volume is similar, the predicted number of FI crashes on 4U highways is much higher than that on rural 2S highways. 2ST highway segments have slightly fewer crashes than 2S highways without TWLTL. For rural two-lane roadways, the HSM SPFs are comparable with the Texas SPFs. For 4U roadways, however, the HSM SPF predicts a higher number of FI crashes than the Texas model because the two models were developed with data from different jurisdictions and/or the base conditions were defined differently (e.g., 8-ft shoulder versus 6-ft shoulder). Thus, the research team recommends calibrating the HSM SPF for 4U roadways before applying to Texas roadway segments.

Table 3. Summary of SPFs Developed in Previous Studies.

Facility Type	SPF	Note
4U	$Y_{4U\_HSM} = \mathrm{L} \times e^{-9.653+1.176 \times ln(AADT)}$	HSM, Total Crash
	$Y_{4U\_FI\_HSM} = \mathrm{L} \times e^{-9.410+1.094 \times ln(AADT)}$	HSM, FI Crash
	$Y_{4U\_FI\_TX} = \mathrm{L} \times \left[0.00749 \times \left(\frac{\mathrm{AADT}}{1000}\right)^{1.63} + 0.109 \times \left(\frac{\mathrm{AADT}}{1000}\right)^{0.631}\right] + n_e \times 0.0169 \times \left(\frac{\mathrm{AADT}}{15000}\right)^{0.738}$	Texas, FI Crash
4M	—	CMF/SPF Unavailable
4T	—	CMF/SPF Unavailable
2S	$Y_{2S\_HSM} = 2 \times 10^{-4} \times \mathrm{AADT} \times \mathrm{L}$	HSM, Total Crash
	$Y_{2S\_FI\_TX} = 0.0349 \times \left(\frac{\mathrm{AADT}}{1000}\right)^{1.20} \times \mathrm{L}$	Texas, FI Crash
2ST	$Y_{2ST\_HSM} = 2 \times 10^{-4} \times \mathrm{AADT} \times \mathrm{L} \times [1.0 - (0.35 \times \frac{0.0047 \times DD + 0.0047 \times DD^2}{1.199 + 0.0047 \times DD + 0.0047 \times DD^2})]$	HSM, Total Crash
	$Y_{2ST\_FI\_TX} = 0.0349 \times \left(\frac{\mathrm{AADT}}{1000}\right)^{1.20} \times \mathrm{L} \times [1.0 - (0.35 \times \frac{0.0047 \times DD + 0.0047 \times DD^2}{1.199 + 0.0047 \times DD + 0.0047 \times DD^2})]$	Texas, FI Crash

Note: — means the SPF for 4U roadway with median buffer or 4T roadway is not available.



Note: The driveway density of 5.0 was considered.

Figure 2. Predicted Number of Fatal and Injury Crashes Based on the SPFs.

2.2.3. Safety Effectiveness of Cross-Section Design Alternatives

Harwood and St John (1985) evaluated operational and safety performance of five operational treatments to alleviate common rural two-lane highway problems (e.g., overdemand, lack of passing opportunities, slow-moving vehicles): (a) passing lanes, (b) short four-lane sections, (c) shoulder driving, (d) turnouts, and (e) TWLTLs. The researchers collected crash data at 22 sites where passing lanes were installed on rural two-lane highways. A before-and-after comparison showed that the total crash rate decreased by 8.7 percent and the FI crash rate was reduced by 17.0 percent. However, neither of the results was statistically significant.
The researchers also compared the distribution of collision types on the treated sites and untreated comparison sites. None of the dominant collision types differed by more than a few percent. In other words, the predominant crash types on two-way segments with a passing lane were the same as those on traditional rural two-lane highways, and their percentages were similar. In addition, the researchers concluded that short four-lane segments showed substantially lower crash rates compared to untreated two-lane highway segments. OD crash rates on short four-lane sections were about half of those on untreated sections. However, the results were not statistically significant due to a low sample size issue (i.e., only nine treated sites).

Harwood and St John (1985) also collected safety data at seven sites with TWLTL on rural two-lane highways and evaluated the safety effectiveness. A comparison between treated and untreated sites indicated that crash rates on the treated segments were 30 percent lower than those on the untreated sites. Before–after analyses revealed that the total crash rate decreased by 85 percent and the FI crash rate was reduced by 67 percent. The result for total crash reduction was statistically significant at the 90 percent confidence level.

Gates et al. (2018) developed SPFs for rural road segments in Michigan. Table 4 provides details of the summary statistics of the key variables that were used for the development of SPFs.

Nearly all Michigan Department of Transportation two-lane two-way rural trunk lines with 55 mph speed limits had continuous milled centerline rumble strips present during the study period. For the CMF development, the curve design speed was chosen as 65 mph to provide an adequate sample of curved segments and to coincide with the new 65 mph maximum statutory speed limit for rural non-freeway highways in Michigan that was enacted in 2017.

Table 4. Michigan Rural Segment Summary Statistics (Gates et al., 2018).

Statistic	2U	4U	4D
Number of segments	1,556	58	55
Segment mileage	5,351.6	95.2	106.7
AADT (vpd)	4,382	9,373	13,518
Average annual segment crashes per mile	2.51	4.19	5.10
Average annual non-deer segment crashes per mile	0.79	1.88	2.51
Deer crashes as proportional of total segment crashes	0.69	0.55	0.51

2.3. OPERATIONAL AND SAFETY EFFECTS OF ROADWAY CONVERSIONS

Council and Stewart (1999) conducted a cross-sectional study to investigate the safety benefit of converting a 2U to a 4U. The researchers collected safety data from the Highway Safety Information System (HSIS) in four states: California, Minnesota, North Carolina, and Washington. State-specific safety prediction models were developed for two-lane and four-lane highways separately. Due to the low sample size issue in other states, only the California four-lane roadway data were used for the crash modeling. Analysis results revealed that there was a 20 percent reduction to a slight increase in crashes after the conversion of four-lane undivided highways from two-lane highways in California (as shown in Figure 3).

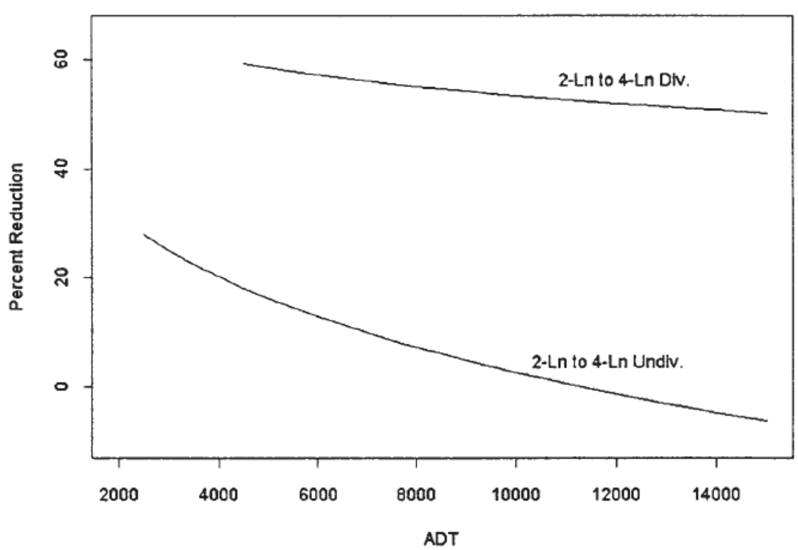


Figure 3. Crash Reduction for Two-Lane to Four-Lane Highway Conversion (California) (Council and Stewart, 1999).

Abdel-Aty et al. (2016) conducted a study to develop Florida-specific CMFs for several roadway conversion treatments. An observational before–after naïve approach was applied on 43 sites totaling 53.561 mi for rural 2U segments that were upgraded to rural four-lane divided (4D) highways. For this conversion, the crash rate dropped from 26.18 crashes per million vehicle miles (MVM) to 16.66 crashes per MVM; the estimated safety effectiveness was 36.35 percent. The same approach was applied to FI crashes only; adding a through lane at each direction of two-lane roadways and separating with a raised median, which reduced FI crashes by 34.98 percent for rural two-lane roadways (see Table 5). This study also showed that conversion of a TWLTL to a divided median reduced crashes significantly (see Table 6).

Table 5. CMFs for Conversion of Rural 2U to 4D (Abdel-Aty et al., 2016).

Florida Specific			**HSM**		
Traffic Volume	**Crash Type**	**CMF**	**Traffic Volume**	**Crash Type**	**CMF**
1,547–139,000	All types (injury)	0.76 (0.12)	Unspecified	All types (injury)	0.88 (0.03)
1,547–139,000	All types (non-injury)	0.75 (0.11)	Unspecified	All types (non-injury)	0.82 (0.03)
1,547–139,000	Head-on (all severity)	0.29 (0.20)	Unspecified	Head-on (All severity)	—

Note: — means not available.

Table 6. CMFs for Conversion of Rural 2T/4T to Rural 2D/4D (Abdel-Aty et al., 2016).

Setting (Road Type)	**Crash Type (Severity)**	**CMF**	**Std. Error**
Rural/urban (undivided roadways)	All types (all severities)	0.53	0.02
	All types (injury)	0.67	0.04
	All types (injury)	0.27	0.07

Note: 2T is a two-lane highway with TWLTL. 2D is a rural two-lane divided highway.

Elvik et al. (2017) conducted an EB before–after evaluation of the conversion of a 2U into a 4D road in Østfold County, Norway. The before period was 1996–2002. The after period was 2009–2015. The study discussed challenges in implementing the EB design when less than ideal data are available. Table 7 and Table 8 show the estimated changes in the number of crashes and injured road users and the standard errors of the estimated changes.

Table 7. Conversion of 2U to 4U (Elvik et al., 2017).

Site Type	Injury Crashes		Killed or Seriously Injured Road Users		Slightly Injured Road Users	
	Before	After	Before	After	Before	After
Treatment	185	123	71	11	403	279
Comparison	59,872	40,580	10,673	6,076	73,659	49,012
EB estimate	183.98	124.70	68.91	39.23	396.50	263.83

Table 8. CMFs for Conversion of 2U to 4U (Elvik et al., 2017).

Method	Injury Accidents		Killed or Seriously Injured Road Users		Slightly Injured Road Users	
	Estimate	Standard Error	Estimate	Standard Error	Estimate	Standard Error
Simple before–after	0.661	0.077	0.153	0.049	0.691	0.054
Before–after with comparison group	0.976	0.113	0.268	0.052	1.038	0.082
EB before–after	0.971	0.112	0.251	0.049	1.05	0.083
EB without variance adjustment	0.986	0.114	0.28	0.056	1.058	0.084

Ahmed et al. (2015) used various observational before–after analyses to evaluate the safety effectiveness of widening 2U to 4D roadways. The results from this study indicated that the conversion from 2U to 4D roadways resulted in a significant reduction in FI crashes of 45 percent on rural roadways. The safety effectiveness was found to be about 30 percent for total and PDO crashes on rural roadways. The crash reduction appeared to be even more effective for sites having a higher AADT. 2U to 4D roadway conversion showed better safety effects on total crashes on rural roadway segments with an AADT > 10,000 vpd. Table 9 and Table 10 list the CMFs and safety effectiveness for this conversion.

Table 9. CMFs for Conversion of Rural 2U to Rural 4D (Ahmed et al., 2015).

Rural Crashes	Naïve Before–After		Before–After with Comparison Group		Before–After with EB				Before–After with Bayesian	
					Simple SPF–Negative Binomial		Full SPF–Negative Binomial		Bayesian	
	CMF	SE	CMF	SE	CMF	SE	CMF	SE	CMF	SE
Total	0.64	0.11	0.73	0.1	0.71	0.09	0.74	0.09	0.71	0.08
PDO	0.61	0.1	0.69	0.1	0.7	0.08	0.71	0.08	0.69	0.08
FI	0.65	0.09	0.54	0.09	0.59	0.09	0.51	0.07	0.55	0.08

Note: SE means standard error.

Table 10. Safety Effectiveness of Conversion of Rural 2U to Rural 4D (Ahmed et al., 2015).

Number of Rural Crash Sites	Traffic Volume After Period (AADT)	Full SPF–NB		Univariate Poisson–Lognormal	
		CMF (Safety Effectiveness)	SE	CMF (Safety Effectiveness)	SE
18	AADT ≥ 10,000 vpd	0.71	0.11	0.71	0.1
25	AADT < 10,000 vpd	0.79	0.18	0.8	0.16

Stamatiadis et al. (2011) developed crash-prediction models and CMFs for multilane rural roads regarding lane width, shoulder width, and median width and type. The models were developed for divided and undivided facilities separately involving both total and injury crashes. FHWA's HSIS database for California and Minnesota was used in this study.

Persaud et al. (2010) evaluated the conversion of road segments from a four-lane to a three-lane cross section with TWLTLs. This conversion is also known as a road diet. The results from EB and full Bayes (FB) indicate highly significant reductions in left-turn, right-angle, and total crashes following signal installation for this conversion. However, rear-end crashes increased around 26 percent in both methods.

2.4. SIMULATION

Traffic analysis software comes in many forms including macroscopic, mesoscopic, and microscopic tools. Macroscopic analysis uses traffic flow theory to mathematically represent performance and typically is used to analyze the big-picture results of a large network (e.g., hundreds of miles of freeway) or in a planning phase of a project. Mesoscopic analyses are in between macroscopic and microscopic tools, where they include more detail than macroscopic tools but focus on an entire corridor or region in their data output. Microscopic analysis tools simulate driver interactions and choices to evaluate the performance of different traffic control or roadway feature alternatives.

Many microsimulation tools are available for evaluation. Aimsun, Traffic Software Integrated System – Corridor Simulation (TSIS – CORSIM), TransModeler, VISSIM, and SwashSim are the available microsimulation options for two-lane highway operations (Aimsun, 2018; Caliper Corporation, 2019; Transportation Institute University of Florida, 2019; Washburn, 2018). Washburn et al. (2018) considered each of these simulation software packages for their two-lane highway capacity analysis. The study included a survey on two-lane highway features of each of these simulation software packages in 2014. The researchers selected SwashSim and TransModeler as their simulation software for their analysis. At that time, only TransModeler and SwashSim enabled simulation of passing in the opposite direction of travel. Since then, VISSIM has incorporated a passing-in-the-opposite-direction-of-travel feature. In this current project, the research team found that no microsimulation is capable of modeling TWLTLs. The research team evaluated these simulation tools based on their ability to model passing in the oncoming lane, ability to model realistic interactions at intersections along the highway, ability to edit driver behavior, cost of the tool, ease of generating user-defined outputs, and developer support/documentation. The research team selected VISSIM as the analysis tool for this research project. VISSIM is capable of modeling overtaking in the opposite direction and gives the user the ability to model complex yield behavior easily through network creation tools.

A limited number of studies are available on modeling the rural cross-section design. A European study done by Cafiso et al. (2016) attempted to analyze safety on rural roadways with alternating passing lanes that they called a 2+1 cross section, which is equivalent to a 2S corridor in Texas. Cafiso et al. (2016) used VISSIM and a safety estimation tool called Surrogate Safety Assessment Model that estimates traffic conflicts from the model and provides an estimate of crashes. They calibrated their driving behavior to match observed percentages of passing vehicles using the desired speed distribution. Cafiso et al. (2016) then calibrated SPFs to match crash data on their 2S facilities. The simulation results indicated that the shortest passing lanes, 1,640 ft (500 m), showed the highest number of conflicts for each traffic volume. Passing lanes of 2,624 ft (800 m) experienced about the same number of conflicts in volumes less than 800 vehicles per hour (vph) per direction. Cafiso et al. (2016) recommended passing lanes longer than 3,280 ft (1,000 m) for volumes greater than 800 vph per direction. However, Cafiso et al. (2016) limited their study to a corridor without intersections. The conflicts for a 2S will increase as a result of intersections.

The finding of Cafiso et al. (2016) is consistent with the minimum passing lengths recommended by other states, as presented by Brewer et al. (2011) in their analysis of 2S facilities in Texas. Brewer et al. (2011) surveyed several states and recommended 1,000-ft passing as a minimum. The authors noted that most passing maneuvers occur in the first mile of a passing lane, and higher volume facilities benefit from longer passing lanes. Padding additional passing lanes provides more benefit than longer passing lanes. The simulations in their study also showed that the effects of average daily traffic (ADT) on operations were more substantial than the effects of terrain or truck percentage for their study corridor.

2.5. SURVEY

The research team conducted an online survey through the Qualtrics portal to identify the locations related to cross-section alternatives constructed by TxDOT that provide similar operational performance as that of four-lane undivided highways. The survey included questions about the locations that were converted or planned for conversion from four-lane undivided highways to other cross-section designs because of safety concerns. After developing the survey, the research team submitted the draft survey questionnaire to TxDOT for review and input. The questionnaire was then revised based on the review. TxDOT assisted the research team in determining and sending the survey to the appropriate person(s) at each district.

The survey questionnaire is included in Appendix A of this report. In total, 30 survey responses were created. However, 16 respondents just looked at the introduction but did not go to the questions. Four respondents looked at the questions but did not provide any responses. As a result, the research team received 10 complete responses. Below is the summary of verbatim responses to every question.

Q1. What are the locations (existing or planned) of rural 4M in your District? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

- Respondent 1: US190/SH6 from OSR to UP Railroad underpass. / SH 36 SB leaving Somerville for approximately 1/2 mile.
- Respondent 2: None.

- Respondent 3: SH 349 from US 87 to Martin county line (South of Lamesa). Just completed the tie into US 87.
- Respondent 4: Existing US 190 / SH 6 Robertson County Hearne to Brazos County line.
- Respondent 5: SH 158 in Sterling County, from US 87 to approximately 9 miles west.
- Respondent 6: In my area I have 1 current location (SH 158 South of Midland Tx) and also 1 currently under construction (SH 349 North of Midland Tx).
- Respondent 7: None in my section.
- Respondent 8: SH0171-DFOs 40.093-40.349. / Just south of Cleburne. There is an OLD concrete median now level with the paved surface. Closest I could find in FTW District.
- Respondent 9: none.
- Respondent 10: SH 300—3 miles north of FM 726 to FM 3358 in Upshur Co.

Q2. What are the locations (existing or planned) of rural 4T in your District? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

- Respondent 1:
  - US 79 between Thorndale and Rockdale in Milam County.
  - SH 21 between Brazos River and FM 50 in Brazos County (short section).
  - SH 21 between Pleasant Hill Rd and FM 2818 (Bryan City Limits).
  - US190/SH6 between the Railroad Underpass and the Hearne CL (Robertson County) Currently under construction.
  - SH 36 in Lyons (about FM 60) to Somerville.
  - SH 36 Washington/Burleson County line to Brenham/BS36, alternates with some turn lanes.
  - US290 from FM 2679 to Brenham CL, Washington County.
- Respondent 2:
  - US 59 from US 259 to Timpson City Limits.
  - SH 94 from LP 287 to FM 2497.
  - US 59 from FM 2021 to LP 287.
  - US 259 from US 59 to SH 204.
  - SH 103 from BU59 to FM 326.
  - US 69N from SH 7 to LP 287.
  - US 69S from LP 287 to FM 326.
  - US 69S from CR 213 to FM 844.
  - SH 19 from SH 94 to 0.8 miles N of FM 3453.
  - US 96 from SH 87 to FM 138.
  - BUS 59 from Pilar St. to LP 224.
  - BUS 59 from LP 287 to US 69.
  - US 59 from 0.1 mi N of US 287 to 0.34 mi S of FM 942.
  - US 59 from Old Hwy 35N to 0.2 mi S of Alexander Creek.
  - BUS 59 from SH 146 to Garner St.
- Respondent 3:
  - SH 114 (19th Street) from Research Blvd (west of Lubbock) 2 miles west.
  - Under Construction.
  - SH 214-0461-09-018, Yoakum County line to Seminole.
  - SH 214-0461-08-023, Plains south to Yoakum county line.

    - SH 214-0461-05-011, Cochran county line to Plains.
- Respondent 4:
    - Many existing sections some of these were considered as a cheaper “divided” section using a “flush median design.”
    - Existing.
    - FM 60 Brazos / Burleson Counties SH 47 to FM 50.
    - SH 36 Burleson Co Lyons to Brenham.
    - Washington US 290 to State school short section.
    - US 290 Washington Co SH 36 to FM 2679.
    - SH 21 Brazos Co FM 2818 to Knife River plant.
    - SH 30 Brazos Co FM 158 to Bird Pond Road.
    - US 79 Milam Co Thorndale to Rockdale.
- Respondent 5:
    - RM 853 in San Angelo (Tom Green County), from US 67 to FM 2288, approximately 1.5 miles.
    - RM 584 in San Angelo, from Loop 306 to the Airport, approximately 4 miles.
    - Similar four lane sections with center turn lane in many urban sections in our smaller towns.
- Respondent 6: FM 307 east of Midland Tx.
- Respondent 7:
    - US 59: From FM 989 To: FM 2148 (existing).
    - US 59: From 2328 To: SH 11 (existing).
- Respondent 8:
    - US0377-DFOs 114.483-115.963.
    - NE-SW of Cresson.
- Respondent 9:
    - US 79 from 4.5 miles E of US 59 to 6.5 miles E of US 59 (Panola County).
    - US 80 from SL 390 to FM 968 (Harrison County).
    - US 59 from Jefferson to 2 miles north of Jefferson.
    - US 59 from SH 149 to FM 2517 (Panola County).
- Respondent 10:
    - US 259—SH 49 to FM 557 in Morris Co. and Upshur Co.
    - SH 49—Mt. Pleasant CL to FM 1735 in Titus Co.

Q3. What are the locations (existing or planned) of rural 2SL in your District? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

- Respondent 1:
    - SH 30 in Walker County from FM 3179 to Didlake Road.
    - SH 105 from Navasota to the Montgomery County lines is a Super 2 section that alternates with a left turn lanes at various location. The turn lanes and passing lanes do not overlap.
- Respondent 2: None.
- Respondent 3: none.
- Respondent 4:
    - SH 30 Walker Co has a Super 2 with a turn lane provided at an FM intersection.

    - We have 3 other Super 2 sections, but due to ROW width we alternate and drop the 3rd lane if we need to provide a turn lane.
- Respondent 5: None.
- Respondent 6: SH 176 west of Andrews Tx. Also some on SH 349 south of Midland Tx.
- Respondent 7: None.
- Respondent 8:
    - SH0114-DFOs 342.365-342.757.
    - NW of Boyd.
- Respondent 9: none.
- Respondent 10: None.

Q4. What are the locations (existing or planned) of rural 2ST in your District? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

- Respondent 1: None.
- Respondent 2: None.
- Respondent 3: none.
- Respondent 4: None.
- Respondent 5: None.
- Respondent 6: None.
- Respondent 7: None.
- Respondent 8:
    - US0180-DFOs 273.793-273.893.
    - Western Mineral Wells-likely not rural.
- Respondent 9: US 79 from FM 31 to Louisiana State line (Panola County) currently under construction.
- Respondent 10: None.

Q5. Of the cross-sections mentioned in Q1 to Q4, which ones were converted (or are planned to be converted) from 4U? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

- Respondent 1: None was converted from a 4U. All were 2-lane, 2-way sections with varying width shoulders.
- Respondent 2: none.
- Respondent 3: none.
- Respondent 4: None.
- Respondent 5: None.
- Respondent 6: None.
- Respondent 7: None.
- Respondent 8: UNSURE.
- Respondent 9: US 79.
- Respondent 10: None.

Q6. What are the locations of Super 2 cross-sections in your district that were converted (or are planned to be converted) from 4U? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

- Respondent 1: None was converted from a 4U. All were 2-lane, 2-way sections with varying width shoulders.
- Respondent 2:
    - SH 7 from Kennard to Crockett.
    - SH 21 from SH 7 to Madison County Line.
    - US 96 from San Augustine to Shelby County Line.
    - US 96 from SH 147 to SH 103.
    - US 96 from SH 103 to SH 184.
    - SH 147 from SH 103 to US 96.
    - SH 150 from Walker County Line to FM 945.
    - US 190 from San Jacinto County Line to FM 2457.
- Respondent 3: none.
- Respondent 4: All of ours were upgraded from 2 lane. / So None.
- Respondent 5: None.
- Respondent 6: I have SH 349 south of Midland Tx.
- Respondent 7: SH 155: SH 8 To: FM 161.
- Respondent 8:
    - UNSURE ON CONVERSION.
    - US0377-DFOs 133.034-134.012.
    - SW of Granbury.
    - Multiple locations between Granbury and Stephenville on US0377.
- Respondent 9: none.
- Respondent 10: None.

Q7. What are the locations of rural 2T in your district that were converted (or are planned to be converted) from 4U? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

- Respondent 1: No rural sections of 4U have been converted to rural 2T. We have converted two sections of 4U recently, on SH 14 in Wortham and SH 75 in Madisonville. We have also recently take a small section of US190/SH36 in Cameron back to 4U from 2T.
- Respondent 2: None.
- Respondent 3: none.
- Respondent 4:
    - We did this but more in an urban setting in Rockdale and Cameron, probably not what you are looking for.
    - Again we have some other urban locations but No current plans to change. Hearne, Calvert, and Huntsville are similar.
- Respondent 5: None.
- Respondent 6: SH 349 south of Midland TX.
- Respondent 7: US 67: From FM 989 To: FM 2148 (Suburban section—converted recently).

- Respondent 8:
    - UNSURE ON CONVERSION.
    - FM1187-DFOs 18.59-20.87.
    - West of Crowley to Chisholm Trail Pkwy.
    - SH 174-DFOs 23.122-23.911.
    - N-S of Rio Vista.
- Respondent 9: none.
- Respondent 10: None.

The final question in the survey asked for respondents to provide their name and email address so that the research team could follow up with them as needed. Nine of the 10 respondents did so.

CHAPTER 3: DATA COLLECTION ACTIVITIES

This chapter discusses the steps considered for collecting the detailed data. The data collection included collecting the crash, roadway, environment, driveway, and speed data for conducting both safety and operational analyses.

The chapter is divided into four sections. The first section describes sampling design used for randomly selecting 4U and 2S highways. The second section presents different phases of data collection. The third section shows the collection of driveway volume data using the Wejo connected vehicle data. The last section provides the steps considered for collecting vehicle speed data.

3.1. SAMPLING FRAMEWORK

Considering the time frame and resources available for this project, the research team decided to develop and use two stratified probability samples: one representing 4U segments and another representing 2S segments. The research team intended to collect detailed data on the segments from the two samples to later merge the samples together for a statistical comparison of safety performance.

Probability sampling is a set of principles and methodologies to select samples systematically in such a way that it is possible to quantify the uncertainty present in the sample with respect to the variable and parameter values in the population from which they were drawn. The key feature of probability sampling is that any one datum in a population has a finite, nonzero probability of being selected into the sample prior to data collection.

3.1.1. Sample Design for 4U Segments

First, the research team developed a probability sample of the 4U roadways that would allow researchers to draw inferences about quantities of interest at the sampled population level (the population being all miles of 4U highways in Texas maintained by TxDOT). The sampling frame for probability sample design can be controlled effectively using key variables available from the Road–Highway Inventory Network Offload (RHINO) database. The research team proposed using a stratified sample balanced for key variables. The stratification criterion was TxDOT’s four regions (north, west, south, and east).

The research team decided to implement cube sampling to produce the stratified sample in order to control for the balancing of key variables that are known to be associated with the safety performance of highways, namely:

- AADT.
- Truck percentage.
- Shoulder width.

The method selected to draw the equal-probabilities sample was an implementation of the fast algorithm proposed by Chauvet and Tillé (2006) based on cube sampling methods.

The sample size was determined via resampling procedures on the data frame. The criteria to determine sample size sufficiency were the sampling SE for two key variables: mean crashes and proportion of serious injury crashes, including fatal (K), incapacitating injury (A), and non-incapacitating injury (B). The researchers retrieved crash data for the years 2015–2019 from the Crash Records Information System (CRIS) database. A minimal target precision was determined at 10 percent, meaning that a sample should be of a size that yields estimates that would be within 10 percent of the statewide value for the two key variables just mentioned. The resampling procedure was performed over the complete population of 7,132 available 4U segments. Many of those segments were extremely short, so the research team discarded segments shorter than 0.025 mi whenever those were drawn. The research team verified that these short segments represented 25 percent of the 7,132 available 4U segments.

Performing this exercise for an increasingly larger number of iterations suggested that approximately 490 segments were needed in order to achieve 10 percent average error in estimating the average crash frequency or the proportion of KAB crashes for these facility types. The plots in Figure 4 represent the results for 200 replications on a sample size set at n=600 roadway segments.

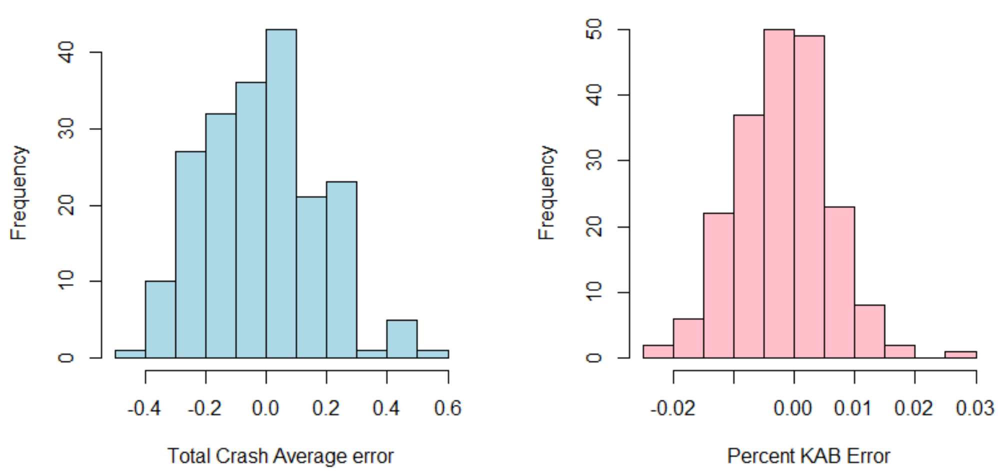


Figure 4. Resampling Results for Total Crash Average and Percent KAB Estimate Errors (n=600, r=200).

Therefore, the research team decided to draw a sample of 500 segments for further data collection. The sample was drawn, as explained earlier, to balance AADT, truck percentage, and shoulder width between the strata and between the overall sample and the population.

3.1.2. Sample Design for 2S Segments

Once a representative sample of 4U segments was drawn, the researchers determined that a complementary sample of 2S segments should be comparable to the 4U sample. A sample of 500 2S segments was drawn from the population of known 2S segments in Texas using the same design parameters as for the 4U segments. However, the research team noted a difference in the distribution of AADTs and shoulder widths, as evidenced in Figure 5.

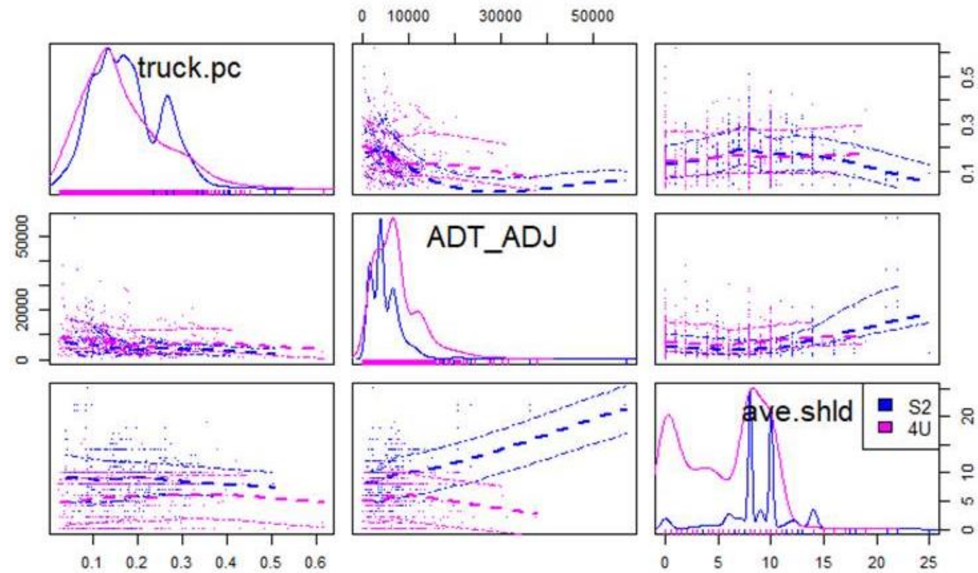


Figure 5. Comparative Plots between 4U and 2S Samples (n1=n2=500).

In the trend plots between pairs of variables, the relationship between AADT and average shoulder is different between the two subsets of sites; for 4U, it is relatively flat throughout its whole range (from 0 up to about 35,000 vpd). The relationship is comparable for 2S from 0 to about 10,000 vpd, but it clearly departs toward a roughly linear increase from about 10,000 up to almost 60,000 vpd.

Because the intent was to use the combined samples in a safety analysis that compared the two facility types, the research team determined that the difference in distribution of shoulder widths is somehow expected because when transforming 4U to 2S, there is additional space that is redistributed between shoulders and medians. However, in order to make a comparison between facilities while controlling for AADT (since AADT is a known safety influential variable), it is preferable to have the two subsets in the comparison exhibit a similar AADT distribution. In that case, the statistical estimate of a safety difference should be robust against distortions due to AADT differences. The same balance is also desirable for truck percentage.

For the reasons described above, the research team repeated the sampling of 2S segments utilizing a propensity score (PS) model fitted to the combined sample initially produced. The purpose was then to use the PS function to produce unequal probabilities of selection in drawing a second sample of 2S segments while controlling for the observed differences in AADT distributions. The PS model explicitly excluded shoulder width, for the reasons discussed earlier. The plot in Figure 6 illustrates the comparison of the 4U sample with a second 2S sample of 500 sites, now selected with unequal probabilities derived from the PS values.

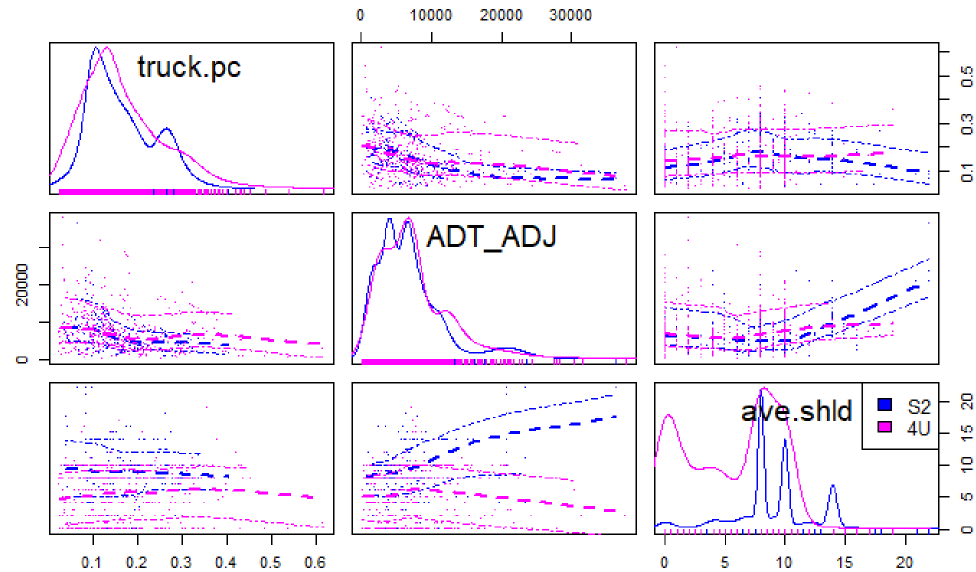


Figure 6**. Comparative Plots between 4U and the Second 2S Sample (n1=n2=500).**

Figure 6 demonstrates that the distribution of AADT and truck percentage now better matches the distribution for those variables in the 4U sample. The research team proceeded to use the second 2S sample to perform additional data collection on those segments.

3.2. DATA COLLECTION

As mentioned earlier, the research team used the stratified random sampling and identified the 4U segments across Texas for data collection. For 4M, 4T, and 2ST, the team used the survey responses in Chapter 2 to identify segments. After an initial investigation, some of the 4U segments had a wider median and were performing similar to 4T, so they were included in the 4T category. Given their rarity, all 4M, 2S, and 2ST segments in the state were considered.

3.2.1. Determination of Area Type

The initial investigation also revealed that the area type populated in the TxDOT RHINO was questionable for some of the segments. The research team refined the area type definition by adopting the following key steps:

- Identify urban areas using the 2017 Census urban area geography and 2017 American Community Survey (ACS) 5-year population estimates (Table B01003).
- Identify urban areas and fringe buffers based on extraterritorial jurisdiction (ETJ) distances as described in the Texas Local Government Code. The area types and fringe buffers were defined as shown in Table 11.

Table 11. Fringe Buffer Definitions by Area Type.

Label	Population Category	Fringe Buffer
Urban—Very Large	> 250K population	5 mi
Urban—Large	100K–250K	5 mi
Urban—Medium	50K–100K	3.5 mi
Urban—Small	25K–50K	2 mi
Urban—Very Small	5K–25K	1 mi
Urban—Very, Very Small	< 5K	0.5 mi
Rural	Everywhere else	—

Note: — means not applicable.

The segments in the medium-to-very-large urban area were excluded. In addition, only segments with a speed limit of 45 mph or higher were considered. Table 12 shows the number of segments and mileage by the cross-section type.

Table 12. Number of Segments by Cross-Section Type.

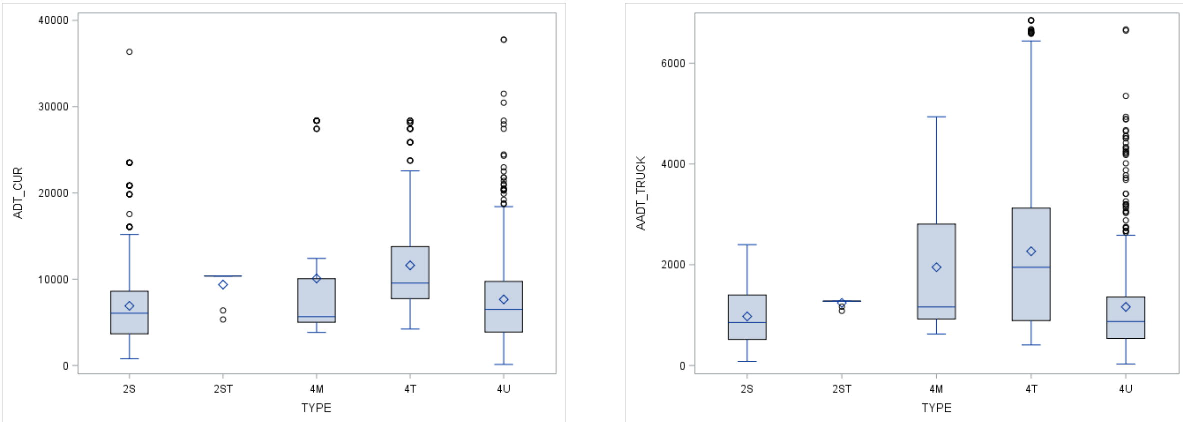
Cross Section	Number of Segments	Total Length (Miles)
4U	131	76.2
4M	95	60.6
4T	536	148.8
2S	463	419.0
2ST	10	9.2

Table 13 shows the summary statistics of AADT and truck volume by the cross section.

Table 13. Summary of AADT and Truck Volumes by Cross-Section Type.

Variable	Type	N Obs.	Mean	Min.	5th Percentile	25th Percentile	75th Percentile	95th Percentile	Max.
AADT	2S	463	4,194	884	1,445	1,984	5,775	8,510	11,715
	2ST	10	9,469	5,336	5,336	10,347	10,375	10,375	10,375
	4M	95	8,709	3,830	3,830	5,015	6,733	28,374	28,374
	4T	536	10,740	504	5,506	7,268	12,461.5	22,559	30,458
	4U	131	5,837	119	1,085	3,012	7,457	13,726	21,804
Truck Volume	2S	463	714	75	204	392	898	1,542	2,458
	2ST	10	1,184	652	652	1,168	1,276	1,276	1,276
	4M	95	1,789	624	831	956	1,896	4,937	4,937
	4T	536	2,044	41	579	839.5	2,909	5,080	6,671
	4U	131	971	30	206	421	1,165	2,526	4,334

Figure 7 shows the box-and-whisker plots for different variables. The black circles represent the extreme observations (i.e., outliers). The box plots show that the median ADT and truck volumes were always higher on 4T highways and lower on 2S and 4U segments.



a) AADT b) Truck Volume

Figure 7. Box-and-Whisker Plots.

Since the RHINO database did not include many of the important variables (such as driveway count, curve advisory speeds, and horizontal curve information), manual data collection was deemed necessary. Fortunately, all data collection could be done in house with the available sophisticated tools and without the need of on-site visits. Therefore, office data collection was adopted by assembling a team of researchers and student workers to assist with the data collection activities.

3.2.2. Office Data Collection

As an initial step, Google Earth (GE) keyhole markup language—zipped (kmz) and shape files—were created showing the geolocations of the selected sites. The data collection process involved collecting the driveway count; intersections; posted speed limits (PSLs) (regulatory, curve advisory, and school zone); passing zone configurations; rumble strips; and roadway design characteristics such as horizontal curve information and lane, and shoulder and median widths. The data collection process was divided into four phases. Facilitating the data collection process in each incremental phase required the use of the following: GE aerial view, GE street view, and Microsoft Excel® spreadsheet. All data collected were recorded in spreadsheets. After a quality control phase, all spreadsheets were merged into a final database for analysis.

Phase 1: Driveway and Intersection Information

The count of driveways and intersections was measured from GE aerial photographs. Both three-legged and four-legged intersections were also counted. The land use served by a driveway was categorized as residential, industrial, business, or office. Table 14 was used to determine the land use associated with each driveway along the subject segment (Bonneson and Pratt, 2009). Two types of driveways were recognized in the count of driveways. A full driveway allowed left and right turns in and out of the property. A partial driveway allowed only right turns in and out of the property. Partial driveways were most commonly found on segments that had double yellow pavement markings that restricted left turns.

Table 14. Adjacent Land Use Characteristics (Bonneson and Pratt, 2009).

Land Use	Characteristics	Examples
Residential or Undeveloped	• Buildings are small • A small percentage of the land is paved • If driveways exist, they have very low volume • Ratio of land use acreage to parking stalls is large	• Single-family home • Undeveloped property, farmland • Graveyard • Park or green-space recreation area
Industrial	• Buildings are large and production-oriented • Driveways and parking may be designed to accommodate large trucks • Driveway volume is moderate at shift change times and is low throughout the day • Ratio of land use acreage to parking stalls is moderate	• Factory • Warehouse • Storage tanks • Farmyard with barns and machinery
Commercial Business	• Buildings are larger and separated by convenient parking between building and roadway • Driveway volume is moderate from mid-morning to early evening • Ratio of land use acreage to parking stalls is small	• Strip commercial, shopping mall • Apartment complex, trailer park • Airport • Gas station • Restaurant
Office	• Buildings typically have two or more stories • Most parking is distant from the building or behind it • Driveway volume is high at morning and evening peak traffic hours; otherwise, it is low • Ratio of land use acreage to parking stalls is small	• Office tower • Public building, school • Church • Clubhouse (buildings at a park) • Parking lot for 8-to-5 workers

Driveways that were unused were not counted. Similarly, driveways leading into fields, small utility installations (e.g., cellular phone tower), and abandoned buildings were not counted. A circular driveway at a residence was counted as one driveway even though both ends of the driveway intersected the subject segment. Similarly, a small business (e.g., gas station) that had two curb cuts separated by only 10 or 20 ft had effectively one driveway.

Phase 2: Speed Limit Pin Placement

The second phase in the data collection process involved checking for the presence of regulatory and advisory speed limit signs either on the segment or in the vicinity of the roadway segment and marking with a pin in the GE. Figure 8 shows an example of markers placed in GE.

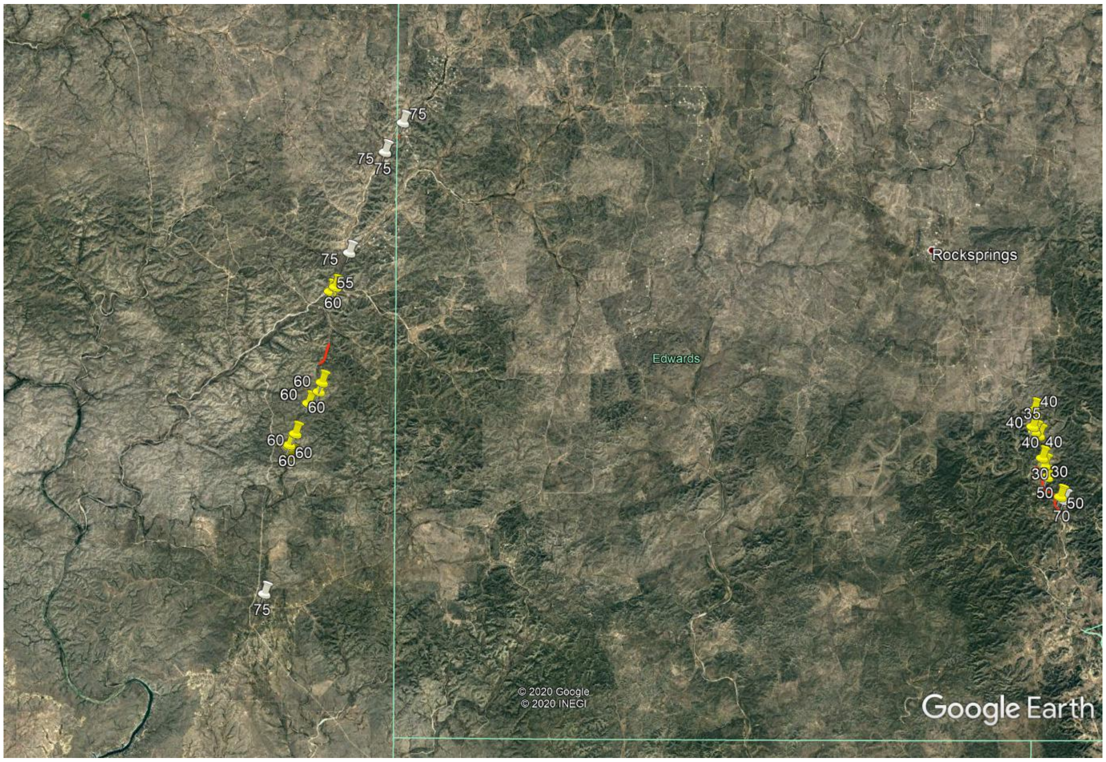
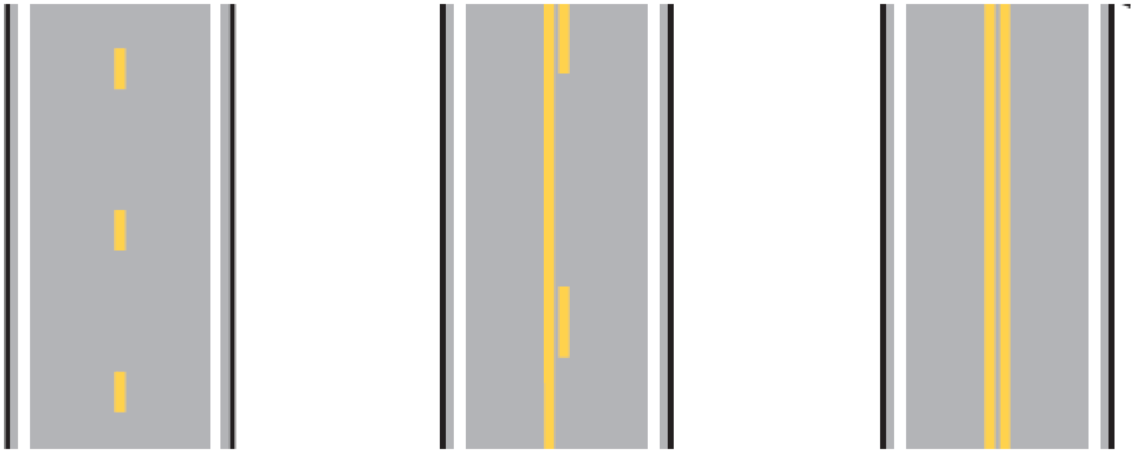


Figure 8. Speed Limit Pins in Google Earth.

Phase 3: Speed Limit, Passing Lane, and Rumble Strip Data Extraction

After the speed limit pins were placed, the information on speed limits was entered into the spreadsheet. The GE street view was used for this task. The information related to the number of curves with advisory speed limits was also obtained. For 2S segments, the passing zone configurations shown in Figure 9 were extracted. The presence of centerline and shoulder rumble strips was extracted from the GE street view as well in this phase.



a) Passing in both directions   b) Passing in one direction only   c) No passing

Figure 9. Passing Lane Configurations.

Phase 4: Cross-Sectional Widths and Horizontal Curve Data
For the fourth phase, the research team collected the cross-sectional data and identified horizontal curve properties on each segment. More specifically, the lane, shoulder, median, and TWLTL widths were measured. The Ruler tool in GE Pro was used to capture linear measurements. The segments had uniform characteristics throughout the segment, but in order to get more accurate values, multiple measurements for the same segment were recorded to calculate the average lane and shoulder widths of the respective segments. The length of the segments was taken from the RHINO database. After confirming the data with a few actual measurements (calculated from GE Pro), the researchers decided that the length of the segment would be directly extracted from RHINO and not measured every time.

For horizontal curve information, the curve on each segment was marked for the calculation of curve radii, chord length, and other horizontal curve calculations. The point of curvature (PC) and point of tangency (PT) were first identified by drawing a straight line along a selected pavement marking. The distance of PC to PT was the curve length and was recorded in the spreadsheet. The middle ordinate measurement was also recorded. Using the curve length and middle ordinate, the other horizontal curve measurements were calculated.

3.2.3. Driveway Volume Data
Traffic volume is one of the most important factors associated with the number of crashes. The AADTs on state-maintained roadways and most of the locally maintained primary roadways were available from TxDOT's RHINO database. However, it was challenging to obtain traffic volumes on driveways.

Wejo Data
In this study, the research team first estimated the traffic volumes on driveways along 10 selected sites using Wejo connected vehicle data that the Texas A&M Transportation Institute purchased. The dataset includes waypoints of vehicles that have been equipped with designated sensors (e.g., global positioning system, braking, acceleration, data connection, etc.) for two months (July 2019 and October 2019) in Texas. The waypoints covered not only major roads but also minor roadways as well as driveways as long as the equipped vehicles travel on them. This made it possible to estimate the AADTs on driveways.

The estimation process included the following four steps:

1. Prepare geographic information system (GIS) format of driveways. In order to estimate the traffic volumes on driveways, it is necessary to identify driveway locations. The research team first drew the driveways on the 10 selected sites using GE. Any driveway that crossed with the selected field sites was identified and drawn in ArcGIS. In total, there were 145 driveways along the field sites. Each driveway was assigned a unique identification number. Figure 10 shows the sites and driveways.

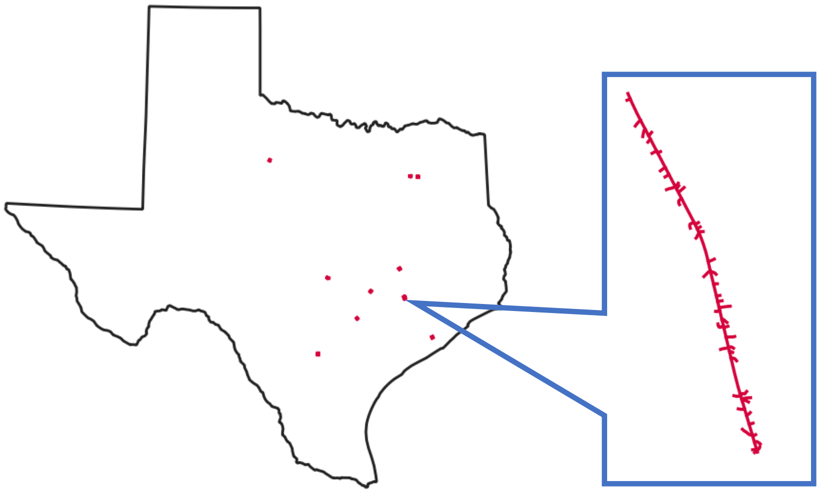


Figure 10. Selected Field Sites and Driveways.

2. Extract vehicle trip waypoints and count trips on selected sites and driveways. The research team created a buffer polygon based on the GIS format of selected sites and driveways. The buffer was set as 500 ft, which is big enough to capture all the waypoints traveling on the roadways. Following that task, Wejo waypoints in the two months were spatially joined with the polygons and extracted to a database server. Figure 11 illustrates the waypoint data along one site as well as its driveways.

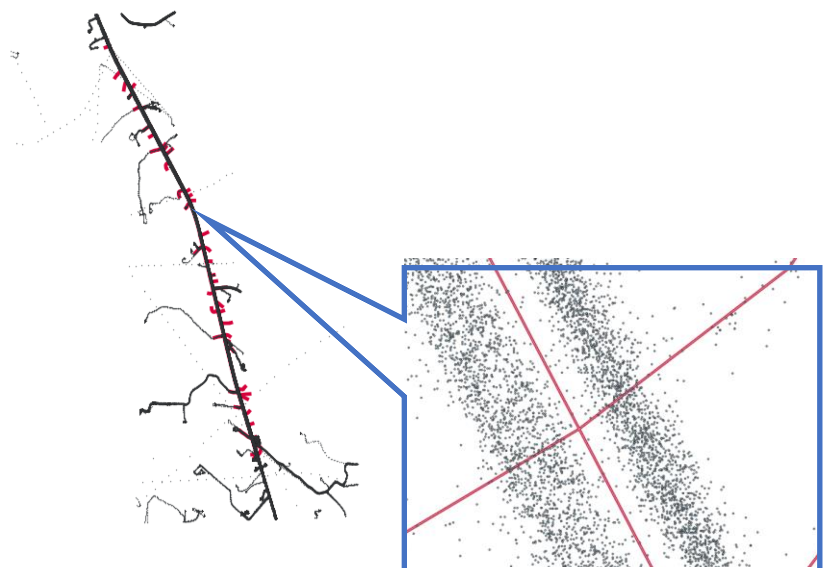


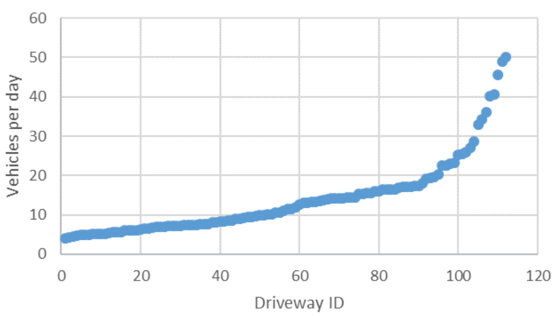
Figure 11. Extracted Waypoint Data on a Site and Its Driveways.

The number of trips (in 60 days) on each of the 10 selected field sites was calculated from the waypoints, as shown in Appendix B (Wejo Trips [60 days] column). Thus, Field01 2S_233 had 15,594 Wejo trips during the 60-day period. Similarly, the number of trips on each driveway was also calculated (see the lower part of Table B1). For example, driveway 2S_233_L01, which is on Field01 2S_233, had 62 Wejo trips during the 60 days.

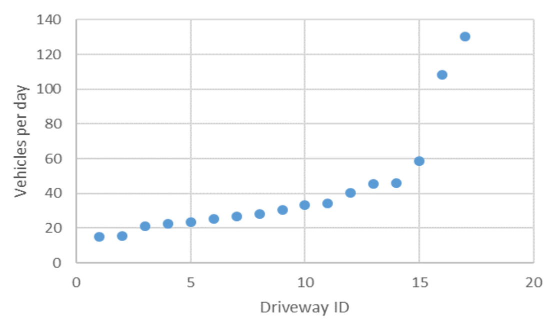
3. Calculate ratio between AADT and Wejo trips. Table B1 also includes the AADT value from RHINO (see AADT from RHINO column). The total number of trips on these segments was 562,206, whereas the total AADT was 131,603. The ratio between AADT and Wejo trips was 1:4.27.

4. Estimate AADT on driveways. Assuming that the ratio between AADT and Wejo was roughly equal on both the selected sites and driveways, the AADT on driveways was estimated based on the observed Wejo trips and the ratio. Taking 2S_233_L01 as an example, there were 62 Wejo trips. The AADT was estimated as 14.5 vpd (i.e., 62/4.27). The detailed Wejo trips and estimated AADT on all the 145 driveways are shown in Appendix B.

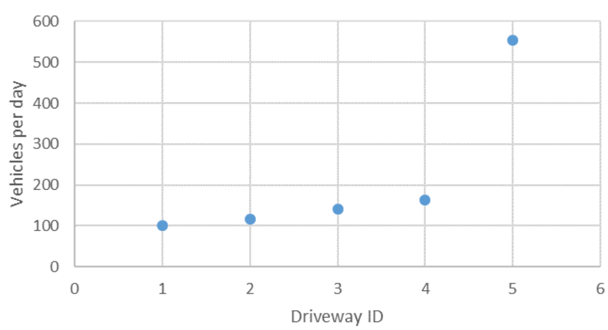
Once the driveway volumes were extracted, the driveways were categorized based on the land use presented in Table 14. Figure 12 shows the volume count for each driveway by different types of driveways. Figure 12a shows that most residential driveway volumes ranged from 5 to 30 vpd. Similarly, Figure 12b and Figure 12c show that most industrial and commercial driveway volumes ranged from 15 to 60 and 100 to 175 vpd, respectively.



a) Residential/Undeveloped



b) Industrial



c) Commercial Business

Figure 12. Driveway Volume Data by Driveway Type.

As shown in Figure 12, the analysis indicated that driveway volume and land use were highly correlated. Recognizing this kind of correlation, Bonneson and Pratt (2009) used land use as a surrogate for driveway traffic volume because these data were not generally available. Similarly, in this project, the research team decided to consider the land use for driveway volumes. The next objective was to convert the industrial and commercial driveways into equivalent residential driveways in terms of vehicular volumes. The second column in Table 15 shows the average volume per driveway when all driveways were considered. The third column shows the average volume when extreme observations were removed. The last column shows the equivalent number of residential driveways in terms of volume.

Table 15. Driveway Volumes by Land Use—Wejo Data.

Land Use	All	Excluding Outliers	Proportion
Residential/Undeveloped	13.9	10.6	1.0
Industrial	41.5	31.1	2.9
Commercial	215.6	131.0	12.1

There were some limitations in the driveway AADT estimate. One of the primary assumptions was that the ratio between AADT and driveways was fixed along the entire analysis area. This may not always have been correct. The Wejo Company collects data from vehicles manufactured by General Motors in recent years. It is possible that the penetration rate of such vehicles in some areas is higher than in others. This analysis only compared the AADT and Wejo trips on 10 selected field sites due to the cost of computation. When the two are collected and compared on a larger scale of area, the ratio may vary.

Video Footage

The research team also collected video footage of driveway activity at four sites to validate the results obtained from the Wejo data. Table 16 provides the counts and types of driveways observed at these sites.

Table 16. Driveway Video Footage Collection.

Highway Number	Nearest Town	Video Data Collection Date(s)	Driveway Count	
			Commercial	Residential
SH 21	Bryan	2/24/2021, 3/3/2021	6	1
SH 36	Kenney	11/19/2020	2	9
SH 36	West Columbia	3/4/2021	0	8
US 80	Edgewood	3/11/2021	0	3

The team collected the video footage for a few hours in the day. Thus, expansion factors were needed to estimate the daily volumes from the partial day counts at the driveways. The expansion factors were derived from FHWA's *Traffic Monitoring Guide* (FHWA, 2016), as shown in Figure 13. Expansion factors were larger for the time period with low driveway volumes and smaller for the time period with high driveway volumes.

A daily volume for a site of interest can be computed using Equation 12:

$$v_d = f_t v_t \tag{12}$$

where:

v_d = daily driveway volume, vpd.
f_t = expansion factor for time period t.
v_t = driveway volume collected in time period t.

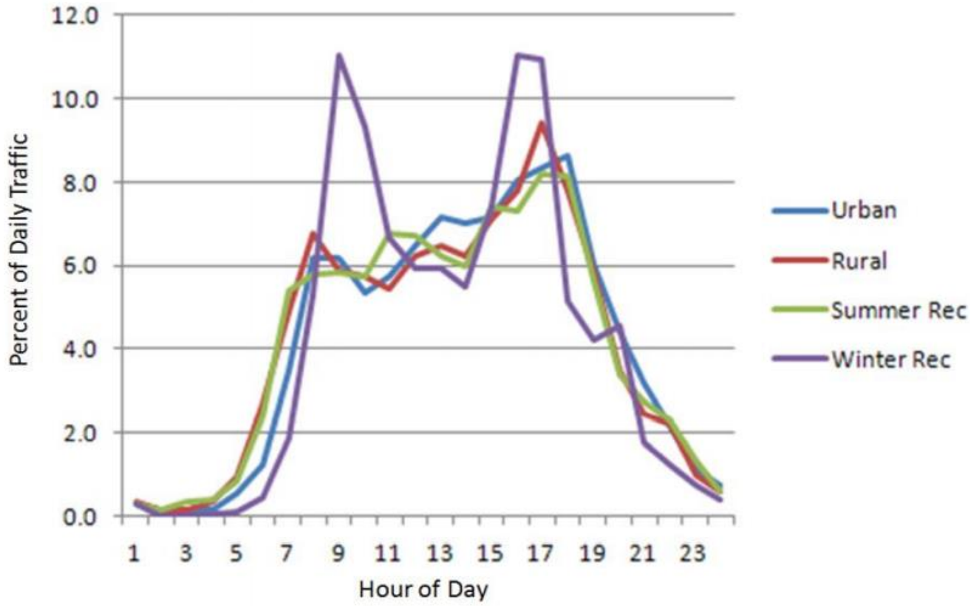


Figure 13. Distribution of Traffic Volumes in the Day.

Table 17 provides the estimated driveway daily volumes based on the volume obtained from the video footage and expansion factors. The second column from the right provides the expansion factors for the time period considered for video footage.

Table 17. Driveway Daily Volumes Estimated from Video Footage.

Highway	Driveway Code	Land Use	Hours Observed	Total Volume	Expansion Factor	Scaled ADT
SH 21 EB	COM_L1	Commercial	3.7	27	6.4	114
SH 21 EB	COM_R1	Commercial	3.7	7	6.4	30
SH 21 EB	COM_R2	Commercial	3.7	66	6.4	280
SH 21 WB	COM_L1	Commercial	3.9	6	6.2	25
SH 21 WB	COM_L2	Commercial	3.9	4	6.2	17
SH 21 WB	COM_R1	Commercial	3.9	17	6.2	70
SH 21 WB	COM_R2	Commercial	3.9	16	6.2	66
SH 36 NB	COM_R1	Commercial	4.3	135	6.2	509
SH 36 NB	COM_R2	Commercial	4.3	130	6.2	490
SH 21 WB	RES_L1	Residential	3.9	3	6.2	12
SH 36 NB	RES_R1	Residential	4.3	0	6.2	0
SH 36 NB	RES_L1	Residential	4.3	5	6.2	19
SH 36 NB	RES_L2	Residential	4.3	1	6.2	4
SH 36 NB	RES_L3	Residential	4.3	12	6.2	45
SH 36 NB	RES_L1	Residential	4.6	0	6.2	0
SH 36 NB	RES_L2	Residential	4.6	0	6.2	0
SH 36 NB	RES_L3	Residential	4.6	20	6.2	71
SH 36 NB	RES_R2	Residential	4.6	1	6.2	4
SH 36 NB	RES_L1	Residential	3.8	0	6.4	0
SH 36 NB	RES_L2	Residential	3.8	0	6.4	0
SH 36 NB	RES_L3	Residential	3.8	0	6.4	0
SH 36 NB	RES_R1	Residential	3.8	2	6.4	8
SH 36 NB	RES_R2	Residential	3.8	2	6.4	8
SH 36 NB	RES_R3	Residential	3.8	1	6.4	4
SH 36 SB	RES_R1	Residential	4.5	11	6.4	38
SH 36 SB	RES_R2	Residential	4.5	40	6.4	137
US 80	RES_L1	Residential	4.3	4	6.2	15
US 80	RES_L2	Residential	4.3	11	6.2	42
US 80	RES_R1	Residential	4.3	2	6.2	8

Once driveway volumes were estimated, the driveways were categorized based on the land use presented in the third column of Table 18. These volumes were then used to convert the commercial driveways into equivalent residential driveways in terms of vehicular volumes. The video collection did not include industrial driveways, so they were not considered in this part of the analysis. The second column in Table 18 shows the average volume per driveway when all driveways were considered. The third column shows the average volume when extreme observations were removed. The fourth column shows the equivalent number of residential driveways in terms of volume. The proportion estimated from the video data is similar to the proportion estimated from the Wejo data for commercial driveways shown in Table 15. These results confirm that the Wejo data can be used for driveway volume estimation. Table 15 shows that the typical industrial driveway generates three times more volume than the typical residential driveway and the typical commercial driveway generates 12 times more volume than the typical residential driveway.

Table 18. Driveway Volumes by Land Use—Video Footage.

Land Use	All	Excluding Outliers	Proportion
Residential	20.7	14.6	1.0
Industrial	—	—	—
Commercial	177.9	177.9	12.2

Note: — means not available.

3.2.4. Vehicle Speed Data

The research team developed a data collection plan to obtain the needed vehicle speed and driveway volume observations. The speed data were used to calibrate the simulation models, and the driveway volume data were used to validate the driveway volume estimates developed by reducing the Wejo data.

The speed data were collected at one or two points at each site using side-fire radar units. The collected data files contained the following observations for each vehicle:

- Timestamp of vehicle arrival.
- Vehicle speed (mph).
- Vehicle length (ft).
- Lane number (counted as 1-*n*, where *n* is the total number of lanes, with lane 1 defined as the closest lane to the radar sensor).

The radar units were deployed for approximately 24 hours at each site. Data collection occurred only during clear-weather conditions. The radar units were attached to portable poles, which in most cases were secured to permanent fixtures such as highway signs, utility poles, or luminaires. In a few cases, the research team drove the portable poles into the ground using temporary mounting bases because there were no permanent fixtures at the location where the radar sensor needed to be deployed. The sensors were placed at locations where no sharp curves or minor-street approaches were present, such that vehicles passing the location were more likely to be free-flow than slowing for a curve or an upcoming turn maneuver. At sites where two sensors were used, the sensors were spaced at least 0.2 mi apart at each site so two separate speed distributions could be derived for the site. Two sensors were used at most sites, but data collection was completed at several sites with only one sensor because of equipment issues or unavailability of suitable roadside locations to deploy a second sensor.

For site selection, the research team queried TxDOT's RHINO database to obtain lists of rural highway segments of the aforementioned configurations (4U, 4M, 4T, 2S, and 2ST). The variables that described roadbed width, surface width, and paved surface width were used to estimate the segments' configurations, and then the research team verified the configurations by locating the segments in aerial photographs and checking the pavement markings. The speed limit variable in the RHINO database was used to group the segments by regulatory speed limit, and then the research team verified the speed limits by locating the posted signs in street-level photographs. After obtaining the verified configurations and speed limits, the research team identified field data collection sites and grouped them as shown in Table 19. The width categories correspond to the possible configuration changes permitted by the pavement width. For example, with a narrow pavement width (such as 50 ft), it is not feasible to implement a 4T configuration, but it is feasible to switch between 2S and 4U categories. Similarly, with a wide pavement width (such as 70 ft), it is feasible to convert a 4U segment to 2ST, 4M, or 4T, but a configuration of 2S is unlikely because a notable amount of pavement width would go unused. At each of the sites in Table 19, the research team collected approximately 24 hours of vehicle speed data using the radar systems.

Table 19. Field Data Collection Sites.

Pavement Width Category	Configuration	Speed Limit (mph)	Highway Number	Nearest Town	Distance between Sensors (mi)	Radar Data Collection Date
Narrow (≤ 55 ft)	2S	60	SH 16	Jourdanton	One sensor	3/29/2021
	4M	55	US 259	Nacogdoches	One sensor	3/23/2021
	4U	55	SH 36	West Columbia	0.2	11/19/2020
Intermediate (56–66 ft)	2S	70	US 183	Gonzales	0.7	9/24/2020
	2ST	65	SH 21	Bastrop	0.3	9/24/2020
		65	US 79	De Berry	One sensor	3/23/2021
	4U	70	SH 71	Marble Falls	0.4	10/6/2020
Wide (> 65 ft)	2ST	70	SH 36	Kenney	0.6	10/1/2020
	4M	65	US 80	Edgewood	One sensor	4/21/2021
	4T	65	SH 21	Bryan	0.4	9/14/2020
	4U	70	US 80	Wills Point	0.8	1/19/2021

CHAPTER 4: SAFETY DATA ANALYSIS

This chapter presents the results of the safety data analysis. The chapter is divided into four sections. The first section describes the exploratory data analysis. The second section presents an analysis of before–after data for the sites where four-lane undivided sections were converted to other cross sections. The third section documents the results of cross-sectional modeling. The last section presents the PS matching analysis that was used to validate the findings of cross-sectional modeling.

4.1. EXPLORATORY DATA ANALYSIS

The research team used the stratified random sampling, as presented in Chapter 3, and identified the 4U segments across Texas for data collection. For 4M, 4T, and 2ST, the team used the survey responses in Chapter 2 to identify the segments. The segments in the medium-to-very-large urban area were excluded. In addition, only segments with a speed limit of 45 mph or higher were considered. Table 20 shows the number of segments and mileage by the cross-section type.

Table 20. Number of Segments by Cross-Section Type.

Cross-Section Type	Number of Segments	Total Length (Miles)
4U	131	76.2
4M	95	60.6
4T	536	148.8
2S	463	419.0
2ST	10	9.2

4.1.2. Crash Rate Analysis

The number of crashes on any given segment was associated with a number of factors, but the length of the segment and traffic volume (which combined were known as exposure) had a great influence on the number of crashes. The segments in this database were of differing lengths and traffic volumes. Therefore, it was desirable to know the crash rate in order to better understand the safety performance of each segment and compare the segments.

The crash rate at each roadway segment was calculated by dividing the number of crashes in any given crash category by the product of length and traffic volume (in this case, the length in miles multiplied by the annual traffic volume, or vehicle miles). Because the number of crashes relative to the number of vehicle miles was very small, the rates were expressed per MVM because the resulting values were more convenient to express and understand.

Crash rates may be interpreted as the probability (based on past events; in this case, what occurred from 2015 to 2019) of being involved in a crash per instance of the exposure measure. The crash rates were developed for different collision types and severity levels, shown in Table 21. The comparison showed that 4U and 4T had the highest rates and 2S and 4M had the lowest. There were no crashes recorded on the 9 mi of 2ST segments in the time period observed, so the crash rate was not provided for this cross section. In addition, the small sample size of 4M highways might have influenced the crash rate calculation, so the results should be interpreted with caution.

Table 21. Crash Rate Comparison.

Type	All	Non-Int.	Int.	Driveway	KABC	KA	SV	MV	OD+ROR
2S	0.26	0.22	0.03	0.01	0.09	0.03	0.18	0.09	0.12
2ST	—	—	—	—	—	—	—	—	—
4M	0.35	0.27	0.06	0.02	0.15	0.04	0.18	0.17	0.15
4T	0.63	0.42	0.14	0.07	0.21	0.04	0.27	0.37	0.23
4U	0.67	0.47	0.12	0.08	0.26	0.07	0.39	0.28	0.29

Note: Non-Int. = non-intersection crashes; Int. = intersection crashes; — = not available.

Table 22 shows the comparison of crash rates for different AADT levels. Two different AADT levels were considered: low (AADT < 10,000 vpd) and high (AADT ≥ 10,000 vpd).

Table 22. Crash Rate Comparison by AADT Levels.

ADT	Type	All	Non-Int.	Int.	Driveway	KABC	KA	SV	MV	OD+ ROR
Low (<10,000 vpd)	2S	0.27	0.22	0.03	0.01	0.09	0.03	0.18	0.09	0.12
	2ST	—	—	—	—	—	—	—	—	—
	4M	0.40	0.31	0.07	0.02	0.15	0.04	0.25	0.16	0.21
	4T	0.61	0.42	0.15	0.04	0.19	0.04	0.30	0.31	0.25
	4U	0.75	0.54	0.14	0.06	0.28	0.08	0.45	0.29	0.35
High (≥10,000 vpd)	2S	0.21	0.18	0.01	0.02	0.07	0.01	0.14	0.07	0.15
	2ST	—	—	—	—	—	—	—	—	—
	4M	0.31	0.23	0.05	0.02	0.14	0.05	0.12	0.18	0.09
	4T	0.65	0.42	0.14	0.08	0.22	0.04	0.25	0.40	0.22
	4U	0.49	0.31	0.05	0.13	0.22	0.06	0.23	0.26	0.16

Note: — means not available.

Table 23 shows the comparison of crash rates for different truck proportion levels. Two different truck proportions were considered: low (proportion <15 percent) and high (proportion ≥15 percent).

Table 23. Crash Rate Comparison by Truck Proportion Levels.

Truck Proportion	Type	All	Non-Int.	Int.	Drive way	KAB	KA	SV	MV	OD+ ROR
Low (<15%)	2S	0.28	0.23	0.03	0.02	0.09	0.02	0.17	0.11	0.14
	2ST	—	—	—	—	—	—	—	—	—
	4M	0.06	0.06	0.00	0.00	0.03	0.00	0.06	0.00	0.06
	4T	0.76	0.45	0.22	0.10	0.26	0.04	0.28	0.48	0.27
	4U	0.75	0.51	0.18	0.06	0.25	0.12	0.42	0.33	0.33
High (≥15%)	2S	0.25	0.21	0.02	0.01	0.08	0.03	0.18	0.06	0.10
	2ST	—	—	—	—	—	—	—	—	—
	4M	0.37	0.28	0.07	0.02	0.15	0.05	0.19	0.18	0.15
	4T	0.52	0.40	0.07	0.04	0.17	0.04	0.25	0.26	0.20
	4U	0.63	0.45	0.08	0.10	0.26	0.04	0.37	0.25	0.27

Table 24 shows the comparison of crash rates for different paved surface width levels. Three different paved surface widths were considered: narrow (<55 ft), intermediate (55–65 ft), and wide (≥65 ft). 4M sections have poor performance at narrow paved surface widths. 4T sections are not possible when the surface width is lower than 60 ft, and 2S is not a feasible option at wider surface widths.

Table 24. Crash Rate Comparison by Paved Surface Width Levels.

Paved Surface Width	Type	All	Non-Int.	Int.	Drive way	KA BC	KA	SV	MV	OD+ ROR
Narrow (<55 ft)	2S	0.28	0.23	0.03	0.02	0.09	0.02	0.19	0.09	0.12
	2ST	—	—	—	—	—	—	—	—	—
	4M	1.21	0.91	0.22	0.07	0.44	0.07	0.58	0.62	0.55
	4T	—	—	—	—	—	—	—	—	—
	4U	0.82	0.63	0.14	0.05	0.32	0.11	0.55	0.27	0.38
Intermediate (55–65 ft)	2S	0.21	0.19	0.01	0.01	0.08	0.03	0.13	0.08	0.11
	2ST	—	—	—	—	—	—	—	—	—
	4M	0.51	0.32	0.13	0.06	0.16	0.13	0.22	0.28	0.13
	4T	0.85	0.30	0.40	0.15	0.17	0.02	0.17	0.68	0.36
	4U	0.63	0.44	0.10	0.08	0.21	0.09	0.31	0.31	0.24
Wide (≥65 ft)	2S	—	—	—	—	—	—	—	—	—
	2ST	—	—	—	—	—	—	—	—	—
	4M	0.29	0.23	0.05	0.02	0.13	0.04	0.15	0.14	0.12
	4T	0.63	0.43	0.14	0.06	0.21	0.04	0.27	0.36	0.23
	4U	0.57	0.34	0.11	0.12	0.26	0.02	0.32	0.25	0.27

Note: — means not available.

The analysis based on crash rate is subject to a few limitations. First, the crash rate method completely depends on the observed crash data—in this case, from law enforcement reports submitted to the state. The issue of data quality and accuracy arises due to the limitations in recording, reporting, and measuring crash data with accuracy and consistency (for example, not every crash will be reported). In addition, not all crashes are geocoded, which makes it difficult to assign them to a particular segment. Second, crash rates presume a linear relationship between crash frequency and the exposure measure, which is typically not true. There are proportionally fewer (or more) crashes per passing vehicles as the traffic flow increases, and thus the crash risk per vehicle diminishes (or increases) when traffic flow increases. Third, the crash rates are highly sensitive to the segment length. Even one crash on a very short segment produces a relatively high crash rate. These results in undue importance placed on short segments.

4.1.3. Crash Characteristics

To understand what crashes are overrepresented on a particular cross-section type, the research team reviewed the roadway segments to examine the characteristics of all crashes and fatal and serious injury (KA) crashes. The team examined SV and MV crashes, light condition, wet pavement, intersection crashes, and tractor-trailer–involved crashes, separately.

Single-Vehicle versus Multi-Vehicle Crashes

2S and 4M roadways had more (69 percent and 54 percent, respectively) SV crashes than MV crashes. The other cross sections had more MV than SV crashes (66 percent and 63 percent). This finding may be related to more limited opportunities for conflicts between vehicles on divided roadways, due to restrictions on turning movements created by the median and the separation of opposing movements. The relationships are shown in Figure 14.

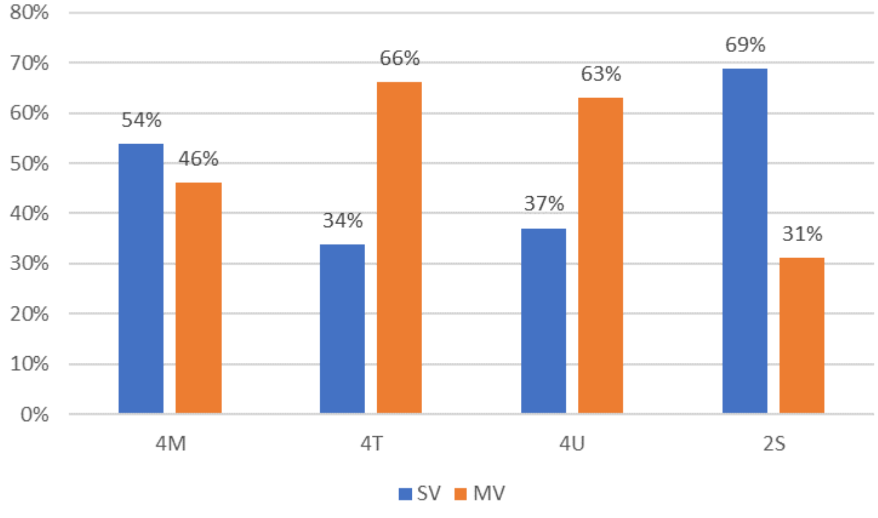


Figure 14. SV versus MV Crashes.

In contrast, more severe crashes tended to involve multiple vehicles for all the cross sections, although this split was more pronounced for 4M and 4T roadways, where 79 percent and 72 percent of KA crashes were MV, respectively. The split was much more balanced for 4U and 2S roads, where 57 percent and 55 percent of the crashes involved more than one vehicle, respectively. Figure 15 depicts these relationships.

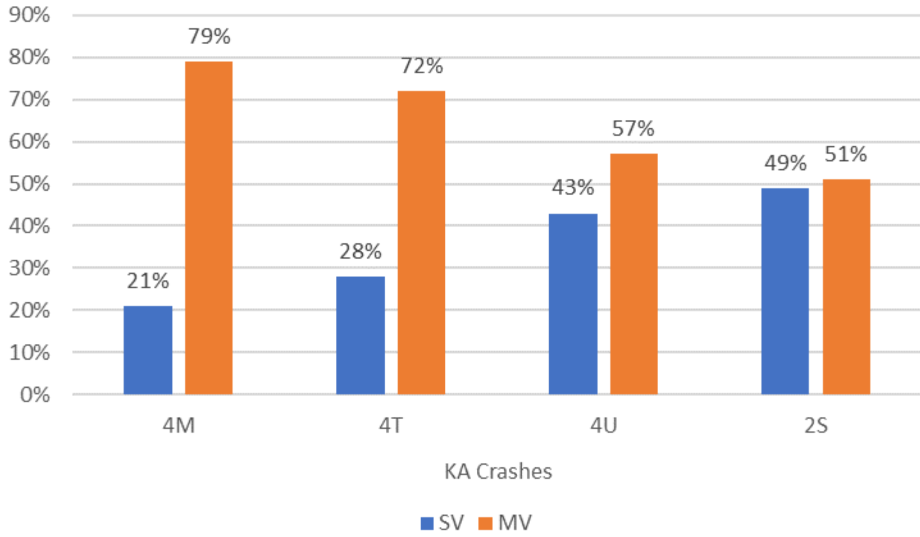


Figure 15**. SV versus MV KA Crashes.**

Light Condition

About half of the SV crashes occurred in dark conditions, regardless of cross section. The MV crashes tended to occur in daylight. Traffic volumes were generally greater during daylight hours, so the opportunities for conflicts between vehicles were higher. This finding also indicated that nighttime SV crashes were overrepresented compared to the proportion of nighttime traffic volume. Figure 16 depicts the relationship between SV and MV crashes during darkness. About 25 percent of MV crashes occurred in the dark, except on 4U roadways, where only 17 percent occurred during darkness.

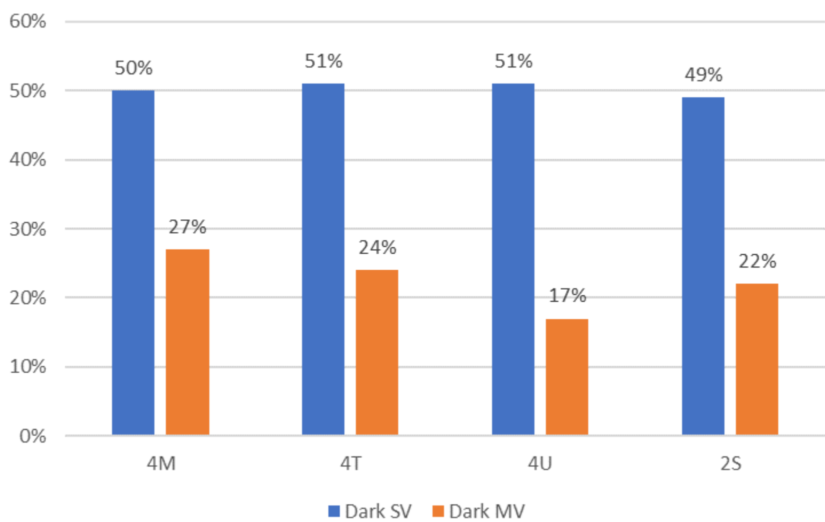


Figure 16**. Crashes Occurring during Darkness.**

In general, this same characteristic was evident for KA crashes, and KA SV crashes tended to be overrepresented at night, although this was less true on 4T roadways, where 40 percent of KA SV crashes occurred in the dark. KA crashes were more likely to occur in periods of darkness than crashes as a whole, in the case of both SV and MV crashes, as can be seen by comparing Figure 17 with Figure 16.

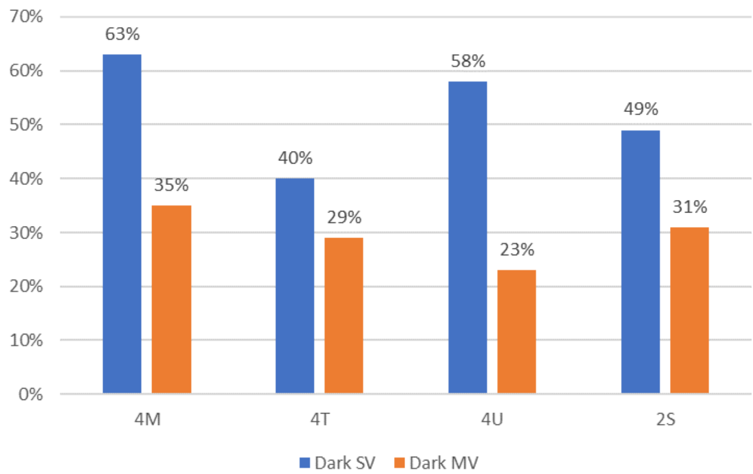


Figure 17**. KA Crashes Occurring during Darkness.**

Intersection Crashes

About half of the MV crashes were recorded at intersections on 4T and 4U roadways. In contrast, 26 percent occurred at intersections on 2S and 4M roadways. As expected, SV crashes were much less likely to be associated with an intersection and ranged from 2 percent to 11 percent. Figure 18 depicts the percentage of SV and MV crashes that occurred at intersections.

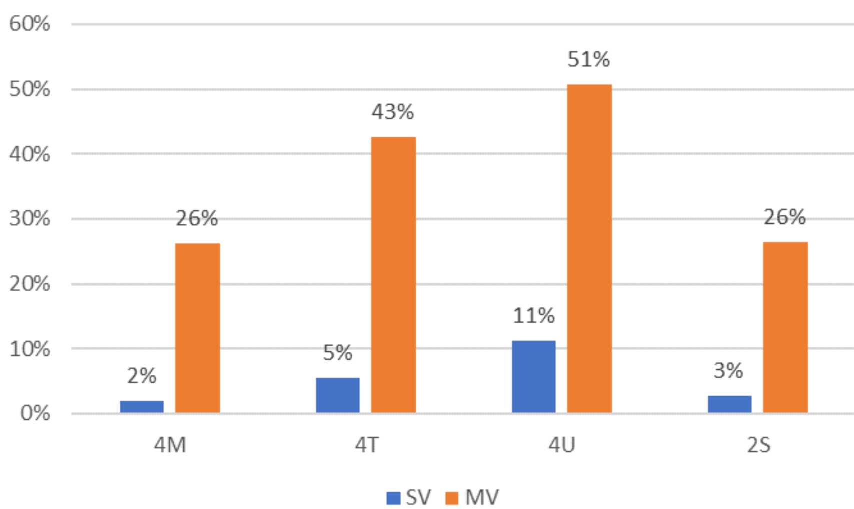


Figure 18**. SV and MV Crashes at Intersections.**

In general, fewer KA crashes occurred at intersections compared to all crashes on 4T and 4U roadways. All of the KA MV crashes occurred at intersections on 4M roadways. The proportion of fatal crashes that occurred at intersections was higher than the proportion of intersection crashes as a whole on 2S roadways. Fewer crashes occurred at intersections on 2S roadways, but they tended to be serious when they did occur. The relationships are depicted in Figure 19.

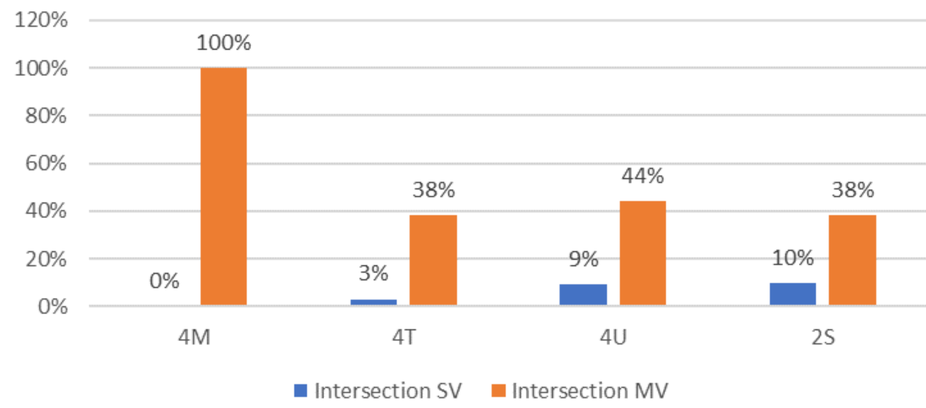


Figure 19**. Percent of KA SV and MV Crashes at Intersections.**

Tractor-Trailer Crashes

The percent of all crashes that involved tractor-trailers ranged from 10 percent to 27 percent. 4M roadways had the highest percent of tractor-trailer–involved crashes (27 percent), with 2S and 4U roads having the least (10 percent and 11 percent, respectively). The percentages were likely related to the amount of truck traffic on each roadway type. Generally, tractor-trailers were more likely to be involved in more serious crashes than crashes in general, except in the case of 4U roadways, which had a lower percent of tractor-trailer involvement in KA crashes than all crashes (11 percent versus 18 percent, respectively). Figure 20 and Figure 21 illustrate the percent of crashes involving tractor-trailers.

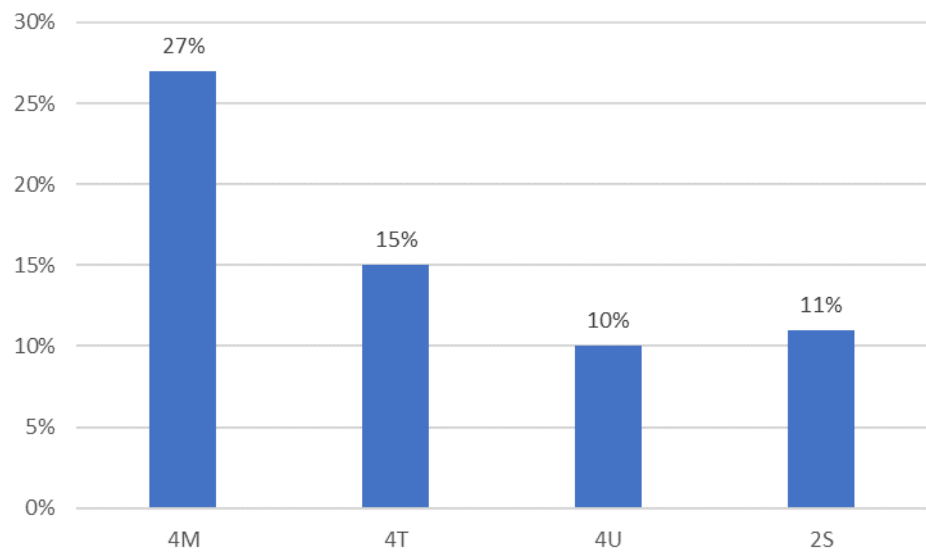


Figure 20**. All Crashes That Involved Tractor-Trailers.**

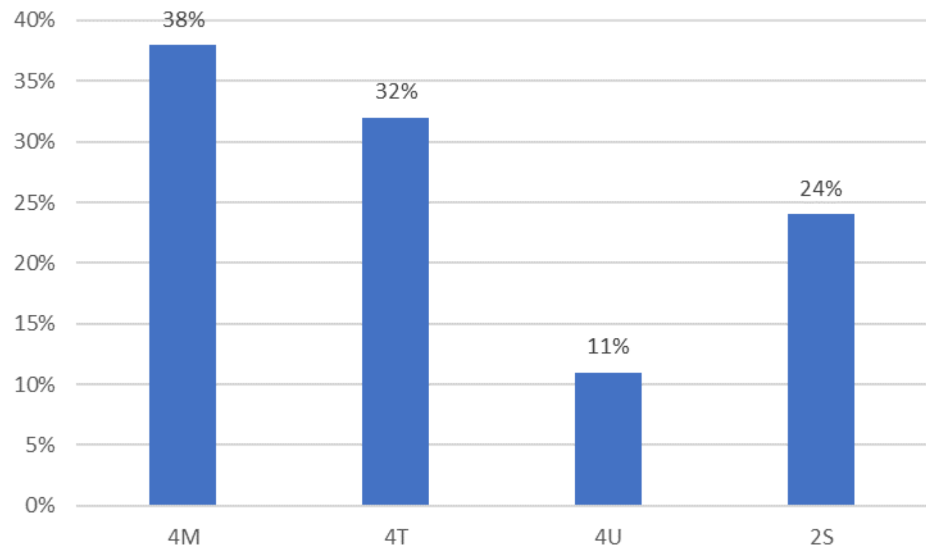


Figure 21**. KA Crashes That Involved Tractor-Trailers.**

Tractor-trailer crashes on 4T and 4U roadways were predominately MV crashes. Crashes on 4M and 2S were more evenly split between SV and MV, with slightly more MV than SV. However, KA crashes involving tractor-trailers were predominantly MV crashes on all cross sections, ranging from 64 percent on 4U roadways to 100 percent on 4T roadways. Figure 22 and Figure 23 depict the percentage of SV and MV crashes on each cross section.

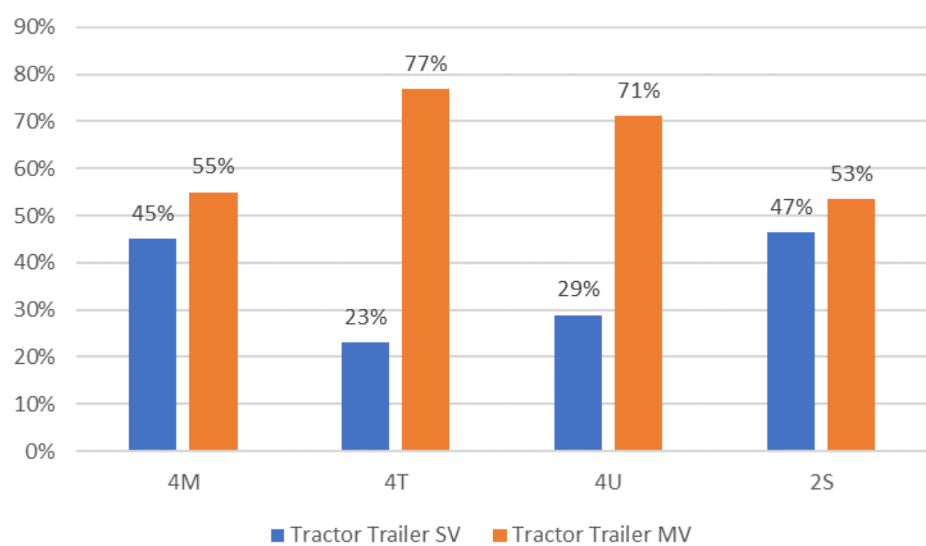


Figure 22**. Percent of SV and MV Crashes.**

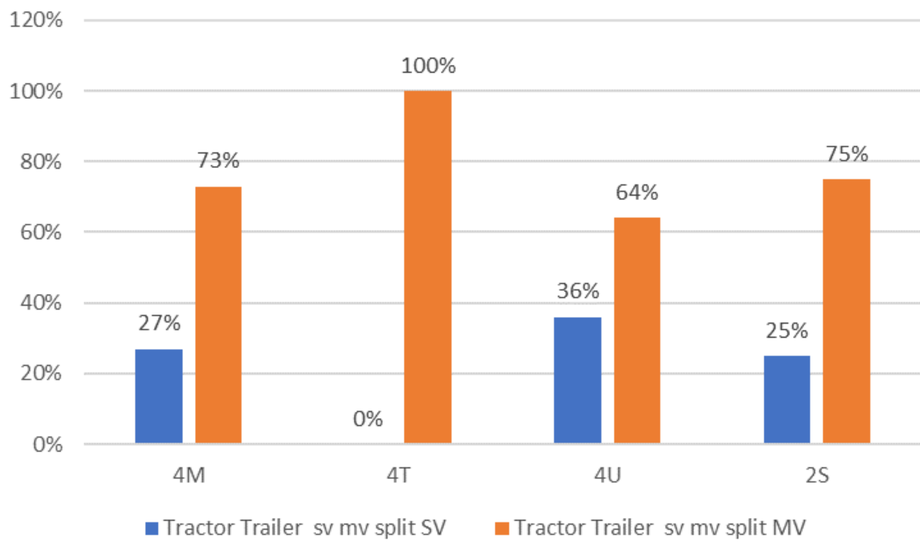


Figure 23**. Percent of KA SV and MV Crashes.**

KA Crashes

The percent of all crashes that were KA ranged from 7 percent (4T) to 14 percent (4M). On 4U roads, 8 percent of all crashes was KA. Figure 24 shows these percentages. The higher percentage of KA crashes as a proportion of all crashes on 4M roadways was an unexpected result that deserves further investigation. Generally, divided roadways are believed to have characteristics that lead to greater safety, and this finding is surprising. However, 4M roadways generally have fewer total crashes per ADT so that, overall, the number of KA crashes as a function of volume may be different than the percentages indicate.

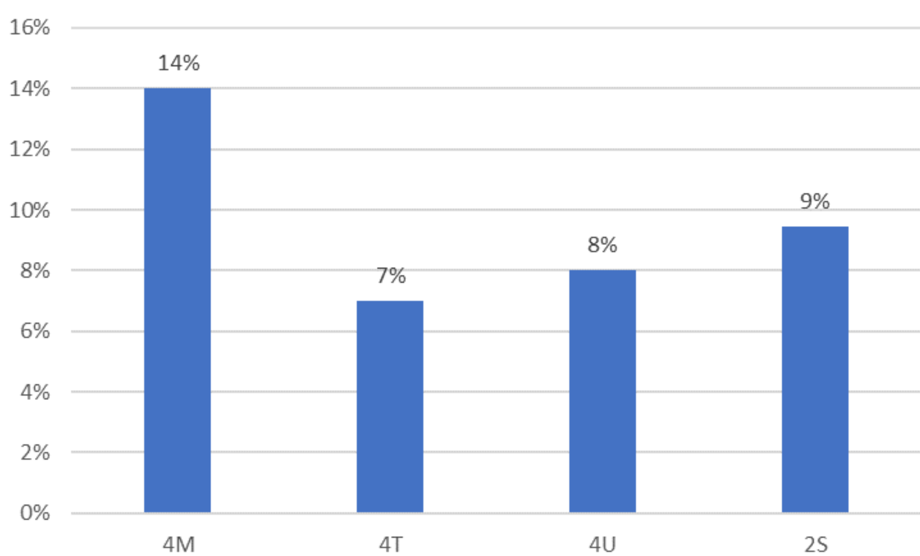


Figure 24**. Percent of All Crashes That Were KA.**

Crash Types

Crashes were divided into SV, head-on, sideswipe, same-direction right turn, angle, left-turn, rear-end, and other categories and then compared within each roadway cross section.

4M Roadways. As Figure 25 depicts, SV crashes comprised just over half of all crashes but only 21 percent of KA crashes. Head-on crashes made up only 8 percent of all crashes but 31 percent of all KA crashes, indicating that the severity of head-on crashes was greater than that of SV crashes on 4M roadways. Rear-end crashes also tended to be severe, comprising 22 percent of crashes but 31 percent of KA crashes. Together, head-on and rear-end crashes made up over 60 percent of all severe crashes. The influence of speed on the severity of head-on and rear-end crashes bears further investigation.

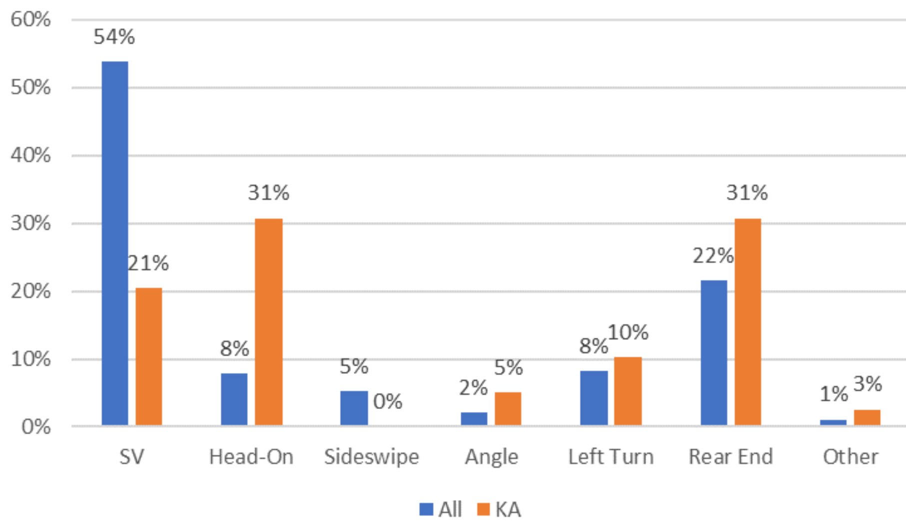


Figure 25**. Crash Type Percentages on 4M Roadways.**

4T Roadways. Figure 26 shows that SV crashes comprised the highest percentage of all crashes and KA crashes. Left-turn crashes comprised the second-highest percentage (22 percent) of KA crashes. Rear-end and angle crashes comprised 18 percent and 15 percent, respectively. Head-on crashes comprised 11 percent of KA crashes but just 3 percent of all crashes, indicating the severe nature of these crashes.

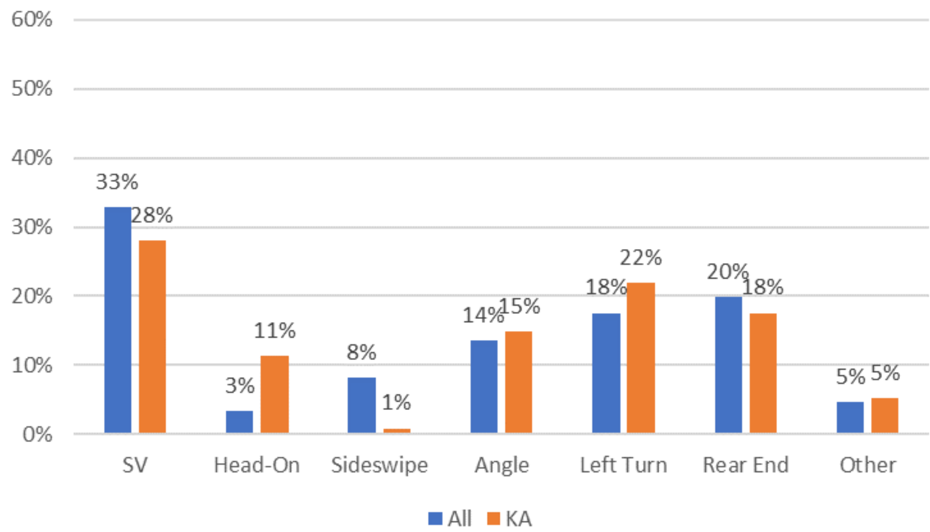


Figure 26**. MV Crash Type Percentages on 4T Roadways.**

4U Roadways. Figure 27 shows that on four-lane undivided roadways, SV crashes constituted about 45 percent of both crashes and severe crashes. Head-on and rear-end crashes each comprised 16 percent of KA crashes. Head-on crashes were once again a greater percentage of severe crashes than all crashes.

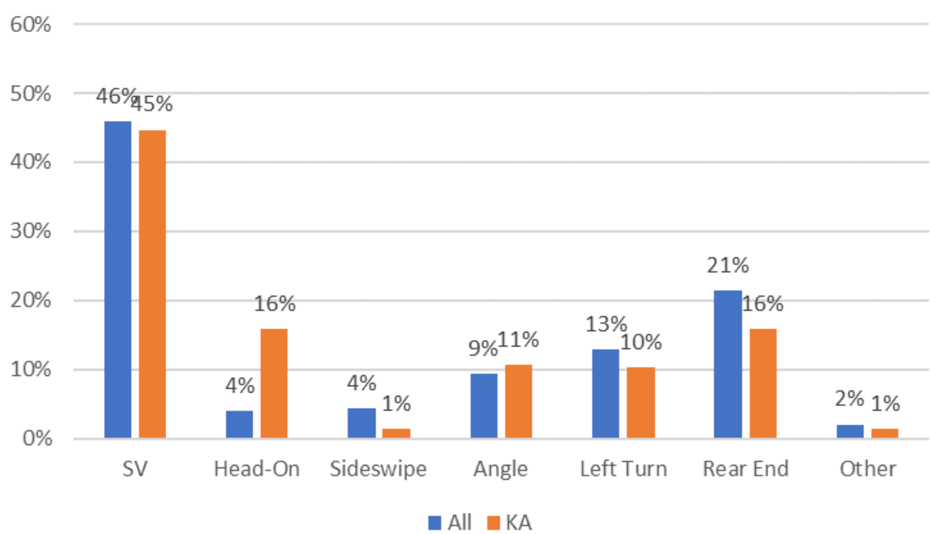


Figure 27**. MV Crash Type Percentages on 4U Roadways.**

2S Roadways. Figure 28 shows that SV crashes comprised the largest percentage of both KA and all crashes, but a lower proportion of the KA crashes were SV. Head-on crashes made up 20 percent of KA but only 6 percent of all crashes. Rear-end crashes comprised the next highest proportion of KA crashes.

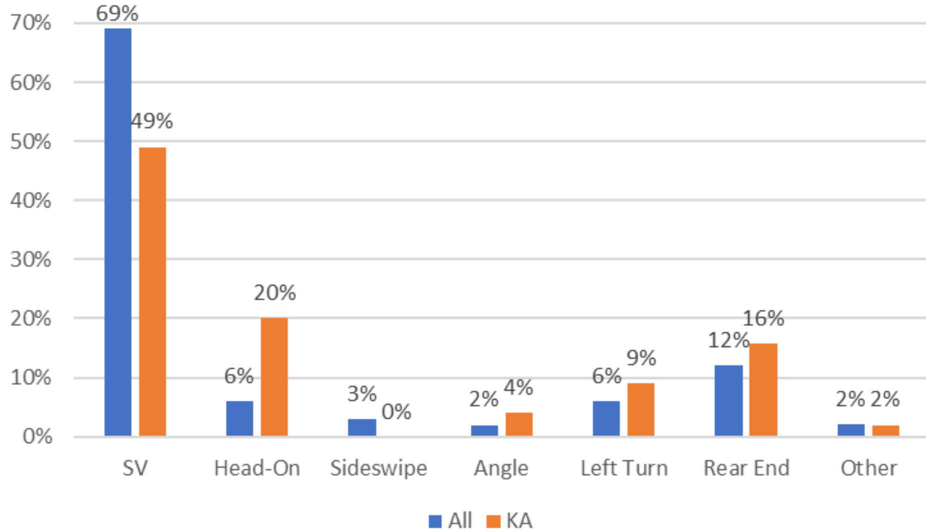


Figure 28**. MV Crash Type Percentages on 2S Roadways.**

Major Findings
The research team's examination of the data resulted in the following major findings:

- The 4U and 4T cross sections had the highest crash rates per vehicle mile traveled (VMT). The 2S cross section had the lowest, and the 4M cross section was the next lowest.
- The serious (KA) crash rates were very similar for all four cross sections.
- The 4T and 4U cross sections experienced more MV than SV crashes. On the 4M and 2S cross sections, there were more SV than MV crashes.
- MV crashes made up the majority of KA crashes on all cross sections. Over 70 percent of KA crashes were MV on 4M and 4T roads. On 4U and 2S roads, 57 percent and 51 percent of KA crashes were MV, respectively.
- KA crashes comprised a higher percentage (14 percent) of all crashes on 4M roadways compared to the other three cross sections, which had 7 percent or 8 percent KA crashes.
- SV crashes were more likely to occur in darkness than MV crashes, and at least 50 percent of SV crashes occurred during darkness regardless of cross-section type.
- Over 50 percent of all SV crashes occurred in dark conditions. Over 50 percent of KA SV crashes occurred in darkness on 4M, 4U, and 2S roadways. Forty percent of SV crashes on 4T roadways occurred at night.
- Intersection crashes accounted for half of the MV crashes on 4M, 4U, and 2S roadways. Intersection crashes comprised 43 percent of MV crashes on 4T roadways and 26 percent on 2S and 4M roads. Intersection crashes comprised a lower percentage of KA crashes compared to all crashes on 4T and 4U, but a higher percentage of KA crashes occurred at intersections than the percentage of all crashes at intersections on 4M and 2S segments.

- 4M roadways had the highest percentage of tractor-trailer crashes (27 percent) and KA crashes (38 percent).
- Tractor-trailer crashes accounted for between 10 and 15 percent of crashes on 4T, 4U, and 2S roadways. The percentage of KA crashes involving tractor-trailers compared to KA crashes was higher on all cross sections except 4U, where 11 percent of KA crashes but 18 percent of all crashes involved tractor-trailers.
- Tractor-trailer crashes on 4T and 4U roadways were predominately MV (70 percent and 80 percent, respectively), whereas crashes were more evenly split between MV and SV crashes on 4M and 2S roadways (although MV crashes still accounted for a slight majority of crashes).
- MV tractor-trailer crashes comprised at least 60 percent of all KA crashes on all cross sections. The percentage was over 70 percent on 4M, 75 percent on 2S, and 100 percent on 4T roadways in the sample. The relationship between these crash statistics and the amount of tractor-trailer traffic on these roads should be examined.
- SV loss-of-control crashes were the predominant crash type and also comprised the highest percentage of severe (KA) crashes, except on 4M roadways, where head-on and rear-end crashes each comprised 31 percent of severe crashes. This was despite the fact that they comprised significantly less of the overall crashes than SV crashes did. Given that the median buffer was primarily aimed at reducing head-on crashes, this finding bears further investigation, particularly in terms of the speed.
- SV crashes comprised over 40 percent of the KA crashes on both 4U and 2S cross sections, which was about twice as much as the percentage on 4M and 4T roadway segments.
- Left-turn crashes comprised the second-greatest percentage of severe crashes on 4T and 2S roadway cross sections.
- Rear-end crashes comprised between 16 percent and 18 percent of severe crashes on 4T, 4U, and 2S cross sections. They comprised nearly twice that percentage of KA crashes on 4M facilities.

4.2. BEFORE–AFTER ANALYSIS

Based on the survey responses in Task 2, four highway corridors were converted from 4U to other cross sections. Table 25 shows these conversions with the highway names, limits, construction dates, and before–after periods considered in this study.

Table 25. Conversions from 4U to Other Cross Sections.

Highway	Limits	Construction Period	Before Start	Before End	After Start	After End*
SH 158 (4U to 4M)	4 mi SE of I-20 to Glasscock County Line	Sep 2017 to May 2018	09/01/2014	08/31/2017	06/01/2018	12/31/2020
SH 349 (4U to 4M)	Midland County Line to Dawson County Line	Sep 2019 to May 2020	09/01/2016	08/31/2019	06/01/2020	12/31/2020
US 79 (4U to 2ST)	LA State Line to FM 31	08/20/2019 to 06/29/2020	08/20/2016	08/19/2019	06/30/2020	12/31/2020
SH 21 (4U to 2ST)	Quarter Horse Loop to S Old Potato Road	08/23/2016 to 01/17/2017	08/23/2013	08/22/2016	01/18/2017	01/17/2020

* 12/31/2020 is the last date on which the complete crash data were available.

Table 26 and Figure 29 show the change in crashes after conversion from 4U to 4M. On both highways, all, OD, and ROR crashes decreased after conversion to 4M. However, severity B crashes increased after the conversion. There was no clear trend for change in other crash types. SH 349 had 7 months of after period only; thus, conclusions cannot be drawn due to a very short after period. In addition, the change in traffic and environmental factors were not considered, so the change in crashes cannot be directly attributed to the change in cross section. The results are presented for informational purposes only.

Table 26. Change in Crashes after Conversion from 4U to 4M.

Hwy	Period	Crash Severity/Type								
		All	KAB	KA	B	Int. related	OD	ROR	SD	Angle
SH 158 (12.23 mi)	Before (36 mo)	130	26	16	10	13	13	67	20	6
	After (31 mo)	70	21	11	10	25	3	15	30	13
	Difference (crashes/mo)	-1.35	-0.04	-0.09	0.04	0.45	-0.26	-1.38	0.41	0.25
	Change in crashes	-37%	-6%	-20%	16%	123%	-73%	-74%	74%	152%
SH 349 (29.11 mi)	Before (36 mo)	171	48	22	26	26	14	63	55	11
	After (7 mo)	26	12	5	7	1	1	11	10	1
	Difference (crashes/mo)	-1.04	0.38	0.10	0.28	-0.58	-0.25	-0.18	-0.1	-0.16
	Change in crashes	-22%	29%	17%	38%	-80%	-63%	-10%	-6%	-53%

Note: SD means same direction crashes.

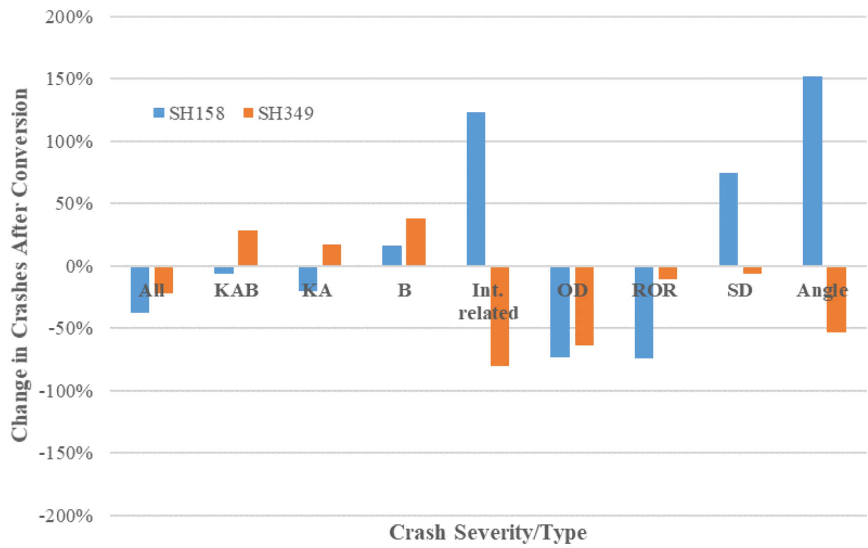


Figure 29. Graphical Representation of Change in Crashes after Conversion from 4U to 4M.

Table 27 and Figure 30 show the change in crashes after conversion from 4U to 2ST. On both highways, KA crashes decreased after conversion to 2ST. However, there was no clear trend for change in other crash types. US 79 had just 6 months of after period, and thus conclusions cannot be drawn due to a very short after period. The results are presented for informational purposes only.

Table 27. Change in Crashes after Conversion from 4U to 2ST.

Hwy	Period	Crash Severity/Type								
		All	KAB	KA	B	Int. related	OD	ROR	SD	Ang.
US 79 (7.03 mi)	Before (36 mo)	68	22	9	13	18	6	20	19	12
	After (6 mo)	14	4	1	3	5	2	4	3	2
	Difference (crashes/mo)	0.44	0.06	-0.08	0.14	0.33	0.17	0.11	-0.03	0.00
	Change in crashes	24%	9%	-33%	38%	67%	100%	20%	-5%	0%
SH 21 (3.71 mi)	Before (36 mo)	49	17	7	10	28	4	10	28	4
	After (36 mo)	27	9	5	4	6	1	6	7	3
	Difference (crashes/mo)	-0.61	-0.22	-0.06	-0.17	-0.61	-0.08	-0.11	-0.58	-0.03
	Change in crashes	-45%	-47%	-29%	-60%	-79%	-75%	-40%	-75%	-25%

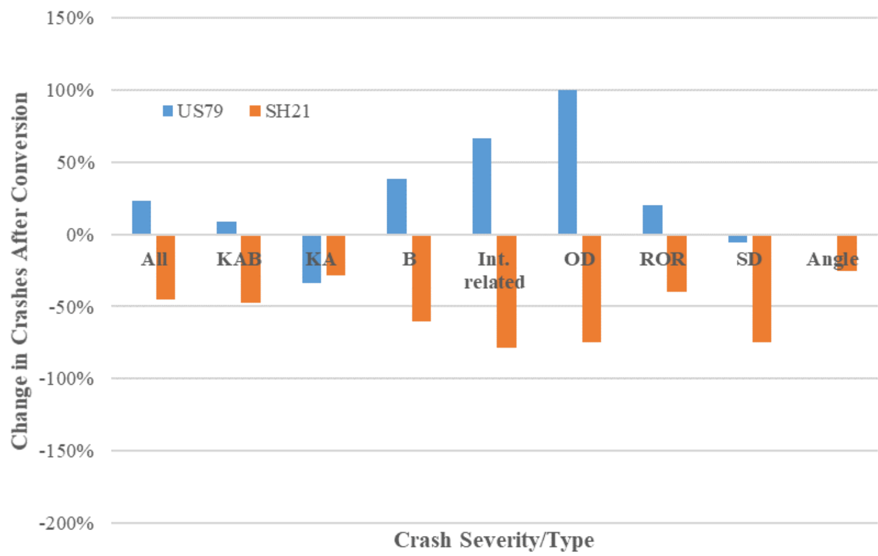


Figure 30. Graphical Representation of Change in Crashes after Conversion from 4U to 2ST.

4.3. CROSS-SECTIONAL MODELING

This section presents the results of the cross-sectional statistical analysis. The primary objective of this task was to develop SPFs to describe the relationship between crash frequency and traffic and geometric variables for horizontal curves in Texas. The development of cross-sectional safety prediction models offers the advantage of quantifying the effects of a range of variables, even if some of the variables are correlated, and yielding insight that is more applicable to a range of sites. In general, a robust safety prediction methodology requires the use of a cross-sectional study approach.

Cross-sectional data have an independent variable value averaged for each site over a particular period of time. The cross-sectional data approach has the following advantages:

- It provides a more robust predictive model than panel data when the year-to-year variability in the independent variables is largely random.
- It minimizes the problems associated with overrepresentation of segments or intersections with zero crashes in model calibration.

Table 28 presents the summary statistics of the variables used for SPF development. The database assembled for calibration included crash frequency as the dependent variable. Geometric design features, traffic control features, and traffic characteristics were included as independent variables. Since there were no reported crashes on 2ST segments, an SPF could not be developed for this cross-section type. The crash data were separated into four categories:

- All crashes.
- KABC crashes.
- Non-intersection crashes.
- Lane-departure (OD and ROR) crashes.

Table 28. Summary Statistics for SPF Development.

Cross Section	Sites	Variable	Min.	Max.	Mean	Std. Dev	Sum
2S	463	ADT	884	11,715	4,193.6	2,354.5	—
		Segment length	0.030	5.98	0.90	0.91	419.0
		Proportion of curve	0	1	0.14	0.24	—
		Driveway density[a]	0	92.4	3.1	5.9	—
		Shoulder width	0	13.5	7.4	2.4	—
		Speed above PSL[b]	—	—	—	—	—
		Total crashes	0	21	1.10	2.74	511
		KABC crashes	0	10	0.36	1.10	167
		Non-int. crashes	0	18	0.93	2.30	430
		Lane-departure crashes	0	13	0.49	1.38	227
4M	95	ADT	3,830	28,374	8,708.8	7,354.3	—
		Segment length	0.001	5.18	0.64	1.11	60.6
		Proportion of curve	0	0.8	0.0	0.1	—
		Driveway density[a]	0	83.3	6.0	13.7	—
		Shoulder width	0	11	7.7	2.2	—
		Speed above PSL	4.5	19	7.5	3.9	—
		Total crashes	0	24	1.93	4.24	183
		KABC crashes	0	12	0.80	2.03	76
		Non-int. crashes	0	17	1.47	3.19	140
		Lane-departure crashes	0	13	0.81	1.99	77
4T	536	ADT	504	30,458	10,739.7	5,314.2	—
		Segment length	0.001	3.79	0.28	0.42	148.8
		Proportion of curve	0	1	0.1	0.2	—
		Driveway density[a]	0	200	15.6	30.0	—
		Shoulder width	0	14	9.0	2.7	—
		Speed above PSL	-2	32.5	10.9	8.4	—
		Total crashes	0	43	2.15	4.57	1152
		KABC crashes	0	15	0.71	1.72	380
		Non-int. crashes	0	30	1.44	2.96	771
		Lane-departure crashes	0	13	0.79	1.62	423
4U	131	ADT	119	2,1804	5,837	3,806.6	—
		Segment length	0.025	5.42	0.58	0.84	76.2
		Proportion of curve	0	1	0.1	0.2	—
		Driveway density[a]	0	196.8182	17.3	33.1	—
		Shoulder width	0	14.5	5.6	3.5	—
		Speed above PSL	0.5	34	11.1	8.4	—
		Total crashes	0	22	2.40	3.82	314
		KABC crashes	0	10	0.92	1.59	121
		Non-int. crashes	0	18	1.69	2.88	221
		Lane-departure crashes	0	12	1.05	1.87	138

Note: — means not available.
[a] Equivalent industrial driveways were considered.
[b] Operating speeds were not available on 2S segments.

An important characteristic associated with the development of statistical relationships is the choice of the functional form linking crashes to the covariates. For this work, the functional form was as follows:

$$N = L \times y \times e^{b_0 + b_{aadt}\ln(AADT) + b_r I_\mathrm{r}} \times CMF_1 \times \ldots \times CMF_k \quad (13)$$

where:

N = estimated annual number of crashes per mile.
L = segment length, mi.
y = number of years of crash data, years.
$AADT$ = Average Annual Daily Traffic, vpd.
I_r = indicator for region r ($r = north, east, west, or\ south$).
b_j = calibrated coefficients.

The research team examined various combinations of variables, and the form presented reflects the findings from several preliminary regression analyses. The predicted crash frequency was calculated as follows:

$$N = L \times y \times e^{b_0 + b_{aadt}\ln(AADT) + b_r I_\mathrm{r}} \times CMF_{hc} \times CMF_{dw} \times CMF_{sw} \times CMF_{spd} \quad (14)$$

with:

$$CMF_{hc} = e^{b_{hc}(p_{hc})}$$
$$CMF_{dw} = e^{b_{dw} \times 0.1 \times (dw - 10)}$$
$$CMF_{sw} = e^{b_{sw}(sw - 6)}$$
$$CMF_{spd} = e^{b_{spd}(SpdFF85 - \mathrm{PSL})}$$

where:

CMF_{hc} = CMF for horizontal curve presence.
CMF_{dw} = CMF for driveway density.
CMF_{sw} = CMF for shoulder width.
CMF_{spd} = CMF for 85th percentile free-flow speed.
p_{hc} = proportion of horizontal curve presence on segment.
dw = equivalent industrial driveway density (=3*residential driveways or 0.25*commercial driveway), driveways/mile.
sw = average shoulder width, ft.
$SpdFF85$ = 85th percentile free-flow speed, mph.
PSL = posted speed limit, mph.

Initially, the models were estimated using the varying dispersion parameter, but the variable coefficient was insignificant. As a result, the models were estimated using a fixed dispersion parameter in the subsequent model development. The NLMIXED procedure in the SAS software was used to estimate the proposed model coefficients. This procedure was used because the proposed predictive model was both nonlinear and discontinuous. The log-likelihood function for the negative binomial distribution was used to determine the best-fit model coefficients.

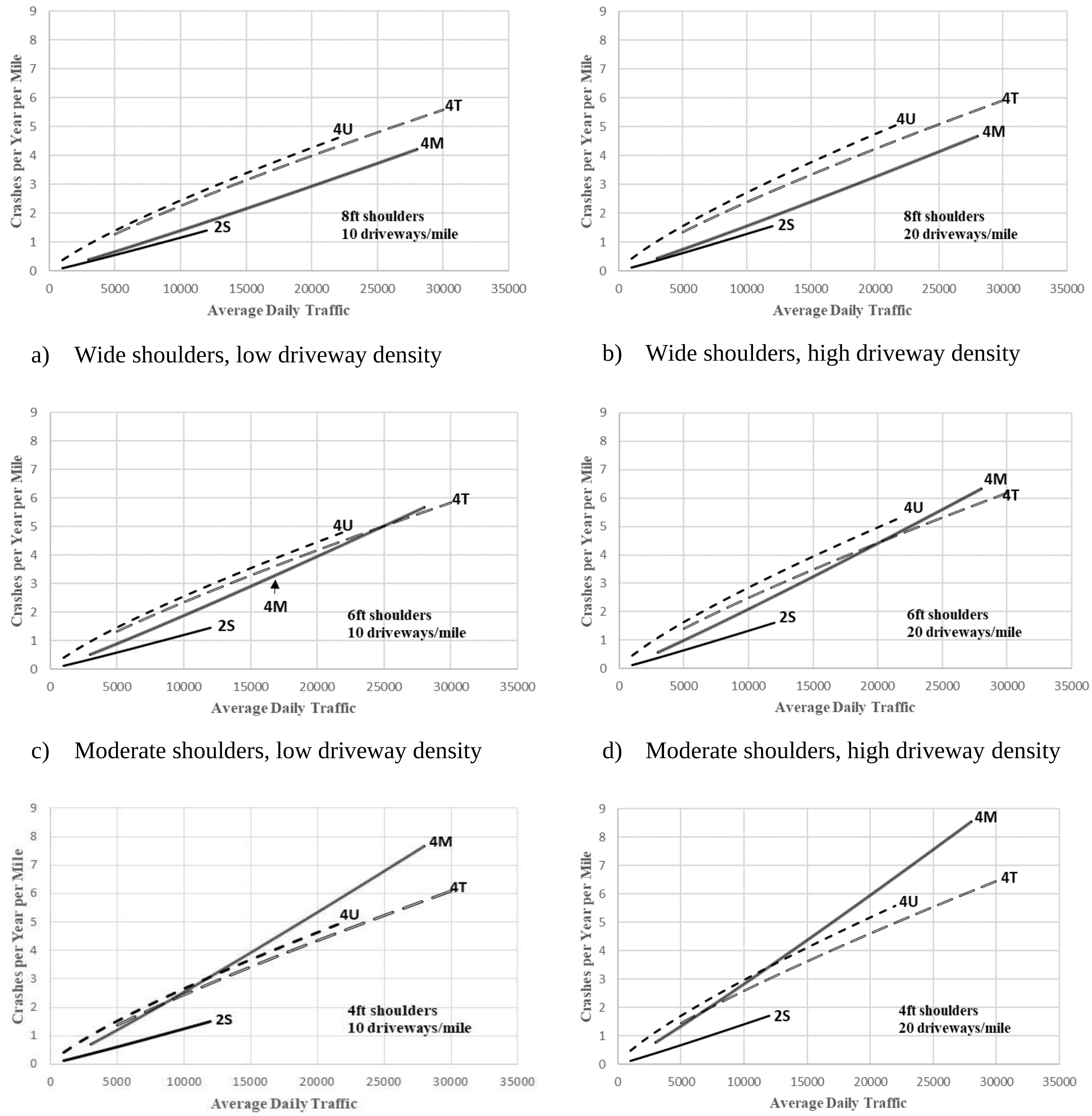
4.3.1. Modeling Results—Total Crashes

Table 29 summarizes the parameter estimates associated with the calibrated SPFs for total crashes. The predictive model calibration process consisted of the simultaneous calibration of SPFs for different cross sections and CMFs using the aggregate model represented by the equations above. The simultaneous calibration approach was needed because several CMFs were common to different cross sections. In general, the sign and magnitude of the regression coefficients in Table 29 are logical and consistent with previous research findings.

Table 29. Calibrated Coefficients for Total Crashes.

Coefficient	Cross Section	Variable	Value	Std. Dev	t-stat.	p-value
b_0	2S	Intercept	-9.518	1.759	-5.41	<.0001
	4U		-6.456	1.410	-4.58	<.0001
	4M		-9.245	1.785	-5.18	<.0001
	4T		-6.786	1.190	-5.7	<.0001
b_{aadt}	2S	AADT	1.053	0.210	5.01	<.0001
	4U		0.803	0.163	4.91	<.0001
	4M		1.073	0.197	5.45	<.0001
	4T		0.829	0.127	6.52	<.0001
b_{hc}	All	Horizontal curve	0.460	0.194	2.37	0.018
b_{dw_l}	All	Driveway density <10	0.241	0.132	1.83	0.0673
b_{dw_h}	2S,4U,4M	Driveway density $\geq$10	0.108	0.032	3.33	0.0009
	4T		0.056	0.024	2.33	0.0199
b_{sw}	2S,4U,4T	Average shoulder width	-0.021	0.017	-1.28	0.2011
	4M		-0.151	0.049	-3.1	0.002
b_{spd}	All	Speed above PSL	0.014	0.006	2.24	0.0252
b_n	All	Effect of north region	-0.258	0.103	-2.51	0.0123
k	2S	Dispersion parameter	4.894	0.685	7.15	<.0001
	4U		0.548	0.164	3.34	0.0008
	4M		0.385	0.162	2.38	0.0176
	4T		0.652	0.095	6.9	<.0001

Figure 31 shows the relationship between the number of total crashes and traffic flow for all cross sections. Four different plots are shown for various shoulder widths and driveway densities. The remaining CMFs are set to 1.0 (representing base conditions). Figure 31 shows that for the same traffic conditions, 2S segments experienced the lowest number of crashes and 4U had the highest. 4M sections provided considerable safety benefits when the shoulder width was 6 ft or higher. For narrow shoulders (i.e., 4 ft or less), they provided poor safety performance. 4T sections were beneficial when the driveway density was higher.



a) Wide shoulders, low driveway density

b) Wide shoulders, high driveway density

c) Moderate shoulders, low driveway density

d) Moderate shoulders, high driveway density

e) Narrow shoulders, low driveway density

f) Narrow shoulders, high driveway density

Figure 31**. Graphical Form of the SPF for Total Crashes.**

4.3.2. Modeling Results—KABC Crashes

Table 30 summarizes the parameter estimates associated with the calibrated SPFs for KABC crashes. Unlike the total crash SPF, only one type of relationship could be established for the driveway densities. Also, the shoulder width was statistically significant for 4M sections only. In addition, a relationship could not be established for vehicle operating speeds.

Table 30. Calibrated Coefficients for KABC Crashes.

Coefficient	Cross Section	Variable	Value	Std. Dev	t-stat.	p-value
b_0	2S	Intercept	-13.323	2.530	-5.27	<.0001
	4U		-6.775	1.610	-4.21	<.0001
	4M		-10.272	2.421	-4.24	<.0001
	4T		-9.400	1.616	-5.82	<.0001
b_{aadt}	2S	AADT	1.357	0.301	4.51	<.0001
	4U		0.716	0.186	3.85	0.0001
	4M		1.074	0.267	4.02	<.0001
	4T		0.981	0.173	5.68	<.0001
b_{hc}	All	Horizontal curve	0.490	0.268	1.83	0.0678
b_{dw}	All	Driveway density	0.084	0.023	3.69	0.0002
b_{osw}	2S,4U,4T	Average shoulder width	—	—	—	—
	4M		-0.156	0.065	-2.39	0.0171
k	2S	Dispersion parameter	5.622	1.175	4.78	<.0001
	4U		0.299	0.186	1.61	0.1087
	4M		0.594	0.315	1.88	0.0597
	4T		0.928	0.195	4.76	<.0001

Note: — means not available.

Figure 32 shows the relationship between the number of KABC crashes and traffic flow for all cross sections. The equations are plotted for the case of all CMFs equal to 1.0 (representing base conditions). Figure 32 shows that for the same traffic conditions, 2S segments experienced the lowest number of crashes and 4U had the highest. 4M had a similar safety performance as 4T until 15,000 vpd but worsened at higher volumes. 4M provided improved safety benefits for shoulders that were 8 ft or higher.

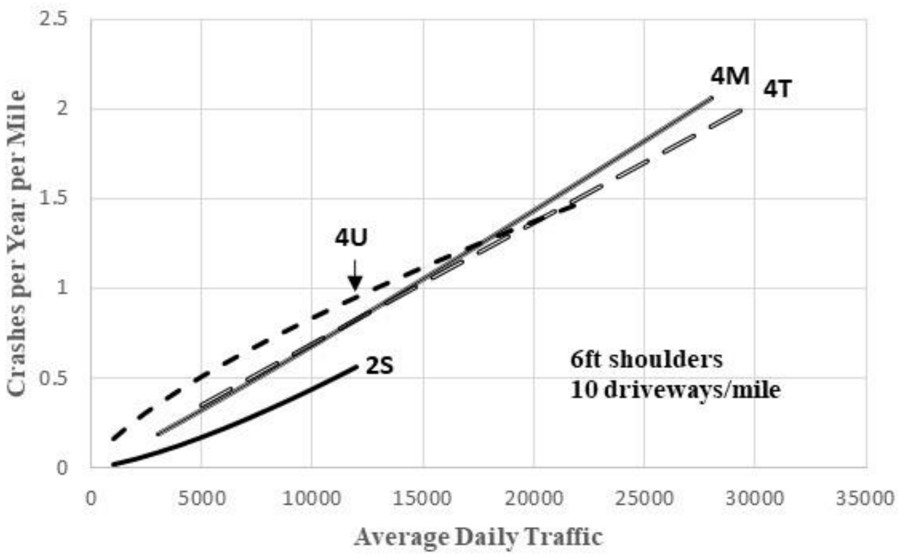


Figure 32**. Graphical Form of the SPF for KABC Crashes.**

4.3.3. Modeling Results—Non-intersection Crashes

Table 31 summarizes the parameter estimates associated with the calibrated SPFs for non-intersection crashes. Similar to the KABC crash SPF, only one type of relationship could be established for the driveway densities. Also, the shoulder width was statistically significant for 4M sections only.

Table 31. Calibrated Coefficients for Non-intersection Crashes.

Coefficient	Cross Section	Variable	Value	Std. Dev	t-stat.	p-value
b_0	2S	Intercept	-8.886	1.732	-5.13	<.0001
	4U		-5.678	1.485	-3.82	0.0001
	4M		-9.260	1.804	-5.13	<.0001
	4T		-7.721	1.091	-7.07	<.0001
b_{aadt}	2S	AADT	0.942	0.208	4.52	<.0001
	4U		0.668	0.173	3.86	0.0001
	4M		1.031	0.199	5.17	<.0001
	4T		0.884	0.116	7.62	<.0001
b_{hc}	All	Horizontal curve	0.473	0.187	2.54	0.0113
b_{dw}	All	Driveway density	0.030	0.019	1.57	0.1158
b_{osw}	2S,4U,4T	Average shoulder width	—	—	—	—
	4M		-0.145	0.050	-2.92	0.0035
b_{spd}	All	Speed above PSL	0.013	0.006	1.98	0.0477
b_n	All	Effect of north region	-0.269	0.101	-2.66	0.0079
k	2S	Dispersion parameter	4.637	0.679	6.83	<.0001
	4U		0.524	0.183	2.87	0.0042
	4M		0.321	0.156	2.06	0.0398
	4T		0.335	0.074	4.53	<.0001

Note: — means not available.

Figure 33 shows the relationship between the number of non-intersection crashes and traffic flow for all cross sections. The equations are plotted for the case of all CMFs equal to 1.0 (representing base conditions). Figure 33 shows that for the same traffic conditions, 2S segments experienced the lowest number of crashes and 4U had the highest until about 12,000 vpd. 4M had better safety performance than 4T. This finding is not unexpected since 4T sections are mostly effective for reducing intersection- and driveway-related crashes.

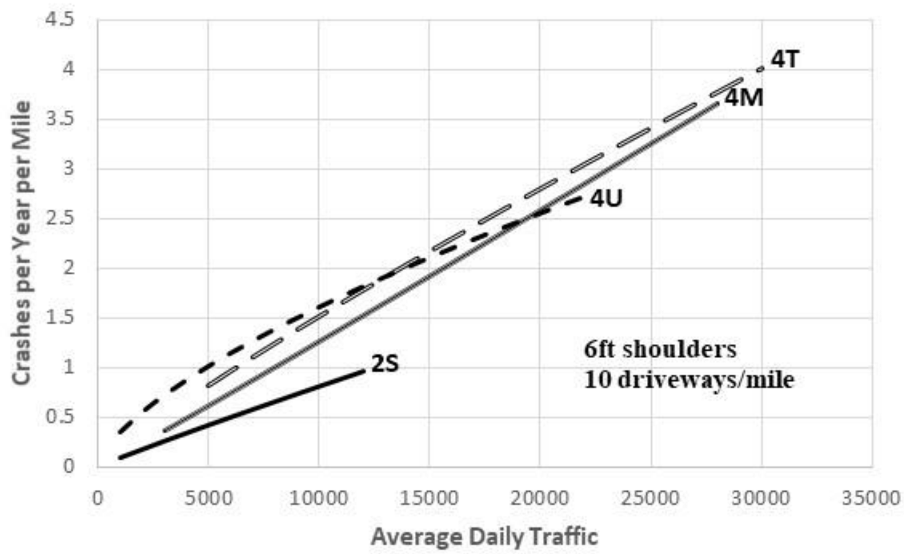


Figure 33. Graphical Form of the SPF for Non-intersection Crashes.

4.3.4. Modeling Results—Lane-Departure Crashes

Table 32 summarizes the parameter estimates associated with the calibrated SPFs for lane-departure (OD and ROR) crashes. Similar to the non-intersection and KABC crash SPFs, only one type of relationship could be established for the driveway densities. Also, the shoulder width was marginally significant for 4M sections in influencing these types of crashes.

Table 32. Calibrated Coefficients for Lane-Departure Crashes.

Coefficient	Cross Section	Variable	Value	Std. Dev	t-stat.	p-value
b_0	2S	Intercept	-13.127	2.211	-5.94	<.0001
	4U		-5.138	1.705	-3.01	0.0026
	4M		-4.023	2.432	-1.65	0.0984
	4T		-6.307	1.528	-4.13	<.0001
b_{aadt}	2S	AADT	1.355	0.261	5.2	<.0001
	4U		0.547	0.198	2.77	0.0057
	4M		0.402	0.266	1.51	0.1313
	4T		0.665	0.163	4.09	<.0001
b_{hc}	All	Horizontal curve	0.702	0.230	3.05	0.0023
b_{dw_l}	All	Driveway density	0.057	0.023	2.48	0.0134
b_{osw}	2S,4U,4T	Average shoulder width	—	—	—	—
	4M		-0.072	0.060	-1.2	0.2314
b_{spd}	All	Speed above PSL	0.016	0.008	1.93	0.0535
b_n	All	Effect of north region	-0.240	0.130	-1.84	0.0655
b_w	All	Effect of west region	-0.375	0.209	-1.79	0.0738
k	2S	Dispersion parameter	3.489	0.667	5.23	<.0001
	4U		0.533	0.214	2.49	0.0129
	4M		0.345	0.211	1.64	0.1022
	4T		0.612	0.138	4.44	<.0001

Note: — means not available.

Figure **34** shows the relationship between the number of lane-departure crashes and traffic flow for all cross sections. The equations were plotted for the case of all CMFs equal to 1.0 (representing base conditions). Figure **34** shows that for the same traffic conditions, 2S segments experienced the lowest number of crashes and 4U had the highest. 4M had much better safety performance than 4U and 4T. This finding was expected since median buffers separated the opposing traffic flows and reduced OD crashes. They were also helpful in reducing lane-departure crashes on the left side.

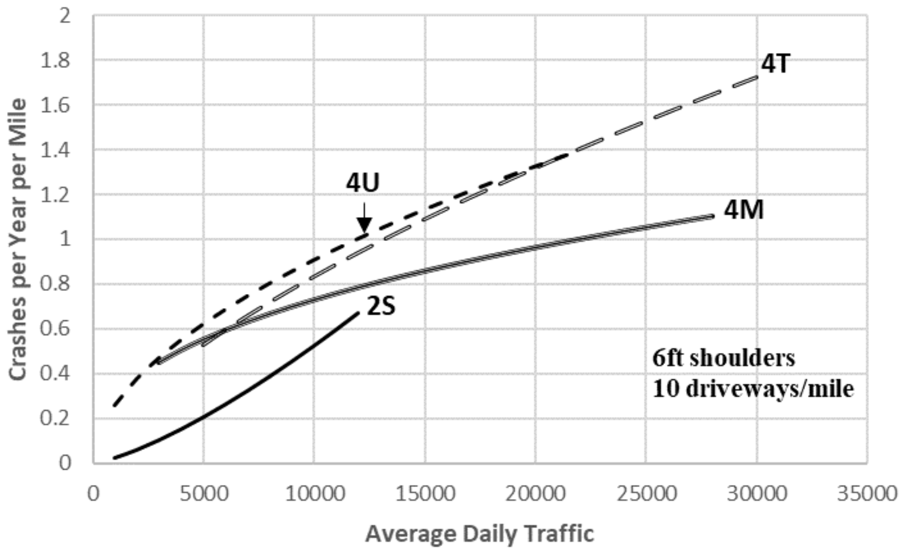


Figure 34**. Graphical Form of the SPF for Lane-Departure Crashes.**

4.3.5. Crash Modification Factors

Several CMFs were calibrated in conjunction with the SPFs. All of them were calibrated using the total crash data. Collectively, they describe the relationship between various operational and geometric factors and crash frequency.

Horizontal Curve CMF

The horizontal CMF is described using Equation 15:

$$CMF_{hc} = e^{0.46(p_{hc})} \tag{15}$$

The base condition for this CMF is no horizontal curves on the segment. Figure 35 shows the truck proportion CMF. The CMF shows that a segment that is completely on a horizontal curve is estimated to experience 58 percent more crashes than a segment without any horizontal curve. The research team attempted to include the radius in the model to capture the horizontal curve sharpness, but the coefficient was not statistically significant.

Figure 35. CMF for Horizontal Curves.

Driveway Density CMF
The driveway density CMF is described using Equation 16:

$$CMF_{dw} = \begin{cases} e^{0.241\times 0.1\times (dw-10)}, \; if\ dw < 10 \\ e^{0.108\times 0.1\times (dw-10)}, if\ dw \geq 10\ and\ for\ 2S, 4U, or\ 4M \\ e^{0.056\times 0.1\times (dw-10)}, if\ dw \geq 10\ and\ for\ 4T \end{cases} \quad (16)$$

The base condition for this CMF is 10 equivalent driveways per mile. The equivalent driveway corresponds to an industrial driveway that generates about 30 vpd. A residential driveway that generates about 10 vpd is equal to 0.33 equivalent driveways. Similarly, a commercial driveway that generates about 120 vpd is equal to four equivalent driveways. Figure 36 shows the driveway density CMF by cross section. The CMF shows that with the increase in driveways, there is an increase in crashes. The increase in crashes on 4T sections was less than on other cross-section types.

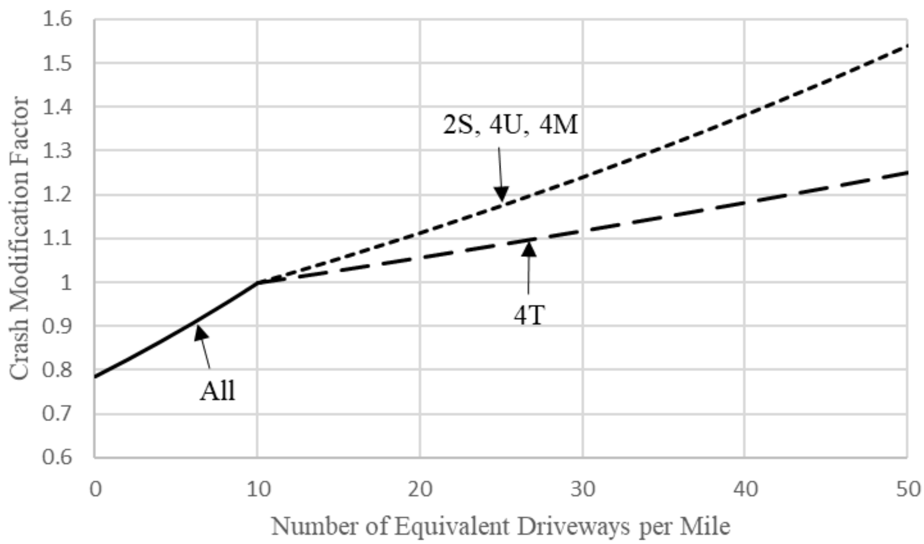


Figure 36**. CMF for Driveway Density.**

Shoulder Width CMF

The shoulder width CMF is described using Equation 17:

$$CMF_{sw} = \begin{cases} e^{-0.151(sw-6)}, if\ 4M \\ e^{-0.021(sw-6)}, if\ 4U, 2S, or\ 4T \end{cases} \quad (17)$$

The base condition for this CMF is a shoulder width of 6 ft. The width used in this CMF is an average for outside shoulders in both directions. The inside shoulder width CMF developed in this study is shown in Figure 37. The inside shoulder widths used to calibrate this CMF ranged from 0 to 14.5 ft. The 4M sections were more sensitive to shoulder widths than the other cross sections.

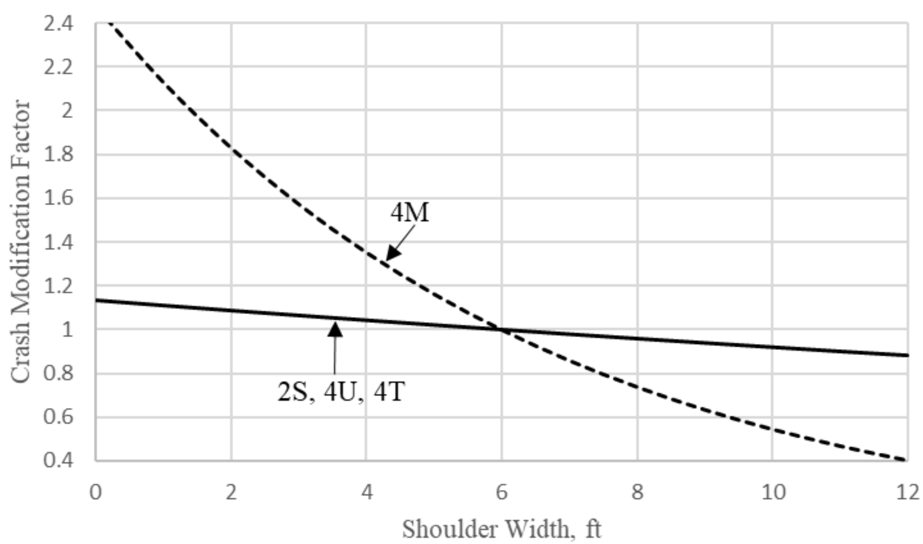


Figure 37**. CMF for Inside Shoulder Width on Freeways.**

Operating Speed CMF
The operating speed CMF is described using Equation 18:

$$CMF_{spd} = e^{0.014(SpdFF85 - \text{PSL})} \quad (18)$$

The base condition for this CMF is that the operating speed (85th percentile free-flow speed) is equal to the PSL. For the sites where the speed information was not available, it was assumed that the operating speed was equal to the PSL. Figure 38 shows the operating speed CMF for the PSL of 65 mph. The CMF shows that the crashes on a highway that has operating speeds more than 10 mph compared to the PSL will experience 15 percent more crashes than a highway with operating speeds equal to the PSL.

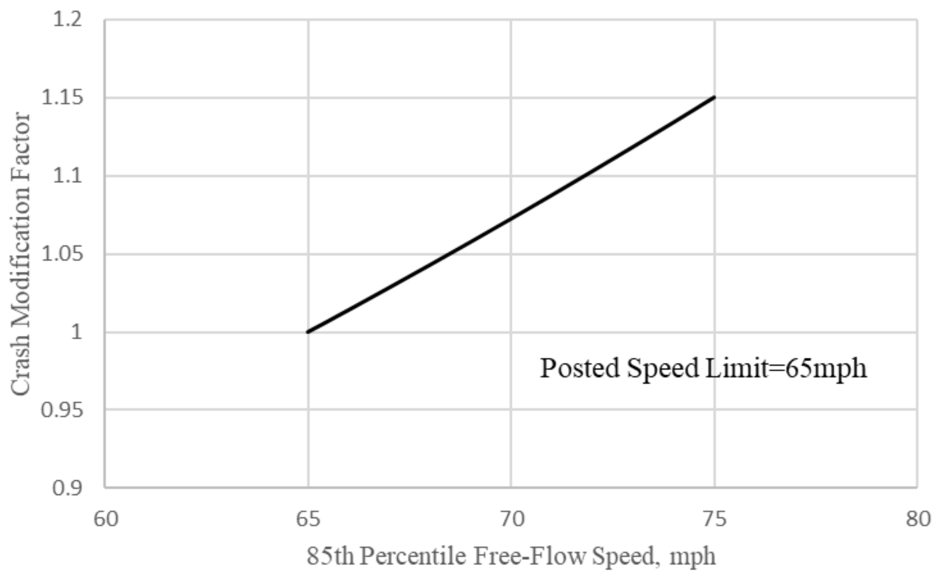


Figure 38**. CMF for Operating Speeds (85th Percentile Free-Flow Speed).**

4.4. PROPENSITY SCORE MATCHING

The previous section presented SPFs for the four cross-section types and estimated the CMFs for a number of roadway geometric and operating features. Although the regression modeling analysis can reveal the association between these features and target crashes (i.e., collision type and severity level), it may not accurately capture the causal-effect relationship between the two. To further examine the effect of different roadway cross-section configurations on safety, this section utilizes the propensity score matching (PSM) method, a causal inference approach, to analyze the crashes on the roadway segments discussed in the previous section.

In causal inference approaches, the effect of a countermeasure or a roadway feature is estimated by comparing the counterfactual crash number to the actual crash number. In the case of this study, when comparing the safety of a 4U roadway against a 4M roadway, the question was what the safety would be if the 4M roadway were converted to a 4U roadway. Due to the confounding factors in the cross-sectional data, it was challenging to estimate the counterfactual safety level of a 4M roadway if it were a 4U roadway. The PSM approach was proposed to estimate the effect of a treatment or a feature by accounting for the variables that predict receiving the treatment or feature. This approach uses the PS to mimic the random selection method, and hence to reduce the bias due to confounding variables that could be found in an estimate of the treatment effect obtained from simply comparing outcomes among units that receive the treatment versus those that do not ("Propensity Score Matching," 2021). Specifically, in this project, the research team considered one facility type as the base condition and another type as the treatment condition, and used the PS between the two to compare their safety performance. Taking 4U and 4M as an example, 4U is the base condition, and 4M is the treatment condition. Through using the PS, the team matched each treatment segment (i.e., 4M) with one or multiple untreated segments (i.e., 4U). The matched pair of segments had similar traffic and roadway characteristics except that one was treated and the other was not. After matching the segments, the team developed a model with the matched segments and examined the different safety levels between the two types of roadways.

Similar to the previous section, this section matches segments between different facility types and analyzes the safety effect at various severity levels. The following paragraph documents the PSM matching process, matched segments, and modeling results for 4U and 4M roadways. The last part documents the PSM results for other types of cross-section configurations.

4.4.1. PSM Analyses—4U and 4M

This section first introduces the matching algorithm, then presents the summary statistics of matched segments, and finally documents the analysis results using matched segments.

Matching Algorithm

As previously mentioned, matching is an important part of the PSM approach. In this project, for estimating the safety difference between 4U and 4M roadways, the project team considered 4U as the untreated or control roadway (denoted as 0) and 4M as the treatment roadway (denoted as 1). In the matching process, a logistic regression model was used to estimate the PS. In the logistic model, the response variable was whether or not a segment was treated (i.e., 1 or 0), and the initial independent variables were as follows:

- Presence of centerline rumble strips.
- Presence of shoulder rumble strips.
- Lane width (ft).
- Average shoulder width (ft).
- Equivalent driveway density (driveways per mile).
- Proportion of length of horizontal curves.
- ADT.
- Segment length (mi).
- VMT.

Ideally, for a given 4M segment, the matching 4U segment(s) should have exactly the same roadway geometric and traffic operation characteristics (e.g., both have the same rumble strip presence, identical length and ADT, the same lane width and shoulder width, and the same proportion of length of horizontal curve). The average shoulder width on 4M segments was artificially modified by subtracting 2 ft from the original shoulder width. This modification was made to ensure that the 2 ft width was extracted from each shoulder of a 4U segment and was used to install a 4-ft median buffer for 4M. Without the shoulder modification, the matching results would compare a 4M segment with a 4U segment having the same lane width and average shoulder width. In this case, the surface of the former would be typically 4 ft wider than the latter since there was a 4-ft median buffer. Because this project studied the trade-offs of reducing shoulder widths, the modification was necessary to keep the same overall surface width.

To eliminate the multicollinearity issue in the logistic regression, the team applied a stepwise model selection algorithm to develop the model. Table 33 shows the logistical model for matching the 4U and 4M segments.

Table 33. Logistic Modeling Results for 4U and 4M Segment Matching.

Variable	Description	Estimate	Std. Error	p-value
(Intercept)	Intercept	**-20.990**	4.284	< 0.001
lw	Lane width	**1.446**	0.336	< 0.001
sw	Shoulder width	**0.461**	0.073	< 0.001
dw	Equivalent driveway density	**-0.033**	0.011	< 0.001
hc	Proportion of horizontal curve	-1.787	1.145	0.119
adt	ADT	0.000	0.000	0.166
L	Segment length	**-0.424**	0.246	0.084
AIC	Akaike information criteria	183.95	—	—

Note: bold = statistically significant at the 90.0 percent level; — = not available.

As can be seen from Table 33, six out of nine independent variables were selected from the stepwise logistic model selection. The estimated parameter for all of them except proportion of horizontal curve length and ADT were statistically significant at the 90.0 percent confidence level. The parameters of the proportion of horizontal curve length and ADT were statistically significant at the 85 percent confidence level. The AIC value of the model was 183.95.

Using the logistic model described in Table 33, the research team calculated the probability of being treated for each control segment (i.e., 4U) and treatment segment (i.e., 4M). This probability was also used as a PS. When a control segment had an identical or similar score with a given treatment segment, they were a matched pair since both shared similar roadway characteristics except that one was treated (i.e., 4M) and the other was not (i.e., 4U). By pairing each of the treatment segments, a subset of untreated segments was identified. The matchit function in the R Package MatchIt was used to pair the segments (Stuart et al., 2011).
The following arguments were used in the matching process:

- Method = nearest (i.e., pairing from the nearest neighbor matching on the PS).
- Replace = FALSE (i.e., no replacement, a control segment can only be matched one time at the most).
- Random = TRUE (i.e., matching takes place in a random order).
- Caliper = 0.1 (i.e., the width of the caliper to use in matching; this is similar to buffer distance).
- Ratio = 2 (i.e., two control segments should be matched to each treatment segment when possible).

Matching Segments

Using the developed logistic model and the matching algorithm described in the previous subsection, the research team successfully matched the 4U segments for the 4M segments. The summary of original segments and matched segments is documented in Table 34.

Table 34. Summary of Original and Matched 4U and 4M Segments.

Facility Type	Segment Number	Crash Frequency (Type)	Total VMT	Crash Rate (Crashes per 100K VMT)
Original Data				
4U	128	254 (Total)	364,825	69.622
	128	98 (FI)	364,825	26.862
	128	175 (Non-Int.)	364,825	47.968
	128	114 (OD+ROR)	364,825	31.248
4M	86	98 (Total)	299,193	32.755
	86	37 (FI)	299,193	12.367
	86	74 (Non-Int.)	299,193	24.733
	86	41 (OD+ROR)	299,193	13.704
Matched Data				
4U	51	106 (Total)	152,090	69.696
	51	38 (FI)	152,090	24.985
	51	71 (Non-Int.)	152,090	46.683
	51	51 (OD+ROR)	152,090	33.533
4M	39	50 (Total)	106,900	46.772
	39	20 (FI)	106,900	18.709
	39	34 (Non-Int.)	106,900	31.805
	39	16 (OD+ROR)	106,900	14.967

As can be seen from Table 34, there were 86 4M segments and 128 4U segments in the original data (since a few filters were used to facilitate the matching algorithm, the number of segments decreased compared to the original data provided in Sections 2 and 4). Among the 86 4M segments, 39 were successfully matched with one or two 4U segments, and the total number of matched 4U segments was 51.

Modeling Result

Based upon the matched segments, the team developed a negative-binomial-based SPF with the following three variables:

- ADT (i.e., traffic volume).
- Segment length (offset variable).
- Facility type (0 = 4U, and 1 = 4M, in this case).

Table 35 shows the modeling results for total crashes.

Table 35. PSM Estimated Coefficients for Total Crashes (4U and 4M).

Coefficient	Variable	Value	Std. Dev	t-stat.	p-value
b_0	Intercept	-8.199	2.542	-3.225	0.001
b_{aadt}	AADT	1.137	0.290	3.915	<0.001
b_{4M}	Indicator for 4M	-0.619	0.313	-1.977	0.048
k	Dispersion parameter	1.239	0.415	—	0.021

Note: 39 matched 4M segments, and 51 matched 4U segments. — means not available.

The parameter for cross-section indicator (i.e., 4M) was −0.619, indicating that compared to 4U roadways, the crashes on 4M roadways were fewer, and this result was statistically significant at the 95 percent level. Specifically, when compared to 4U, 4M roadways had 46.2 percent fewer crashes for the same length and traffic volume.

The modeling results for FI (i.e., KABC), non-intersection, and lane-departure (OD+ROR) crashes are shown in Table 36.

Table 36. PSM Estimated Coefficients for Other Types of Crashes (4U and 4M).

Coefficient	Variable	Value	Std. Dev	t-stat.	p-value
FI Crashes					
b_0	Intercept	-5.039	2.858	-1.763	0.078
b_{aadt}	AADT	0.636	0.327	1.946	0.052
b_{4M}	Indicator for 4M	-0.192	0.344	-0.559	0.576
k	Dispersion parameter	2.333	1.442	—	0.106
Non-intersection Crashes					
b_0	Intercept	-6.567	2.227	-2.948	0.003
b_{aadt}	AADT	0.882	0.253	3.482	0.000
b_{4M}	Indicator for 4M	-0.397	0.270	-1.467	0.142
k	Dispersion parameter	3.769	2.669	—	0.158
OD+ROR Crashes					
b_0	Intercept	-6.204	2.579	-2.406	0.016
b_{aadt}	AADT	0.804	0.293	2.739	0.006
b_{4M}	Indicator for 4M	-0.846	0.333	-2.537	0.011
k	Dispersion parameter	4.149	3.693	—	0.261

Note: 39 matched 4M segments, and 51 matched 4U segments. — means not available.

For all three types (i.e., FI, non-intersection, and lane departure), crashes were lower on 4M roadways compared to 4U roadways. The crash reduction factors were 17.5 percent, 32.7 percent, and 57.1 percent, respectively. However, the reduction factors for FI and non-intersection crashes were not statistically significant. The factor for lane-departure crashes was statistically significant at the 95.0 percent confidence level. The trend of crashes was overall consistent with that discussed in Section 4.1. 4M roadways had fewer total and lane-departure crashes than 4U roadways. FI and non-intersection crashes showed a decreasing trend, albeit statistically insignificant, in the PSM analysis, and the same conclusions were drawn from the SPF analyses for certain AADT and/or shoulder width ranges.

4.4.2. PSM Results—Other Cross Sections

Following the same procedure described above, the research team conducted PSM analyses for the other three pairs of roadway comparisons: (a) 4U versus 4T, (b) 4M versus 4T, and (c) 4U versus 2S. The results are documented in Table 37 to Table 39, respectively.

Table 37. PSM Estimated Coefficients for 4U and 4T.

Coefficient	Variable	Value	Std. Dev	t-stat.	p-value
Total Crashes					
b_0	Intercept	-5.576	1.718	-3.247	0.001
b_{aadt}	AADT	0.872	0.196	4.444	<0.001
b_{4T}	Indicator for 4T	-0.360	0.241	-1.496	0.135
k	Dispersion parameter	0.901	0.194	—	<0.001
FI Crashes					
b_0	Intercept	-6.695	2.215	-3.023	0.003
b_{aadt}	AADT	0.883	0.252	3.503	<0.001
b_{4T}	Indicator for 4T	-0.403	0.305	-1.320	0.187
k	Dispersion parameter	0.837	0.263	—	0.001
Non-intersection Crashes					
b_0	Intercept	-6.308	1.627	-3.876	0.000
b_{aadt}	AADT	0.896	0.185	4.845	0.000
b_{4T}	Indicator for 4T	-0.341	0.222	-1.534	0.125
k	Dispersion parameter	1.720	0.575	—	0.003
OD+ROR Crashes					
b_0	Intercept	-5.053	1.894	-2.668	0.008
b_{aadt}	AADT	0.695	0.216	3.221	0.001
b_{4T}	Indicator for 4T	-0.160	0.261	-0.614	0.539
k	Dispersion parameter	1.398	0.528	—	0.008

Note: 64 matched 4T segments, and 88 matched 4U segments. — means not available.

Table 38. PSM Estimated Coefficients for 4M and 4T.

Coefficient	Variable	Value	Std. Dev	t-stat.	p-value
Total Crashes					
b_0	Intercept	-6.454	2.043	-3.159	0.002
b_{aadt}	AADT	0.855	0.225	3.808	<0.001
b_{4T}	Indicator for 4T	0.349	0.245	1.427	0.154
k	Dispersion parameter	1.130	0.275	—	<0.001
FI Crashes					
b_0	Intercept	-6.129	2.983	-2.055	0.040
b_{aadt}	AADT	0.713	0.328	2.176	0.030
b_{4T}	Indicator for 4T	0.148	0.357	0.414	0.679
k	Dispersion parameter	0.649	0.226	—	0.004
Non-intersection Crashes					
b_0	Intercept	-6.476	1.933	-3.350	0.001
b_{aadt}	AADT	0.826	0.211	3.910	<0.001
b_{4T}	Indicator for 4T	0.194	0.232	0.836	0.403
k	Dispersion parameter	1.826	0.627	—	0.004
OD+ROR Crashes					
b_0	Intercept	-3.988	2.279	-1.750	0.080
b_{aadt}	AADT	0.485	0.250	1.938	0.053
b_{4T}	Indicator for 4T	0.264	0.270	0.977	0.329
k	Dispersion parameter	1.868	0.843	—	0.027

Note: 76 matched 4T and 76 matched 4M segments (1:1 matching ratio due to data sample); a negative crash reduction percentage indicated that the crash number increased. — means not available.

Table 39. PSM Estimated Coefficients for 4U and 2S.

Coefficient	Variable	Value	Std. Dev	t-stat.	p-value
Total Crashes					
b_0	Intercept	-5.079	1.748	-2.906	0.004
b_{aadt}	AADT	0.765	0.207	3.703	<0.001
b_{2S}	Indicator for 2S	-1.239	0.269	-4.612	<0.001
k	Dispersion parameter	0.697	0.167	—	<0.001
FI Crashes					
b_0	Intercept	-5.155	2.054	-2.510	0.012
b_{aadt}	AADT	0.663	0.242	2.741	0.006
b_{2S}	Indicator for 2S	-1.496	0.323	-4.638	<0.001
k	Dispersion parameter	0.972	0.380	—	0.010
Non-intersection Crashes					
b_0	Intercept	-5.222	1.858	-2.810	0.005
b_{aadt}	AADT	0.756	0.220	3.444	0.001
b_{2S}	Indicator for 2S	-1.202	0.285	-4.221	<0.001
k	Dispersion parameter	0.652	0.166	—	<0.001
OD+ROR Crashes					
b_0	Intercept	-4.947	1.900	-2.604	0.009
b_{aadt}	AADT	0.651	0.224	2.907	0.004
b_{2S}	Indicator for 2S	-1.172	0.289	-4.053	<0.001
k	Dispersion parameter	1.042	0.367	—	0.005

Note: 74 matched 2S and 82 matched 4U segments. — means not available.

Table 37 to Table 39 demonstrate that when comparing 4U and 4T roadways, all of the target crashes reduced (i.e., 4T roadways had fewer crashes), but none of them were statistically significant. Similarly, crashes showed an increasing trend from 4M to 4T roadways (i.e., 4T roadways had a higher number of crashes), but none of the results were statistically significant. The comparison between 2S and 4U indicated that 2S roadways had notably fewer crashes. Specifically, total, FI, non-intersection, and OD+ROR crashes reduced by 71.0 percent, 77.6 percent, 69.9 percent, and 69.0 percent, respectively, compared to 4U roadways. The results were statistically significant at the 99.9 percent confidence level. All of the observations were similar to the findings from the cross-sectional modeling.

4.4.3. Summary of the PSM Results

In this section, the research team presented the evaluation of the safety effect of different cross-section configurations using the causal inference approach—PSM. Roadway segments between different pairs of configurations were matched and then modeled using negative binomial regression models. The PSM results can be summarized as follows:

- Total crashes reduced by 46.2 percent on 4M roadways compared to 4U roadways.
- Lane-departure crashes reduced by 57.1 percent on 4M roadways compared to 4U roadways.
- FI and non-intersection crashes showed a decreasing trend on 4M roadways compared to 4U roadways.

- All the target crashes (total, FI, non-intersection, and OD+ROR) showed a decreasing trend on 4T roadways compared to 4U roadways.
- All the target crashes (total, FI, non-intersection, and OD+ROR) showed an increasing trend on 4T roadways compared to 4M roadways.
- All the target crashes decreased on 2S roadways compared to 4U roadways. The reduction factors were 71.0 percent, 77.6 percent, 69.96 percent, and 69.0 percent for total, FI, non-intersection, and OD+ROR crashes, respectively.

CHAPTER 5: OPERATIONAL DATA EVALUATION

This chapter presents the evaluation results of operational data. The data considered include historical speed data, radar speed data, and driveway video footage collected in the field.

The chapter is divided into four sections. The first section shows the data analysis related to historical speeds and radar speeds collected in the field. The second section shows the analysis of variance (ANOVA) test results that compared speeds on different cross sections. The third section presents the speed data model calibration results. The fourth section describes the change in speeds after the conversion of four-lane undivided highways to other cross sections. The last section provides a description of Vissim simulation results.

5.1. SPEED DATA ANALYSIS

In addition to the speed data collected in the field using radar systems, the research team also acquired historic speed data to obtain more insights into the performance of different cross sections.

5.1.1. Historic Speed Data

Three readily accessible options exist for capturing historic speed information on Texas roadways: the National Performance Management Research Data Set (NPMRDS),[1] the recently released Performance Network[2] from FHWA, and the INRIX XD™ network. The NPMRDS consists of a static GIS file and a database file. The GIS shapefile that contains static roadway information was used to relate the travel time information to each traffic message channel (TMC) segment. The GIS shapefile was used to visualize and geo-reference the NPMRDS data for different maps. The TMC file contains TMC segment-level geometry information with operating speed measures at 5-minute epochs.

Operating Speed Differences among Cross Sections

To acquire the operating speed measures for the selected cross sections, the research team conflated the NPMRDS segments on the linework of the selected roadway networks. Since the NPMRDS contains speed measures for national highway system (NHS) roadways only, a small subset of the segments falls within the range of NHS roadways. The research team used three years (2017–2019) of NPMRDS 5-minute interval operating speed data for the analysis.
For speed data, NPMRDS provides speed data for three vehicle categories: (a) all vehicles, (b) car only, and (c) truck only. The research team used operating speed measures for all vehicles.

Table 40 lists the segment counts on NHS roadways, number of epochs for each segment (for example, 4M had one segment with speed measures, which contains 1×3×365×24×12=315,360 epochs), and the associated average of operating speed measures for these facilities by the PSL of each roadway facility. 2S was not included in this analysis because none of the selected 2S facilities were on the NPMRDS network. The speed measure values indicated that 4M roadways experienced a higher operating speed on 65 mph and 75 mph PSL roadways. For 70-mph PSL

[1] https://npmrds.ritis.org/analytics/help/#npmrds
[2] https://www.fhwa.dot.gov/policyinformation/tables/performancenetwork/

roadways, the average of 85th percentile free-flow operating speed for 4M, 4T, and 4U roadways was within closer range. For 45-mph PSL roadways, 4U showed the higher average operating speed compared to 4T and 4M roadways.

Table 40. Facilities on NHS Roadways with Operating Speed Measures.

Cross Section	45 mph	50 mph	55 mph	60 mph	65 mph	70 mph	75 mph
Count of Segments (NHS Roadways)							
2ST	—	—	—	—	7	—	—
4M	1	—	—	—	2	9	11
4T	13	20	33	19	4	65	37
4U	6	3	7	3	9	15	22
Number of Epochs							
2ST	—	—	—	—	2,207,520	—	—
4M	315,360	—	—	—	630,720	2,838,240	3,468,960
4T	4,099,680	6,307,200	10,406,880	5,991,840	1,261,440	20,498,400	11,668,320
4U	1,892,160	946,080	2,207,520	946,080	2,838,240	4,730,400	6,937,920
Average of 85th Percentile Free-Flow Operating Speed							
2ST	—	—	—	—	75.00	—	—
4M	64.00	—	—	—	81.00	75.50	81.55
4T	69.42	70.54	74.05	72.72	74.33	76.17	76.25
4U	72.38	79.00	72.29	73.33	75.06	76.59	80.32

Note: — means not available.

It is important to know the impact of operating speed measures on traffic crashes. Because these facilities differ in cross-sectional and design properties, an analysis of operating speeds can provide insights into the operational impact. The research team conducted statistical analyses and produced graphical data using RStudio (Version 1.4.1106) with package "ggstatsplot" (Patil and Powell, 2018; R Core Team, 2013). Figure 39 through Figure 42 provide graphical and statistical analysis results using a violin plot, a box plot, or a mix of the two for between-group or between-condition comparisons with results from statistical tests in the subtitle of each of these figures. Figure 39 displays the distribution of operating speed measures for four facility types and shows the number of segments for each facility. The display of four *p*-measures indicates that operating speed varied by facility type. The *p*-measures are shown only when the results were statistically significant. The in-between differences were not significant for all combinations. For example, the difference between the speed distribution on 2ST roadways and 4M or 4U roadways was statistically significant. However, this difference was not statistically significant between 2ST and 4T roadways.

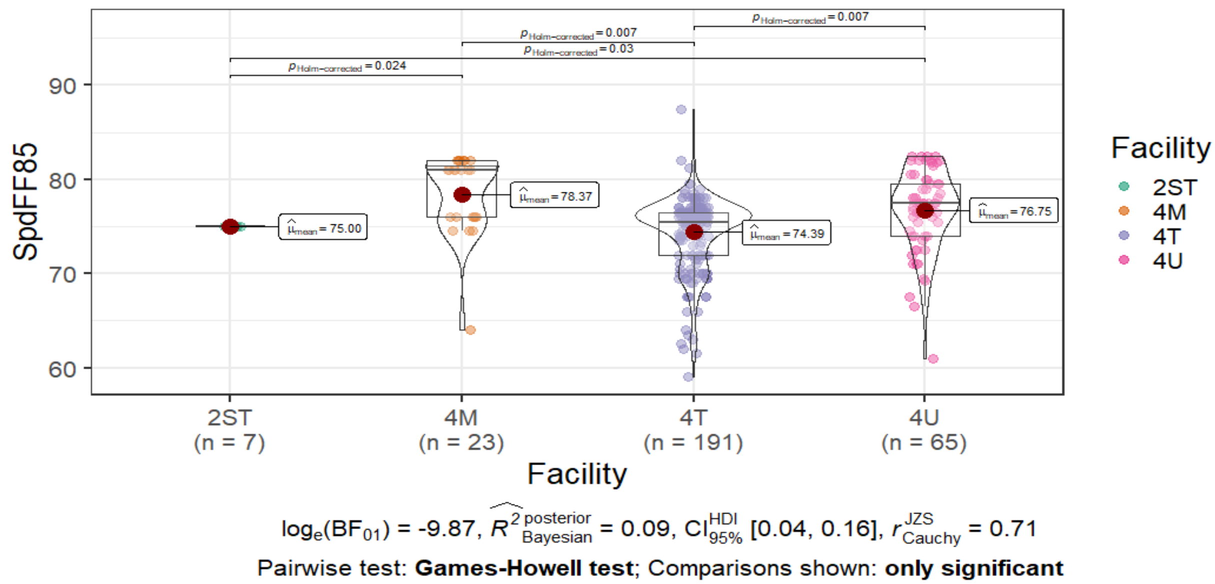


Figure 39**. Distribution of 85th Percentile Free-Flow Operating Speed Measures by Cross Section.**

Since the PSL varies in between each of these cross sections, additional in-between comparisons between different PSL roadways under the same roadway cross section can shed additional insights on the operational perspective of these roadways. Figure 40, Figure 41, and Figure 42 illustrate the results of three cross-section types: 4M, 4T, and 4U, respectively. The results show that operating speed measures varied in between two different PSL roadways under the same roadway cross-section type. The differences were not statistically significant for all combinations.

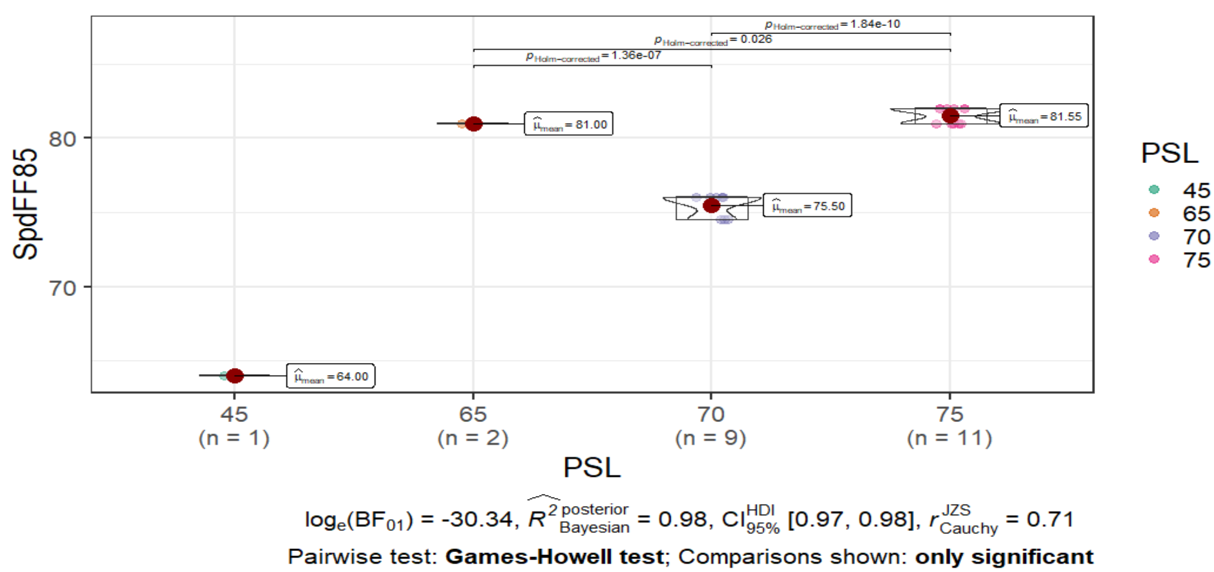


Figure 40**. Distribution of 85th Percentile Free-Flow Operating Speed Measures by Posted Speed Limit (4M Roadways).**

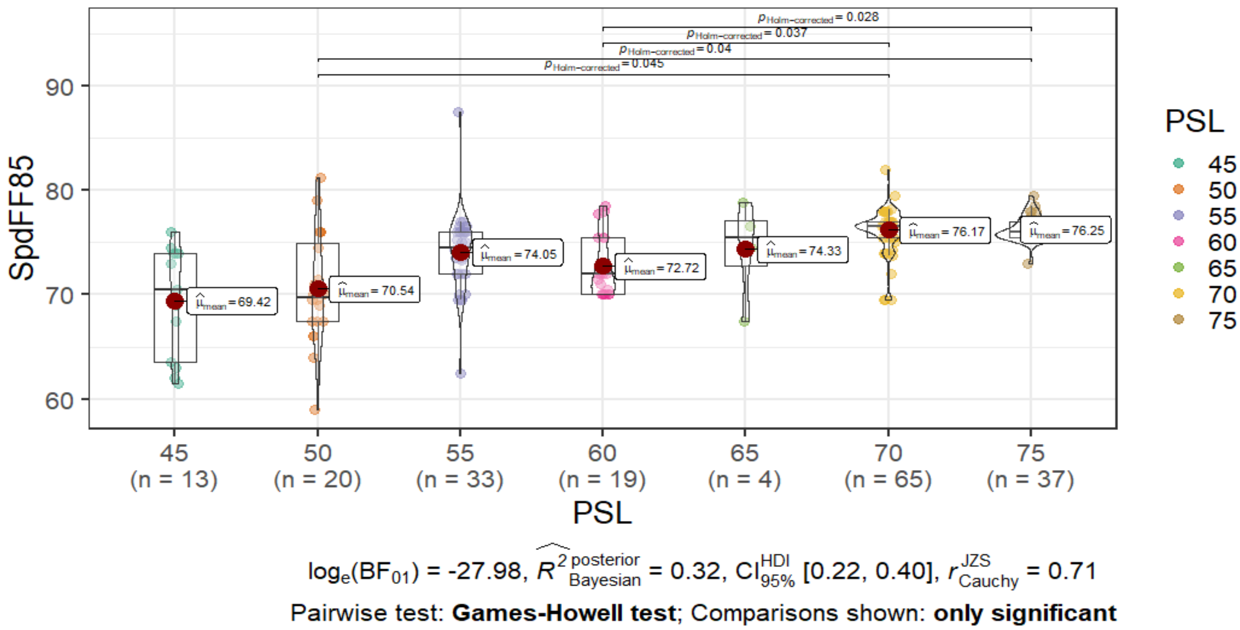


Figure 41**. Distribution of 85th Percentile Free-Flow Operating Speed Measures by Posted Speed Limit (4T Roadways).**

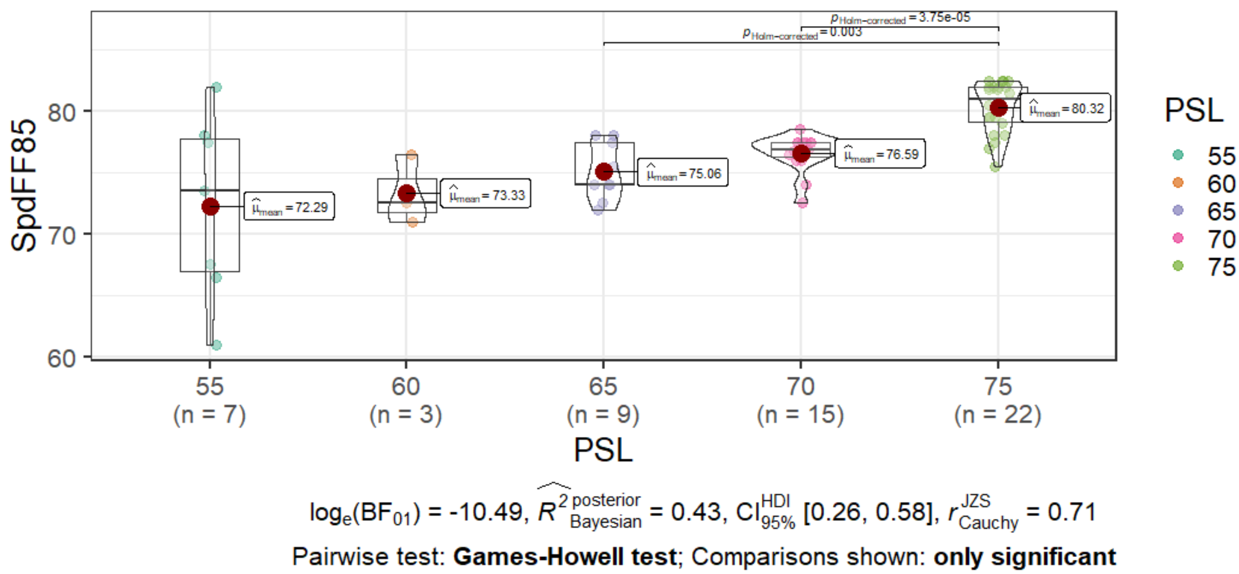


Figure 42**. Distribution of 85th Percentile Free-Flow Operating Speed Measures by Posted Speed Limit (4U Roadways).**

Operating Speed before and after Crash Occurrences

Due to the cross-sectional differences of the roadway facilities considered in this analysis, it was expected that the impact of operating speed would differ by cross-section type. Table 41 and Table 42 list the average operating speed measures before and after crash occurrences at three temporal clusters (2 hours, 4 hours, and 6 hours) by cross-section type and severity type, respectively. Although the PSL varied between these facilities, the general consensus showed that 2ST and 4M roadways experienced higher average operating speed measures in both the before and after periods. It is interesting to see that the operating speed did not reach the level of pre-crash operating speed even after 6 hours after the crash occurrence. The incident clearance time varied by location and facility type. If the clearance is associated with hazardous materials, the clearance time will be greater than normal clearance time. A previous study mentioned that non-hazardous clearance takes between 3 and 5 hours, and the clearance time ranges between 5 and 7 hours for hazardous site clearance (Balke et al., 2014).

Table 41. Average Operating Speeds before and after Crash Occurrences (by the Roadway Facilities).

Cross Section	Average of Operating Speed (mph)					
	6 Hours before Crash	4 Hours before Crash	2 Hours before Crash	2 Hours after Crash	4 Hours after Crash	6 Hours after Crash
2ST	61.17	61.04	60.78	59.12	58.62	58.42
4M	64.24	64.16	63.87	59.72	61.08	61.91
4T	55.12	55.08	54.85	52.84	53.79	54.18
4U	50.62	50.52	50.37	47.9	48.77	49.28

A similar table (see Table 42) shows the operating speed measures at different temporal clusters by different severity types. It shows that operating speed measures were higher on these roadways before the occurrence of fatal or severe injury crashes.

Table 42. Average Operating Speeds before and after Crash Occurrences (by Crash Severity Type).

Severity Type	Average of Operating Speed (mph)					
	6 Hours before Crash	4 Hours before Crash	2 Hours before Crash	2 Hours after Crash	4 Hours after Crash	6 Hours after Crash
K	59.69	59.75	59.44	48.03	49.84	51.72
A	57.88	57.68	57.62	50.19	53	54.32
B	54.06	53.92	53.79	50.6	51.76	52.25
C	47.57	47.44	47.09	45.66	46.47	46.72
O	50.91	50.81	50.69	49.69	50.07	50.29

5.1.2. Radar Speed Data

The research team post-processed and analyzed the radar-measured speed data at the aforementioned sites. Vehicles were included in this analysis only if they were free flow, which was defined as having the leading and trailing headways of at least 7 seconds. The distribution of the raw speed observations at each site and sensor was examined to identify outlier speed values. Speed values were deemed outliers if they were more than three standard deviations away from the mean value. After discarding the outliers, the research team recalculated the averages and standard deviations, as well as 85th percentiles and ranges, at each site and sensor. These statistics are provided in Table 43.

At most sites, the speed statistics were similar at the two sensor locations. The following exceptions are noted:

- At the Kenney and Gonzales sites, notably fewer vehicles were observed at one of the sensor locations than at the other. The research team determined that one of the radar units had a flawed power connector, so the radar unit was intermittently powering off and missing vehicles. Despite this problem, the speed statistics between the two sensor locations at these sites were similar.
- At the Bastrop site, speeds were slightly higher at the southwestern end than at the northeastern end. The difference was 4.1 mph for the average speed and 5.0 mph for the 85th percentile speed. An examination of the speed statistics by lane showed that the speed differences were highest in the outermost two lanes, but the connection to site characteristics is unclear.
- At the Bryan site, speeds were slightly higher at the western end than at the eastern end. The difference was 4.7 mph for the average speed and 4.0 mph for the 85th percentile speed. This site was located at the western edge of the city of Bryan, just outside the city limit. Speeds were likely lower at the eastern end of the site because that end was closer to the city limit where the speed limit decreases.

Table 43. Speed Data Statistics.

Site	Sensor Location	Number of Vehicles	Average Vehicle Length (ft)	Speed Statistic (mph)			
				Average	Standard Deviation	85th Percentile	Range
SH 16 (Jourdanton)	One sensor	2,022	29.8	64.8	6.6	71	37–89
SH 21 (Bastrop)	NE end	3,546	32.2	62.5	6.7	69	42–83
SH 21 (Bastrop)	SW end	3,775	33.5	66.6	6.9	74	44–89
SH 21 (Bryan)	E end	6,326	34.4	58.6	6.5	65	38–79
SH 21 (Bryan)	W end	6,084	32.4	63.3	6.3	69	40–85
SH 36 (Kenney)	N end	2,329	29.1	69.5	6.3	76	47–89
SH 36 (Kenney)	S end	3,818	27.6	70.8	6.2	77	50–91
SH 36 (West Columbia)	N end	4,943	26.9	54.1	6.1	60	35–73
SH 36 (West Columbia)	S end	5,090	28.1	53.5	6.2	60	31–75
SH 71 (Marble Falls)	E end	5,416	29.7	68.1	6.6	75	48–88
SH 71 (Marble Falls)	W end	5,315	28.9	69.8	6.3	76	49–90
US 183 (Gonzales)	NW end	1,043	34	70	5.9	76	52–89
US 183 (Gonzales)	SE end	2,692	32.4	70.8	5.4	76	52–88
US 259 (Nacogdoches)	One sensor	5,792	27.2	56.8	5.4	62	39–74
US 79 (De Berry)	One sensor	3,862	45.5	66.9	5.4	73	49–85
US 80 (Edgewood)	One sensor	4,107	28.1	62.2	6.6	69	39–83
US 80 (Wills Point)	E end	4,617	28.6	69.4	6.6	76	49–90
US 80 (Wills Point)	W end	4,948	30.2	70.3	6.9	77	48–92
	Overall:	75,725	30.8	64.0	8.6	73	31–92

Table 44 provides a count of vehicles by site, categorized as passenger car or truck, aggregated across both sensors at each site. The overall dataset consists of about 15 percent trucks. For this tabulation, a truck was defined as any vehicle with a length greater than 36 ft. (The side-fire radar units report length but not axle count or FHWA vehicle classification number.)
The threshold value of 36 ft was based on the distribution of vehicle lengths across the sites, which is shown in Figure 43. The radar sensors tended to overestimate vehicle speeds consistently but gave the expected distribution of lengths. The distribution in Figure 43 shows a large peak consisting of shorter vehicles, centered at about 24 ft (passenger cars), and a long tail of longer vehicles, with a smaller peak centered at about 70 ft (trucks). A similar distribution was observed in the speed dataset from Research Project 0-6960 (Pratt et al., 2019), which consisted of more than 300,000 vehicles, and ground-truth observations using video footage in that project that verified the vehicle classifications based on the radar-measured vehicle lengths.

Table 44. Vehicle Counts by Site and Type.

Site	Vehicle Count by Type		
	Passenger Car	**Truck**	**Total**
SH 16 (Jourdanton)	1,730	292	2,022
SH 21 (Bastrop)	5,916	1,405	7,321
SH 21 (Bryan)	9,794	2,616	12,410
SH 36 (Kenney)	5,530	617	6,147
SH 36 (West Columbia)	9,242	791	10,033
SH 71 (Marble Falls)	9,317	1,414	10,731
US 183 (Gonzales)	3,064	671	3,735
US 259 (Nacogdoches)	5,512	280	5,792
US 79 (De Berry)	2,264	1,598	3,862
US 80 (Edgewood)	3,695	412	4,107
US 80 (Wills Point)	8,498	1,067	9,565
Total	64,562	11,163	75,725

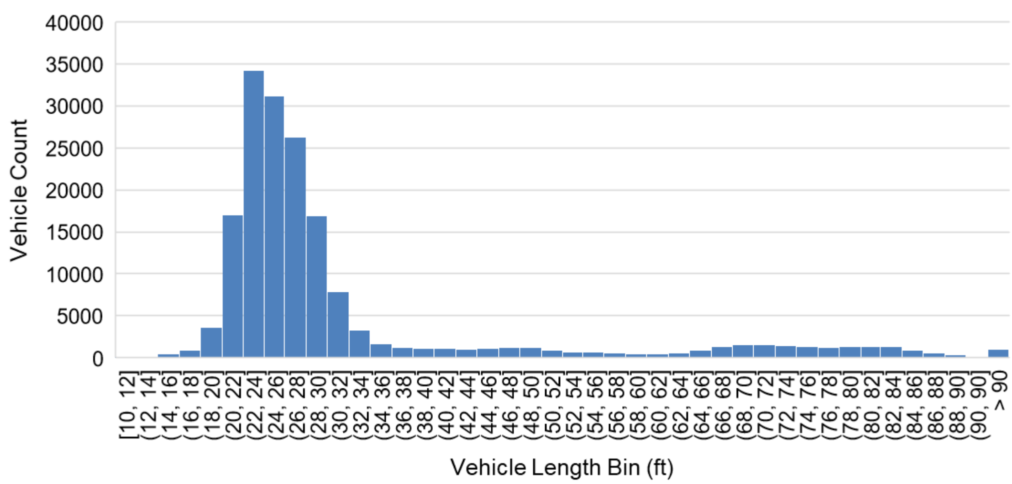


Figure 43. Vehicle Length Distribution.

5.2. ANOVA TEST

ANOVA is used to compare means of two or more independent groups. ANOVA provides a statistical test of whether or not the means of multiple groups are all equal. When there are only two means to compare, the *t*-test and the ANOVA are equivalent. The ANOVA test is based on the assumption that the observations within each group are independent of each other. The other assumption is that the data are approximately normally distributed and should not contain any outliers. Additionally, it relies on the homogeneity of variance, which means that the variance among the groups is approximately equal. ANOVA is dependent on two variances—variation within group observations and variation among groups.

In an ANOVA test, the total variation in the data is separated into a portion due to random error (quantified by sum of squares for error [SSE]) and portions due to the treatment (quantified by sum of squares total [SST]). To conduct the significance test, the F value is calculated as a ratio of mean square treatment (MST) and mean square error (MSE). If the calculated F value is significantly larger than the critical value in the F distribution table, which is based on the chosen significance level and the degrees of freedom for treatment and error, the null hypothesis of equal means is rejected. The corresponding p-value can also be used for a significance test. If the p-value is greater than the chosen significance level, then the null hypothesis is rejected. If the ANOVA reveals that there is a significant difference between sample means, then the Newman–Keuls or Student–Newman–Keuls (SNK) method, which is a stepwise multiple comparisons procedure, can be used to compare differences between the group with the largest mean and the group with the smallest mean.

In this study, the ANOVA test was used to analyze the differences in operating speeds among different cross sections within the same pavement width categories. The following descriptions provide more detail for each of the comparisons.

5.2.1. 4U versus 4M

Table 45 shows the ANOVA results for the difference in mean speeds between the 4U and 4M cross sections within the narrow pavement width category and a PSL of 55 mph. The p-value in the ANOVA table shows there was a significant difference between the mean speeds. The SNK grouping test results presented in Table 46 show that the speeds on 4M were higher than on 4U, and the result was statistically significant.

Table 45. ANOVA Test—4U versus 4M.

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	1	32,995.8	32,995.8	954.23	<.0001
Error	15823	547,138.3	34.6		

Table 46. SNK Test—4U versus 4M.

SNK Grouping*	Mean	N	Cross Section
A	56.8	5,792	4M
B	53.8	10,033	4U

* Means with the same letter were not significantly different.

5.2.2. 4M versus 4T

Table 47 shows the ANOVA results for the difference in mean speeds between the 4M and 4T cross sections within the wide pavement width category and a PSL of 65 mph. The *p*-value in the ANOVA table shows there was a significant difference between the mean speeds. The SNK grouping test results presented in

Table 48 show that the speeds on 4M were higher than on 4T, and the result was statistically significant.

Table 47. ANOVA Test—4M versus 4T.

Source	DF	Sum of Squares	Mean Square	*F* Value	Pr > *F*
Model	1	5,027.0	5,027.0	110.61	<.0001
Error	16515	750,577.3	45.4		

Table 48. SNK Test—4M versus 4T.

SNK Grouping*	Mean	N	Cross Section
A	62.2	4,107	4M
B	60.9	12,410	4T

* Means with the same letter were not significantly different.

5.2.3. 4U versus 2S

Table 49 shows the ANOVA results for the difference in mean speeds between the 4U and 2S cross sections within the intermediate pavement width category and a PSL of 70 mph. The *p*-value in the ANOVA table shows there was a significant difference between the mean speeds. The SNK grouping test results presented in Table 50 show that the speeds on 2S were higher than on 4U, and the result was statistically significant.

Table 49. ANOVA Test—4U versus 2S.

Source	DF	Sum of Squares	Mean Square	*F* Value	Pr > *F*
Model	1	6,986.2	6,986.2	177.41	<.0001
Error	14464	569,565.0	39.4		

Table 50. SNK Test—4U versus 2S.

SNK Grouping*	Mean	N	Cross Section
A	70.6	3,735	2S
B	68.9	10,731	4U

* Means with the same letter were not significantly different.

5.2.4. 4U versus 2ST

Table 51 shows the ANOVA results for the difference in mean speeds between the 4U and 2ST cross sections within the wide pavement width category and a PSL of 70 mph. The *p*-value in the ANOVA table shows there was a significant difference between the mean speeds. The SNK grouping test results presented in Table 52 show that the speeds on 2ST were higher than on 4U, and the result was statistically significant.

Table 51. ANOVA Test—4U versus 2ST.

Source	DF	Sum of Squares	Mean Square	*F* Value	Pr > *F*
Model	1	694.4	694.4	16.11	<.0001
Error	15710	676,991.7	43.0930		

Table 52. SNK Test—4U versus 2ST.

SNK Grouping*	Mean	N	Cross Section
A	70.3	6,147	2ST
B	69.8	9,565	4U

* Means with the same letter were not significantly different.

5.2.5. Narrow Pavement Width

Table 53 shows the ANOVA results for the difference in mean speeds between the 4U, 4M, and 2S cross sections within the narrow pavement width category. Since the PSLs were different, speed differential (operating speed minus PSL) was considered. The *p*-value in the ANOVA table shows there was a significant difference between the mean speed differentials. The SNK grouping test results presented in Table 54 show that 2S had the highest speeds, followed by 4M and then 4U, and the result was statistically significant.

Table 53. ANOVA Test—Narrow Pavement Width.

Source	DF	Sum of Squares	Mean Square	*F* Value	Pr > *F*
Model	2	75,201.6	37,600.8	1,054.84	<.0001
Error	17844	636,069.1	35.6		

Table 54. SNK Test—Narrow Pavement Width.

SNK Grouping*	Mean	N	Cross Section
A	4.8	2,022	2S
B	1.8	5,792	4M
C	-1.2	10,033	4U

* Means with the same letter were not significantly different.

5.2.6. Intermediate Pavement Width

Table 55 shows the ANOVA results for the difference in mean speeds between the 4U, 2S, and 2ST cross sections within the intermediate pavement width category. Since the PSLs were different, speed differential (operating speed minus PSL) was considered. The *p*-value in the ANOVA table shows there was a significant difference between the mean speed differentials. However, the SNK grouping test results presented in Table 56 show that there was no significant difference between speeds on 2S and 2ST, but those two highways had higher speeds than the 4U.

Table 55. ANOVA Test—Intermediate Pavement Width.

Source	DF	Sum of Squares	Mean Square	*F* Value	Pr > *F*
Model	2	13,480.1	6,740.0	162.07	<.0001
Error	25,646	1,066,551.0	41.6		

Table 56. SNK Test—Intermediate Pavement Width.

SNK Grouping*	Mean	N	Cross Section
A	0.6	3,735	2S
A	0.4	11,183	2ST
B	-1.0	10,731	4U

* Means with the same letter were not significantly different.

5.2.7. Wide Pavement Width

Table 57 shows the ANOVA results for the difference in mean speeds between the 4U, 4M, 4T, and 2ST cross sections within the wide pavement width category. Since the PSLs were different, speed differential (operating speed minus PSL) was considered. The *p*-value in the ANOVA table shows there was a significant difference between the mean speed differentials. The SNK grouping test results presented in Table 58 show that 2ST highways had the highest speeds, followed by 4U and 4M, and then 4T.

Table 57. ANOVA Test—Wide Pavement Width.

Source	DF	Sum of Squares	Mean Square	*F* Value	Pr > *F*
Model	3	123,322.396	41,107.465	927.93	<.0001
Error	32,225	1,427,568.984	44.300		

Table 58. SNK Test—Wide Pavement Width.

SNK Grouping*	Mean	N	Cross Section
A	0.2780	6,147	2ST
B	-0.1527	9,565	4U
C	-2.8468	4,107	4M
D	-4.1232	12,410	4T

* Means with the same letter were not significantly different.

5.3. MODEL CALIBRATION

The research team developed models to predict 85th percentile speeds at the study sites. The authors used the NLIN procedure in the SAS program. Equation 19 shows the functional form for the speed model. Table 59 displays the results of the calibration for the 85th percentile free-flow speed.

$$v_{85} = b_0\sqrt{v_{sl}} \times e^{b_{lw}(LW)+b_{sw}(SW)+b_{tk}(I_{tk})} \tag{19}$$

where:

V_{85} = 85th percentile vehicle speed, mph.
V_{sl} = regulatory speed limit, mph.
LW = lane width, ft.
SW = shoulder width, ft.
I_{tk} = indicator variable for trucks (=1 if truck speed; 0 if car speed).
b_n = calibration coefficients.

Table 59. Speed Model Calibration Results.

Model Statistics				
	R^2	0.81		
	Observations	11 sites (64,562 passenger cars and 11,163 trucks)		
Range of Model Variables				
Variable	**Variable Name**	**Units**	**Minimum**	**Maximum**
v_{85}	85th Percentile Vehicle Speed	mph	59.0	77.0
v_{sl}	Regulatory Speed Limit	mph	55	70
LW	Lane Width	ft	11	12
SW	Shoulder Width	ft	1	12
Calibrated Coefficient Values				
Coefficient	**Coefficient Definition**	**Value**	**Std. dev**	**t-statistic**
b_0	Intercept	3.114	0.777	4.0
b_{lw}	Effect of Lane Width	0.079	0.022	3.5
b_{sw}	Effect of Shoulder Width	0.005	0.003	1.8
b_{tk}	Effect of Trucks	-0.033	0.019	-1.8

The calibrated model is provided in Equation 20. A comparison of predicted and observed vehicle speed values for the proposed model is provided graphically in Figure 44.

$$v_{85} = 3.114\sqrt{v_{sl}} \times e^{0.079(LW)+0.005(SW)-0.033(I_{tk})} \tag{20}$$

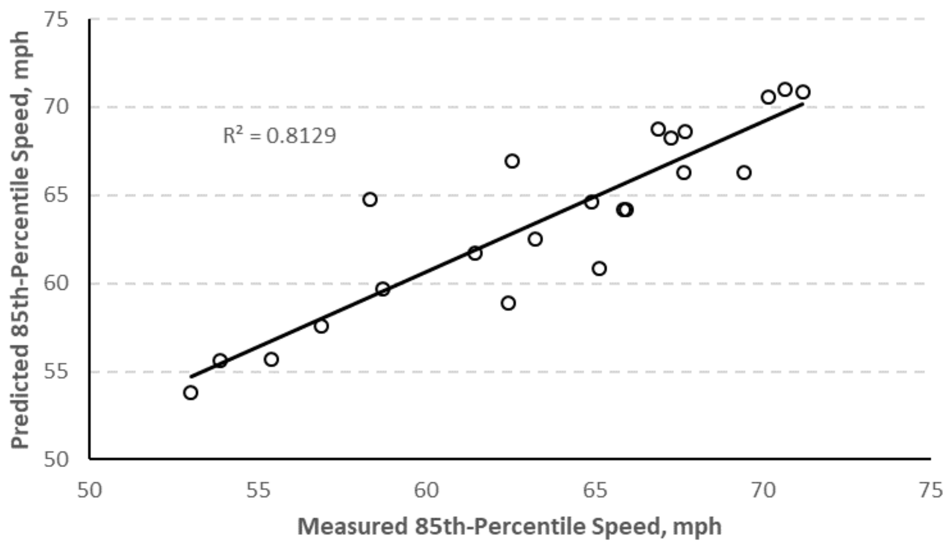


Figure 44. Comparison of Measured and Predicted 85th Percentile Free-Flow Speeds.

The values for coefficients b_{lw} and b_{sw} show that drivers chose higher speeds on highways with wider lanes and shoulders. The speed on a 12-ft lane highway was about 8 percent higher than on a highway with 11-ft lanes. Similarly, for every 2-ft increase in shoulders, there was an approximate 1 percent increase in speeds. Figure 45 shows the free-flow speeds for different lane and shoulder widths on a highway with a PSL of 70 mph. The values for coefficient b_{tk} show that truck drivers generally chose lower speeds than passenger car drivers.

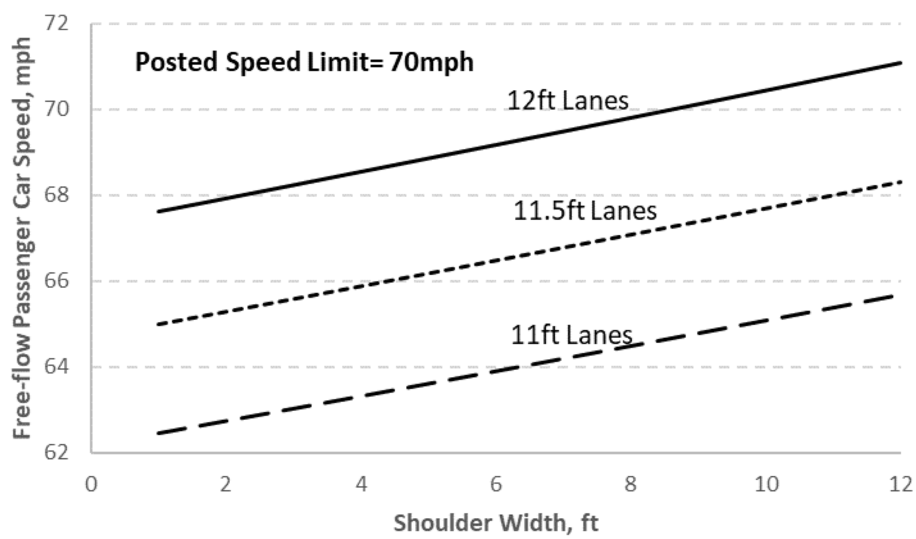


Figure 45**. 85th Percentile Speeds for Different Lane and Shoulder Widths.**

5.4. BEFORE–AFTER ANALYSIS

Based on the survey responses in Chapter 2, it was determined that four highway corridors were converted from 4U to other cross sections. These conversions with the highway names, limits, construction dates, and before–after periods considered in this study, are shown in Table 60. For some highways, the after period was too short to obtain any crash data.

Table 60. Conversions from 4U to Other Cross Sections for Operational Evaluation.

Highway	Limits	Construction Period	Before Start	Before End	After Start	After End
SH 158 (4U to 4M)	4 mi SE of I-20 to Glasscock county line	Sep 2017 to May 2018	09/01/2014	08/31/2017	06/01/2018	08/31/2020
SH 349 (4U to 4M)	Midland county line to Dawson county line	Sep 2019 to May 2020	09/01/2016	08/31/2019	06/01/2020	10/31/2020
US 79 (4U to 2ST)	LA state line to FM 31	08/20/2019 to 06/29/2020	08/20/2016	08/19/2019	06/30/2020	10/31/2020
SH 21 (4U to 2ST)	Quarter Horse Loop to S Old Potato Road	08/23/2016 to 01/17/2017	08/23/2013	08/22/2016	01/18/2017	01/17/2020

The research team evaluated the operational (i.e., speed) changes after the conversion. To assign speed measures, the following steps were taken:

1. Conflate the NPMRDS roadway network and the RHINO network.
2. Collect 5-minute interval raw speed data from NPMRDS.
3. Calculate the speed measure for each roadway segment.

The conflation was conducted by integrating the segment shapefile and NPMRDS 2018 network shapefile. Table B1 in Appendix B shows the roadway segments and corresponding traffic management center numbers.

For each highway, raw speed data (5-minute intervals) were collected for three time periods—(a) before construction, (b) during construction, and (c) after construction—based on the dates presented in Table 60. For SH 349 and US 79, the after periods considered were until October 31, 2020, since that was the last date that the speed measures were available.

The speed measure variables listed in Table B2 were calculated for all traffic management center numbers. After calculating the speed measure data, the research team aggregated these data at the highway level.

Table 61 shows the change in free-flow vehicle average, all-vehicle average, and standard deviation in speeds after conversion from 4U. The analysis shows that there was no significant change in the speeds after conversion.

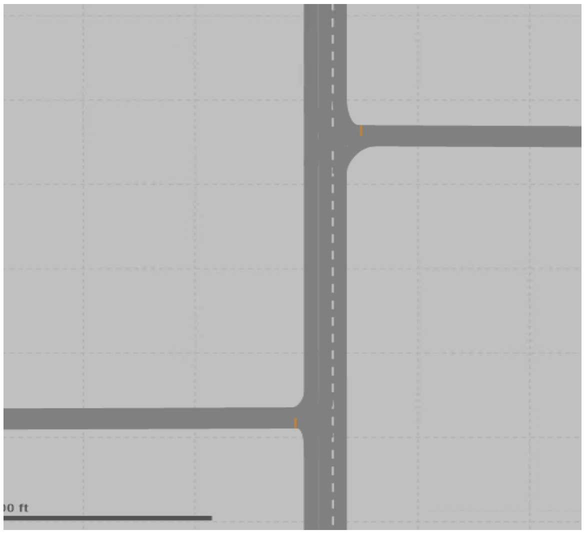
Table 61. Change in Speed Measures after Conversion.

Highway	Free-Flow Vehicle Average Speeds (mph)		All-Vehicle Average Speed (mph)		Standard Deviation in Speeds (mph)	
	Before	**After**	**Before**	**After**	**Before**	**After**
SH 158	78.53	78.89	67.27	66.74	6.12	6.77
SH 349	79.10	79.30	66.30	68.53	8.47	9.57
US 79	69.64	69.44	60.83	61.22	4.92	4.63
SH 21	72.84	72.69	62.14	61.71	6.34	5.87

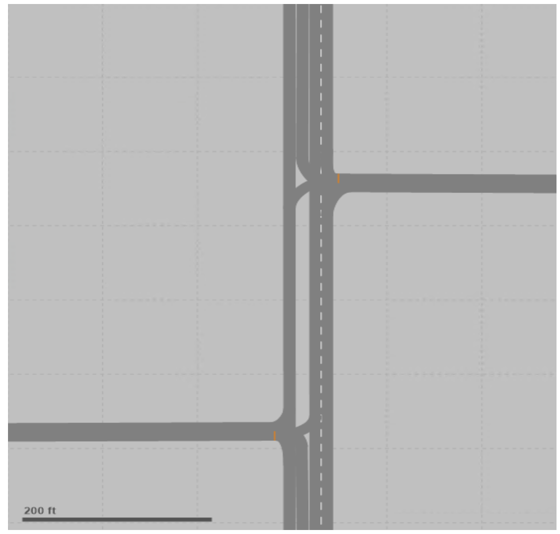
5.5. SIMULATION

The research team developed a VISSIM model to evaluate the operational impacts of the different cross sections considered in the project. This geometry consists of an approximately 21-mile model of a hypothetical facility designed to represent a generic Texas rural highway.
The 21-mile length allows for three sets of 3-mile passing lanes in each direction, consistent with analyses conducted in other TxDOT projects on 2S design projects, and a 1-mile section with a single lane in each direction on both ends of the facility to allow the randomly generated vehicles to form platoons before loading into the facility.

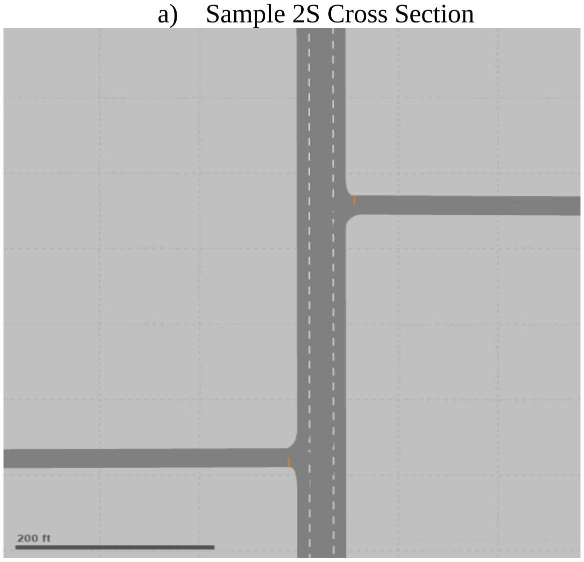
The geometry differences between the cross sections were coded into modifications for the VISSIM model. The cross sections considered for this model coincided with the cross sections considered for field data collection. Figure 46 shows each cross section with a screenshot of a segment of the facility from two of the closely spaced intersections.



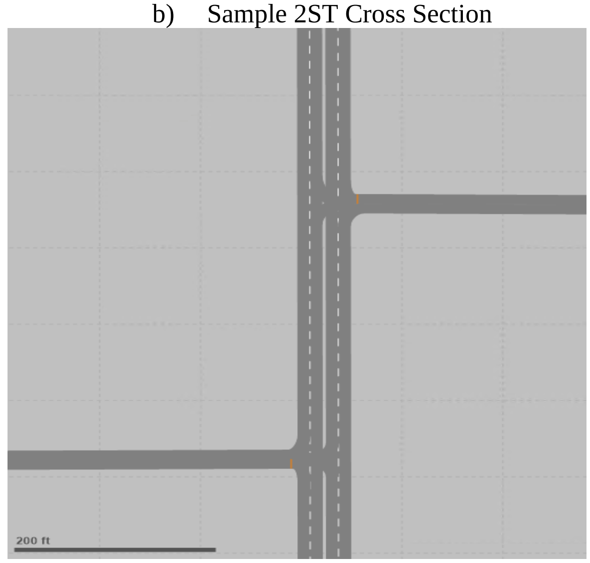
a) Sample 2S Cross Section



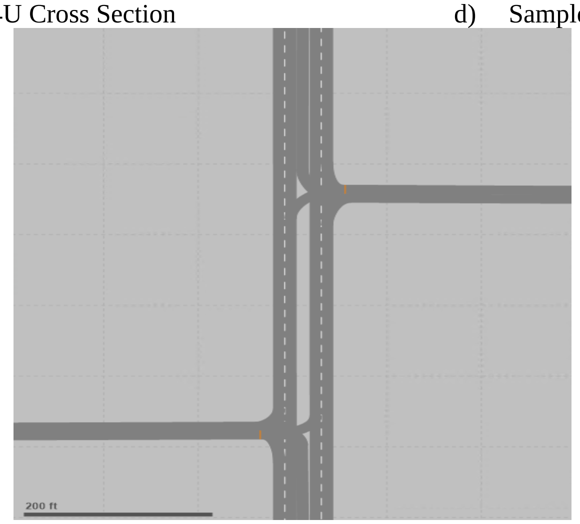
b) Sample 2ST Cross Section



c) Sample 4U Cross Section



d) Sample 4M Cross Section



e) Sample 4T Cross Section

Figure 46. Different Cross Sections Coded in VISSIM.

Notice that the TWLTL cross sections have left-turn bays. This was done because the simulation cannot model a TWLTL where vehicles can enter the left-turn lane at any location. However, on a facility of this type, a TWLTL can be simulated with a left-turn lane long enough to represent the entire length of the TWLTL a vehicle would use to turn into a driveway. This type of simulation was representative of the behavior observed on the facility since few of the intersections overlapped such that there would have been a head-on conflict for two vehicles trying to execute left-turning maneuvers. The inability to model this type of conflict can be considered a limitation of the simulation method.

The following sections describe the generation of the simulation experiment to determine the operational differences between the various cross sections considered in this project and the results.

5.5.1. Simulation Model Inputs

Driveway data collection in the previous tasks of this project indicate that rural facility driveway density can range from 5 to 23 driveways per mile on average. Given that coding driveways into the simulation model is labor intensive and introduces the potential for random misbehaviors to skew the simulation results because of unrealistic yielding behavior, the research team also compared the volumes for different land uses corresponding to the driveways, shown in Table 15.

Given that the residential driveways have very low daily volumes, the research team decided to model industrial and commercial driveways only. Of course, these can be converted into equivalent residential driveways based on the proportion in Table 15. The research team took the driveway density data and the volumes and converted the driveways in the simulation model to represent either three industrial driveways per mile, three commercial driveways per mile, or six commercial driveways per mile to represent the various rural highways based on these data. Each driveway modeled in the corridor was a stop-controlled intersection with appropriate yield behavior and routing for analysis within this research effort. Two of the intersections in the model represented intersections with other rural highways and would have greater volumes than the other intersections in the model. Figure 47 shows the entire VISSIM model with a three-driveway-per-mile configuration, where the black boxes represent the locations of the two highway intersections. The northern highway intersection has four legs, and the southern highway intersection has three legs.

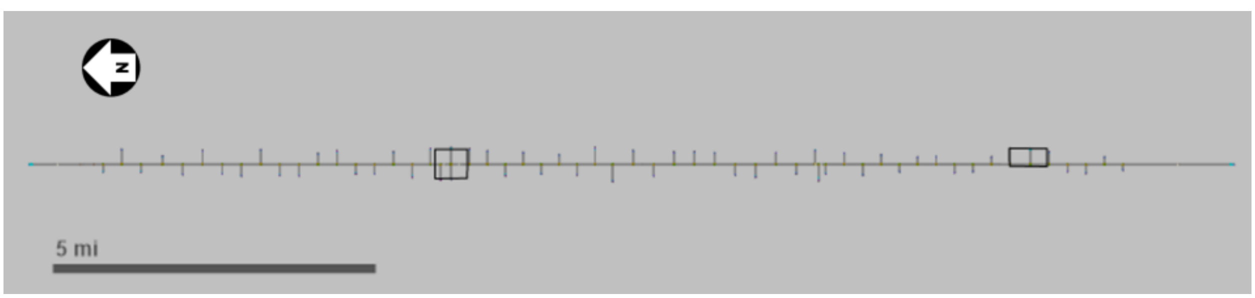


Figure 47**. High-Level View of Entire Rural Cross-Section VISSIM Model.**

Each driveway has a 50/50 directional split for left- and right-turning vehicles. Vehicles generated at the driveways traveled the remaining length of the main facility. The same number of vehicles turned into each driveway from the main facility as turned out of the driveway during the simulation to balance the volumes in the model.

Another key piece of information for the model coming from the field data was the speed distribution for the different cross sections. The team utilized radar speed data collected from the field to represent the known differences in operating speeds based on the roadway striping. The 2ST cross-section data on SH 36 near Kenney, Texas, and the 4U cross-section data from US 80 near Wills Point, Texas, represented the respective wide configurations for their respective cross sections directly. Other configurations utilized the same desired speed curves as the 2ST or 4U wide configuration, but their speeds were adjusted according to a representative cross section assumed for simulation. The adjustments to the speed profiles used the regression model presented in Table 59 that was developed to adjust the desired speed profile based on the representative lane widths and shoulder widths. The desired speed profiles as a starting point for the 2S and four-lane facilities are in Figure 48 and Figure 49, respectively.

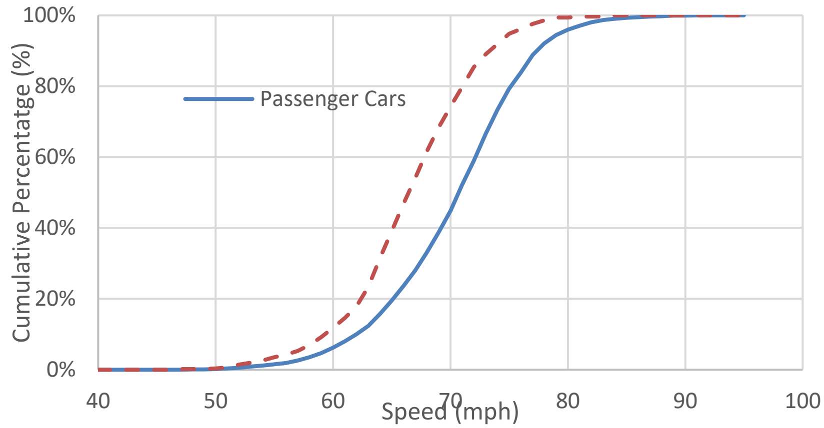


Figure 48. 2S Cross Section Desired Speed Curve Shape (from 2ST Wide at SH 36 near Kenney, Texas).

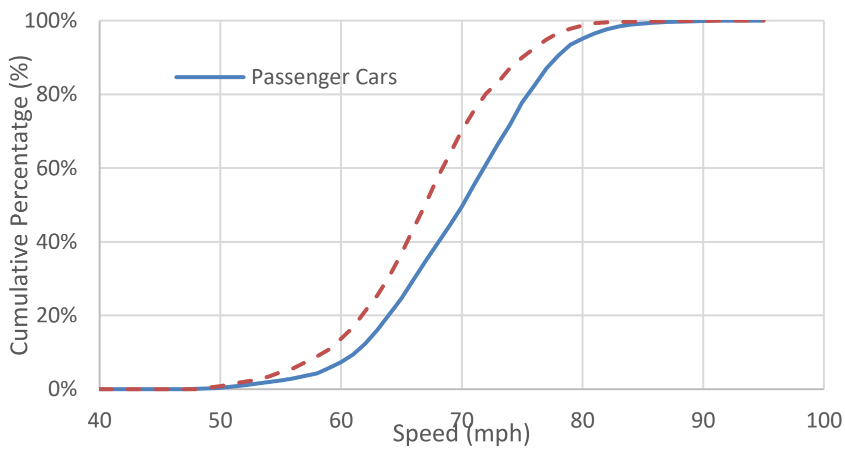


Figure 49. Four-Lane Cross Section Desired Speed Curve Shape (from 4U Wide at US 80 near Wills Point, Texas).

The research team determined a representative cross section for each facility for the intermediate and wide width facilities modeled to apply the speed adjustment regression model.

The normalized intermediate cross-sectional pavement width was 58 ft, and the normalized wide width was 70 ft. Table 62 shows the design of each cross section including the corresponding adjustment factors for the desired speed distribution and the corresponding average desired speed for the resulting cross section.

Table 62. Representative Cross-Section Widths and Corresponding Speed Adjustments to Desired Speed Profiles.

Configuration	Lane Width	Shoulder Width	Median Width	Total Width	Mean Desired Speed Adjustment from Field Data	Passenger Car Average Speed (mph)
2S-I	12	11	0	36 + 22 = 58	-0.32	70.3
2ST-I	12	5	12	36 + 10 + 12 = 58	-2.25	68.4
4U-I	11.5	6	0	46 + 12 + 0 = 58	-4.20	66.0
4M-I	11.5	4	4	46 + 8 + 4 = 58	-4.85	65.3
2ST-W[a]	12	11	12	36 + 22 + 12 = 70	-0.32	70.3
4U-W[b]	12	11	0	48 + 22 + 0 = 70	0.16	70.3
4M-W	12	9	4	48 + 18 + 4 = 70	-0.48	69.7
4T-W	11	6	14	44 + 12 + 14 = 70	-6.85	63.3

[a] The 2ST configuration at SH 36 near Kenney, Texas, represents the shape of any 2S configuration.
[b] The 4U configuration at US 80 near Wills Point, Texas, represents the shape of any four-lane configuration.

The research team used the calibration from TxDOT Project 0-6997 for this research effort. The 0-6997 project explored the capacity of 2S corridors and calibrated for passing lane usage. The driver model utilized for this simulation was the Wiedemann 99 driving behavior model due to overall high speeds along the corridor. Vissim defaults to the Wiedemann 74 model, a model designed by the simulation creators, which is intended to model vehicles traveling at low speeds. The Wiedemann 99 model was developed later to provide a more accurate representation of vehicles traveling at high speeds. The research team edited the lane-change parameters within the model from their default values as part of the calibration process so that vehicles would have lane usage in passing lanes more consistent with observations in the field. Table 63 shows the lane-change parameters used for the simulation corridor.

Table 63. Lane-Change Parameters Used in Simulation Corridor.

Lane-Change Parameter	Default Value	Calibrated Value
Maximum deceleration of lane-changing vehicle (ft/s^2)	-13.12	-13.6
Maximum deceleration of trailing vehicle (ft/s^2)	-9.84	-9.84
Accepted deceleration of lane-changing vehicle (ft/s^2)	-3.28	-3.28
Accepted deceleration of trailing vehicle (ft/s^2)	-1.64	-1.64
Safety distance reduction factor	0.6	0.8
To slower lane if collision time is above (sec)	11	30
Maximum deceleration for cooperative braking (ft/s^2)	-9.84	-13
Cooperative lane change	No	Yes
Cooperative lane change—maximum speed difference (mph)	6.7	10
Cooperative lane change—maximum collision time (sec)	10	10

5.5.2. Simulation Matrix and Measures of Performance

The rural cross-section simulation model helps answer the question of which cross sections are acceptable or ideal given generic geometries and traffic. The analysis team generated modifications to parameters in the simulation to represent different operational conditions for analysis so the team could evaluate cross sections based on operations. This section presents the parameters varied between simulation scenarios and lists the performance metrics the team used to analyze the various scenarios.

The parameters varied in this simulation study were meant to represent different alternatives for operational conditions for a given cross-section width. The pavement width categories were narrow, intermediate, and wide, which represented cross sections of less than or equal to 55 ft, between 56 and 65 ft, and greater than or equal to 66 ft, respectively. The speed data collected in the field indicated that narrow and intermediate widths did not cause statistically different impacts in speed. Therefore, the simulations did not include any narrow width representations since the team expected the narrow facility results to be consistent with the intermediate width results. The simulation considered changes in cross section, volume conditions, and vehicle compositions while keeping all other parameters constant. The simulation parameters varied in the simulation experiment were as follows:

- Cross section—Eight values.
  - 2S—Intermediate.
  - 2ST—Intermediate.
  - 4U—Intermediate.
  - 4M—Intermediate.
  - 2ST—Wide.
  - 4U—Wide.
  - 4M—Wide.
  - 4T—Wide.
- Driveway density—Three values.
  - Three industrial driveways per mile.
  - Three commercial driveways per mile.
  - Six commercial driveways per mile.
- Major road ADT—Five values (Note: 10,000 and 15,000 vpd were not applicable in the six commercial driveways per mile scenario).
  - 10,000 vpd.
  - 15,000 vpd.
  - 17,500 vpd.
  - 20,000 vpd.
  - 25,000 vpd.
- Percentage of trucks in the traffic volume—Three values.
  - 10 percent.
  - 20 percent.
  - 40 percent.

The total number of simulation scenarios resulting from this simulation matrix was 312. This was calculated by multiplying the eight cross sections by three driveway density levels and five traffic volumes and three truck percentages and then subtracting 48 scenarios to account for volume scenarios (i.e., 10,000 and 15,000 vpd) that were impossible in the scenario for six commercial driveways per mile. Each simulation scenario was simulated for the peak four hours of the day based on an average percent of ADT per hour distribution from the TxDOT Super 2 Capacity Project (0-6997).

Each of these simulation scenarios collected a set of performance metrics. These performance metrics enabled the team to assess the operational performance of each alternative cross section. The research team focused the study on the network-wide performance results, which summarized the performance of all vehicles, regardless of origin or destination.

5.5.3. Simulation Results

This section summarizes the results of the 312 unique scenarios analyzed in this simulation experiment. The research team focused on the average minutes of delay and the average speeds of each vehicle in the network across the four-hour simulation period.

Average Minutes of Delay

Table 64 to Table 67 show the average minutes of delay for each vehicle in each intermediate width cross section. Following are some key findings from the tables:

- For all cross sections:
    - From an average delay standpoint, each cross section experienced less than 2 minutes of total delay per vehicle in the 10,000 and 15,000 vpd scenarios.
    - In the scenarios with three industrial or three commercial driveways per mile, the 10 percent truck scenarios experienced less than 3 minutes of delay per vehicle for less than 25,000 vpd.
    - At an ADT of 25,000 vpd, all intermediate width facilities experienced large amounts of delay.
- The 2S cross section experienced large amounts of delay with six commercial driveways in all scenarios analyzed.
- For the 2ST cross section:
    - There was less than 5 minutes of delay for six commercial driveways at 17,500 vpd and at 20,000 vpd, both with 10 percent trucks, and at 17,500 vpd with 20 percent trucks.
    - Three and six commercial driveways/mile scenarios experienced 5 or more minutes of delay with 20 percent truck scenarios at 20,000 vpd.
    - At 40 percent trucks, the 2ST cross section experienced large amounts of delay at 20,000 vpd and greater.
- For four-lane cross sections:
    - Both 4U and 4M cross sections experienced similar performance with some signs of instability at 20 and 40 percent trucks and 17,500 and 20,000 vpd.
    - The four-lane cross sections experienced less delay on average than the 2ST, but the difference between the four-lane and 2ST facilities was not large in any condition with ADTs of 17,500 vpd or less.

Table 68 to Table 71 show the average delay-per-vehicle results for the wide pavement facilities. Key findings are as follows:

- Similar to the intermediate width facilities, all the cross sections experienced less than 2 minutes of delay per vehicle on average in the 10,000 and 15,000 vpd scenarios for each truck percentage and driveway configuration analyzed.
- At 10 and 20 percent trucks, each wide cross section had less than 5 minutes of average delay per vehicle at 17,500 vpd, and many cross sections had less than 1 minute.
- For 10 percent trucks and 20,000 vpd, each cross section had low average delays per vehicle, keeping in mind that the 5.1 minutes of delay per vehicle on average for the 4M scenario was likely an artificially high number caused by randomness in the model.
- The total delays began to increase for 20 percent truck scenarios with 20,000 vpd for 2ST, 4M, and 4U scenarios, keeping in mind that the 4M delays were assumed to be artificially low. The research team assumed that the 4M delays for the three driveways/mile scenarios were low because the 4U cross section showed delays around 5 minutes per vehicle, and the research team assumed that the two cross sections would have similar results.
- At 40 percent trucks, the total delay performance became unstable and started to increase at the 17,500 vpd scenarios for the 2ST, 4U, and 4M cross sections.
- The 4T cross section was the only cross section to have average delays less than 10 minutes per vehicle in all 25,000 vpd scenarios, and those values were less than 2 minutes.

Table 64. Average Minutes of Delay per Vehicle for Intermediate 2S Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	0.6	1.2	1.4	1.9	32.7	0.7	1.3	1.7	3.6	70.8	0.8	1.7	5.5	56.3	112.0
3 Commercial	0.8	1.1	1.4	2.3	39.8	0.9	1.3	1.6	5.8	79.9	1.1	1.7	8.9	69.2	126.3
6 Commercial	—	—	7.2	26.0	80.9	—	—	21.2	43.2	113.0	—	—	64.1	95.0	129.7

Note: — means not available.

Table 65. Average Minutes of Delay per Vehicle for Intermediate 2ST Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	0.6	1.2	1.3	1.7	12.0	0.6	1.2	1.4	3.7	28.4	0.7	1.5	4.3	21.0	46.6
3 Commercial	0.8	1.0	1.2	1.7	14.0	0.8	1.1	1.3	5.0	29.4	1.0	1.4	3.4	23.8	53.1
6 Commercial	—	—	2.4	4.1	17.7	—	—	3.3	9.0	34.8	—	—	10.8	26.9	56.2

Note: — means not available.

Table 66. Average Minutes of Delay per Vehicle for Intermediate 4U Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	0.2	0.3	0.4	0.5	23.7	0.2	0.3	0.4	4.3	38.4	0.3	0.5	9.6	9.7	44.6
3 Commercial	0.4	0.4	0.5	2.7	18.0	0.4	0.5	1.4	4.1	18.7	0.5	1.7	2.8	15.8	51.8
6 Commercial	—	—	3.0	2.2	18.5	—	—	4.3	8.3	28.8	—	—	8.0	15.9	46.8

Note: — means not available.

Table 67. Average Minutes of Delay per Vehicle for Intermediate 4M Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	0.2	0.3	0.4	0.5	11.3	0.2	0.3	0.4	1.9	26.9	0.3	0.5	11.7	10.6	38.6
3 Commercial	0.4	0.4	0.7	0.6	22.9	0.4	0.5	0.5	9.7	26.2	0.5	0.6	9.0	21.0	44.2
6 Commercial	—	—	1.2	3.9	14.1	—	—	2.2	13.6	38.1	—	—	10.1	17.0	42.5

Note: — means not available.

Table 68. Average Minutes of Delay per Vehicle for Wide 2ST Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	0.6	1.2	1.3	1.7	12.0	0.6	1.2	1.4	3.7	28.4	0.7	1.5	4.3	21.0	46.6
3 Commercial	0.8	1.0	1.2	1.7	14.0	0.8	1.1	1.3	5.0	29.4	1.0	1.4	3.4	23.8	53.1
6 Commercial	—	—	2.4	4.0	18.1	—	—	3.0	7.8	32.8	—	—	10.8	26.8	56.2

Note: — means not available.

Table 69. Average Minutes of Delay per Vehicle for Wide 4U Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	0.2	0.3	0.4	1.9	18.6	0.2	0.3	2.5	4.9	21.0	0.3	0.4	10.7	7.3	47.5
3 Commercial	0.4	0.4	0.5	0.6	23.5	0.4	0.5	0.5	5.5	30.4	0.5	0.4	3.6	7.0	40.9
6 Commercial	—	—	2.0	2.9	21.0	—	—	2.8	9.5	27.8	—	—	10.9	23.9	30.5

Note: — means not available.

Table 70. Average Minutes of Delay per Vehicle for Wide 4M Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	0.2	0.3	0.4	0.6	14.7	0.2	0.3	0.4	0.8	36.7	0.3	0.5	5.2	22.0	43.0
3 Commercial	0.4	0.4	0.5	0.6	21.0	0.4	0.5	0.5	0.8	17.8	0.5	0.6	3.2	26.8	29.7
6 Commercial	—	—	1.3	5.1	23.6	—	—	4.4	11.6	36.3	—	—	11.2	21.7	49.9

Note: — means not available.

Table 71. Average Minutes of Delay per Vehicle for Wide 4T Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	0.2	0.3	0.3	0.4	0.6	0.2	0.3	0.4	0.7	0.7	0.2	0.4	0.5	0.6	0.9
3 Commercial	0.4	0.4	0.4	0.6	0.8	0.4	0.4	0.5	0.7	0.9	0.5	0.5	0.6	0.8	1.1
6 Commercial	—	—	0.9	0.7	1.7	—	—	0.8	1.2	1.5	—	—	1.0	1.4	1.8

Note: — means not available.

Average Travel Speeds

The research team also recorded the average speeds of all vehicles on the facility. The average speeds were dependent on the desired speeds, which varied between the cross sections. However, the average operating speed still worked as an intuitive metric to determine if the facility was experiencing a breakdown in operations. Table 72 to Table 75 show the average speeds for each intermediate width cross section. The color coding conforms with the 2016 *Highway Capacity Manual* level-of-service (LOS) cutoffs for a two-lane rural highway: a green cell corresponds to LOS A, a yellow cell corresponds to LOS C, and a red cell corresponds to LOS E and lower (AASHTO, 2016). Key results from the tables are listed as follows:

- Each intermediate cross section maintained speeds close to 60 mph at the 10,000 and 15,000 vpd volumes in each truck percentage.
- At 10 percent trucks, all cross sections had near LOS A speeds for the three industrial and three commercial driveways.
- The intermediate width 2S cross section had very low speeds during all of the scenarios with six commercial driveways, corresponding to a failing level of service.
- The speeds for the 2ST, 4U, and 4M corresponded to approximately an LOS B for 10 percent trucks and 17,500 and 20,000 vpd.
- The 2ST, 4U, and 4M intermediate cross sections each had speeds about 50 mph or higher at 20 percent trucks and 17,500 vpd at all driveway densities. These three cross sections also performed well at 20 percent trucks, 20,000 vpd, and a density of three industrial driveways/mile. However, at the higher driveway densities and 20 percent trucks, the 2ST, 4U, and 4M intermediate cross sections became unstable, where high speeds were dependent on random turning events.
- Each cross section had either LOS F speeds or unstable traffic with 40 percent trucks and ADTs exceeding 17,500 vpd. All intermediate cross sections had failing LOS speeds at all driveway densities and truck percentages with 25,000 vpd.

Table 72. Average Speed (mph) for Intermediate 2S Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	64	63	62	61	18	64	62	61	55	7	62	60	48	9	4
3 Commercial	63	62	61	57	12	62	61	59	45	5	60	58	40	6	2
6 Commercial	—	—	40	20	3	—	—	25	9	2	—	—	5	2	1

Note: — means not available.

Table 73. Average Speed (mph) for Intermediate 2ST Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	63	62	61	60	36	62	61	60	53	20	61	59	50	25	13
3 Commercial	61	61	60	58	30	61	60	59	46	17	59	58	50	19	9
6 Commercial	—	—	52	46	23	—	—	48	33	12	—	—	29	15	7

Note: — means not available.

Table 74. Average Speed (mph) for Intermediate 4U Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	64	65	64	64	27	64	64	64	53	16	63	63	39	47	14
3 Commercial	63	63	63	57	33	63	63	59	51	32	62	58	54	35	10
6 Commercial	—	—	52	55	25	—	—	49	42	19	—	—	35	25	9

Note: — means not available.

Table 75. Average Speed (mph) for Intermediate 4M Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	64	64	64	63	39	63	64	63	59	22	63	63	39	44	16
3 Commercial	63	63	62	62	25	62	62	62	42	20	61	61	38	31	12
6 Commercial	—	—	58	48	29	—	—	53	32	12	—	—	40	26	11

Note: — means not available.

Table 76 to Table 79 show the average speeds for each wide cross section with the same color-coding based on LOS. The results summarized in the tables indicate the following:

- The 4U and 4M wide cross sections maintained speeds close to 70 mph at the 10,000 and 15,000 vpd volumes in each truck percentage, while the 2ST and 4T wide scenarios had speeds around 60 mph in the same conditions.
- At 10 percent trucks, all cross sections had LOS A speeds for each driveway density for the 17,500 and 20,000 vpd scenarios with the exception of the 2ST at 20,000 vpd and six commercial driveways, which had an average speed of 47 mph corresponding to LOS C.
- All the wide cross sections had speeds corresponding to about LOS B or LOS A at 20 percent trucks and 17,500 vpd at all driveway densities.
- All the wide cross sections performed well at 20 percent trucks, 20,000 vpd, with three industrial driveways.
- The 4M and 4T wide cross sections each had LOS A at 20 percent trucks, 20,000 vpd, and three commercial driveways per mile. In the same conditions, the 4U was at LOS B and the 2ST was at LOS C.
- The 2ST, 4U, and 4M scenarios each had LOS F at 20 percent trucks, 20,000 vpd, and six commercial driveways.
- The 2ST, 4U, and 4M cross sections had either LOS F speeds or unstable traffic with 40 percent trucks and ADTs exceeding 17,500 vpd. These three wide cross sections had failing LOS speeds at all driveway densities and truck percentages with 25,000 vpd.
- The 4T wide cross section had LOS A for all scenarios simulated.

Table 76. Average Speed (mph) for Wide 2ST Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	63	62	61	60	36	62	61	60	53	20	61	59	50	25	13
3 Commercial	61	61	60	58	30	61	60	59	46	17	59	58	50	19	9
6 Commercial	—	—	53	47	22	—	—	51	36	13	—	—	29	15	7

Note: — means not available.

Table 77. Average Speed (mph) for Wide 4U Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	68	69	68	63	36	68	68	60	56	33	67	67	45	48	14
3 Commercial	67	67	67	67	29	66	67	67	52	19	65	67	56	49	13
6 Commercial	—	—	58	55	21	—	—	54	38	19	—	—	33	18	18

Note: — means not available.

Table 78. Average Speed (mph) for Wide 4M Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	68	68	68	67	41	67	68	67	66	16	66	67	51	31	14
3 Commercial	66	67	67	66	28	66	66	66	65	30	65	65	58	24	23
6 Commercial	—	—	61	54	24	—	—	49	37	13	—	—	34	24	8

Note: — means not available.

Table 79. Average Speed (mph) for Wide 4T Cross Section.

Truck Percentage	10					20					40				
ADT (vpd)	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000	10000	15000	17500	20000	25000
3 Industrial	62	62	62	62	61	61	62	62	60	60	61	61	61	60	59
3 Commercial	61	61	61	60	60	60	61	60	60	59	59	60	59	59	58
6 Commercial	—	—	57	59	55	—	—	58	56	55	—	—	56	55	53

Note: — means not available.

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

This study presented the results of the statistical analyses conducted on crashes that occurred on study segments. The cross-sectional modeling was conducted with a primary objective of developing SPFs to describe the relationship between crash frequency and traffic and geometric variables. Horizontal curve presence, driveway density, shoulder width, and operating speed have been identified as key influential variables. The PSM method was used to validate the results obtained from the cross-sectional modeling.

The key points of the safety modeling results are as follows:

- There is no one best cross section for all circumstances, although it is clear that the 4U cross section generally has the worst safety performance of all the cross sections considered.
- The 2S cross section has the best safety performance in all circumstances at volumes up to 12,000 vpd. The sites considered in this study had traffic volumes at 12,000 vpd or less. It was previously shown that 2S cross sections perform well until at least 15,000 vpd. 2S may also be appropriate for volumes greater than 15,000 vpd and needs further investigation.
- The shoulder width and driveway density have varying effects on different cross sections. Mainly, the effect of shoulder width on the safety performance of 4M roadways is substantial, with shoulders of less than 6 ft significantly increasing crashes. Above 6 ft, safety performance on 4M continues to improve up to 12 ft shoulders.
- 4M cross sections are highly effective in reducing lane-departure crashes. They produce excellent safety performance at volumes above 15,000 as long as the cross section has at least 6 ft shoulders and driveway density is low (10 driveways per mile or less).
- 4T sections provide better safety performance when the driveway density is higher. At driveway densities above 10 per mile, increases in driveway density drive crashes up more on 4M roadways than on 4T.
- When reviewing the findings, it is important to recognize that 4T roadways are generally built-in areas with a significant amount of land use activity, turning, and crossing volumes, and the others may not be. Therefore, the results for the 4T roadway may reflect the complex situations (and crash levels associated thereof) rather than the performance of the cross section itself.
- As operating speeds increase, so do the number of crashes. For example, a roadway with a PSL of 65 mph and with an average operating speed of 75 mph will experience 15 percent more crashes than a highway with an operating speed of 65 mph.

The following guidelines are recommended based on the safety analysis:

- A 2S cross section is recommended when the volumes are lower than 15,000 vpd and the driveway density is less than 10 driveways per mile. 2ST may be recommended for higher driveway densities, but not enough data are available yet to quantify the safety performance of these roadways.

- 4T is recommended when the volumes are higher than 25,000 vpd, irrespective of the driveway density. For higher driveway density (greater than 10 driveways per mile), 4T is recommended, particularly when shoulders of 6 ft cannot be provided.
- 4M is recommended when the volume is between 15,000 and 20,000 vpd and the total paved surface width is at least 64 ft. 4M is not recommended for sections with less than a 6 ft shoulder.
- Widening the highway is a feasible option when the above conditions are not met.

The simulation experiment explored intermediate (58 ft) and wide (70 ft) pavement width facilities with different cross-sectional operations varying from 2S to four-lane facilities and with and without turning lanes. The narrow width (≤ 50 ft) is not considered since 2S is the only possible alternative. Each facility was analyzed with varying truck percentages, volume levels, and driveway densities. The intermediate facilities considered were 2S, 2ST, 4U, and 4M cross-sections (4T is not a possible option for this cross-sectional width). The 2S is not advisable in any scenario with a density of six commercial driveways per mile. All intermediate width facilities are not advisable at 25,000 vpd in any combination, nor are they advisable above 15,000 vpd with 40 percent trucks. Table 80 to Table 82 show the advisable intermediate cross sections for each truck percentage scenario based solely on operations. (Note: The 10,000 and 15,000 vpd levels are not possible in a scenario with six commercial driveways per mile and were not simulated.)

Table 80. Advisable Cross Sections for Intermediate Width—10 Percent Trucks.

Driveway Density	**ADT, vpd**				
	10000	**15000**	**17500**	**20000**	**25000**
3 Industrial	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	None
3 Commercial	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	None
6 Commercial	—	—	2ST, 4U, or 4M	2ST, 4U, or 4M	None

Note: — means not available.

Table 81. Advisable Cross Sections for Intermediate Width—20 Percent Trucks.

Driveway Density	**ADT, vpd**				
	10000	**15000**	**17500**	**20000**	**25000**
3 Industrial	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	None
3 Commercial	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	None	None
6 Commercial	—	—	2ST, 4U, or 4M	None	None

Note: — means not available.

Table 82. Advisable Cross Sections for Intermediate Width—40 Percent Trucks.

Driveway Density	ADT, vpd				
	10000	**15000**	**17500**	**20000**	**25000**
3 Industrial	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	None	None	None
3 Commercial	2S, 2ST, 4U, or 4M	2S, 2ST, 4U, or 4M	None	None	None
6 Commercial	—	—	None	None	None

Note: — means not available.

The wide cross sections considered in this simulation study were a 2ST, 4U, 4M, and 4T. The 4T scenario maintained LOS A type speeds under all operating scenarios and was the only cross section advisable for 25,000 vpd among those considered in this study. Table 83 to Table 85 show the advisable wide pavement width cross sections for each truck percentage operating scenario based solely on operations.

Table 83. Advisable Cross Sections for Wide Width—10 Percent Trucks.

Driveway Density	ADT, vpd				
	10000	**15000**	**17500**	**20000**	**25000**
3 Industrial	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	4T
3 Commercial	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	4T
6 Commercial	—	—	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	4T

Note: — means not available.

Table 84. Advisable Cross Sections for Wide Width—20 Percent Trucks.

Driveway Density	ADT, vpd				
	10000	**15000**	**17500**	**20000**	**25000**
3 Industrial	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	4T
3 Commercial	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	4U, 4M, and 4T	4T
6 Commercial	—	—	2ST, 4U, 4M, and 4T	4T	4T

Note: — means not available.

Table 85. Advisable Cross Sections for Wide Width—40 Percent Trucks.

Driveway Density	ADT, vpd				
	10000	**15000**	**17500**	**20000**	**25000**
3 Industrial	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	4T	4T
3 Commercial	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	2ST, 4U, 4M, and 4T	4T	4T
6 Commercial	—	—	4T	4T	4T

Note: — means not available.

6.1. Guidelines for Selecting Cross Sections

Based on the findings from research conducted in this project, the research team recommends the following guidelines for selecting cross sections for rural highways:

- 4U cross sections have poor safety performance and mediocre operational performance compared to other alternatives and should be avoided.
- For existing 4U roadways:
  - 4U sections with 15,000 ADT or less should be reviewed for conversion to 2S. These sections can be restriped as 2S roadways to significantly reduce traffic crashes without creating any operational issues. It may be necessary to add turn lanes (i.e., a 2ST cross section) with higher levels of driveway activity.
  - Sections above 15,000 ADT should be reviewed for adding a 4-ft median buffer (i.e., a 4M cross section). Adding a buffer to a 4U roadway results in significant safety improvement if shoulders of 6 ft or more are provided and foreslopes are not reduced. If driveway activity is high, a center turn lane (i.e., a 4T cross section) may be necessary.
- For all roadways, traffic volume (ADT), shoulder width, truck percentage, and driveway activity all play significant roles in safety and operational performance. When considering the potential widening of a 2U roadway or changing the cross section of any other rural highway, these effects should be considered. Based on these effects, as identified in this research project, preferred cross sections for key combinations of rural highways are summarized in Table 86.

Table 86. Guidelines for Selecting Cross Sections Based on Safety and Operational Performance.

Nominal Pavement Width (range)	AADT	Driveway Activity Index[a] per Mile	Truck Percentage	Preferred Cross Section
50 ft (≤55 ft)	≤15,000	≤30	Any	Super 2
		>30	Any	Widen to Super 2 with TWLTL
	>15,000	≤30	Any	Widen to Four Lanes with 4-ft Median Buffer[b]
		>30	Any	Widen to Four Lanes with TWLTL
60 ft (56–65 ft)	≤15,000	≤30	Any	Super 2
		>30	≤15%	Super 2 with TWLTL
			15–25%	Super 2 with TWLTL
			>25%	Widen to Four Lanes with TWLTL
	15,000–20,000	≤30	≤15%	Four Lanes with 4-ft Median Buffer[b]
			15–25%	Four Lanes with 4-ft Median Buffer[b]
			>25%	Widen to Four Lanes with TWLTL
		>30	≤15%	Four Lanes with 4-ft Median Buffer[b]
			15–25%	Four Lanes with 4-ft Median Buffer[b]
			>25%	Widen to Four Lanes with TWLTL
	>20,000	Any	Any	Widen to Four Lanes with TWLTL
70 ft (≥66 ft)	≤15,000	≤30	Any	Four Lanes with 4-ft Median Buffer[b]
		>30	Any	Super 2 with TWLTL
	15,000–20,000	≤30	≤15%	Four Lanes with 4-ft Median Buffer[b]
			15–25%	Four Lanes with 4-ft Median Buffer[b]
			>25%	Four Lanes with TWLTL
		>30	≤15%	Four Lanes with 4-ft Median Buffer
			15–25%	Four Lanes with TWLTL
			>25%	Four Lanes with TWLTL
	>20,000	Any	Any	Four Lanes with TWLTL

[a] Driveway activity index is the number of residential driveways. It is equal to three times the number of industrial driveways, or 12 times the number of commercial driveways (measured per mile).
[b] 6-ft minimum shoulder width. Greater widths are desirable.

6.2. OPPORTUNITY FOR IMPLEMENTATION

In terms of implementing these guidelines, there is a great opportunity to make improvements to a large number of centerline miles to provide these safety and operational benefits. Based on the sample of 131 segments of 4U roadways used in this research, the research team estimates a distribution of improvements as shown in Table 87.

Table 87. Distribution of 4U Sample Roadways for Cross-Section Improvements.

Nominal Pavement Width (range)	AADT	Driveway Activity Index per Mile	Truck Percentage	Preferred Cross Section	% Based on Sample	Estimated Existing Miles
50 ft (≤55 ft)	≤15,000	≤30	Any	Super 2	26.0%	463
		>30	Any	Widen to Super 2 with TWLTL	11.5%	204
	>15,000	≤30	Any	Widen to Four Lanes with 4-ft Median Buffer	1.5%	27
		>30	Any	Widen to Four Lanes with TWLTL	0.0%	0
60 ft (56–65 ft)	≤15,000	≤30	Any	Super 2	26.7%	476
		>30	≤15%	Super 2 with TWLTL	4.6%	82
			15–25%	Super 2 with TWLTL	6.1%	109
			>25%	Widen to Four Lanes with TWLTL	5.3%	95
	15,000–20,000	≤30	≤15%	Four Lanes with 4-ft Median Buffer	0.0%	0
			15–25%	Four Lanes with 4-ft Median Buffer	0.0%	0
			>25%	Widen to Four Lanes with TWLTL	0.0%	0
		>30	≤15%	Four Lanes with 4-ft Median Buffer	0.0%	0
			15–25%	Four Lanes with 4-ft Median Buffer	0.0%	0
			>25%	Widen to Four Lanes with TWLTL	0.0%	0
	>20,000	Any	Any	Widen to Four Lanes with TWLTL	0.0%	0
70 ft (≥66 ft)	≤15,000	≤30	Any	Four Lanes with 4-ft Median Buffer	11.5%	204
		>30	Any	Super 2 with TWLTL	6.9%	122
	15,000–20,000	≤30	≤15%	Four Lanes with 4-ft Median Buffer	0.0%	0
			15–25%	Four Lanes with 4-ft Median Buffer	0.0%	0
			>25%	Four Lanes with TWLTL	0.0%	0
		>30	≤15%	Four Lanes with 4-ft Median Buffer	0.0%	0
			15–25%	Four Lanes with TWLTL	0.0%	0
			>25%	Four Lanes with TWLTL	0.0%	0
	>20,000	Any	Any	Four Lanes with TWLTL	0.0%	0

When described simply by the preferred improved cross section, the distribution of 4U sample sites can be summarized as follows:

- Overlay and restripe existing 4U to:
    - 2S: 939 mi (52.7 percent).
    - 2S + TWLTL: 313 mi (17.6 percent).
    - Four-lane + 4-ft buffer: 204 mi (11.5 percent).
- Widen and restripe existing 4U to:
    - 2S + TWLTL: 204 mi (11.5 percent).
    - Four-lane + 4-ft buffer: 27 mi (1.5 percent).
    - Four-lane + TWLTL: 95 mi (5.3 percent).

The distribution of the sample 4U roadways is not necessarily representative of all 4U roadways on the state highway system, but it does provide an illustration of the potential for improvement. More than half of the 4U roadways in the sample can be improved for safety and operations with overlay and restriping (i.e., without widening). The restriping to make configuration changes can be coordinated with the repaving schedule. An implementation project that incorporates a more detailed review of additional 4U roadways, in conjunction with details on construction costs, value of delay reduction, and value of crash and injury reduction, will provide a straightforward method of determining specific benefit-cost ratios and prioritizing which roadways to improve first.

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APPENDIX A—SURVEY QUESTIONNAIRE

We would appreciate your help in identifying the location of rural 4-lane undivided highways with median buffers, along with other cross-sections that might be suitable alternatives.

This questionnaire is part of a task on TxDOT Research Project 0-7035, "Examine Trade-Offs Between Center Separation and Shoulder Width Allotment for a Given Roadway Width".
The project has the following technical objectives:

1. Determine the safety and operational benefits of providing median buffer by reducing lane or shoulder widths on a four-lane undivided highway (4U).

2. Evaluate the safety and operational performance of other similar cross-section alternatives when compared to a traditional 4U.

The 0-7035 research team is led by Srinivas Geedipally at TTI, and the TxDOT Project manager is Tom Schwerdt. If you have questions about this questionnaire or the project in general, you may contact them as follows:

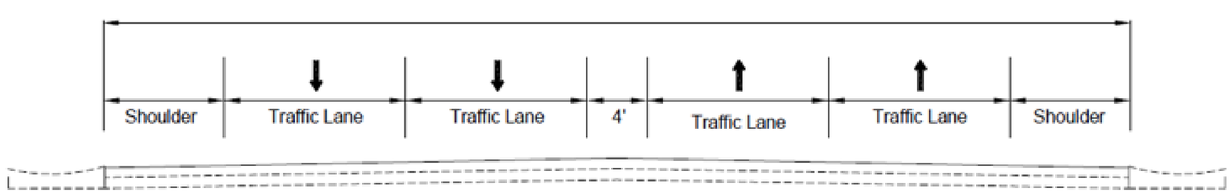
- Srinivas Geedipally – srinivas-g@tti.tamu.edu
- Tom Schwerdt – Tom.Schwerdt@txdot.gov

We would appreciate your help in compiling that information through your responses to the following questions.

Thank you for your input!

Question 1:

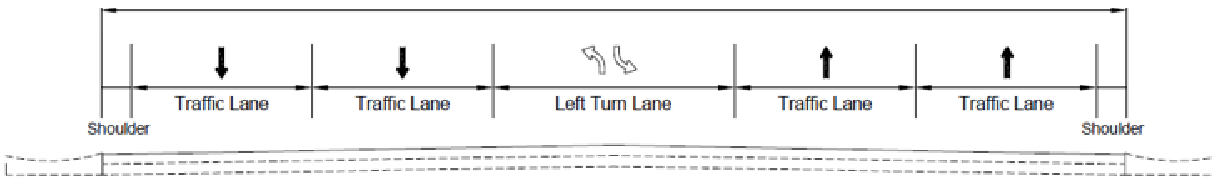
The figure below shows an example of a four-lane undivided highway with median buffer (4M). Median buffers are paved, painted, and unprotected with width between 4 and 6 ft.



What are the locations (existing or planned) of rural 4M in your District? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

Question 2:

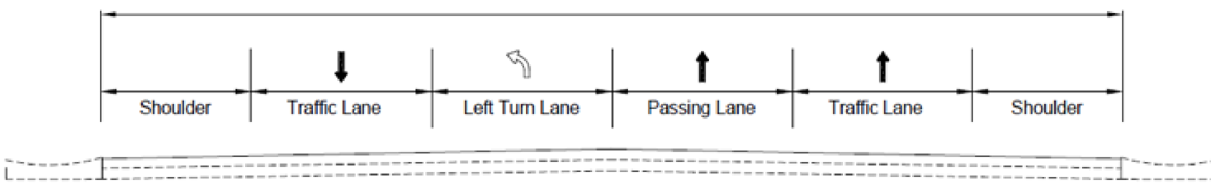
The figure below shows an example of a four-lane undivided highway with continuous two-way left-turn lane (4T).



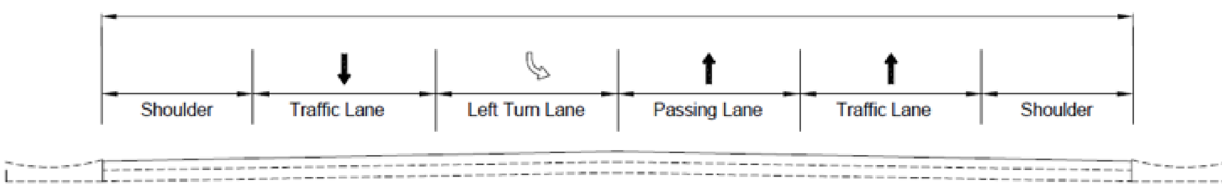
What are the locations (existing or planned) of rural 4T in your District? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

Question 3:

The figure below shows examples of a Super 2 cross-section with occasional left-turn lane (2SL).



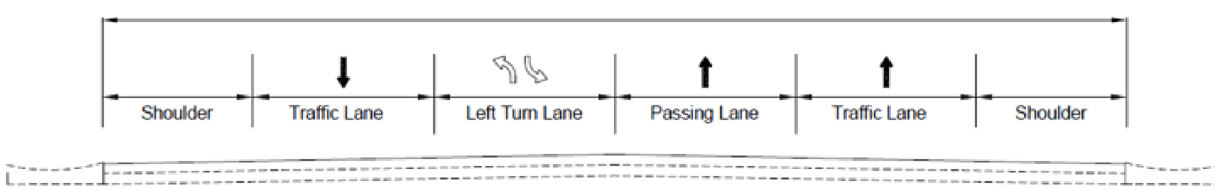
Or



What are the locations (existing or planned) of rural 2SL in your District? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

Question 4:

The figure below shows an example of a Super 2 cross-section with continuous two-way turn lane (2ST).



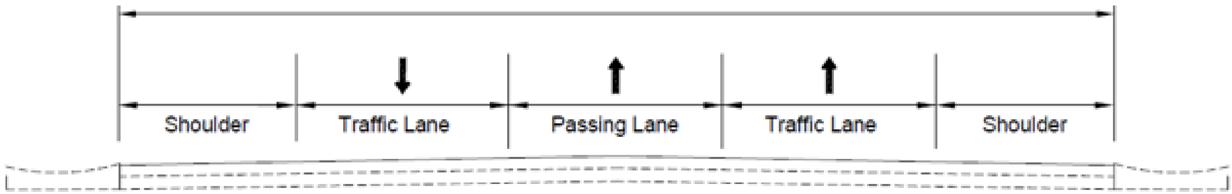
What are the locations (existing or planned) of rural 2ST in your District? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

Question 5:

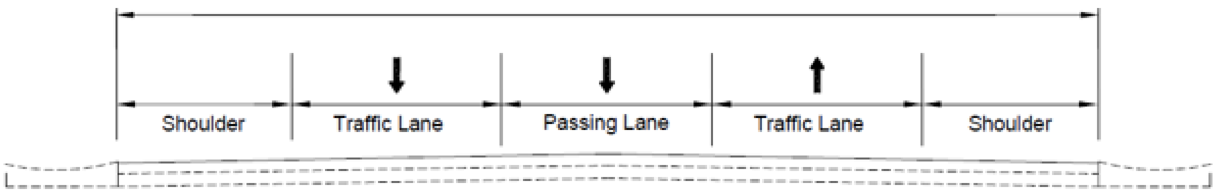
Of the cross-sections mentioned in Q1 to Q4, which ones were converted (or are planned to be converted) from 4U? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

Question 6:

The figure below shows examples of a Super 2 cross-section.



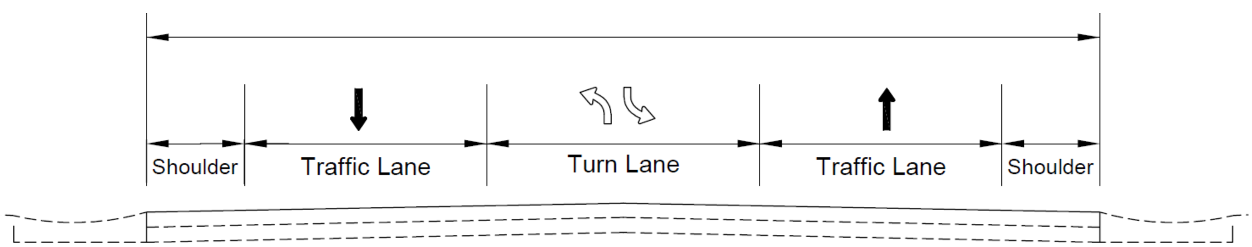
Or



What are the locations of Super 2 cross-sections in your district that were converted (or are planned to be converted) from 4U? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

Question 7:

The figure below shows an example of a two-lane undivided highway with continuous turn lane (2T).



What are the locations of rural 2T in your district that were converted (or are planned to be converted) from 4U? (*Open-ended text box for responses*) (*Enter none if you do not have any*)

If you have any of the cross-sections discussed in the previous questions, may we follow up with you to request additional details about those locations? If so, please provide your name and e-mail address.

- Name (open-ended text box)
- E-mail Address (open-ended text box)

Thank you again for your participation!

APPENDIX B—WEJO DRIVEWAY DATA

Table B1. Trips and AADT on 10 Selected Sites.

Segment/Driveway ID	Field Site ID	Wejo Trips (60 Days)	AADT from RHINO
Selected Field Sites			
2S_233	Field01	15,594	4,040
4U_278	Field02	30,265	7,910
2S_133	Field03	34,995	6,832
2S_856	Field03	34,901	6,832
2S_2281	Field03	34,984	6,832
2ST_003	Field04	51,529	10,375
2ST_005	Field04	51,447	10,375
2ST_033	Field04	50,274	10,375
4M_035	Field05	26,442	4,889
4M_074	Field05	27,376	4,889
4M_098	Field05	27,016	4,889
4U_034	Field06	52,639	13,482
2ST_001	Field07	21,488	6,385
4M_082	Field08	16,900	5,015
4M_101	Field08	17,240	5,015
4T_208	Field09	41,843	14,514
4U_088	Field10	27,273	8,954
Total		**562,206**	**131,603**
Driveways (partial)			
2S_233_L01	Field01	62	—
2S_233_L02	Field01	18	—
2S_233_L03	Field01	23	—
2S_233_L04	Field01	42	—
2S_233_L05	Field01	21	—

Note: The actual number of segments is greater than the number of sites (i.e., one site may contain multiple segments). — means not available.

Table B2. Estimated AADT on Driveways.

Driveway ID	Field Site ID	Wejo Trips (60 Days)	Estimated AADT
2S_233_L01	Field01	62	14.5
2S_233_L02	Field01	18	4.2
2S_233_L03	Field01	23	5.4
2S_233_L04	Field01	42	9.8
2S_233_L05	Field01	21	4.9
2S_233_L06	Field01	17	4.0
2S_233_L07	Field01	30	7.0
2S_233_R01	Field01	26	6.1
2S_233_R02	Field01	22	5.1

Driveway ID	Field Site ID	Wejo Trips (60 Days)	Estimated AADT
2S_233_R03	Field01	19	4.4
2S_233_R04	Field01	36	8.4
2S_233_R05	Field01	196	45.9
2S_233_R06	Field01	194	45.4
2S_233_R07	Field01	114	26.7
4U_278_L01	Field02	61	14.3
4U_278_L02	Field02	54	12.6
4U_278_L03	Field02	45	10.5
4U_278_L04	Field02	65	15.2
4U_278_L05	Field02	49	11.5
4U_278_L07	Field02	57	13.3
4U_278_L08	Field02	66	15.4
4U_278_L06	Field02	57	13.3
4U_278_L09	Field02	141	33.0
4U_278_L10	Field02	70	16.4
4U_278_L11	Field02	70	16.4
4U_278_L12	Field02	96	22.5
4U_278_L13	Field02	703	164.6
4U_278_L14	Field02	73	17.1
4U_278_L15	Field02	49	11.5
4U_278_L16	Field02	43	10.1
4U_278_R01	Field02	65	15.2
4U_278_R02	Field02	56	13.1
4U_278_R03	Field02	62	14.5
4U_278_R04	Field02	56	13.1
4U_278_R05	Field02	99	23.2
4U_278_R06	Field02	68	15.9
4U_278_R07	Field02	58	13.6
4U_278_R08	Field02	146	34.2
4U_278_R09	Field02	68	15.9
4U_278_R10	Field02	47	11.0
4U_278_R11	Field02	70	16.4
4U_278_R12	Field02	501	117.3
4U_278_R13	Field02	2,368	554.3
2S_133_L01	Field03	82	19.2
2S_133_L02	Field03	74	17.3
2S_133_R01	Field03	60	14.0
2S_856_L01	Field03	61	14.3
Q5: 003_L02	Field04	108	25.3
Q5: 003_L01	Field04	84	19.7
Q5: 003_R01	Field04	61	14.3

Driveway ID	Field Site ID	Wejo Trips (60 Days)	Estimated AADT
Q5: 003_R02	Field04	87	20.4
Q5: 033_R01	Field04	83	19.4
4M_074_L01	Field05	556	130.2
4M_074_R01	Field05	109	25.5
4M_098_R01	Field05	73	17.1
4U_034_L01	Field06	74	17.3
4U_034_R01	Field06	1,696	397.0
4U_034_L02	Field06	1,181	276.5
2ST_001_L01	Field07	21	4.9
2ST_001_L18	Field07	35	8.2
2ST_001_L19	Field07	122	28.6
2ST_001_L20	Field07	214	50.1
2ST_001_L21	Field07	33	7.7
2ST_001_L22	Field07	28	6.6
2ST_001_L23	Field07	24	5.6
2ST_001_L24	Field07	31	7.3
2ST_001_L25	Field07	35	8.2
2ST_001_L26	Field07	431	100.9
2ST_001_L27	Field07	603	141.2
2ST_001_L28	Field07	38	8.9
2ST_001_L04	Field07	37	8.7
2ST_001_L02	Field07	172	40.3
2ST_001_L03	Field07	42	9.8
2ST_001_L05	Field07	24	5.6
2ST_001_L06	Field07	26	6.1
2ST_001_L07	Field07	22	5.1
2ST_001_L08	Field07	22	5.1
2ST_001_L09	Field07	28	6.6
2ST_001_L10	Field07	26	6.1
2ST_001_L11	Field07	36	8.4
2ST_001_L12	Field07	37	8.7
2ST_001_L13	Field07	29	6.8
2ST_001_L14	Field07	918	214.9
2ST_001_L15	Field07	26	6.1
2ST_001_L16	Field07	33	7.7
2ST_001_L17	Field07	32	7.5
2ST_001_R01	Field07	43	10.1
2ST_001_R11	Field07	40	9.4
2ST_001_R02	Field07	111	26.0
2ST_001_R03	Field07	32	7.5
2ST_001_R04	Field07	173	40.5

Driveway ID	Field Site ID	Wejo Trips (60 Days)	Estimated AADT
2ST_001_R05	Field07	59	13.8
2ST_001_R06	Field07	96	22.5
2ST_001_R07	Field07	21	4.9
2ST_001_R08	Field07	38	8.9
2ST_001_R19	Field07	39	9.1
2ST_001_R18	Field07	195	45.6
2ST_001_R17	Field07	33	7.7
2ST_001_R16	Field07	70	16.4
2ST_001_R15	Field07	45	10.5
2ST_001_R14	Field07	30	7.0
2ST_001_R13	Field07	30	7.0
2ST_001_R12	Field07	98	22.9
2ST_001_R10	Field07	32	7.5
2ST_001_R09	Field07	32	7.5
2ST_001_R20	Field07	73	17.1
4M_082_L01	Field08	20	4.7
4M_101_L01	Field08	209	48.9
4M_101_L02	Field08	40	9.4
4M_101_L03	Field08	24	5.6
4M_101_L04	Field08	27	6.3
4M_101_L05	Field08	31	7.3
4M_101_L06	Field08	31	7.3
4M_101_R01	Field08	31	7.3
4M_101_R02	Field08	22	5.1
4M_101_R03	Field08	72	16.9
4T_208_R01	Field09	66	15.4
4T_208_L01	Field09	116	27.2
4T_208_L02	Field09	462	108.1
4T_208_L03	Field09	96	22.5
4T_208_L04	Field09	101	23.6
4T_208_L05	Field09	146	34.2
4T_208_L06	Field09	41	9.6
4T_208_L07	Field09	120	28.1
4T_208_L08	Field09	90	21.1
4T_208_L09	Field09	65	15.2
4T_208_L10	Field09	173	40.5
4T_208_R02	Field09	143	33.5
4T_208_R03	Field09	109	25.5
4T_208_R04	Field09	130	30.4
4T_208_R05	Field09	62	14.5
4T_208_R06	Field09	66	15.4

Driveway ID	Field Site ID	Wejo Trips (60 Days)	Estimated AADT
4T_208_R07	Field09	251	58.8
4U_088_L01	Field10	720	168.5
4U_088_R01	Field09	154	36.0
4U_088_R02	Field09	77	18.0
4U_088_R03	Field09	51	11.9
4U_088_R04	Field09	61	14.3

Table B3. Roadway Segments and Corresponding Traffic Management Center Numbers.

Highway	Segment ID	Traffic Management Center Number
SH 158	4M_058	112+06722; 112-08641
SH 158	4M_032; 4M_107	112+08977; 112-08640
SH 158	4M_083; 4M_102; 4M_066	112+08641; 112-08977
SH 349	4M_004; 4M_024; 4M_109; 4M_077; 4M_027	111+16283; 111-11719
US 79	Start Coordinate: 32°18'15.58"N 94° 9'53.34"W End Coordinates: 32°22'22.02"N 94° 2'51.25"W	111+18963; 111-18962
SH 21	Q5: 4U to 2ST 003; Q5: 4U to 2ST_005; Q5: 4U to 2ST_012; Q5: 4U to 2ST_023; Q5: 4U to 2ST_029	112+09084; 112-09083
SH 21	Q5: 4U to 2ST_033	112+06055; 112-09084

Table B4. Speed Measure Variables.

Speed Measure	Definition
SpdAve	Average speed determined for year using all data
SpdStd	Standard deviation of speed determined for year using all data
Spd85	85th percentile speed determined for year using all data
SpdAveDay	Average speed determined for year ($6 \leq$ hour ≤ 18) using all data
SpdStdDay	Standard deviation of speed determined for year ($6 \leq$ hour ≤ 18) using all data
SpdAveNight	Average speed determined for year ($19 \leq$ hour ≤ 23 and $0 \leq$ hour ≤ 5) using all data
SpdStdNight	Standard deviation of speed determined for year ($19 \leq$ hour ≤ 23 and $0 \leq$ hour ≤ 5) using all data
SpdAveMTWT	Average speed determined for year (Mon, Tues, Wed, Thurs) using all data
SpdStdMTWT	Standard deviation of speed determined for year (Mon, Tues, Wed, Thurs) using all data
SpdAveFSS	Average speed determined for year (Fri, Sat, Sun) using all data
SpdStdFSS	Standard deviation of speed determined for year (Fri, Sat, Sun) using all data
SpdFFAve	Average speed determined for year using speed data during $1 \leq$ hour ≤ 4
SpdFF85	85th percentile speed determined for year using speed data during $1 \leq$ hour ≤ 4

APPENDIX C—VALUE OF RESEARCH ANALYSIS

OVERVIEW

The research team conducted a value of research (VOR) analysis of TxDOT Research Project 0-7035 to produce an estimate of the benefit that the project will likely yield for TxDOT. The temporal scope for this analysis is an 11-year period (labeled as years 0–10) starting with the beginning of the 3-year project. The value of the project is described in terms of net present value (NPV) and cost-benefit ratio (CBR), which are computed using economic discounting formulas.

The primary objective of TxDOT Research Project 0-7035 is to develop a framework that road design engineers can use in making decisions on cross sections for new and resurfaced roadway segments. The project quantifies the safety (in terms of reduced crash frequency) and operational (in terms of reduced delays) benefits that can be obtained by converting 4U highways to other cross-sectional designs. Since the difference in delays is not significantly different between cross-sectional alternatives, the research team focused the VOR analysis on the safety benefits of the conversions and the resulting cost savings that can be obtained by improving this knowledge.

METHODOLOGY

The research team used a VOR template provided by TxDOT to compute the NPV and CBR measures. The template requires the following items:

- Project budget: $258,085 ($9,096 in year 0 + $129,970 in year 1 + $119,019 in year 2).
- Project duration: 2.08 years.
- Expected value duration: 10 years (convention chosen by TxDOT).
- Discount rate: 3 percent (default value assumed by TxDOT).
- Expected value per year: $283,738.

The project's expected value per year is estimated based on savings obtained from reduced crashes. The analysis method is described in the following sections.

Concept

An analysis method that can be used to estimate the benefit of conducting a research project on a safety treatment is documented in National Cooperative Highway Research Program (NCHRP) Report 756 (Pratt et al., 2014; Zegeer, 2013).

To conduct a VOR analysis, it is necessary to conduct the following steps:

1. Identify target sites where a treatment can be implemented.
2. Determine the total number of crashes at these sites.
3. Determine the reduced crash frequency by severity due to conversion of 4U highways to other cross-sectional alternatives.
4. Apply the procedure to estimate the expected VOR.

For TxDOT Research Project 0-7035, the treatment of interest is restriping the four-lane rural highways to other cross sections. Studies have shown that 4U roadways have poor safety performance compared to 4D and 2D cross sections. 4U highways experience relatively high crash frequencies—especially as traffic volume increases—resulting in conflicts with high-speed OD vehicles. However, there is not always sufficient space within the available right of way to accommodate a traditional 4D cross section. Some states, including Texas, have started providing a narrow centerline buffer area that is separated by longitudinal pavement markings. This additional buffer area shifts the lateral placement of vehicles and introduces a greater physical separation between approaching vehicles. However, the provision of centerline buffer comes at a cost of reduced lane or shoulder widths. Other cross sections such as 2S with and without TWLTL and 4T are also possible alternatives to 4U roadways.

Conducting a research project is expected to yield a better understanding of the benefits of center separation, as well as lane and shoulder combinations, to designers who make decisions on cross sections for new and resurfaced roadway segments. This improved knowledge will reduce losses that TxDOT would otherwise incur due to poor performance of 4U roadways in terms of increased crashes.

Input Data

The VOR analysis method documented in NCHRP Report 756 is implemented using a spreadsheet program called Safety Research Prioritization Worksheet (SRPW), which is available from NCHRP and described in a user manual (Zegeer, 2013). The required input data, values, and sources are listed in Table C1. The input data provide information about the candidate sites for treatment, safety knowledge of the treatment, crash cost, and treatment cost. Safety knowledge is described in terms of CMFs.

Sites

The research team queried the Texas Reference Marker (TRM) database to obtain an estimate of the total mileage of rural four-lane highways in Texas. Almost 94 percent of the segments have a traffic volume less than 15,000 vpd. About 43 mi have a traffic volume of more than 20,000 vpd. This query also revealed that the average ADT was 7,630 vpd and the average segment length was 0.252 mi.

Table C1. VOR Analysis Input Data and Sources.

Topic	Input Data	Value(s)	Source/Notes
Sites	Target highway miles	1783	Query of TRM database, including all 4U highways
	Average AADT, vpd	7630	Query of TRM database
	Average length of segment, mi	0.252	Query of TRM database
Safety Knowledge	SPF coefficients	b_0 = 0.00157, b_1 = 0.803	TxDOT Research Project 0-7035
	Inverse dispersion parameter	0.548	TxDOT Research Project 0-7035
	Mean CMF value (effect of countermeasure) for 2S	0.761	Bonneson and Pratt (2009) for conversion of four-lane to two-lane based on crash predictions and Park et al. (2012) for adding passing lane (i.e., two-lane to 2S)
	Lowest and highest likely CMF values for 2S	0.533, 1.028	Used default assumptions of 70 percent and 135 percent of mean value for SRPW
	Mean CMF value (effect of countermeasure) for 2S	0.942	Dixon et al. (2018) and the proportion of OD and total crashes from 0-7035
	Lowest and highest likely CMF values for 2S	0.659, 1.271	Used default assumptions of 70 percent and 135 percent of mean value for SRPW
Crash Cost	Crash distribution by severity	K = 0.0350, A = 0.0764, B = 0.1465, C = 0.1274, PDO = 0.6146	Query of TRM and CRIS database
	Cost of K, A, B crash	\$3.7 million, \$3.7 million, and \$520,000, respectively	TxDOT's Highway Safety Improvement Program guidelines Dixon and Avelar (2015)
	Costs of C and PDO crashes	\$155,000, and \$51,000, respectively	National Safety Council 2019 estimates
	Limiting benefit-cost ratio	2.5	NCHRP Report 756 (Zegeer, 2013)
Treatment Cost	Treatment implementation level of 2S, 2ST, and four-lane with 4-ft median buffer	53 percent, 18 percent, 12 percent of sites, respectively	Estimated based on sample in 0-7035
	Countermeasure service life	10 years	Assumed
	Initial cost of project of 2S, 2ST, and four-lane with 4-ft median buffer	\$23,000, \$29,000, and \$32,000 per mile, respectively	Estimated based on the costs provided by TxDOT
	Annual maintenance cost of project	\$0 per mile	No added maintenance costs compared to the do-nothing alternative

Safety Knowledge

The SPF for total crashes on rural highways for different cross sections is described by Equation 21.

$$N = L \times y \times e^{b_0 + b_{aadt}\ln(AADT) + b_r I_r} \times CMF_{hc} \times CMF_{dw} \times CMF_{sw} \times CMF_{spd}; \qquad (21)$$

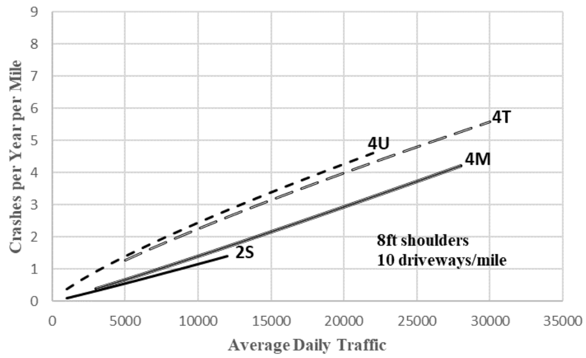
with:

$$CMF_{hc} = e^{b_{hc}(p_{hc})}$$
$$CMF_{dw} = e^{b_{dw} \times 0.1 \times (dw-10)}$$
$$CMF_{sw} = e^{b_{sw}(sw-6)}$$
$$CMF_{spd} = e^{b_{spd}(SpdFF85-\mathrm{PSL})}$$

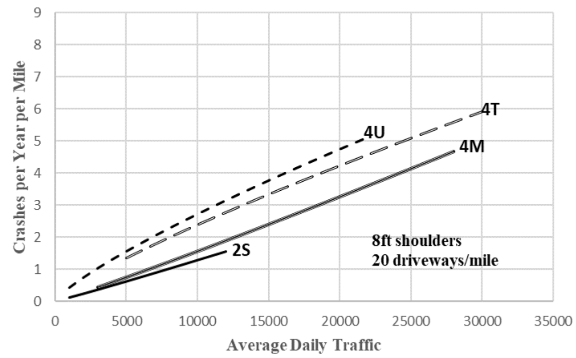
where:

CMF_{hc}	=	CMF for horizontal curve presence.
CMF_{dw}	=	CMF for driveway density.
CMF_{sw}	=	CMF for shoulder width.
CMF_{spd}	=	CMF for 85th percentile free-flow speed.
p_{hc}	=	proportion of horizontal curve presence on segment.
dw	=	equivalent driveway density, driveways/mile.
sw	=	average shoulder width, ft.
$SpdFF85$	=	85th percentile free-flow speed, mph.
PSL	=	posted speed limit, mph.

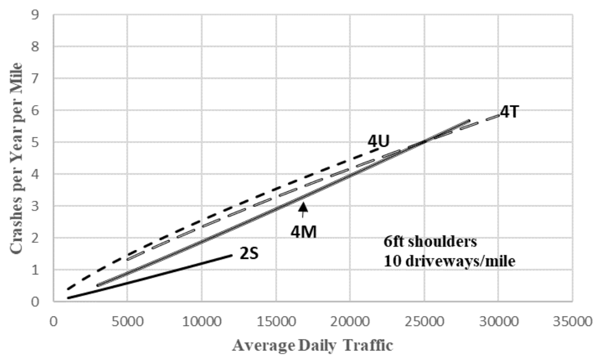
Figure C1 shows the relationship between the number of total crashes and traffic flow for all cross sections. Six different plots are shown for various shoulder widths and driveway densities. The remaining CMFs are set to 1.0 (representing base conditions).



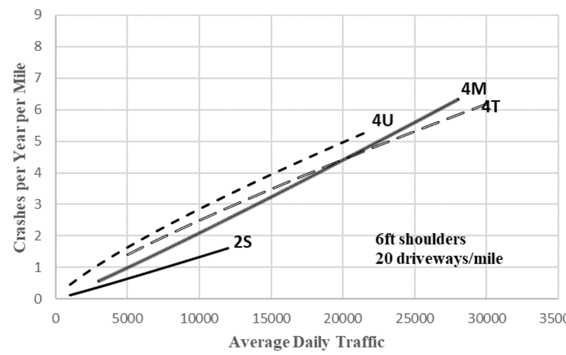
a) Wide shoulders, low driveway density



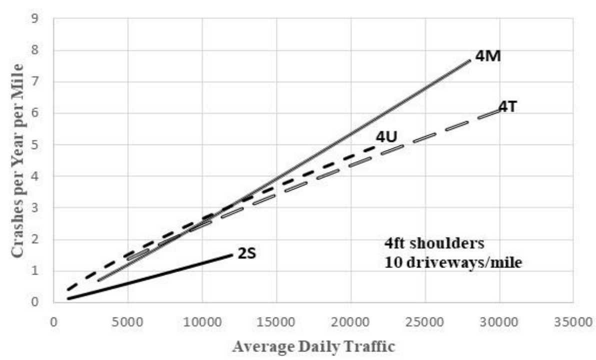
b) Wide shoulders, high driveway density



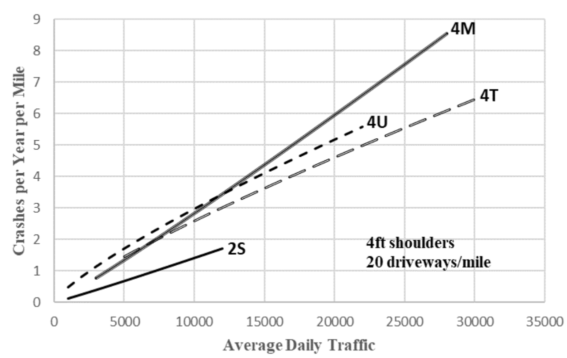
c) Moderate shoulders, low driveway density



d) Moderate shoulders, high driveway density



e) Narrow shoulders, low driveway density



f) Narrow shoulders, high driveway density

Figure C1. Graphical Form of the SPF for Total Crashes.

To obtain an estimate of crashes on 4U highways, the research team used the predictions provided by Equation 21. For computing the CMF value for converting 4U highways to 2S, the research team used two sources. First, based on the SPFs provided in Bonneson and Pratt, (2009), crash frequency is first estimated for the 2U and 4U highways. The ratio of the crash predictions is calculated to get the CMF for converting 4U to 2U highways. The CMF for converting two-lane to 2S highways is provided in Park et al. (2012). The resulting CMF value is 0.761. The same CMF value is also assumed for 2ST. Based on the default values provided in the SRPW program, the lowest and highest likely values of the CMF are 0.533 and 1.028, corresponding to 70 percent and 135 percent of the mean CMF value, respectively. The CMF for four-lane highways with a 4-ft median buffer is computed from Dixon et al. (2018) for OD crashes. It is converted into CMF for total crashes based on the proportion of OD crashes on 4U highways in Texas. The CMF for 4T highways is computed as 0.942. Based on the default values provided in the SRPW program, the lowest and highest likely values of the CMF are 0.659 and 1.271, corresponding to 70 percent and 135 percent of the mean CMF value, respectively.

Crash Cost

The research team derived crash severity distribution proportions from the sample considered in TxDOT Research Project 0-7035. These proportions are as follows:

- K: 3.50 percent.
- A: 7.64 percent.
- B: 14.65 percent.
- C: 12.74 percent.
- PDO: 61.46 percent.

To estimate the costs of crashes on 4U highways, the research team chose two sources. First, the research team used crash costs from TxDOT's Highway Safety Improvement Program guidelines Dixon and Avelar (2015). The crash value is $3.7 million for K (fatal) and A (incapacitating injury) crashes. The B (non-incapacitating injury) crash value is $520,000. Second, the National Safety Council values of $155,000 and $51,000 for C (possible injury) and O (no injury) respectively are used.

The research team used the default limiting benefit-cost ratio of 2.5 that was suggested by Zegeer (2013). This ratio represents the minimum benefit-cost ratio that typical agencies will deem adequate to justify implementing a proposed safety treatment.

Treatment Cost

The research team estimated the existing mileage based on the sample considered in TxDOT Research Project 0-7035 where the treatments are applicable. It is estimated that 4U highways can be restriped to 2S, 2ST, and four-lane width with a 4-ft median buffer configurations on as much as 53 percent, 18 percent, and 12 percent of the rural highway mileage in Texas, respectively. These restriped configurations are chosen based on the 0-7035 guidance, particularly relating to the consideration of traffic volumes and delay; however, the value of reduced delay is not explicitly computed. The research team further assumed a service life of 10 years and treatment cost of $23,000, $29,000, and $32,000 for 2S, 2ST, and four-lane width with a 4-ft median buffer configurations, respectively, based on costs provided by TxDOT for typical pavement marking replacement projects and an additional source for the cost of installing (or moving) shoulder rumble strips (MDT, 2020). The research team used an annual maintenance cost of $0 for analysis based on the assumption that TxDOT would provide the same amount of periodic maintenance and monitoring for the reconfigured sites as in the existing condition (or the do-nothing alternative).

RESULTS

The research team conducted the VOR analysis using the SRPW program and obtained an annual VOR estimate of $283,738. This value represents the benefit that can be obtained if (a) the results of the research project are used to analyze all 1,783 mi of 4U highways that were identified in the TRM database query, and (b) the treatments are installed at all sites found to be deserving of treatment. A site is considered deserving of treatment if the benefit-cost ratio of the treatment is found to be greater than or equal to the limiting benefit-cost ratio of 2.5.

A summary of the VOR calculations is shown in Figure C2. The payback period for Research Project 0-7035 is found to be 0.91 years, and the CBR is found to be 6.10.

The findings shown in Figure C2 are limited as follows:

- The benefits in the VOR calculations included only those incurred by TxDOT. In reality, other agencies (e.g., local and county agencies within Texas, other state departments of transportation) will be able to implement and benefit from the published findings from the project.
- The estimated benefits included only crash reduction, which will occur when the safety prediction model is applied to evaluate the rural four-lane highways in TxDOT's jurisdiction. TxDOT will likely incur additional benefits that are more difficult to quantify. These benefits may include reduced delays and reduced tort liability because TxDOT will be able to use the guidance to defend design decisions related to rural highways.
- The VOR analysis focused on rural four-lane highways. In reality, urban roadways may also realize similar benefits from the application of these project results. The scope of this project was rural highways, so the research team did not include urban highways in the VOR analysis. The estimated VOR, NPV, and CBR would increase if these sites were included in the analysis.

Project #	0-7035		
Project Name:	Examine Trade-Offs Between Center Separation and Shoulder Width Allotment for a Given Roadway Width		
Agency:	TTI	**Project Budget**	$ 258,085
Project Duration (Yrs)	2.08	**Exp. Value (per Yr)**	$ 283,738
Expected Value Duration (Yrs)	10	**Discount Rate**	3%

Economic Value

Total Savings:	$ 2,011,818.00	**Net Present Value (NPV):**	$ 1,574,490
Payback Period (Yrs):	0.91	**Cost Benefit Ratio (CBR):**	6.10

Years	Expected Value
0	-$9,096
1	-$129,970
2	-$119,019
3	$201,997
4	$295,415
5	$295,415
6	$295,415
7	$295,415
8	$295,415
9	$295,415
10	$295,415

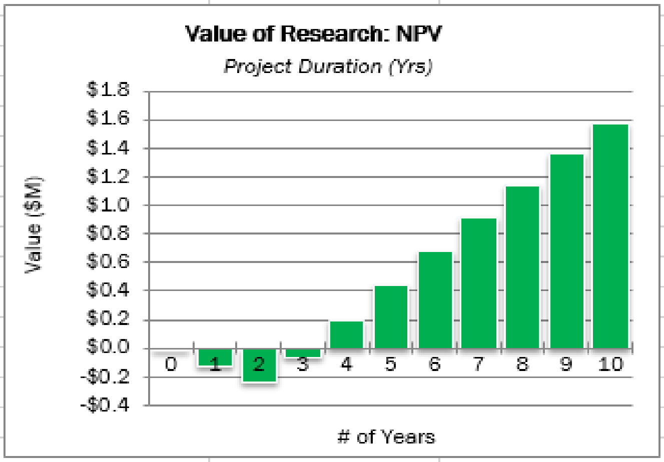


Figure C2. VOR Analysis Results.