Rural Intersections within Passing Lanes

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16. Abstract

Super 2 highways are used across Texas to provide passing opportunities and increase capacity on rural two-lane roads, but passing lanes are a treatment for through vehicles and do not address the needs of turning vehicles. This project investigated the operational and safety benefits of turning treatments on two-lane and Super 2 corridors, as well as best practices for designing these treatments. In this project, the research team used existing conditions on two-lane and Super 2 corridors with and without turning treatments for traffic simulation models to evaluate the effectiveness of such treatments on operations and safety. Research findings revealed that 40 percent of through drivers will move onto the shoulder to pass a left-turning vehicle, and they will generally do so at a distance of 210 ft or less. When a passing lane ends within 1500 ft of an intersection (or within 2640 ft for higher-volume conditions), intersection delay is higher than if the passing lane is not present; merging the passing and through lanes should be avoided within those distances. A statistical evaluation was conducted on intersection and intersection-related crashes at 283 intersections located within Super 2 corridors. These rural intersections all had stop control on the minor legs and included both three-leg and four-leg intersections. The findings from the evaluation and from the literature illustrate that the presence of either a left-turn lane or a passing lane is associated with fewer crashes at the intersection. Researchers used the research results to update guidelines that the Texas Department of Transportation can use to implement turning treatments on sections in or near passing lanes. Guidelines address content related to geometric design, access management, and traffic control devices.

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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 presented herein. The contents do not necessarily reflect the official view or policies of FHWA or TxDOT. This report does not constitute a standard, specification, or regulation.

This report is not intended for construction, bidding, or permit purposes. The engineer in charge of the project was Kay Fitzpatrick, P.E. #86762.

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CHAPTER 1: INTRODUCTION

BACKGROUND

The Super 2 concept is being used across the state to provide passing opportunities and increase capacity on rural two-lane roads, but passing lanes are a treatment for through vehicles and do not address the needs of turning vehicles. Current guidance in the Texas Department of Transportation (TxDOT) *Roadway Design Manual* (RDM) (*1*) provides the following discussion:

> When evaluating the termination of a passing lane at an intersection, consideration should be given to traffic operations turning and weaving movements, and intersection geometrics. If closure of the passing lane at the intersection would result in significant operational lane weaving, then consideration should be given to extending the passing lane beyond the intersection.

In recent years, a number of rural two-lane highway corridors have experienced high volumes in conjunction with high numbers of turning movements at access points (e.g., corridors in energy exploration areas where the number of vehicles, especially trucks, has increased greatly and driveways and intersections are frequent and are serving much larger volumes than originally intended). In these corridors, providing periodic passing lanes could be useful for allowing faster through traffic to pass slower right-turning traffic, but passing lanes could also provide unintended consequences for traffic turning left from a passing lane, and the increased number of access points makes it difficult to avoid them in determining passing lane placement.

A concept has been discussed on previous TxDOT projects as an alternative to the Super 2 cross-section. This alternative, colloquially named “Super 3,” provides turn lanes at key locations in addition to passing lanes on rural two-lane highways to better separate slower turning traffic from high-speed through traffic. This type of cross-section would help improve operations and safety on high-volume two-lane corridors that have frequent access points with many turning movements, but additional research is needed to determine the specific characteristics of this alternative that would produce the most favorable results.

PURPOSE OF THE PROJECT

This project investigated the operational and safety benefits of turning treatments on two-lane and Super 2 corridors, as well as best practices for designing these treatments. In this project, the research team used existing conditions on two-lane and Super 2 corridors with and without turning treatments to develop traffic simulation models for evaluating the effectiveness of such treatments on operations and safety. Researchers used the evaluation results to update guidelines that TxDOT can use to implement turning treatments on sections in or near passing lanes.

ORGANIZATION OF THIS REPORT

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

- Chapter 2 summarizes the findings from a review of current practices, operational and economic influences, and evaluation tools and techniques related to intersections on rural two-lane highways or within Super 2 corridors.
- Chapter 3 describes the research team’s activities in identifying study sites and collecting field data.
- Chapter 4 provides a description of the development of the traffic simulation model to conduct the operational analysis, as well as the results of that analysis.
- Chapter 5 describes the findings from the economic analysis.
- Chapter 6 presents the suggested guidelines developed using the findings from this research.
- Chapter 7 summarizes the researchers’ findings and conclusions, and provides recommendations for future action.

CHAPTER 2: REVIEW OF RELEVANT INFLUENCES ON PERFORMANCE

The chapter summarizes the review of existing guidance materials and findings from previous research. This review focused on features that affect performance on rural, high-speed, two-lane highways and access points found on those highways. For access points, the emphasis was on those with stop control on the minor approaches only (i.e., no all-way stop and no signalization). This review considered intersections with and without left-turn lanes or bypass lanes and how the added complexity of a left turn within a passing lane can be treated to minimize safety issues and improve operations.

DESIGN FEATURES

A variety of design features can influence the operational and safety performance of rural two-lane highways and the intersections and driveways that provide access to them. The following sections summarize findings from literature that describe those influences.

Influences on Roadway Operations

TxDOT Project 0-4064, conducted by Wooldridge et al. (*2*), developed initial guidelines for Super 2 corridors in Texas. As part of that project, the research team conducted simulation of Super 2 passing lanes of various lengths and spacing for two-way volumes between 400 and 1000 vph. The research indicated that passing lane lengths should be between 1.5 and 2.0 mi for average daily traffic (ADT) > 3550 on level terrain and > 3000 on rolling terrain. The distance between passing lanes for those volumes was 3.5 to 4.0 mi, with the guidance that passing lanes should be located to best fit existing terrain and field conditions and avoid major intersections.

The more recent TxDOT-sponsored Project 0-6135, by Brewer et al. (*3*, *4*), reviewed the 0-4064 recommendations in the context of higher ADTs and simulated additional Super 2 operational scenarios. The 0-6135 research found that passing lane lengths over 2 mi show less incremental benefit than higher frequency of lanes, particularly for ADT less than 10,000 vpd. Among the conclusions of the research was that providing additional passing lanes in a Super 2 corridor is preferable to adding length to a given passing lane. Much of the passing activity in a passing lane takes place in the first 1.0 to 1.5 mi, even if additional length is provided. More frequent passing lanes result in reduced delay compared to longer passing lanes. Regardless of volume, it was recommended that passing lanes be 2 mi or less in general, longer than 3 mi be used sparingly, and lengths of more than 4 mi be avoided. The traffic conditions simulated on Project 0-6135 compared scenarios with 10 percent truck volume and 20 percent truck volume. Results indicated that the truck percentage and terrain types (i.e., level or rolling) showed negligible impact on the measure of percent time spent following, but their impacts on the measure of average delay and number of passes were more pronounced. In most scenarios, the number of passes for 20 percent truck volume was higher than that for 10 percent trucks.

Most recently, the TxDOT-sponsored Project 0-6997 by Brewer et al. (*5*) investigated the operational and economic benefits of Super 2 corridors compared to traditional four-lane and two-lane cross-sections. Researchers analyzed the operational performance of a simulated 40-mi corridor with varying ADT; heavy vehicle volumes; length, number, and spacing of passing lanes; and access to identify operational benefits in key scenarios. Operational and benefit-cost inputs formed the basis of a model to quantify the relative economic benefits of Super 2 corridors. Operational analysis showed that, at volumes up to 17,000 vpd, Super 2 cross-sections provided higher average minimum speeds and lower delay than other options with fewer than four lanes, though the two-lane cross-section with left-turn lanes had similar performance as Super 2 for volumes of 15,000 vpd or higher. The cross-section with left-turn lanes was more stable at the highest volumes, suggesting that as volume and truck percentage increase, accommodating turning vehicles outside the through lane can improve operations more than an additional through lane or a passing lane. The economic analysis showed that Super 2 had the highest benefit-cost ratios (BCRs) in all ADT and truck percentage configurations, and had the best net present value in most cases. In both the economic and operational analyses, the findings agreed with previous research that adding shorter passing lanes to a Super 2 corridor is more beneficial than providing fewer but longer passing lanes.

Influences on Roadway Safety

Harwood et al. (*6*) in 2000 developed an algorithm for the Federal Highway Administration (FHWA) to predict the safety performance of a rural two-lane highway. The crash prediction algorithm consisted of base models and crash modification factors (CMFs) for both roadway segments and at-grade intersections on rural two-lane highways. The base models provided an estimate of the safety performance of a roadway or intersection using a safety performance function (SPF) for a set of assumed nominal or base conditions. The CMFs adjusted the base model predictions to account for the effects of changes from baseline values. When variations represent conditions that are associated with fewer crashes, the CMF is less than 1.0. A CMF of more than 1.0 indicates an increase in crashes.

The variables considered in the 2000 Harwood et al. study for the baseline value and the CMFs are summarized in Table 1. These variables formed the basis of the *Highway Safety Manual* (HSM) (*7*) rural two-lane highway chapter.

Table 1. CMF Variables in Safety Performance Model for Rural Two-Lane Highways.

	Variable	**Baseline Value**	**Adjustment in Crashes**
Roadway Segments	Lane Width	12 ft	Increase as width decreases
	Shoulder Width	6 ft	Increase as width decreases
	Shoulder Type	Paved	Increase with gravel, composite, turf
	Horizontal Curve	NA	Function that considers curve length (mi), radius (ft), and presence of spiral transition
	Superelevation	AASHTO *Green Book* values	Increase as superelevation decreases
	Grade	0 percent	Increase as absolute value of grade increases
	Driveway Density	5 driveways/mi	Increase as density increases
	Passing Lane	No passing lane (traditional two-lane cross-section)	CMF = 0.75 for passing lane in one direction, 0.65 for passing lane in both directions
	Two-Way Left-Turn Lanes	5 driveways/mi	Function that considers driveway-related crashes, left-turn crashes, and driveway density
	Roadside Design	3 on the Zegeer roadside hazard rating (1 to 7)	Function based on ratio of predicted crashes for actual rating to predicted crashes for rating of 3
Intersections	Number of Legs	NA	Separate base models for three-leg intersections and four-leg intersections
	Skew Angle	0 degrees (intersection angle = 90 degrees)	Function that increases as skew angle increases
	Traffic Control	Stop control on minor road only	CMF = 0.53 for warranted all-way stop control
	Left-Turn Lane on Major-Road Approaches	No left-turn lane	CMF = 0.78 for approach on three-leg intersection, 0.76 for one approach at four-leg intersection, 0.58 for both approaches at four-leg intersection
	Right-Turn Lane on Major-Road Approaches	No right-turn lane	CMF = 0.95 for one approach, 0.90 for both approaches
	Intersection Sight Distance	AASHTO *Green Book* value	CMF increases by 0.05 for each quadrant with limited sight distance

Source: Compiled from (*6*), pp. 29–49. AASHTO = American Association of State Highway and Transportation Officials; NA = not applicable.

In 2009 Bonneson and Pratt developed safety design guidelines and evaluation tools using Texas crash data for TxDOT designers to use in a variety of contexts. The *Roadway Safety Design Workbook* (*8*) contains information describing the relationship between various highway geometric design components and crash frequency. The workbook is intended to be used by engineers for the purpose of explicitly evaluating the potential safety trade-offs associated with various design alternatives. For rural highways, the researchers developed a procedure to quantify the safety associated with an existing rural highway facility or with a proposed design, where safety was defined as the expected frequency of injury (plus fatal) crashes. Similar to the national HSM, the crash frequency for a typical segment is computed from a base model, and that frequency is then adjusted using various CMFs to tailor the resulting estimate to a specific highway segment. The base model includes variables for traffic volume, segment length, and access point frequency. CMFs are used to account for factors found to have some correlation with crash frequency, typically of a more subtle nature than the main factors. The CMFs are multiplied by the base crash frequency to obtain an expected crash frequency for the subject highway segment. The base conditions considered in the Texas *Roadway Safety Design Workbook* that apply to two-lane rural highways are listed in Table 2 and result in a CMF of 1.0. For some characteristics, the crash modification was a defined multiplicative factor, while others involved a function that required additional calculation.

Table 2. Base Conditions for Rural Highway CMFs.

Characteristic	**Base Condition**
Rural Highways—Two or Four Lanes	
Horizontal curve radius	Tangent (no curve)
Grade	Flat (0% grade)
Lane width	12 ft
Outside shoulder width	8 ft
Rigid or semi-rigid barrier	Not present
Horizontal clearance	30 ft
Side slope	1V:4H
Rural Highways—Two Lanes	
Spiral transition curve	Not present
Shoulder rumble strips	Not present
Centerline rumble strip	Not present
Median type	Undivided
Superelevation	No deviation
Passing lane	Not present
Driveway density	5 driveways/mi

Source: Adapted from (*8*), Table 3-2.

A paper by Stokes et al. (*9*) in 2016 described a study in South Carolina to improve driveway safety and enhance access management practices. The intent of the study was to determine the potential safety and operational consequences of individual driveways and their specific characteristics so that informed decisions can be made when granting or denying a particular access point permit application. Statistical analysis identified several significant independent variables that influenced crash rates either positively or negatively, among which was changing driveway spacing. The researchers used results from the statistical analysis to develop CMFs for different driveway characteristics. The CMF associated with the changing driveway spacing is shown in the following equation.

$$CMF = e^{(-0.0004154\times(DS_a - DS_b)} \qquad (1)$$

Where:

DS_a = driveway spacing in feet after modification
DS_b = driveway spacing in feet before modification

Hamzeie et al. (*10*) conducted a study to provide a quantitative evaluation of how crash risk on multilane and two-lane highways varies with respect to access spacing in support of the development of a revised access management policy. Data were obtained for approximately 1,247 mi and 5,795 mi of segments across multilane and two-lane highways, respectively. Crash data were obtained for a 5-year period from 2012 to 2016, and a series of random effect negative binomial regression models were estimated for each facility to examine the association between crash frequency, access point spacing, and traffic volume. For both facility types, crashes were found to increase consistently as the average spacing of access points along road segments decreased. Crash rates were highest when consecutive accesses were within 150 ft of one another, and the frequency of crashes decreased substantively as spacing was increased to 300 ft and, particularly, 600 ft. With spacing beyond 600 ft, crash rates continued to decrease, though these improvements were less pronounced than at the lower range of values. These findings were generally consistent on multilane and two-lane highways. Reported crash reduction factors were described in the context of increasing density of access points and ranged from −17 percent to −292 percent (i.e., crashes increased as access density increased).

Influences on Intersection Operations

Left-Turn Lanes

Although many procedures are in use by organizations to determine the need for left-turn lanes, several are either very similar or identical. The oldest research found on evaluating the need for left-turn lanes at unsignalized intersections was that of M. Harmelink (*11*) in a paper published in 1967. His research provided the foundation for many subsequent left-turn guidelines, with more current guidelines being based on benefit-cost evaluations.

Koepke and Levinson in the 1992 National Cooperative Highway Research Program (NCHRP) Report 348 discussed several considerations and two methods for determining the need for left-turn lanes (*12*). They stated that in most cases, left-turn lanes should be provided where there are more than 12 left turns per peak hour. They also stated that "left-turn lanes should be provided when delay caused by left-turning vehicles blocking through vehicles would become a problem." They emphasized the fact that separate left-turn lanes not only increase intersection capacity but also increase vehicle safety.

A pair of NCHRP projects by Fitzpatrick et al. (*13*, *14*) developed guidance on left-turn lanes based on delay and other operational metrics. NCHRP Project 3-91 developed a process for selecting left-turn accommodations at unsignalized intersections and provided guidance on the design of those accommodations. The results of that effort produced a design guide (*13*) and a research report documenting the activities and methodologies from the project (*15*). NCHRP Project 3-102 (*14*) extended that work, reviewing existing literature and ongoing research projects and identifying issues meriting further study to validate, enhance, and expand then-current AASHTO *Green Book* guidance. Field studies were also conducted to assess the operation of double left-turn lanes and deceleration lanes. The research team then developed practical guidance for designers on auxiliary lanes, including recommendations for improving the AASHTO *Green Book*. Warrants associated with that guidance are described in the section on "Influences on Intersection Safety" later in this chapter.

Right-Turn Lanes

Warren et al. (*16*) investigated the delay effects of right turns on through traffic. They recorded speed-and-distance profiles of vehicles decelerating in through lanes before turning right into a driveway, with study sites consisting of multilane urban arterial roadways with speed limits of 35, 40, or 45 mph. In developing the deceleration models based on their data, the researchers found three separate deceleration areas, described through piecewise regression models. The upstream area was best described by a linear function, the middle interval by a quadratic curve, and the area closest to the driveway by a cubic function, the combination of which describes a process of increasing deceleration as the vehicle approaches the access point. They also found the data fit into three speed groups by the average speed of through vehicles: 38 mph, 42 mph, and 46 mph. Distance boundaries for the three deceleration areas for each speed group can be found in Table 3. Each combination of speed and distance range had a unique equation to predict speed for a given distance upstream of the driveway. The researchers noted that there was only one site with observed speeds in the 46-mph speed group.

Table 3. Distance Ranges for Models Predicting Speeds during Deceleration.

Speed Group (mph)	First Distance Range (ft)	Second Distance Range (ft)	Third Distance Range (ft)
38	−620 to −430	−430 to −210	−210 to 0
42	−620 to −450	−450 to −230	−230 to 0
46	−620 to −470	−470 to −250	−250 to 0

Source: Compiled from Warren et al. (*16*).

In addition to speed prediction, Warren et al. (*16*) also calculated total delay to through vehicles caused by right-turning vehicles by determining the added time needed for a right-turning vehicle to complete the turn compared to traveling through at constant speed. Those delay values are shown in Table 4.

Table 4. Total Delay to Through Vehicle from Right-Turn Maneuver.

Speed Range (mph)	Delay Before Right Turn (s)	Right-Turn Speed (mph)	Desired Through Speed (mph)	Time to Accelerate (s)	Time if Unimpeded (s)	Difference (s)	Total Delay (sum of “Before” and “Difference”) (s)
38	3.29	8.6	37.7	11.86	7.28	4.57	7.86
42	2.81	10.2	41.7	12.85	7.99	4.86	7.67
46	2.66	13.7	46.0	13.18	8.55	4.64	7.30

Source: Adapted from Warren et al. (*16*), Table 5, p. 119.

Based on the predictive equations, Warren et al. (*16*) calculated deceleration lane lengths for right turns using the threshold of a 10-mph difference between through and turning vehicles. Because they found that standard deviations of turning vehicle speeds were largely within 4 mph, they adjusted their speed difference to a 10-mph difference between the average speed of through vehicles and the average minus 4 mph of turning vehicles. This resulted in deceleration lane lengths of 213 ft for 38 mph, 232 ft for 42 mph, and 237 ft for 46 mph, not inclusive of storage.

Gemar et al. (*17*) developed guidelines for the use of right-turn slip lanes. The rural design guidance emphasized facilitating mobility through the slip lane, compared to accommodating pedestrian activity at urban intersections. Accordingly, the design promotes larger sweeping turns, the use of acceleration lanes, unpaved channelizing islands, and a flatter angle of entry into the cross street. The key considerations for the design of a rural right-turn slip lane included angle of entry, radius and lane width, channelization, acceleration lane, deceleration lane, drainage, lighting, pole placement (e.g., signal poles, utility poles), signage, and pedestrian/bicycle facilities (e.g., crosswalk placement and orientation, bicycle lane, and other considerations not typically included at a rural intersection).

The review of previous studies in NCHRP 3-102 also included right-turn installation guidelines. Hadi and Thakkar (*18*) investigated the use of through and right-turn movement volumes and speeds as criteria for installing right-turn deceleration lanes. To evaluate the need for right-turn lanes based on these criteria, researchers used as a surrogate the percentage of through vehicles behind right-turning vehicles in the outside (right) lane that performed evasive maneuvers because of the presence of right-turning vehicles. However, this measure could not be estimated from traffic simulation models if these models were used to evaluate the need for right-turn deceleration lanes. They determined that the speed differential between through vehicles affected by right-turning vehicles and those not affected by these turns could be used to determine the need for right-turn deceleration lanes at unsignalized intersections; this was accomplished by using speed differential as a surrogate to safety and related to crash involvement. To determine the total speed differential caused by right-turning vehicles in the outside lane, two variables were needed: the number of through vehicles in the outside lane affected by right-turning vehicles and the average drop in speed of affected vehicles. The research used these variables to determine critical right-turn volumes that created a speed differential sufficient to necessitate installation of a deceleration lane based on a benefit-cost threshold.

Potts et al. (*19*) used an economic analysis procedure to identify where installation of right-turn lanes at unsignalized intersections and major driveways would be cost effective. The researchers discussed results with respect to right-turn deceleration lanes. They conducted a traffic simulation study of motor vehicles and pedestrians at right-turn lanes to determine their operational effects. The researchers determined that right-turn maneuvers from a two-lane arterial at an unsignalized intersection or driveway could delay through traffic by 0 to 6 seconds per through vehicle where no right-turn lane was present. Delays to through traffic due to right turns in the same situation on a four-lane arterial were substantially lower, in the range of 0 to 1 second per through vehicle. They concluded that pedestrians at unsignalized intersections or driveways could have a substantial impact on delay to through vehicles as right-turning vehicles slow to yield to pedestrians, but provision of a right-turn lane could reduce pedestrian-related delays to through traffic by as much as 6 seconds per through vehicle, depending on pedestrian volume.

The procedure from NCHRP Project 3-72 (*19*) indicated combinations of through volumes and right-turn volumes for which provision of a right-turn lane would be recommended. Researchers stated that their economic analysis procedure could be applied by highway agencies using site-specific values for ADTs, turning volumes, crash frequency, and construction cost for any specific location (or group of similar locations) of interest. The procedure was used to develop plots that indicated combinations of through volumes and right-turn volumes for which provision of a right-turn lane would be recommended.

Influences on Intersection Safety

Fitzpatrick et al., in a 2002 study of treatments to reduce crashes on rural two-lane roads (*20*), described left-turn bays as generally the key auxiliary lane at an intersection. They create the opportunity to separate and avoid speed differences between turning vehicles and through vehicles. They also decrease the delay that can be experienced by through vehicles behind a turning vehicle. By increasing the operational efficiency of the intersection, the capacity and safety are also increased. In addition, left-turn lanes can provide to the opposing traffic increased visibility of the turning vehicle.

A 2002 study by Harwood et al. (*21*) found that the addition of a left-turn lane can result in reductions of crashes ranging from 7 to 48 percent. The researchers gathered geometric design, traffic control, traffic volume, and traffic crash data for 280 improved sites under the jurisdiction of the participating states, as well as 300 similar intersections that were not improved during the study period. The types of improvement projects evaluated included installation of added left-turn lanes, installation of added right-turn lanes, installation of added left- and right-turn lanes as part of the same project, and extension of the length of existing left- or right-turn lanes.

Persaud et al. (*22*) evaluated the effectiveness of center two-way left-turn lanes (TWLTLs) on two-lane roads to reduce the frequency of head-on or rear-end crashes involving a turning vehicle. They obtained geometric, traffic, and crash data for 78 sites (21.3 mi) in North Carolina, 10 sites (6.0 mi) in Illinois, 31 sites (6.8 mi) in California, and 25 sites (13.2 mi) in Arkansas. An empirical Bayes analysis revealed a statistically significant reduction in total and rear-end crashes where TWLTLs were added in each of the four states; an aggregate analysis for each state showed reductions in rear-end crashes ranged from 21.7 to 49.9 percent, with a combined effect over all four states of 38.7 percent. A disaggregate analysis indicated that rural installations of TWLTLs were more effective than urban installations at reducing total crashes in each state. Researchers reported that there were few head-on crashes reported in their sample of data, so they did not have conclusive findings for those crashes compared to rear-end crashes. The findings from the available data, however, led the researchers to conclude that lower-cost installations of TWLTLs can be a cost-effective treatment for two-lane rural locations, especially those with a high frequency of rear-end collisions involving a lead vehicle desiring to make a turn. Based on the conservative lower 95 percent confidence limit of the safety effect estimates, they concluded that reductions of at least 29 percent, 19 percent, and 36 percent can be expected in total, injury, and rear-end crashes, respectively, at rural installations.

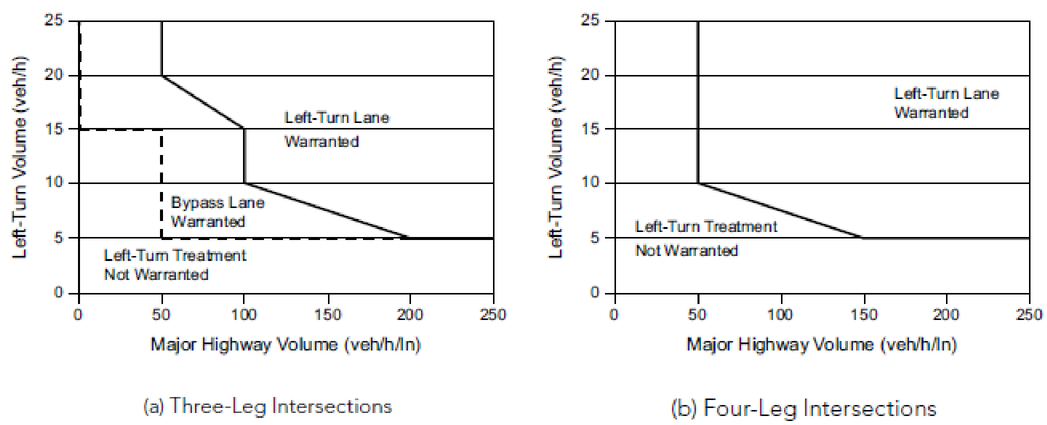
A pair of successive NCHRP projects, NCHRP Project 3-91 (*13*) and NCHRP Project 3-102 (*14*), developed warrants for installing left-turn lanes at unsignalized intersections. Using a benefit-cost approach that quantified the effects of delay, crashes, and construction costs, the researchers from those projects used the results to generate a series of graphs to display warrant criteria for a variety of scenarios (*23*). Those guidelines have since been included in the 2018 AASHTO *Green Book* (*24*); the guidelines pertaining to rural two-lane roads are shown in Table 5 and Figure 1.

Fitzpatrick et al. (*20*) in 2002 also described shoulder bypass lanes as a low-cost alternative to intersection turn lanes for reducing delays to through vehicles caused by left-turning vehicles. Where a side road intersects a two-lane highway at a three-leg or T-intersection, a portion of the paved shoulder opposite the intersection may be marked as a lane for through traffic to bypass vehicles making a left turn. Where an adequate paved shoulder is already available, installation of a shoulder bypass lane may be as simple as remarking the highway edge line. Thus, provision of a shoulder bypass lane is often much less expensive than construction of a left-turn lane.

Table 5. Suggested Left-Turn Treatment Guidelines for Intersections on Two-Lane Highways in Rural Areas.

Left-Turn Lane Peak-Hour Volume (veh/h)	Three-Leg Intersection, Major-Road Two-Lane Highway Peak-Hour Volume (veh/h/ln) that Warrants a Bypass Lane	Three-Leg Intersection, Major-Road Two-Lane Highway Peak-Hour Volume (veh/h/ln) that Warrants a Left-Turn Lane	Four-Leg Intersection, Major-Road Two-Lane Highway Peak-Hour Volume (veh/h/ln) that Warrants a Left-Turn Lane
5	50	200	150
10	50	100	50
15	< 50	100	50
20	< 50	50	< 50
25	< 50	50	< 50
30	< 50	50	< 50
35	< 50	50	< 50
40	< 50	50	< 50
45	< 50	50	< 50
50 or more	< 50	50	< 50

Note: These guidelines apply where the major road is uncontrolled and the minor-road approaches are stop- or yield-controlled. Both the left-turn peak-hour volume and the major-road volume warrants should be met as shown in Figure 1.
Source: AASHTO *Green Book* (*24*), Table 9-25, p. 9-107.



Source: AASHTO *Green Book* (*24*), Figure 9-36, p. 9-107.

Figure 1. Suggested Left-Turn Treatment Warrants for Intersections on Two-Lane Highways in Rural Areas.

Dissanayake and Shams conducted multiple investigations (*25*, *26*, *27*) into the effects of shoulder bypass lanes at unsignalized rural intersections in Kansas. They found that expected and observed crash frequency and crash rate typically declined at three-leg intersections with bypass lanes, compared to those without bypass lanes, both for crashes labeled as intersection-related and for crashes that occurred within 300 ft of an intersection, although those reductions were not always significant at a 90 percent confidence interval (*26*). They also found similar decreases for crashes within 300 ft of four-leg intersections, and those reductions were significant at a 95 percent confidence interval (*26*), though the use of bypass lanes on four-leg intersections is not allowed or recommended by all guidelines. For example, Chapter 9 of the 2018 AASHTO *Green Book* (*24*) provides guidance for bypass lanes only for three-leg intersections.

Bonneson and Pratt (*8*) in 2009 produced models and CMFs for rural intersections similar to those for rural highways. For some characteristics, the modification was a defined multiplicative factor, while others involved a function that required additional calculation.

Harwood and Bauer (*28*) in 2015 conducted a safety study on the effects of stopping sight distance (SSD) near crest vertical curves. They analyzed crash data from 452 crest vertical curves on rural two-lane highways in Washington and found statistically significant differences in crash frequency between sites with SSD above existing design criteria and sites below criteria. Those differences were explained, however, with the inclusion of a factor to account for the presence of horizontal curves, intersections, or driveways that were hidden from an approaching driver's view due to the sight obstruction caused by the vertical curve. Specifically, crash rates for crest vertical curves with SSD lower than AASHTO criteria were 27 percent higher than curves that exceeded AASHTO SSD criteria. Furthermore, a crest vertical curve with SSD above AASHTO criteria along with a horizontal curve, intersection, or driveway had a crash risk 41 percent higher than a similar location with no horizontal curves or access points; the corresponding crash risk was 71 percent greater if the curve did not meet AASHTO SSD criteria, and 87 percent higher if the horizontal curve or access point was not visible to opposing drivers prior to the crest of the vertical curve. Thus, the presence of a hidden feature was found to be highly statistically significant in conjunction with SSD.

Park et al. (*29*) in 2012 reviewed crash data on existing Super 2 highways in Texas. The study used crashes with fatal and injury severities, the KABC crashes on the KABCO scale. Empirical Bayes analysis of crash data indicated that the installation of passing lanes led to a statistically significant KABC crash reduction of 35 percent for segment-only crashes and 42 percent for segment and intersection KABC crashes on the study corridors. These findings were statistically significant above the 95 percent confidence level, which indicated that the reduction in crashes could be attributed to the Super 2 treatments with a high degree of certainty. The results were consistent with findings of previous safety-related studies of Super 2 corridors, which showed improvements in safety with installation of passing lanes, even at traffic volumes higher than those considered under previous guidance in Texas.

Biancardo et al. (*30*) in 2019 reviewed data from 240 crashes on 35 unsignalized three-leg intersections connecting rural two-lane, two-way roads and minor roads with a stop control on the approaches. They used an 8-year period of study: 5 years to calibrate a new SPF, and the remaining 3 years, not included in the first dataset, to validate the results. The authors concluded that a skew angle of greater than 10 degrees was related to higher crash frequency, and this effect was compounded if the intersection was located on a circular curve instead of a tangent segment. They also concluded that the presence of left- and right-turn lanes on the major road at these intersections provided a benefit greater than that predicted in the HSM (*7*), recommending the calculation of a local CMF for such intersections.

Stapleton et al. (*31*) in 2019 reported on a study to develop SPFs for rural, low-volume, minor-road stop-controlled (three-leg and four-leg) intersections in Michigan. They reviewed traffic crashes, volumes, roadway classification, geometry, cross-sectional features, and other site characteristics from 2,023 intersections statewide for the period 2011–2015. Their approach developed separate models for fatal/injury crashes, property damage crashes, and select target crash types. Their analysis found that skew angles of greater than 5 degrees led to significantly greater crash occurrence for both three-leg and four-leg intersections, while greater than two driveways near the intersection led to significantly greater angle crashes at four-leg intersections. They found other factors had little impact on crash occurrence. Comparing their models with the HSM base models showed that the HSM models predicted more crashes on four-leg intersections and on three-leg intersections with volumes between 1,200 and 2,000 vpd.

Influences from Driveway Design

Frawley et al. (*32*) reviewed then-existing guidelines on driveway design to accommodate turning maneuvers of heavy vehicles. They developed recommendations to best incorporate these required turning maneuvers in concert with other critical driveway factors. The following were among those listed as critical factors influencing design:

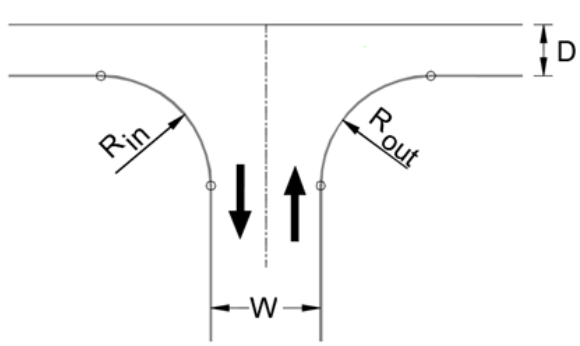
- Vertical Profile: For rural applications, the driveway slope can match the cross-slope of the highway shoulder and then have a grade change to match existing terrain. The use of larger grade changes, particularly when sharp sag or crest curves result from the transition, can introduce a risk of vehicles dragging.
- Driveway Entry/Exit Angle: The angle of a two-way driveway between the driveway centerline and the centerline for the adjacent road is typically designed to be 90 degrees whenever possible; however, a 15-degree variation (from 75 to 105 degrees) may be considered for commercial driveway locations. All turning maneuvers that do not adhere to the 90-degree configuration should be designed on a case-by-case basis by examining the specific design vehicle turn paths in order to confirm that the vehicle can navigate the driveway.
- Horizontal Geometry: The horizontal design of a commercial or industrial driveway where heavy vehicles are present may consist of three components: a dedicated turn lane, the throat/lane width, and a radius return/curb return. Locations with a dedicated right-turn lane on the approach to the driveway will enable the driver of a heavy vehicle to shift the truck from the adjacent through lane into the turn lane and then execute the turn maneuver. By providing this refuge space, there is less opportunity for a rear-end crash on the adjacent travel way and more opportunity for the truck drivers to take their time to properly navigate the right turn into the driveway. The provision of a dedicated right-turn lane is important if simultaneous truck movements cannot be achieved. A shoulder with a pavement section designed to accommodate heavy vehicles can provide additional navigational space for turning trucks.

After their review of existing state-of-the-practice techniques and engineering and probability assessments of configurations, Frawley et al. (*32*) recommended two driveway configurations, a traditional driveway format and a hybrid driveway design, as summarized in Figure 2. The hybrid designs apply to locations that must accommodate a few heavy vehicles but where primary traffic consists of passenger cars and single-unit (SU) trucks (SU vehicles). They specifically considered a WB-67 heavy vehicle for rural applications. Their tips to avoid poor driveway horizontal or vertical placement included the following:

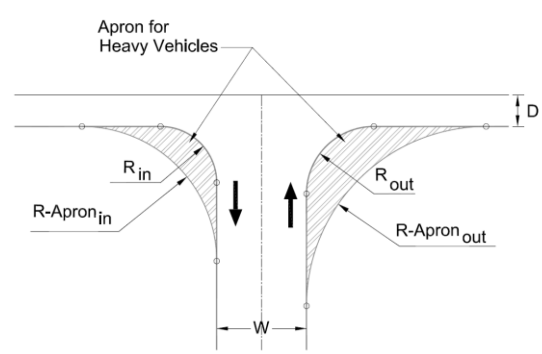
- If a driveway must be located on the outside of a horizontal curve, it should be positioned as close to the point of intersection for the horizontal curve as practical. This placement will enable visibility from both directions of travel.
- Driveways positioned on the outside of horizontal curves at locations where the main road is superelevated may require more than one vertical grade break along the driveway so that no more than a 3 percent grade change occurs at any one time. Steeper vertical grade breaks may cause low-boy trailers to drag.
- Placing a driveway near the point of curvature or point of tangency for sharp horizontal curve locations should be avoided because this placement creates visibility issues. At locations where this placement cannot be avoided, any obstacles or vegetation that disrupts the line of sight should be removed or relocated.
- Driveways positioned on the inside of a horizontal curve should have similar sight distance considerations as those on the outside of the curve.
- Constructing a driveway at a vertical curve sag location should be avoided. This placement can create drainage issues that may result in standing water during inclement weather conditions.
- In rural areas, the initial vertical grade of a driveway at the intersection with the main road should match the shoulder grade. Any grade breaks should occur away from the edge of the shoulder and should not exceed 3 percent. In some cases, more than one grade break may be needed. At these locations, designers should consider using smoothing vertical curves with minimum lengths of 20 ft.
- Where practical, the intersection of a two-way driveway and the main road should be 90 degrees. If terrain or geometry will not permit this configuration, the angle should be within a 75- to 105-degree range.

Brewer et al. (*33*) in 2020 studied a highway corridor with high volumes, high truck percentages, high speeds, high turning volumes, and high demand for access, with an emphasis on application of access management principles. The study team reviewed the applicable guidelines and policies for driveway spacing in Texas, as well as relevant findings from other guidelines and research, to develop a set of recommended driveway spacing values for cars and for trucks on high-speed rural roads. Based on a design philosophy to minimize the interaction of vehicles accelerating from a driveway with those vehicles decelerating to a downstream driveway, the recommended spacing between driveways was set equal to the sum of vehicles' acceleration and deceleration lengths along with the necessary taper lengths. The study team then developed spacing recommendations corresponding to two different vehicle types, passenger cars and heavy trucks. Recognizing that rural highways can have different functions, the study team created two roadway tiers (see Table 6) that reflect when the highway has the function of local access, long haul, or connectivity. Primary routes provide regional connectivity, serve as haul roads for energy-sector activities, and provide local access. Secondary routes predominantly serve haul and local access needs.

Scenario	D (ft)	W (ft)	R_{in} (ft)	R_{out} (ft)	R-Apron$_{in}$ (ft)	R-Apron$_{out}$ (ft)
Driveway with ≤ 4 SU vehicles per hour and no heavy vehicles, site has other driveways designed for heavy vehicles	14	28	25	25	NA	NA
Driveway with ≤ 2 heavy vehicles per hour, site has other driveways designed for heavy vehicles	14	30	30	30	NA	NA
Driveway with ≤ 2 heavy vehicles per hour, site does not have any other driveways, located in urban environment (Hybrid Design)	14	40	25	30	60	80
Driveway with ≤ 2 heavy vehicles per hour, site does not have any other driveways, located in rural environment (Hybrid Design)	12	40	25	35	70	95
Driveway with ≥ 3 heavy vehicles per hour, located in urban environment, no dedicated right-turn lane	14	40	60	80	NA	NA
Driveway with ≥ 3 heavy vehicles per hour, located in rural environment, no dedicated right-turn lane	12	40	70	95	NA	NA



Traditional Driveway Configuration



Hybrid Driveway Configuration

Note: NA = not applicable.

Source: Frawley et al. (*32*), Table 13, p. 40.

Figure 2. Recommended Driveway Configurations.

Table 6. Summary of Access Management Recommendations.

Classification	Function	Passing Lanes	Turn Lanes	Driveway Spacing Based on PC[a]	Driveway Spacing Based on Heavy Trucks[b]	Minimum Offset[c]	Hybrid Driveway Design[d]
Tier 1	Connectivity Haul Local Access	Ok	Yes	Minimum 1/2 mi	Minimum 1 mile	Minimum 1/4 mi	Yes
Tier 2	Haul Local Access	No	Yes	Minimum 1/3 mi	Minimum 3/4 mi	Minimum 1/8 mi	Yes

[a] Minimum spacing between access on the same side of road for consideration of passenger cars.
[b] Minimum spacing between access on the same side of road for consideration of heavy trucks.
[c] Minimum offset between access on opposite sides of road if access points are not aligned directly across from each other to create a four-leg intersection, which is preferred.
[d] Hybrid driveway design shown in Figure 2 from Frawley et al (*32*).
Note: Spacing and offset distances greater than the minimums shown are encouraged.
Source: Adapted from Brewer et al. (*33*), Table 10, p. 13.

SIGNS AND MARKINGS

Influences from Signs and Markings—Previous Research

Boonsiripant et al. (*34*) in 2010 calculated acceleration and deceleration rates and zone lengths for vehicles approaching and leaving intersections under likely uncongested conditions. The intersections studied were controlled by either stop signs or traffic signals. Data were obtained through speed profiles recorded by global positioning system instrumentation. Using a threshold of 1 mph/s (1.47 ft/s^2) to determine the beginning of the maneuver, the researchers calculated average acceleration and deceleration zones. They compared the observed deceleration rates with recommendations from the AASHTO *Green Book* and found that the threshold method produced acceleration rates significantly lower than AASHTO values (resulting in longer acceleration zones), but rates calculated using the achieved constant rate of observed vehicles compared much more closely to AASHTO values. Similar trends were found for deceleration rates.

As part of their study on treatments to reduce crashes on rural two-lane roads in Texas, Fitzpatrick et al. (*20*) listed several traffic control device treatments for use at intersections, including advance warning signs and markings, intersection control beacons, and illumination.

For advance warning signs, the researchers noted that the combination of infrequent intersections and high speeds along a rural two-lane highway creates a situation where conflicts are unexpected (*20*). Providing advance signs and markings informs drivers of upcoming conditions. Adding beacons to those signs can provide additional benefit, either through beacons that flash continuously or through beacons that are activated as traffic approaches.

The objective of a fixed lighting system is to supplement the headlights of automobiles and to make distant, complex, or low-contrast objects more visible to motorists and pedestrians. Because of costs, continuous lighting systems are not generally employed in rural areas; however, illumination can improve safety at isolated, rural at-grade intersections (*20*). Wortman et al. (*35*) recommended that lighting should be considered at a rural intersection if the average number of nighttime crashes per year exceeds the average number of day crashes per year divided by 3. A benefit-cost analysis (BCA) should then be performed to determine if the benefits of lighting the intersection exceed the cost of providing the lighting system (*35*).

Signs and Markings—Reference Documents

Several documents provide information on the signing and marking of intersections, including left-turn lanes. The *Texas Manual on Uniform Traffic Control Devices* (Texas MUTCD) (*36*) provides information for signing and marking of turn lanes; however, specific guidance for turn lanes at rural two-lane highway intersections is not identified. The guidance provided for lane transition taper length may be relevant, and the minimum lane transition taper length should be 100 ft in urban areas and 200 ft in rural areas (Section 3B.02, paragraph 15, also illustrated in Texas MUTCD Figure 3B-15).

With respect to intersection control beacons, the Texas MUTCD (*36*) states that intersection control beacons may be used at intersections where traffic or physical conditions do not justify conventional traffic control signals, but crash rates indicate the possibility of a special need (Section 4L.02, paragraph 08). Intersection control beacons are used in conjunction with stop signs at isolated intersections or intersections having sight distance obstructions. Research findings recommend that they not be used at Y-intersections, offset intersections, or intersections with more than four legs because the geometry of these intersections frequently does not provide an adequate line of sight from all intersection legs to a center-mounted beacon (*20*).

The TxDOT *Sign Guidelines and Applications* (*37*) does not include any discussion on signing for left-turn lanes. With respect to rural intersections, it references the TxDOT *Sign Crew Field Book* (*38*). For county road intersections with state highways, the TxDOT *Sign Guidelines* *and Applications* (*37*) document provides information about installing and maintaining stop and yield signs. The document does include information on intersections; however, none relates to left-turn lanes.

The TxDOT *Sign Crew Field Book* (*38*) provides examples of guide sign arrangement patterns such as the use of left arrows below the route sign.

In addition to the advice provided by Wortman et al. (*35*), other sources providing warrants for roadway lighting include AASHTO's *Roadway Lighting Design Guide* (*39*) and the FHWA *Lighting Handbook* (*40*).

DRIVING ON THE SHOULDER AT RURAL INTERSECTIONS

Paved shoulders in Texas are used by drivers for refuge and for passing slower-moving vehicles. A 1981 study (*41*) investigated the usage of shoulders using field measurements. The researchers conducted drive-throughs for 18 sites for a 6-hour period at each site and recorded vehicle behavior using onboard radar and other equipment. For each vehicle encountered, its type, speed, lane position (travel or shoulder), and longitudinal position were recorded. Data were collected for over 21,000 vehicles. For two-lane highways with paved shoulders, the shoulder was used by between 5 and 13 percent of the vehicles. A practice in use at that time was to repave two-lane highways with wide shoulders to four-lane undivided highways (sometimes called a poor-boy design). The authors also studied these facilities and considered vehicles driving on the outside lane as using the shoulder. They found between 65 and 75 percent of the vehicles drove on the shoulder that had been converted to a lane. Furthermore, to improve the traffic flow in rural two-lane highways, they concluded that a paved shoulder should be warranted when the volume-to-capacity ratio of the updated two-lane highways reaches between 0.54 and 0.66 (*42*). However, previous studies did not provide details regarding the distance that drivers move onto the shoulder upstream of an intersection to avoid colliding with the left-turn vehicle.

GAP ACCEPTANCE AT RURAL INTERSECTIONS

Most of the rural two-lane highways have higher posted speed limits than urban highways. Thus, the gap acceptance and critical gap of urban and rural facilities differ significantly. While numerous studies have evaluated gap acceptance in urban areas, a limited number of studies are available for rural two-lane highways. According to the AASHTO *Green Book* (*24*), the overall critical gap for high-speed two-way roadways for passenger cars is 5.5 seconds, while that of trucks is 7.5 seconds. Moreover, the reported critical gap for left turns from major roadways in urban regions ranges between 4.21 and 6.44 seconds, compared to between 6.00 and 8.00 seconds for suburban areas (*43*). Similarly, older drivers (above 70 years) are more likely to reject a shorter gap than young drivers.

Among the challenges to establish critical gap for high-speed two-way roadways is the difficulty of attaining an adequate sample size (*44*). As a result, the equilibrium of probabilities method was developed to simplify the computation efforts (*45*, *46*). A review of available methodologies for critical gap estimation (*44*) revealed that the equilibrium of probabilities method requires fewer data points to produce comparable results from other methodologies. One previous study (*47*) developed an equation to calculate critical gaps based on a number of site characteristics. For rural two-lane highways, the critical gap was found to be between 5.85 and 6.21 seconds.

POTENTIAL VARIABLES FOR FIELD STUDY AND SIMULATION

Overall, the literature contains significant efforts to understand the operational characteristics of the added left-turn lane, passing lane, or both on rural two-lane highways. However, limited information is available on driver behavior when approaching a slowing or stopped vehicle turning left from a rural two-lane highway. Furthermore, relatively limited information is available on gap acceptance at rural intersections. Additionally, the traffic volume at which the addition of the left-turn lane, passing lane, or both improves traffic operations at the unsignalized T-intersection on rural two-lane highways has not been extensively explored.

Based on the findings from previous research, the following items were identified as potential variables to consider for use in the simulation and in-field analysis activities on this project:

- At intersection:
  - Volume.
    - Approach volume on major road.
    - Turning volume from major road to minor road.
    - Opposing volume on major road.

        - Crossroad volume.
        - Percent heavy trucks on major and on minor roads.
    - Intersection angle, including whether it is skewed.
    - Number of intersection legs.
    - Intersection sight distance for left turns from the major road (compare available to values calculated for posted speed limit or 85th percentile operating speed).
    - SSD to the minor road along the major road (compare available to values calculated for posted speed limit or 85th percentile operating speed).
    - Presence of turn lane.
        - Length (deceleration length and storage length).
        - Taper length.
    - Lane width.
    - Shoulder width and type (paved vs. not paved).
    - Distance to beginning/end of passing lane if intersection is located within passing lane.
    - Whether the passing lane is going in the same direction as the left-turning vehicle or the opposing direction.
    - Markings at the intersection (e.g., is it a true left-turn lane or is a bypass lane present?).
- On corridor.
    - Presence of passing lane.
    - Horizontal and vertical curvature at the intersection, especially whether the sight distance to the intersection is below SSD or intersection sight distance.
    - Access density within the corridor.
    - Presence of advance signing and marking.
    - Illumination at and/or in advance of the intersection.

CHAPTER 3: FIELD STUDY

This chapter presents the activities completed as part of the field study. The field study task began with the researchers identifying potential study sites along with developing the protocol for collecting the operations data. These study sites were then reviewed for their potential to provide insights into existing operations at rural intersections along with their usefulness in calibrating a traffic simulation model. Next, researchers completed data collection and reduction activities for each site. Findings from the speed and video data are presented together with the overall assessment of the field data. Researchers also conducted a crash analysis on a larger set of intersections to identify influences on performance; the final portion of this chapter documents activities and findings from that analysis.

IDENTIFICATION OF CORRIDORS

The research team used the list of known Super 2 corridors obtained from previous TxDOT projects (e.g., 0-6997, 0-6135, 0-6806, and 0-4064) along with other corridors provided by TxDOT for the corridor selection process.

A research team member visually reviewed these corridors using Google Earth, identifying intersections with cross streets that had either a street name or a number or intersections with driveways to a business that would generate several left turns in a day. Intersections with driveways to residential homes were not identified. Researchers initially identified over 300 intersections during this effort to further review for details of intersection characteristics.

CODING OF INTERSECTION CHARACTERISTICS

The research team developed an iterative process to identify six characteristics of intersection: intersection geometry, left-turn configuration, passing lane configuration, centerline marking type, right-turn lane configuration, and lane additions. Upon identification of an intersection in Google Earth, researchers coded the information into a text string. Each of these categories of intersection information and elements of the text string are discussed in greater detail in the remainder of this section.

Category I: Intersection Geometry

The start of the classification of the intersection was the number of legs. For site selection, intersections with three or four legs had the higher priority, and intersections with more than four legs or heavily skewed intersections were dropped from consideration. Beyond the number of legs, the research team also determined which direction of traffic could make a left turn. At four-leg intersections, the direction assigned to the intersection was that of the left turn with the assumed higher volume. The classification of the minor roads (e.g., farm-to-market (FM) road versus county road [CR], if legs had different classifications) was used to make the determination of the direction along with the judgment of the reviewer. The primary purpose of assigning a direction to the intersection was to ensure that information regarding lane configuration was accurately captured.

Category II: Left-Turn Configuration

Super 2 corridor intersections can have a variety of left-turn configurations. While left turns are generally permitted at both three- and four-leg intersections along the corridors, they are not always accompanied by dedicated left-turn lanes. Consequently, the configuration of the left-turn lane at the intersection is a unique characteristic that needed to be captured. The left-turn configurations identified during the manual review of Super 2 corridors included the following:

- None (no left-turn lane, coded as LTL: no). The simplest intersection, in terms of coding for the left-turn treatment, that occurs along Super 2 corridors has no left-turn lanes present.
- Left-turn lane in same direction as passing lane (coded as LTL: S). The direction associated with the left-turning vehicles (Category I) is critical. The word *Same* indicates that a left-turn lane is present at an intersection in the same direction as the primary left-turn movement. At three-leg intersections, this is obviously the only direction that left turns can be made. At four-leg intersections, this indicates that a left-turn lane is present at what was estimated to be the higher volume of the two left-turn options.

- Left-turn lane in the opposite direction of the passing lane (coded as LTL: O). *Opposite* is used to designate when one of the left-turn lanes present at an intersection runs in the opposite direction of primary left-turn movement at the intersection.
- Both approaches have a left-turn lane (coded as LTL: S, O). This type of left-turn lane condition is available only at four-leg intersections.
- Two-way left-turn lane (coded as LTL: T). The other potential left-turn configuration at rural intersections involves a TWLTL. TWLTL design allows for multiple intersections to be serviced by a turn lane without specific markings for each intersection.

Category III: Passing Lane Configuration

The research team also wanted to consider the proximity of the nearest passing lane and how its presence might affect intersection operations. To accomplish this, researchers identified the distance to merge (M) and diverge (D) points. Researchers scanned upstream (up) and downstream (down) from the intersection for 2 mi to identify where the nearest passing lane begins (diverge) and ends (merge). Distances were measured from the intersection to the merge point and to the diverge point for the passing lane.

Category IV: Centerline Marking Type

The centerline marking type conveys information about the intersection including if passing is allowed along the segment at the intersection location:

- Marking Types. When a passing zone is present in the same direction as the primary left-turn direction, the intersection is coded as CM: PZ-S (passing zone same). Conversely, when the intersection is located in a passing zone servicing the opposite direction of travel as the primary left-turn lane, the intersection is coded as CM: PZ-O (passing zone opposite). When passing is permitted in both directions of traffic, the intersection is coded as CM: PZ-B. Finally, when no passing is permitted, the intersection is coded as CM: DY (double yellow). In addition to these marking styles, a flush median may be present. This scenario generally occurs when a left-turn lane is present at a three-leg intersection and a flush median is on the approach opposite of the left-turn lane. This type of centerline marking is coded as CM: FM.

- Modifiers. At higher-volume intersections where no left-turn lanes are present, the centerline marking will generally break. In this case, a "-B" modifier is added to the centerline marking coding (e.g., CM: DY-B).

Category V: Right-Turn Configuration

Right-turn lanes are sometimes present at intersections along Super 2 corridors. Due to their infrequency, the right turn is only coded when present. For example, when a right-turn lane is present in the opposite direction of the primary left-turn movement (the typical geometric configuration), the intersection is coded as RTL: O. However, when no right-turn lane is present, the RTL code is omitted.

Category VI: Lane Additions

Another category of information that is only included when present is that of lane additions. Lane additions occur when an acceleration lane or some other auxiliary lane is present. Acceleration lanes are sometimes provided for right-turning vehicles from the minor street when entering the mainline due to the high speed of mainline traffic.

AVERAGE DAILY TRAFFIC

While the intersection may have the geometric design layout of interest, it also needs to have sufficient traffic volume. ArcMap and Excel were used to extract traffic volume data from the TxDOT Roadway Inventory file (RHINO). Volume for both the major (two-lane rural highway) and minor roads was sought. Volume was not always available within RHINO for the minor approaches, and in those cases, the research team considered the likelihood of turning volume based on the quantity and type of developments accessing the minor road.

SELECTING POTENTIAL FIELD STUDY SITES

The intersection characteristic codes along with ADT were used to help identify sites of interest. Of highest priority were sites where the left-turning vehicle could be stopped in the passing lane (see example in Figure 3). Researchers also desired sites with a left-turn lane (either left-turn bay or TWLTL) to provide a comparison between operations with and without the turn lane. While existing travel volumes at the intersection had a large influence on site identification, COVID-19 travel restrictions were also influential since sites within a reasonable travel distance from Texas A&M Transportation Institute (TTI) headquarters were given greater consideration.

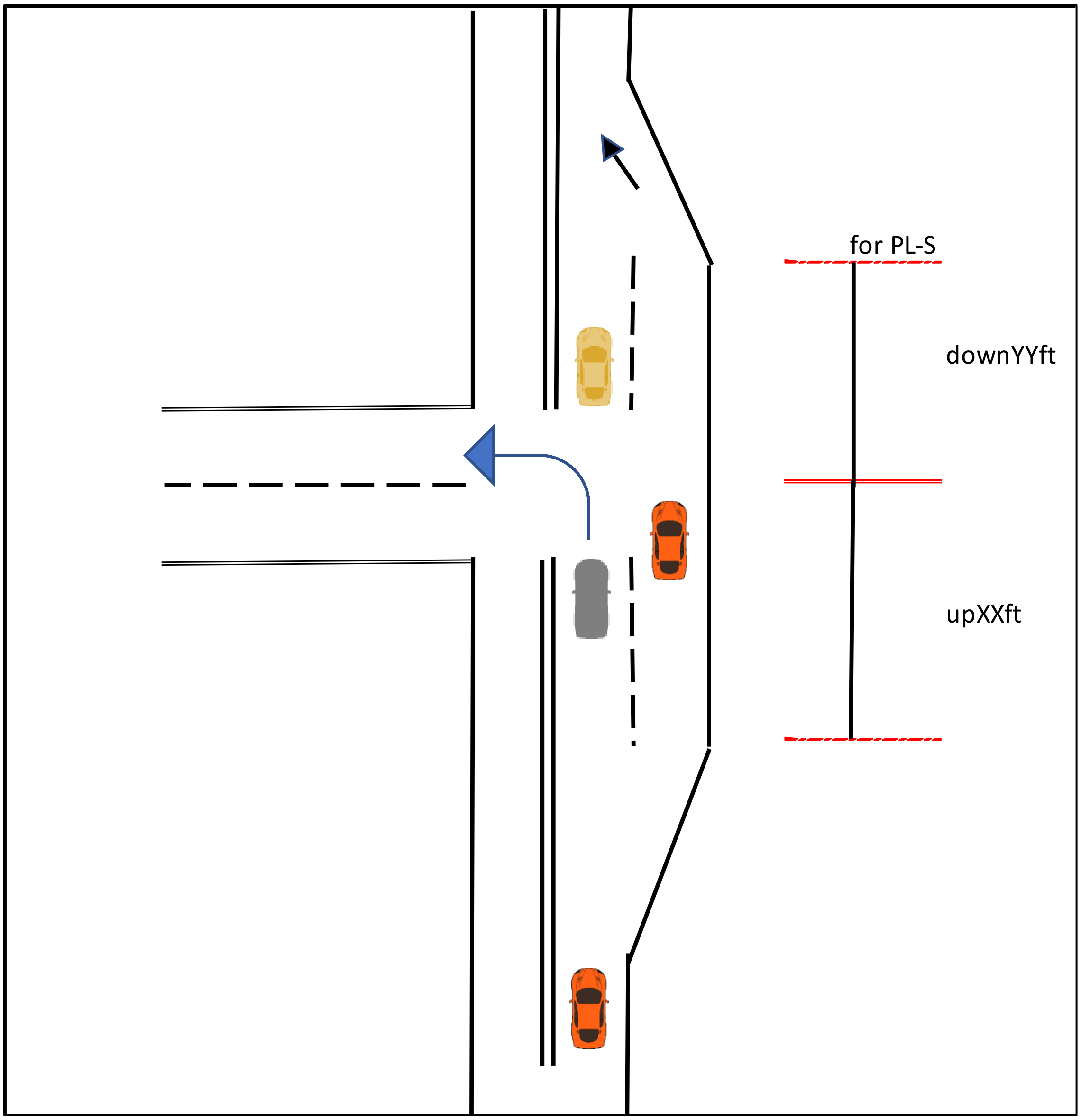


Figure 3. Example of Left-Turning Vehicle Turning Left from the Passing Lane.

The database of identified rural intersections was filtered to identify sites with no left-turn lane and a passing lane present at the intersection, a condition similar to the intersection configuration shown in Figure 3. The most promising intersections were those with higher volumes and those close to the research team that allowed data collection during a day trip or single overnight trip. The research team used intersection characteristics gathered and ADT to identify the sites of interest and then selected five sites for study.

CHARACTERISTICS OF SELECTED FIELD STUDY SITES

Table 7 presents the characteristics of the five sites selected for study. The sites were located in five different counties: Madison, Grimes, Austin, Washington, and Colorado Counties. Four sites had a posted speed limit of 70 mph on the major approach, while the remaining intersection (SH 105 @ FM 159) had a posted speed limit of 65 mph. Diagrams summarizing the lane configuration at each intersection are shown in Figure 4.

Table 7. Characteristics of the Study Sites.

Study site	**SH 21 @ FM 1372**	**SH 30 @ Loma Rd**	**SH 36 @ Loop 497**	**SH 71 @ CR 102**	**SH 105 @ FM 159**
County	Madison	Grimes	Austin	Colorado	Washington
Number of legs	3	3	3	4	3
Main road speed limit (mph)	70	70	70	70	65
Major approach ADT (vpd)	6795	4043	5336	5267	6861
Minor approach ADT (vpd)	520	69	628	1025	534
Number of through lanes, both directions	2	3	3	3	2
Presence of passing lane	No	Yes	Yes	Yes	No
Presence of a left-turn lane on the major approach	No	No	No	No	No
Presence of a right-turn lane on the major approach	No	No	No	No	No
Average lane width (ft)	12	12	12	12	12
Average shoulder width (ft)	10	8	12	10	10
Main road direction	E/W	E/W	N/S	E/W	E/W
Passing lane length for sites with a passing lane at intersection (mi)	NA	2.03	1.5	2.15	NA
Passing lane upstream	Drop	Add	Add	Add	NA
Distance upstream to passing lane (ft)	3170	8900	5290	5490	NA
Passing lane downstream	NA	Drop	Drop	Drop	NA
Distance downstream to passing lane (ft)	NA	1820	2650	5860	NA

Note: E/W = east/west; NA= not applicable.

The selected intersections all have an average lane width of 12 ft, with shoulder widths ranging from 8 to 12 ft. The ADT for major approaches varies between 6700 vpd and 4100 vpd, and minor approach ADT varies between 1000 vpd and 60 vpd. Three sites (SH 30 @ Loma Rd, SH 36 @ Loop 497, and SH 71 @ CR 102) have the passing lanes that are added upstream of the intersection and end (drop) downstream of the intersection in the same direction as those vehicles making a left turn (Figure 4). The remaining two sites (SH 21 @ FM 1372 and SH 105 @

FM 159) do not have a passing lane at the intersection. SH 21 @ FM 1372 is within a Super 2 corridor and has a passing lane that ends about 3170 ft prior to the study intersection.

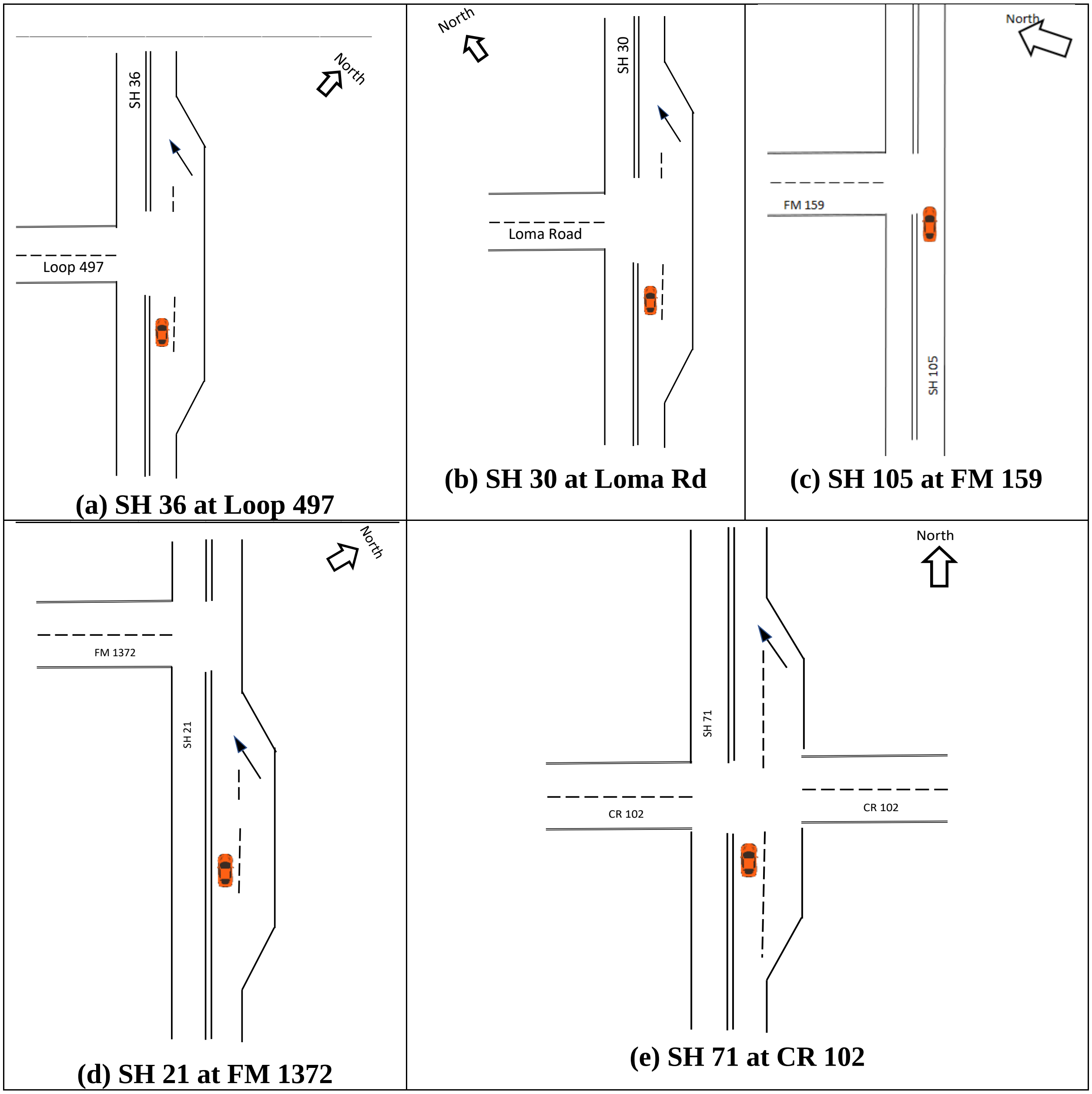


Figure 4. Passing Lane Configurations at the Study Sites.

FIELD DATA COLLECTION

The field data collection objectives were to document vehicle operations at the intersection, especially regarding how the presence of the passing lane affected drivers' behavior. The behaviors of interest were gap acceptance for turning vehicles and shoulder use for vehicles behind the left-turn vehicle. Furthermore, the research team was interested in traffic data such as speed, volume, and vehicle classification.

Initially, the research team collected data for about 5 to 7 hours at each site between July 23 and November 12, 2020 (Table 8). The initial assessment of the data revealed that SH 21 @ FM 1372 had a relatively higher number of turning vehicles. Therefore, researchers chose this site for additional data collection, which was performed on February 24 and 25, 2021.

Table 8. Data Collection Schedule and Duration.

Study Site	Date	Time	Duration (h:mm)
SH 36 @ Loop 497	July 23, 2020	12:25 p.m.–6:45 p.m.	6:20
SH 30 @ Loma Rd	August 03, 2020	2:20 p.m.–8:00 p.m.	5:40
SH 21 @ FM 1372	August 19, 2020	11:25 a.m.–6:30 p.m.	7:05
SH 71 @ CR 102	August 28, 2020	1:05 p.m.–6:45 p.m.	6:40
SH 105 @ FM 159	November 12, 2020	12:50 p.m.–6:20 p.m.	5:30
SH 21 @ FM 1372	February 24, 2021	11:00 a.m.–6:00 p.m.	7:00
SH 21 @ FM 1372	February 25, 2021	7:45 a.m.–11:25 a.m.	3:40

Equipment

Two types of field data collection equipment were deployed: digital video recording equipment and Wavetronix radar. Two digital video cameras (equipment shown in Figure 5) were used to record the vehicle interactions during left-turn maneuvers. The research team set one digital video camera to record a distant view, while the second recorded a closer view. The team selected the distance from the intersection to the data collection equipment so that the video cameras could record the turning vehicles and upstream shoulder use by vehicles avoiding the turning vehicle. In the initial data collection effort (July 23–November 12, 2020), the digital video cameras and the Wavetronix were set approximately 100 ft to 300 ft from the intersection in the direction where turning vehicles approached the intersection. However, after reviewing the videos, the team observed that some of the vehicles were moving onto the shoulder before the camera captured them. Therefore, the research team increased the camera distance to 580 ft for the data collected on February 24 and 25, 2021. This setup was able to capture the motorist behavior for the turning vehicles, opposing vehicles, and vehicles behind the left-turn vehicles. The behaviors of interest included braking/slowing down, lane changing, gap acceptance, and shoulder use.

The radar recorded the speed, length, and lane of each vehicle traveling through the intersection during the study period. In turn, the collected information was used to determine traffic volume, speed, headway, and vehicle classification. The data collection activities at each site lasted for 5 to 8 hours. In total, the research team collected about 40 hours of video documenting over 350 left-turn vehicles.

Figure 5. TTI Portable Video Data Collection Equipment.

Speed Data

Wavetronix radar collects and stores data in .DAT file format, which needs to be converted to other more easily usable formats such as .csv, .txt, or .xls. Further, Wavetronix creates a new file every hour. Therefore, the research team translated the .DAT file to comma-separated text (.csv) files using a program developed within TTI. Figure 6 shows the sample output of the .csv file deciphered from the .DAT file.

	A	B	C	D	E	F
1	UTCDateTime	LaneID	Speed	Duration	Length	OriginalMessage
2	29743288	1	56	126	25.9308	XA01C5D8B81007E00381~
3	29743288	3	67	102	25.11495	XA01C5D8B83006600431~
4	29744500	3	64	100	23.52	XA01C5DD743006400401~
5	29745308	3	59	102	22.11615	XA01C5E09C30066003B1~
6	29746116	3	67	113	27.823425	XA01C5E3C43007100431~
7	29746116	2	73	98	26.29095	XA01C5E3C42006200491~
8	29746520	3	64	125	29.4	XA01C5E5583007D00401~
9	29746520	2	78	90	25.7985	XA01C5E5582005A004E1~
10	29752176	1	67	123	30.285675	XA01C5FB701007B00431~
11	29752580	1	72	97	25.6662	XA01C5FD041006100481~

Figure 6. Sample Output Comma-Separated Text Files.

As shown in Figure 6, the data collected using Wavetronix radar included the following six variables:

- UTCDateTime is the timestamp, measured in units of 2.5-ms increments.
- LaneID represents the lane number occupied by the vehicle being recorded. The lane closest to the Wavetronix radar is given a value of 1, the next lane is given a value of 2, etc.
- Speed is the vehicle speed measured in miles per hour.
- Duration is the length of time (in milliseconds) that the vehicle was present in the detection zone.
- Length represents the length of the vehicle in feet.
- OriginalMessage represents the data message for the vehicle, as recorded by the sensor, in hexadecimal format.

The research team used the collected speed data to determine speed distribution under free-flow traffic conditions for both passenger vehicles and trucks, headway, vehicle composition, and traffic volume.

Video Data

The team collected about 17 hours of video data at SH 21 @ FM 1372 and between 5 and 7 hours for each of the other sites. The videos captured gap acceptance behavior for turning vehicles and shoulder use behavior for vehicles behind the left-turn vehicle.

FINDINGS BASED ON SPEED DATA

The research team used the speed data collected using Wavetronix to determine four traffic flow parameters: headway, vehicle distribution, speed distribution, and traffic volume. The detailed information related to these traffic flow parameters is presented in the next section.

Headway

For this study, vehicles were identified as operating at free-flow speed when they had at least a 3-second headway. The headway was computed using UTCDateTime and LaneID variables. This is the time difference between two consecutive vehicles traveling in the same direction on the same lane. Figure 7 shows the distribution of free-flow headways per study site for up to 125 seconds. The data plot for each site indicates that the majority of vehicles were traveling at headways between 3 and 25 seconds, irrespective of the study site.

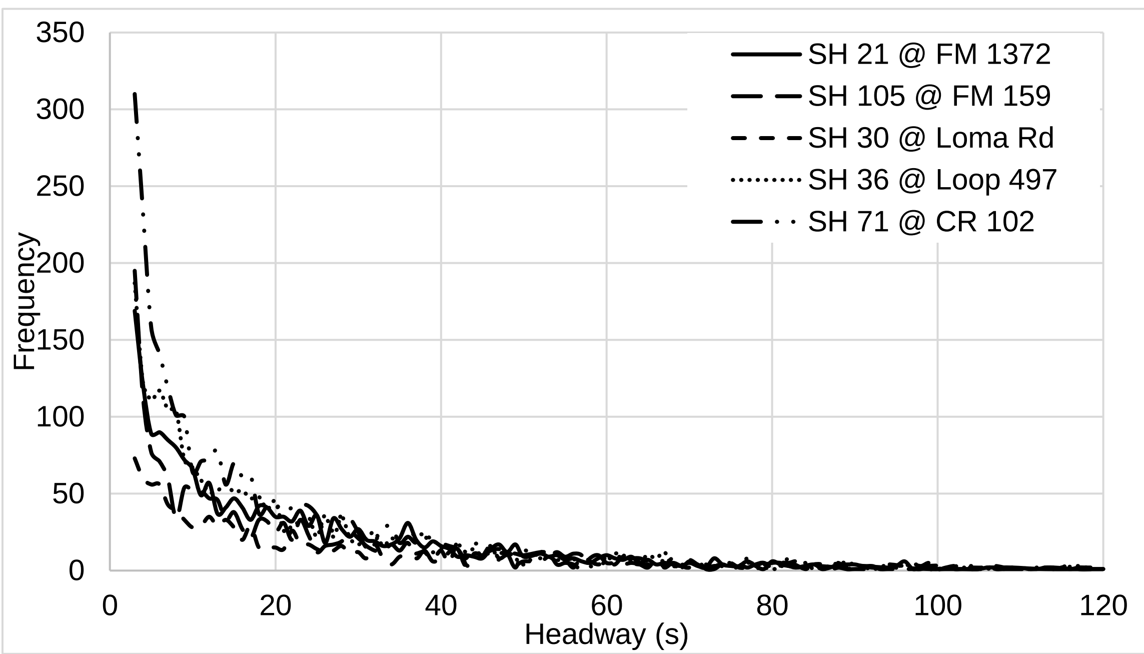


Figure 7. Density Plot of the Headways per Study Site.

Vehicle Distribution

The research team used the Length variable from the Wavetronix data to determine the type of vehicle. According to FHWA vehicle classification (*48*), any vehicle greater than 35 ft can be considered either a combination vehicle or a multi-trailer vehicle. In this study, the two categories were identified as "trucks." On the other hand, vehicles 35 ft or shorter are classified as SU vehicles or passenger vehicles. In this study, the SU vehicles and passenger vehicles were grouped to form the "passenger cars" category. Therefore, the research team used 35 ft to distinguish between passenger cars and trucks in accordance with the FHWA vehicle classification. Table 9 and Figure 8 show the number and percentage distribution of the vehicle types per site of data collection, respectively. It can be observed that the proportion of trucks ranges from 11 percent at SH 36 @ Loop 497 to 17 percent at SH 71 @ CR 102.

Table 9. Number of Vehicles by Type per Study Site.

Vehicle Type	Study Site				
	SH 21 @ FM 1372	**SH 30 @ Loma Rd**	**SH 36 @ Loop 497**	**SH 71 @ CR 102**	**SH 105 @ SH 159**
Passenger car	1710	1137	2040	2268	1424
Heavy truck	332	161	250	468	231
Total	2042	1298	2290	2736	1655

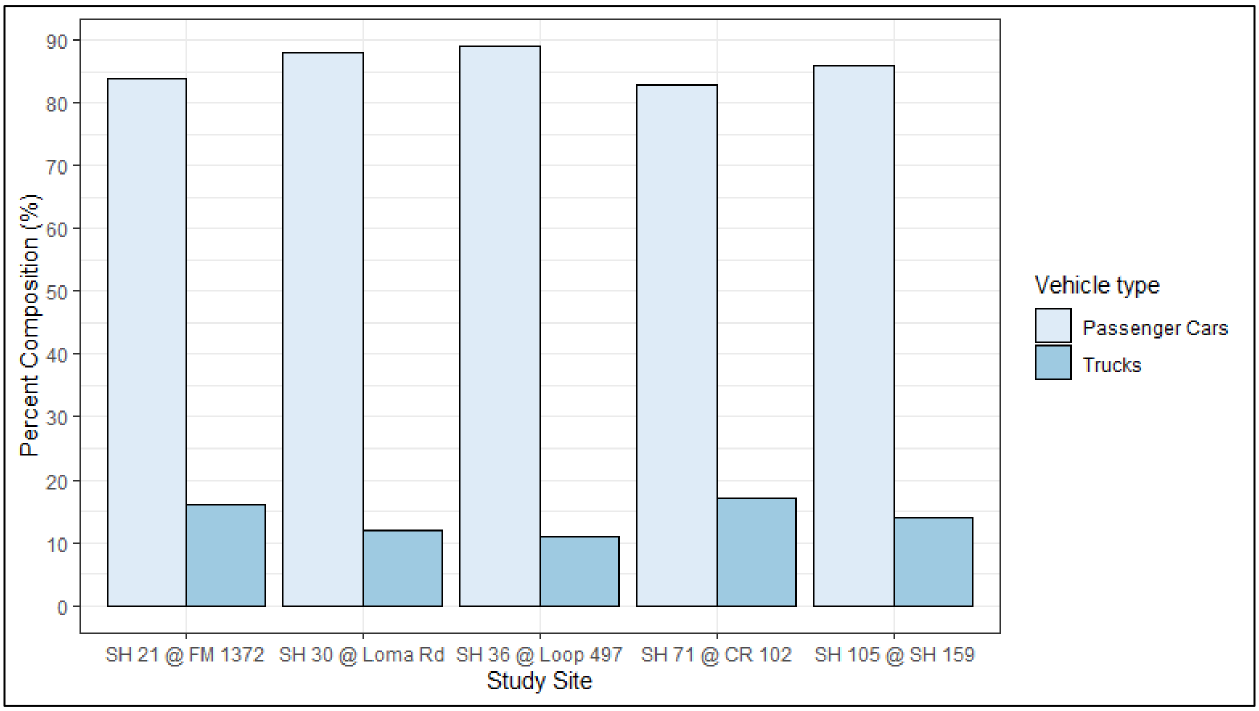


Figure 8. Percentage Composition of Vehicles per Site.

Speed Distribution

The research team developed the major-road traffic speed distribution per site. The speed distribution represents the cumulative percentage of recorded speed for passenger cars and trucks. The team used the 3-second headway to obtain free-flow traffic speed, as described earlier. Moreover, the research team removed from the dataset the vehicles traveling at speeds slower than 40 mph because most of these vehicles might have been either the turning vehicles or merging vehicles that stopped at the intersection. In addition, vehicle speeds greater than 90 mph were also removed from the dataset under the assumption of radar detection errors.

Figure 9 and Figure 10 present the free-flow speed distribution for passenger cars and trucks, respectively, for the five sites. Table 10 presents the speed distribution metrics for study sites by vehicle types.

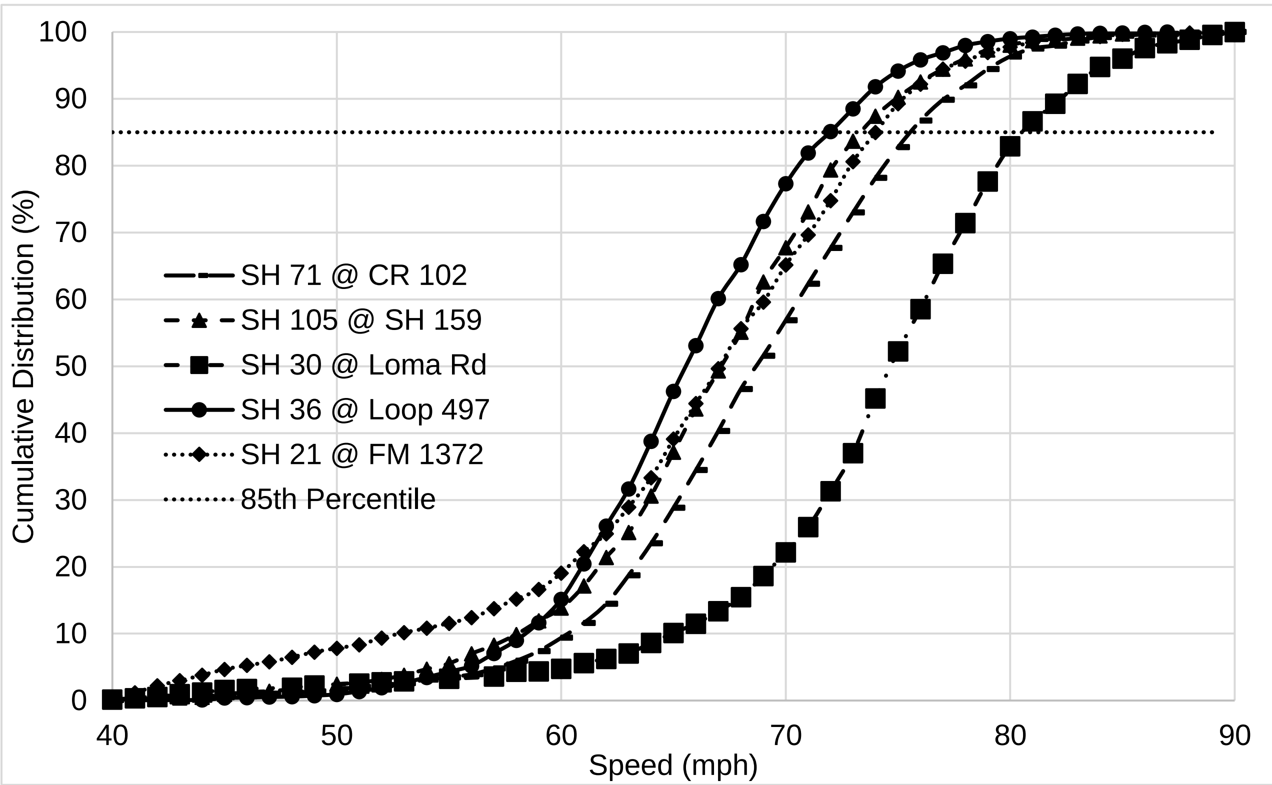


Figure 9. Speed Distribution for Passenger Cars across Study Sites.

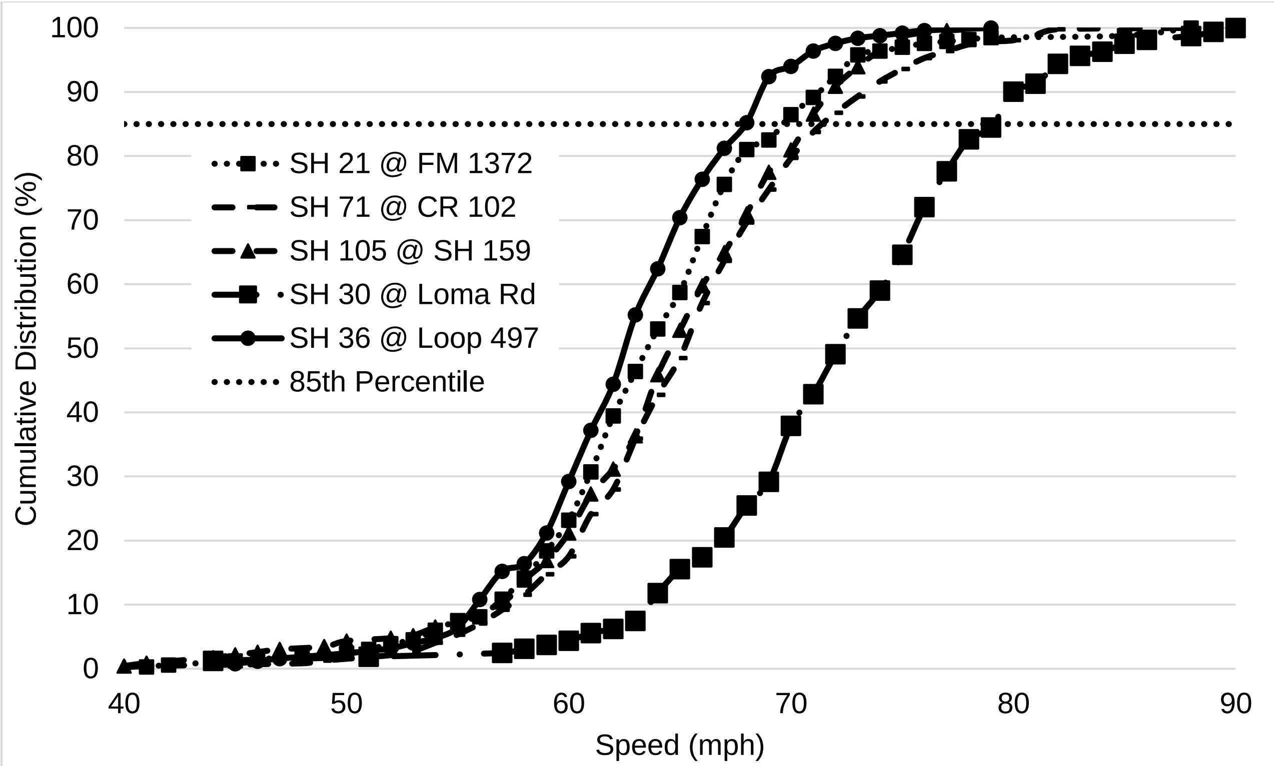


Figure 10. Speed Distribution for Trucks across Study Sites.

Table 10. Speed Distribution Metrics for Study Sites.

Site	**Vehicle Count by Type**		**Speed Distribution Metrics (mph)**					
			Min	**Mean**	**Max**	**Median**	**85th Percentile**	**Standard Deviation**
SH 21 @ FM 1372	Passenger cars	1710	40	66	90	68	75	8.9
	Trucks	332	41	64	88	64	70	6.5
	Combined	2042	40	66	90	67	74	8.6
SH 36 @ Loop 497	Passenger cars	2040	41	66	87	66	72	6.1
	Trucks	250	45	63	79	63	68	5.2
	Combined	2290	41	66	87	66	72	6.1
SH 71 @ FM 102	Passenger cars	2268	40	69	90	69	76	7.1
	Trucks	468	40	66	88	66	72	6.4
	Combined	2736	40	68	90	68	76	7.1
SH 30 @ Loma Rd	Passenger cars	1137	40	74	90	75	81	7.8
	Trucks	161	44	72	90	73	80	7.2
	Combined	1298	40	74	90	75	81	7.7
SH 105 @ SH 159	Passenger cars	1424	40	67	90	68	74	7.0
	Trucks	231	40	65	79	65	71	6.6
	Combined	1655	40	67	90	67	73	7.0

The data in Table 10 indicate that there was a slight variation of travel speed across data collection sites and among the vehicle types. A comparison across sites shows that the SH 30 @ Loma Rd site had the fastest traveling vehicles, while SH 36 @ Loop 497 had the slowest vehicles. Further, as expected, passenger cars traveled at a higher speed than trucks. In general, the 85th percentile speed for trucks varied between 68 mph and 80 mph, while that of passenger cars varied between 72 mph and 81 mph (Table 10). Likewise, the vehicle speeds' standard deviation tended to be higher for passenger cars than for trucks.

Traffic Volume

The research team extracted traffic volume data from speed data collected using Wavetronix radar. Table 11 and Figure 11 show the hourly traffic volume distribution for the study sites. The table and figure show that the SH 71 @ CR 102 site had the largest observed traffic volume, while SH 30 @ Loma Rd had the lowest traffic volume. These statistics, however, are not reflected in the ADT data available in TxDOT's database (*49*); the major approach's ADT retrieved from TxDOT's database showed that the SH 105 @ SH 159 site had the largest traffic volume (Table 11). The likely reason for this difference is that the hourly traffic volumes in Table 11 and Figure 11 were collected for a few hours, while the ADT used data collected for multiple days.

Table 11. Traffic Volume for the Study Sites.

	Study Site				
	SH 21 @ FM 1372	**SH 30 @ Loma Rd**	**SH 36 @ Loop 497**	**SH 71 @ CR 102**	**SH 105 @ SH 159**
Data collection time	11:25 a.m.– 6:30 p.m.	2:20 p.m.– 8:00 p.m.	12:25 p.m.– 6:45 p.m.	1:05 p.m.– 6:45 p.m.	12:50 p.m.– 6:20 p.m.
Number of hours from radar detector	7:05	5:40	6:20	6:40	5:30
Total number of vehicles from radar detector	2654	1480	2903	3714	2292
Maximum number of vehicles per hour from radar detector	426	351	549	874	482
Average hourly volume (vph) from radar detector	380	260	461	554	416
Major approach ADT (vpd) from TxDOT database (*49*)	6795	4043	5336	5267	6861
Minor approach ADT (vpd) from TxDOT database (*49*)	520	69	628	1025	534

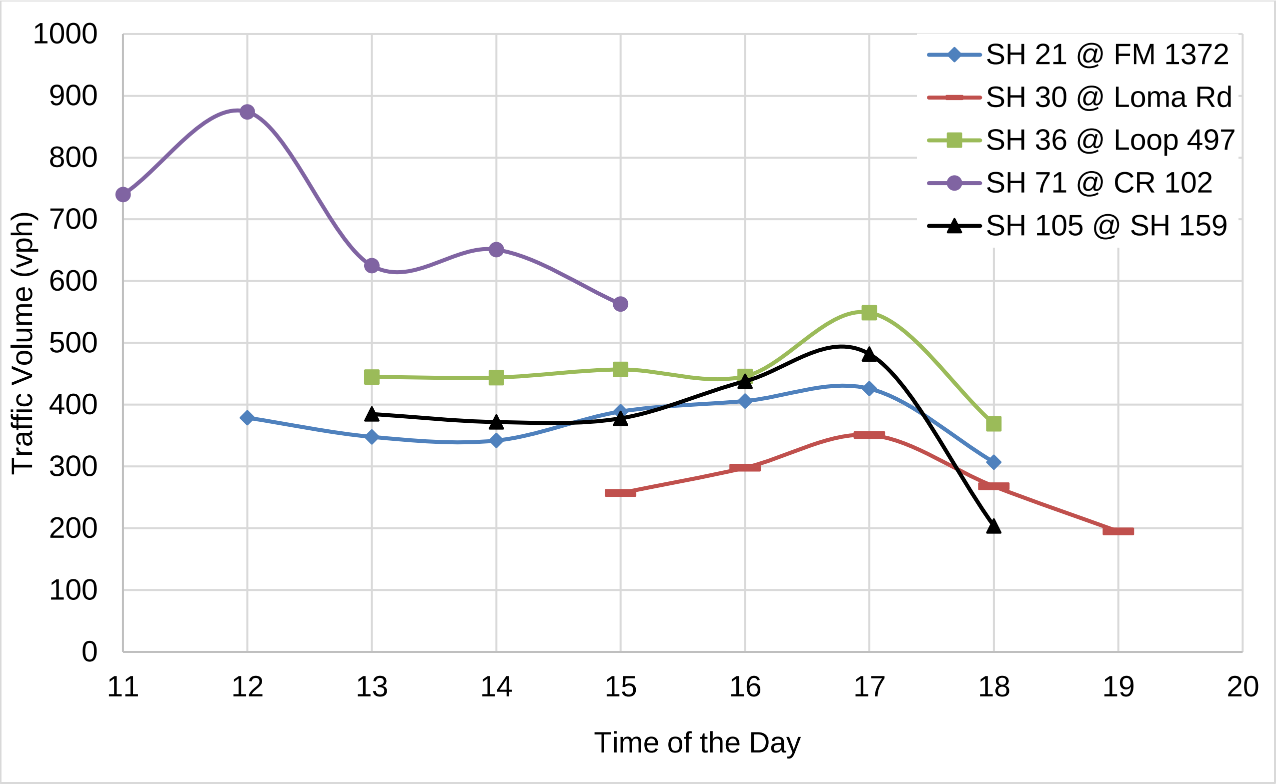


Figure 11. Observed Traffic Volume per Hour for the Five Study Sites.

FINDINGS BASED ON VIDEO DATA

The research team reviewed over 17 hours of video recordings from SH 21 @ FM 1372 and between 5 and 7 hours for other sites to extract driver behaviors. The behaviors of interest were gap acceptance for turning vehicles and shoulder use for vehicles behind the left-turn vehicles. The team extracted the information from the video and stored it in spreadsheets for analysis. Table 12 presents the turning volumes for all five study sites.

Table 12. Turning Volume for the Study Sites.

	Study Site				
	SH 21 @ FM 1372	**SH 30 @ Loma Rd**	**SH 36 @ Loop 497**	**SH 71 @ CR 102**	**SH 105 @ SH 159**
Data collection duration (hours)	17:45	5:40	6:20	6:40	5:30
Total number of turning vehicles	266	3	12	28	47
Maximum turning vehicles per hour	27	1	4	14	14
Average turning vehicles per hour	15	0.5	1.9	4.2	8.5
Vehicles accepted/rejected a gap	44	0	0	0	12
Total number of available gaps	82	0	0	0	14

Among the five sites, only two sites (SH 21 @ FM 1372 and SH 105 @ SH 159) had a substantial number of turning vehicles. On average, 15 vph turned left at SH 21 @ FM 1372 and 8.5 vph turned left at SH 105 @ SH 159. Conversely, the other three sites had between 3 and 28 total turning vehicles. Further, the two key sites had a total of 56 vehicles whose drivers had to either accept or decline the 96 available gaps.

Shoulder Use by Vehicles behind the Left-Turn Vehicle

Drivers of vehicles upstream of the left-turn vehicles may move onto the shoulder to avoid having to slow or stop behind the turning vehicles. The research team used video data from two sites (SH 21 @ FM 1372 and SH 105 @ FM 159) to evaluate the shoulder use by vehicles approaching the left-turn vehicle from the rear. Over 17 and 5 hours of video recording for SH 21 @ FM 1372 and SH 105 @ FM 159, respectively, were reviewed. The objective was to determine whether each upstream vehicle moved onto the shoulder and the distance at which the movement took place.

The team identified strategic features at the study sites to facilitate the extraction of the needed information. The team identified an imaginary stop bar (see Figure 12), which was assumed to be parallel to the shoulder line of the intersecting minor road. Then, the team measured the actual distances (on the ground) from an imaginary stop line to the key identifiable features. Such features include the power poles, chevrons, trees, and driveways (see Figure 12). The measured distances were later used for cross-checking the distances measured using Google Map tools.

Figure 12. Key Identifiable Features at the Study Site at SH 21 @ FM 1372.

The team reviewed the videos to observe the movements of vehicles onto the shoulders. Figure 13 and Figure 14 show the typical scenarios where the rear approaching vehicle moved onto the shoulder. Researchers identified the point where the right front tire of the vehicle moving onto the shoulder crossed the solid white edgeline. The identified point in the video was matched to the corresponding point on the Google Map, and the distance to the imaginary stop line was measured using Google Map tools. The measured distance from the Google Map was compared to the known distance measured on site for quality checks.

Figure 13. Vehicle Moved onto the Shoulder at the Study Site at SH 21 @ FM 1372.

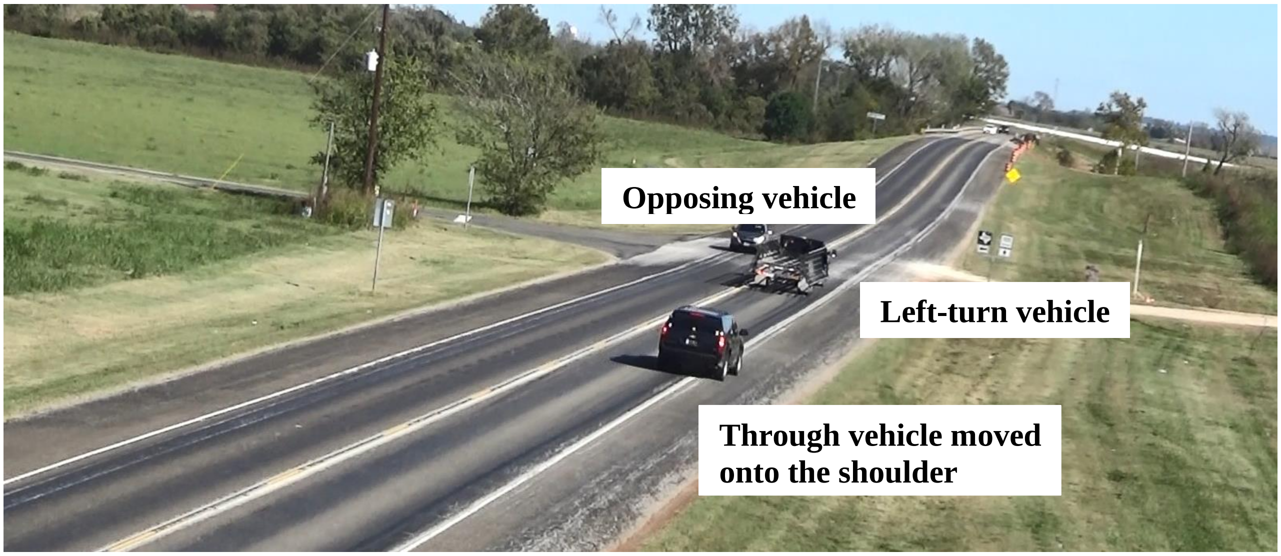


Figure 14. Vehicle Moved onto the Shoulder at the Study Site at SH 105 @ FM 159.

After reviewing all video recordings, the team observed 313 vehicles turning left, which included 266 vehicles at SH 21 @ FM 1372 and 47 vehicles at SH 105 @ FM 159 (Table 13). A large proportion of vehicles (87 percent at SH 21 @ FM 1372 and 81 percent at SH 105 @ FM 159) did not stop before turning left.

Table 13. Number of Vehicles Arriving behind the Left-Turn Vehicle by Study Sites.

	Study Site	
	SH 21 @ FM 1372	**SH 105 @ FM 159**
Data collection duration (hours)	17:45	5:30
Total left-turn vehicles observed	266	47
Interactions for all through vehicles arriving behind the left-turn vehicle		
All through vehicles arriving behind the left-turn vehicle that needed to either stop, slow, or move onto shoulder in order to avoid the left-turn vehicle	108	45
All through vehicles that stopped behind left-turn vehicle	0 (0%)	1 (2%)
All through vehicles that slowed behind left-turn vehicle	71 (66%)	31 (69%)
All through vehicles that moved to the shoulder to avoid left-turn vehicle	37 (34%)	13 (29%)
Interactions when only one through vehicle arrived behind the left-turn vehicle		
Number of interactions when only one through vehicle arrived behind the left-turn vehicle before left-turn vehicle completed turn	50	13
All through vehicles that stopped behind left-turn vehicle	0 (0%)	1 (8%)
Number of through vehicles that slowed behind left-turn vehicle	31 (62%)	6 (46%)
Number of through vehicles that moved to the shoulder to avoid left-turn vehicle	19 (38%)	6 (46%)

Table 13 provides the number of through vehicles that needed to react to the slowing or stopped left-turn vehicle. Considering all cases when a through vehicle arrived behind a left-turn vehicle and needed to change speed to avoid the left-turn vehicle, there were 108 and 45 vehicles arriving behind the left-turn vehicles at SH 21 @ FM 1372 and SH 105 @ FM 159, respectively. Among those trailing vehicles, 34 percent (37 vehicles) for SH 21 @ FM 1372 and 29 percent (13 vehicles) for SH 105 @ FM 159 moved onto the shoulder, while the rest slowed enough for the left-turn vehicle to clear the intersection before reaching the intersection. The left-turn vehicles associated with the slowed through vehicles did not take long to clear the intersection; as a result, the through vehicles did not need to move onto the shoulder to avoid stopping. On average, 2.1 vph moved onto the shoulder at SH 21 @ FM 1372, compared to 2.3 vph at SH 105 @ FM 159.

Some of the left-turn and through vehicle interactions involved more than one vehicle arriving behind the left-turn vehicle. Focusing on those interactions involving only one through vehicle arriving behind the left-turn vehicle removed those cases when the through vehicle driver may have been making a decision due to another through vehicle rather than only to the left-turn vehicle. Figure 15 subdivides the number of through vehicles arriving behind a left-turning vehicle by the number of through vehicles arriving for a specific left turn. The proportion of incidents when the through vehicle moved onto the shoulder when only one trailing through vehicle arrived was 38 percent (19/50) at SH 21 @ FM 1372 and 46 percent (6/13) at SH 105 @ FM 159.

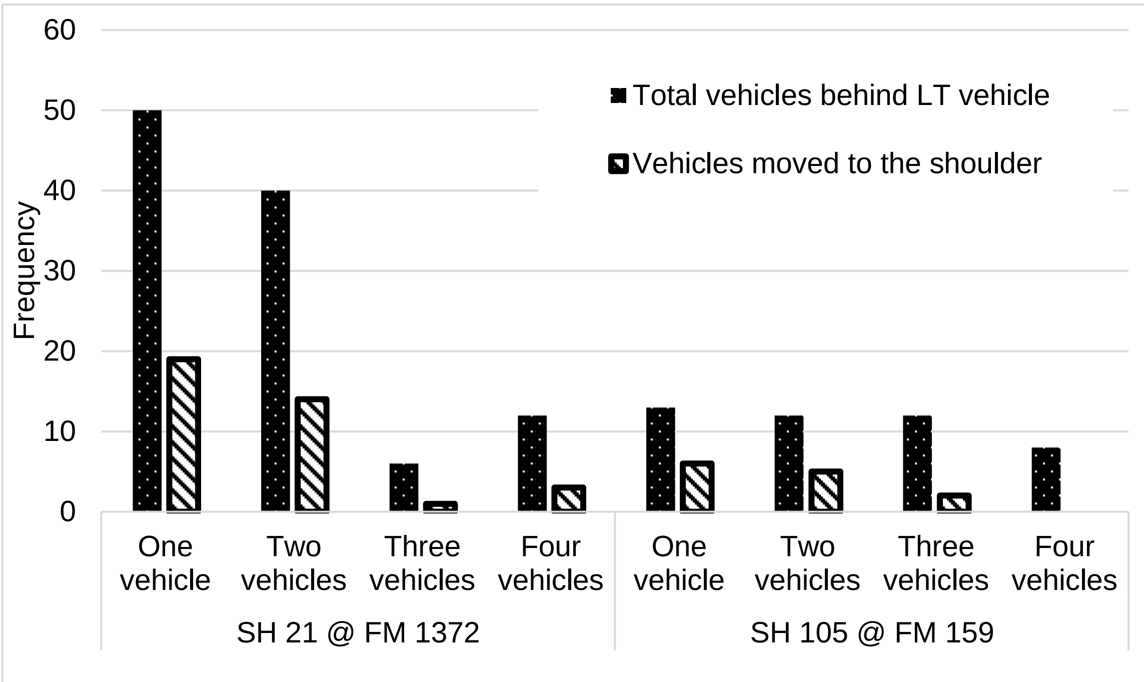


Figure 15. Distribution of Vehicles behind the Left-Turn Vehicle.

Figure 16 provides the cumulative distribution of the distance upstream of the intersection vehicles that were observed moving onto the shoulder. Results show that the two sites differed notably in when a driver would move onto the shoulder. Figure 16 shows that drivers at SH 105 @ FM 159 were more likely to move to the shoulder at a closer distance than at site SH 21 @ FM 1372. For comparison, the 85th percentile of the distance that the vehicles moved to the shoulder for SH 21 @ FM 1372 (250 ft) was more than twice the distance at SH 105 @ FM 159 (100 ft). Further, the distance that 50 percent of the vehicles moved to the shoulder at SH 21 @ FM 1372 (190 ft) was more than three times that of SH 105 @ FM 159 (55 ft). The possible reason for such a variation in the distances is that there is more available sight distance at SH 105 @ FM 159. SH 21 @ FM 1372 is located on a horizontal curve; therefore, drivers may be hesitant to move onto the shoulder at a longer upstream distance.

Furthermore, during the initial data collection period, the cameras were positioned at a closer distance to the intersection. As a result, three vehicles at SH 21 @ FM 1372 and one vehicle at SH 105 @ FM 159 were already on the shoulder when they entered the camera's field of view, and their distances were estimated based on what could be seen in the video.

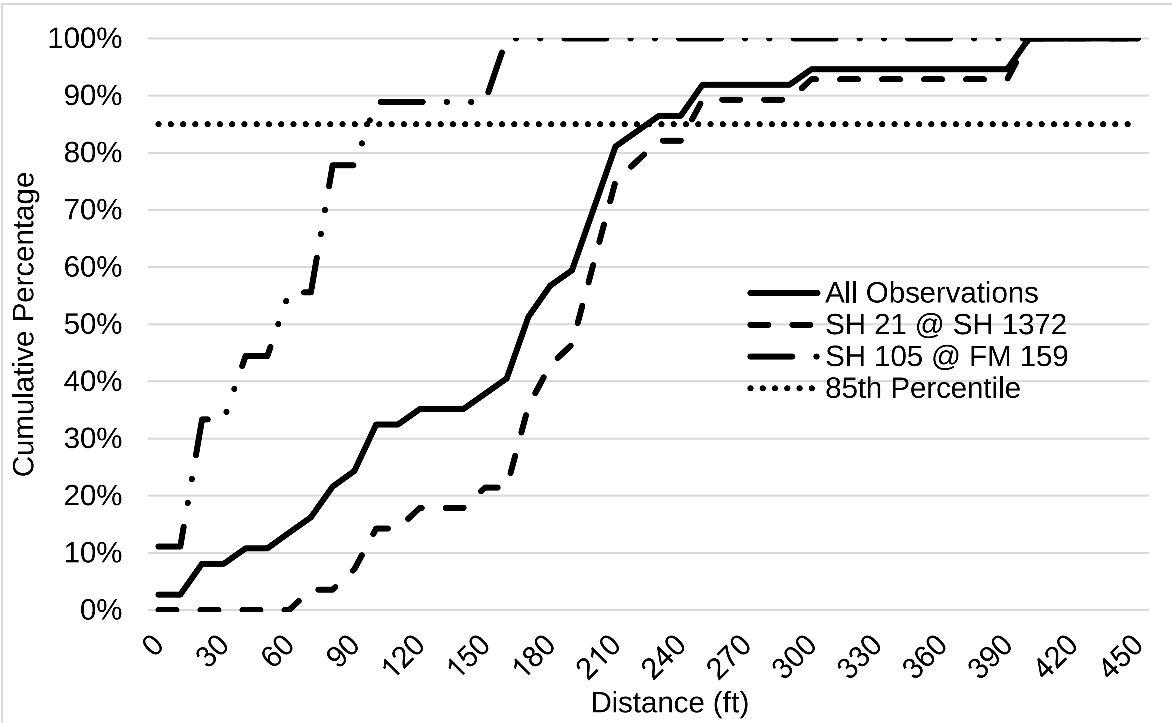


Figure 16. Cumulative Distribution of Distance Upstream from Intersection That Vehicles Moved onto Shoulder to Avoid a Left-Turn Vehicle.

Gap Acceptance

Gap acceptance plays a vital role in evaluating the safety and operational performance of an unsignalized intersection. Due to the small number of turning vehicles for the other three study sites (SH 30 @ Loma Rd, SH 36 @ Loop 497, and SH 71 @ CR 102), the gap acceptance analysis was performed using data from two sites only (SH 21 @ FM 1372 and SH 105 @ FM 159). The two sites experienced a total of 313 turning vehicles for the data collection period. Among 313 turning vehicles, about 18 percent (56 vehicles) had a chance to either accept or reject the available gap. The total number of gaps available was 96, of which SH 21 @ FM 1372 had 82 gaps and SH 105 @ FM 159 had 14 gaps.

The average, minimum, and maximum accepted and rejected gaps are shown in Table 14, which shows that the rejected gaps varied between 1 and 6 seconds, while the accepted gaps varied between 6 and 151 seconds. More variabilities are observed in the accepted gaps than rejected gaps, as revealed by the standard deviation values.

Table 14. Characteristics of Accepted and Rejected Gaps.

Gap acceptance	Average gap (s)	Minimum gap (s)	Maximum gap (s)	Standard deviation gap (s)
Accepted	32	6	151	26.3
Rejected	2	1	6	1.2

To determine a critical gap, the research team applied the equilibrium of probabilities method. This method was developed to simplify the computation efforts from previous approaches (*45*, *46*). According to a study that reviewed available methodologies for critical gap estimation (*44*), the equilibrium of probabilities method requires fewer data points to produce similar results to the Raff model, average accepted gap, Troutbeck model, and cumulative acceptance methods, among others. Thus, the selection of equilibrium of probabilities method was based on the available small sample size and simplified computations while yielding acceptable results. The equilibrium of probabilities method takes into account all relevant gaps, not only the maximum rejected gap considered in the Troutbeck model.

All available gaps (rejected and accepted) were extracted from video data and stored in a spreadsheet to compute the critical gap. After arranging the gaps in ascending order and identifying the accepted and rejected gaps, the accumulated frequencies of rejected and accepted gaps were determined. The probability distribution functions (PDFs) of the accepted and rejected gaps were computed. Using the PDFs of the accepted and rejected gaps, the researchers computed the critical gap's PDF for each observation as shown in Equation 2:

$$F_{tc} = \frac{F_a}{F_a + (1 - F_r)} \qquad (2)$$

Where:

F_{tc} = critical gap's PDF
F_a = accepted gap's PDF
F_r = rejected gap's PDF

Further, the estimated frequency of critical gap (p_{tc}), which is the difference between two consecutive estimated critical gap PDFs, was computed, followed by the class mean value (t_d), which is the average of two consecutive gaps. Finally, the average value of the critical gap and the variance were computed using Equations (3) and (4):

$$t_{c,average} = \sum(p_{tc} * t_d) \quad (3)$$

$$variance\ (\sigma^2) = \sum(p_{tc} * {t_d}^2) - (\sum(p_{tc} * t_d))^2 \quad (4)$$

Using the equations above, the average critical gap was found to be 5.8 seconds, with a variance of 0.06 seconds. Because there were only six heavy vehicles (i.e., four pickup trucks with a trailer and two 18-wheelers) among 56 vehicles that had an opportunity to accept or reject the gap, the research team combined both heavy vehicles and passenger cars for computing critical gap.

The computed critical gap is close to the values reported in the literature for similar roadway facilities. A previous study (*47*) developed an equation to calculate critical gaps based on a number of site characteristics. For conditions similar to sites included in this study, the critical gap would be between 5.85 and 6.21 seconds. According to AASHTO (*24*), the overall critical gap for high-speed two-way roadways for passenger cars is 5.5 seconds, while that of single-unit and combination trucks is 6.5 and 7.5 seconds, respectively. Moreover, several other studies suggested a variation in critical gap acceptance at unsignalized intersections by the drivers' demographic characteristics, posted speed limit, and land-use setting, among others. For instance, the reported critical gap for left-turn movements from major roadways in urban regions ranges between 4.21 and 6.44 seconds, while the gap in suburban regions ranges between 6.00 and 8.00 seconds (*43*). Similarly, older drivers (above 70 years) are more likely to reject a shorter gap than young drivers. Thus, although a small sample size (56 left-turn vehicles and 96 gaps) was used in this study, the computed critical gap does not differ significantly from the ones found in the literature. The observation suggests that the computed critical gap can be used with confidence.

CRASH ANALYSIS

Site Selection

Using the more than 300 intersections identified at the beginning of Task 3 in the corridor review, researchers identified a total of 289 sites for potential use in a crash analysis. Six of those intersections had unique features such as left-turn restrictions, proximity to an interchange or another intersection, or all-way stop control and were eliminated from consideration, leaving

283 sites to include in the crash analysis of the unsignalized intersections on Super 2 corridors. These 283 intersections were geocoded for further data integration and analysis.

Crash Data

The research team used 5 years (2014–2019) of crash data. To match the crashes with the intersections, the team plotted both intersections and crashes using geocoordinates (i.e., latitudes and longitudes). A buffer of 250 ft was used to identify and assign crashes to a given intersection. Further, the street names were used to correctly locate the appropriate intersection for intersections within close proximity. Upon associating crashes and intersections, researchers found a total of 576 crashes that occurred on the 283 unsignalized intersections in the 5 years. Of those 576 crashes, 140 were intersection crashes, 174 were intersection-related crashes, 37 were driveway crashes, and 225 were non-intersection crashes. In this study, the team used only the 314 intersection and intersection-related crashes for analysis. The next section presents the descriptive analysis of the crashes.

Descriptive Analysis

Table 15 presents the descriptive statistics of the variables considered in the statistical evaluation, and Table 16 provides the ADT range for the intersections. The descriptive review of the crashes focused on the distribution of crashes per intersection by the number of legs, presence/absence of the left-turn lane, and presence/absence of the passing lanes. The data show that of the 283 intersections, 224 were three-leg and 59 were four-leg. On average, the number of annual crashes at four-leg intersections was about twice the number of crashes at three-leg intersections.

A relatively small number of the intersections in this database (54) had left-turn lanes. Statistics showed that intersections with left-turn lanes experienced more crashes per year (0.46) than intersections without left-turn lanes (0.17); however, these crash statistics did not include vehicle volume. More crashes per year were observed at the intersections without passing lanes (0.40) than with passing lanes (0.16).

Table 15. Descriptive Results of the Variables Included in the Statistical Model.

Variable	Variable Category	Number of Intersections	Number of Intersection and Intersection-Related Crashes	Crashes per year per Intersection
Number of legs	Three	224	208	0.19
	Four	59	106	0.36
Left-turn lane present	No	229	189	0.17
	Yes	54	125	0.46
Passing lane present	No	71	141	0.40
	Yes	212	173	0.16

Table 16. ADT Range by Number of Legs.

Values	3-Leg Intersections	4-Leg Intersections	All Intersections
Number of intersections	224	59	283
Min of major volume	884	884	884
Average of major volume	5,316	4,252	5,094
Max of major volume	11,715	10,466	11,715
Min of minor volume	7	10	7
Average of minor volume	210	193	207
Max of minor volume	4408	1348	4408

The research team experimented with several variables to reflect the change in a passing lane. Researchers initially considered how to account for the presence of a passing lane at the intersection and the distance from the intersection to the nearest passing lane addition taper and passing lane-drop taper. Because of the number of possible combinations of passing lane presence and distance, researchers also considered categorical grouping of the passing lane variable. The distance to the nearest lane addition or lane drop presented challenges in terms of whether to have a cap on the distance. Should a passing lane that begins a mile from the intersection be considered? Should a distance of 0.5 mi be considered? Should the influential distance be less? The simulation evaluation discussed in Chapter 4 revealed a difference in operations when the passing lane was within 750 and 1500 ft; therefore, the research team created a categorical variable to identify those intersections where a passing lane was added or was dropped within 1500 ft. None of the variables associated with a passing lane change were found to be statistically significant.

Statistical Model Results and Discussion

The team developed a negative binomial model to evaluate the influence of passing lanes and left-turn lanes on the intersection crash frequency in Super 2 corridors. Additionally, the influence of the distance to a passing lane change (either being added or being dropped) upstream and downstream of the intersection was considered but found to not be significant for this database. Traffic volume and the number of intersection legs were used as the control variables.

The team made several attempts to determine the best-fitting model. The statistical significance of the variables at a 90 percent confidence level was used to determine the variables to be included in the final model. The attempts included the consideration of the passing lane within 1500 ft of the intersection and passing lane and left-turn lane interactions. However, neither passing lane distance nor interacted variables were statistically significant at a 90 percent confidence level. Therefore, the team only included the left-turn lane and passing lane variables.

Table 17 presents the negative binomial model results for intersection and intersection-related crash frequency in the Super 2 corridors. The negative coefficient in the model indicates that the presence of a passing lane or a left-turn lane at the intersection was associated with a reduction in crash frequency. The presence of a passing lane was associated with a decrease in 38 percent (determined as 1-exp(coefficient) or 1-exp(−0.472)=0.376) of intersection and intersection-related crashes for the same time period.

The left-turn lane at the intersection was associated with a decrease in 7 percent (1-exp(−0.074)=0.0713) of intersection and intersection-related crashes per year. The finding was not statistically significant for this set of Texas sites; however, other research has found a statistically significant benefit of left-turn lanes in rural areas. For example, Abdel-Aty et al. (*50*) found a 27 percent reduction for rural crashes with the installation of a left-turn lane, and Harwood et al. (*21*) found a 42 percent reduction. Although the left-turn lane variable was not statistically significant at a 90 percent level for this Texas model, it was retained in the model due to its importance for intersection configuration. The lack of statistical significance is attributed by the small sample size.

In addition to the passing lane and left-turn lane, the number of legs and traffic volume variables were included in the model. The model results in Table 17 show that the increased number of legs was associated with increased intersection and intersection-related crash frequency. Similarly, the increase in major and minor approach traffic volume was associated with an increase in intersection and intersection-related crash frequency.

Table 17. Negative Binomial Model Results.

Variable	Estimate	Adjustment Factor	Standard Error	Z-Statistic	P-value
Intercept	−17.060	0.00	2.237	−6.907	<0.001
Ln(Major Volume [vpd])	1.119	3.06	0.224	5.005	<0.001
Ln(Minor Adjusted Volume [vpd])	0.559	1.75	0.090	6.238	<0.001
Number of Legs	1.057	2.88	0.228	4.631	<0.001
Left-Turn Lane	−0.074	0.93	0.251	−0.296	0.767
Passing Lane	−0.472	0.62	0.217	−2.179	0.029

Note: Number of observations = 283, AIC = 685, dispersion parameter = 0.852.

CHAPTER 4: SIMULATION STUDIES

The research team used the collected field data to develop the simulation analysis of a three-leg intersection with stop control (3ST) within a Super 2 corridor. The team designed several geometric configurations of the corridor around the intersection. The experiment involved varying parameters such as traffic volumes and turning and truck percentages to understand the performance of the corridor and intersection. This chapter documents the research team's effort to develop simulation models, including the findings from the literature and the field studies considered in the simulation. The chapter ends with a summary of the results.

SIMULATION ANALYSIS AND DISCUSSION

In this study, the research team used VISSIM 2020 (*51*) for simulation analysis of the 3ST intersections in Super 2 corridors. The selection of VISSIM 2020 was based on its capability to:

- Accurately replicate car-following behavior and lane-changing maneuvers.
- Track individual vehicles as they travel through the system.
- Allow shoulder use maneuvers in case a left-turn vehicle occupies a through lane.
- Produce various measures of effectiveness (average speed, travel time, headway, number and location of passing maneuvers for various passing lane lengths, and conflict potential in passing lanes).

In addition, VISSIM 2020 allows for adjustment in speed, vehicle type, and yielding behaviors, among other parameters. VISSIM 2020 is the only available microsimulation software that can model passing in the opposite direction, a situation that occurs on the rural two-lane facilities.

The operational effects were measured using intersection and corridor delays. The key purpose of simulation was to estimate the changes in delay for various passing lane configurations and the presence or absence of a left-turn lane. The next section presents the model inputs and the summary of the simulation results.

Model Inputs

The research team designed five basic 3ST intersection scenarios with generic 10.5-mi rural two-lane highway corridors characteristics. The five intersection testbed scenarios were established as follows:

1. Basic rural two-lane highway with no passing lane and one three-leg intersection.
2. Basic rural two-lane highway with no passing lane and one three-leg intersection with a left-turn lane.
3. Rural two-lane highway where the intersection, which does not have a left-turn lane, is located within the passing lane. Passing lane is in the same direction of travel as left-turning vehicles.
4. Rural two-lane highway where the intersection, which does have a left-turn lane, is located within the passing lane. Passing lane is in the same direction of travel as left-turning vehicles.
5. Super 2 highway where the three-leg intersection, which does not have a left-turn lane, is located in the two-lane section that is between passing lanes.

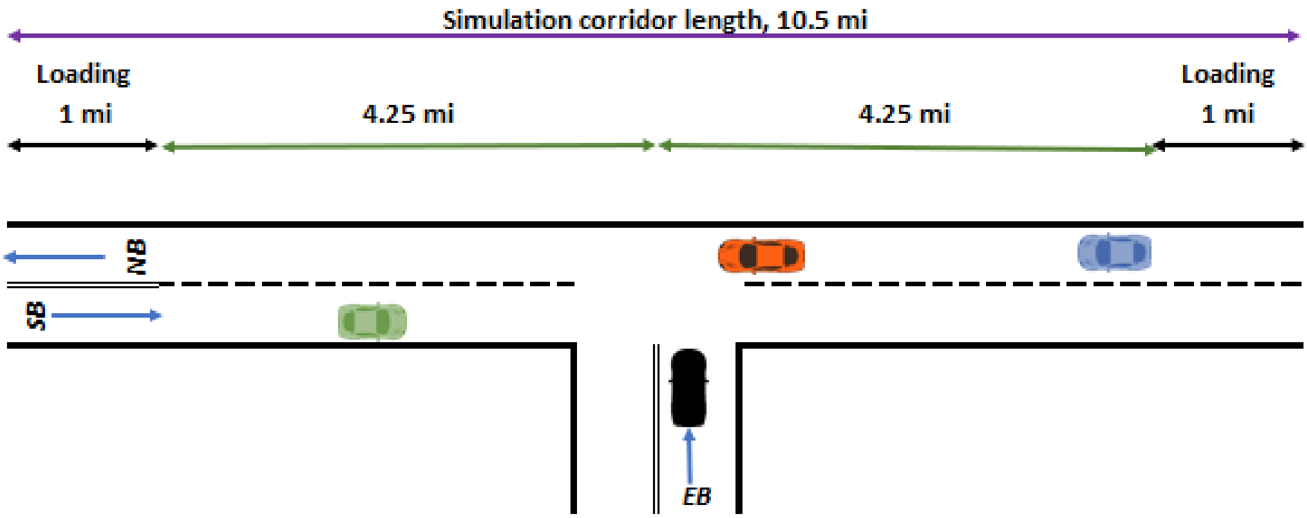
The testbeds had varying configurations in terms of the presence of left-turn lane, presence of passing lane, and distance from the intersection to the beginning or ending of the passing lane. To investigate the influence of passing lane beginning and ending point locations in the above scenarios, a total of 21 testbeds were designed. Table 18 provides the geometric details for these testbeds, and Figure 17 through Figure 20 provide diagrams of configurations for selected testbeds. The testbeds had the major approaches oriented north-south and a span of 10.5 mi, while the minor approach was included as a south leg with a 700-ft length. One-mile segments at each entry point of vehicles for eastbound and westbound directions were reserved for simulation loading so platoons could form from the randomly generated traffic.

Table 18. Geometric Variables of Interest for Simulation Analysis.

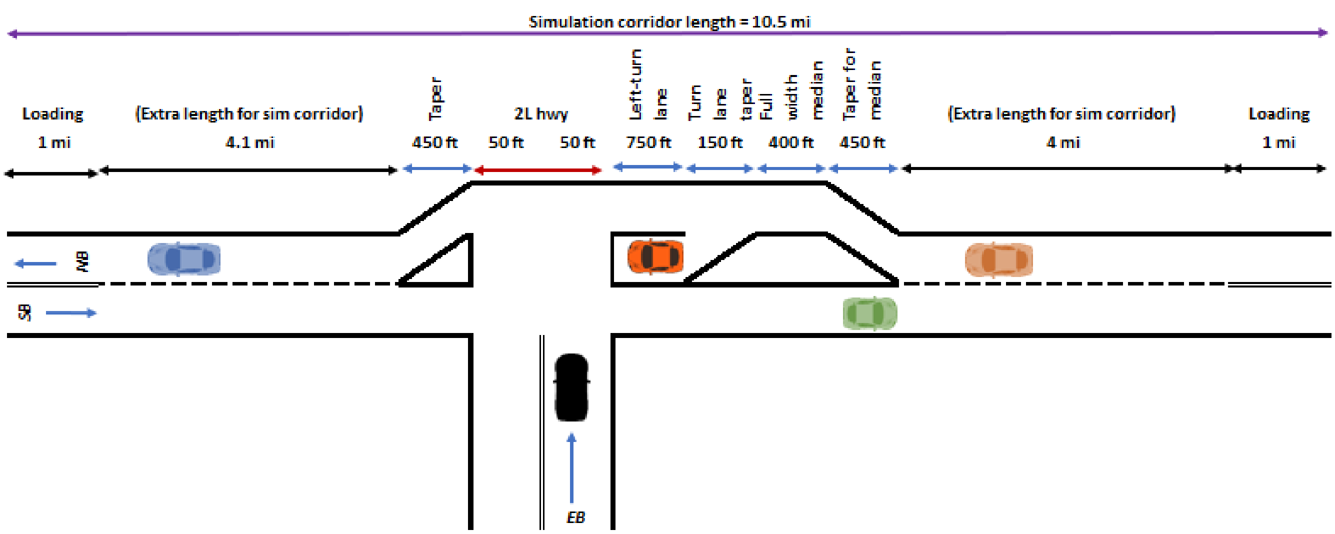
Testbed	LTL	PL at Intersections	Downstream Start Next PL	Downstream PL Length	Upstream PL Length	Upstream End Previous PL
1.1	No	No	2 mi	NA	NA	2 mi
2.1	Yes	No	NA	NA	NA	NA
3.1	No	Yes, SD	NA	2 mi	2 mi	NA
3.2	No	Yes, SD	NA	2 mi	750 ft	NA
3.3	No	Yes, SD	NA	2 mi	2150 ft	NA
3.4	No	Yes, SD	NA	750 ft	2 mi	NA
3.5	No	Yes, SD	NA	1500 ft	2 mi	NA
3.6	No	Yes, SD	NA	2640 ft	2 mi	NA
4.1	Yes	Yes, SD	NA	2 mi	2 mi	NA
4.2	Yes	Yes, SD	NA	2 mi	750 ft	NA
4.3	Yes	Yes, SD	NA	2 mi	2150 ft	NA
4.4	Yes	Yes, SD	NA	750 ft	2 mi	NA
4.5	Yes	Yes, SD	NA	1500 ft	2 mi	NA
4.6	Yes	Yes, SD	NA	2640 ft	2 mi	NA
5.1	No	No	2 mi	NA	NA	2 mi
5.2	No	No	2 mi	NA	NA	500 ft
5.3	No	No	2 mi	NA	NA	1000 ft
5.4	No	No	500 ft	NA	NA	2 mi
5.5	No	No	1000 ft	NA	NA	2 mi
5.6	No	No	2640 ft	NA	NA	2 mi
5.7	No	No	1 mi	NA	NA	2 mi

Note: NA = not applicable; PL = passing lane; LTL = left-turn lane; SD = same direction. Gray shading provides separation between groups of Testbeds.

The baseline testbeds were Testbed 1.1, which had no passing lane or left-turn lane at the intersection, as shown in Figure 17(a); and Testbed 2.1, which had a left-turn lane at the intersection, as shown in Figure 17(b). The Testbed 3.0 series considered the scenarios where a passing lane was present in the corridor and included a different beginning or ending of the passing lane upstream and downstream of the intersection, varying that distance between 750 ft and 2 mi. The typical layout of the 3.0 series is presented in Figure 18, which shows that Testbed 3.2 had a passing lane starting 750 ft upstream and Testbed 3.4 had a passing lane ending 750 ft downstream of the intersection. The Testbed 4.0 series included both a passing lane in the corridor and a left-turn lane at the intersection. Examples of two of the Testbed 4.0 layouts are shown in Figure 19. The Testbed 5.0 series, which is a typical Super 2 design, considered the impacts to an intersection located between two alternating passing lanes. The example of the Testbed 5.0 series is presented in Figure 20.

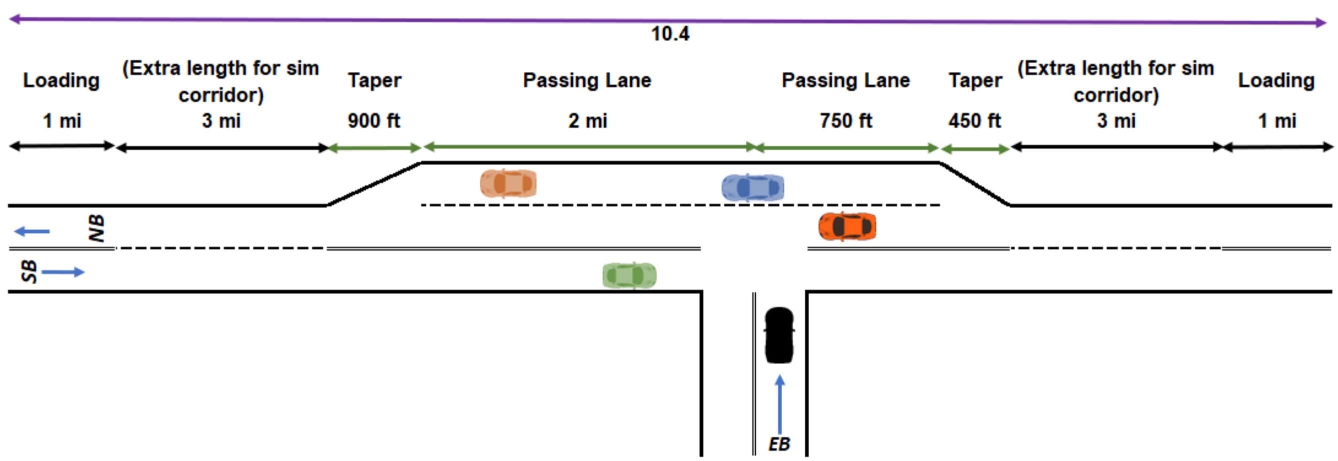


(a) Intersection without left-turn lane (Testbed 1.1)

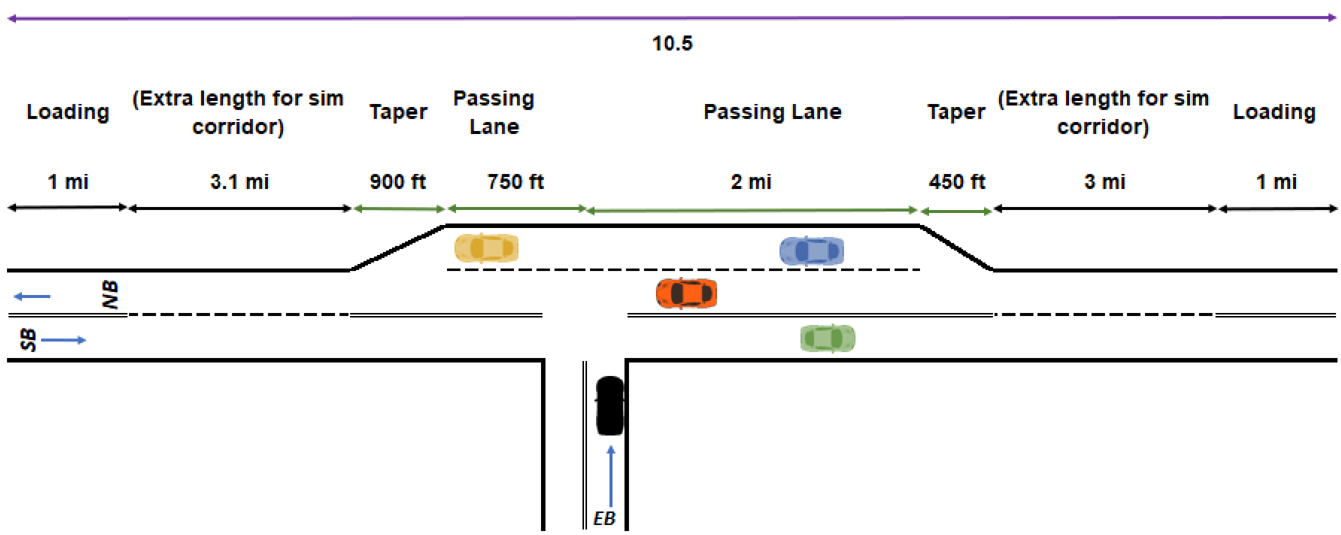


(b) Intersection with left-turn lane (Testbed 2.1)

Figure 17. Rural Two-Lane Highway with Unsignalized Intersection with and without Left-Turn Lane.

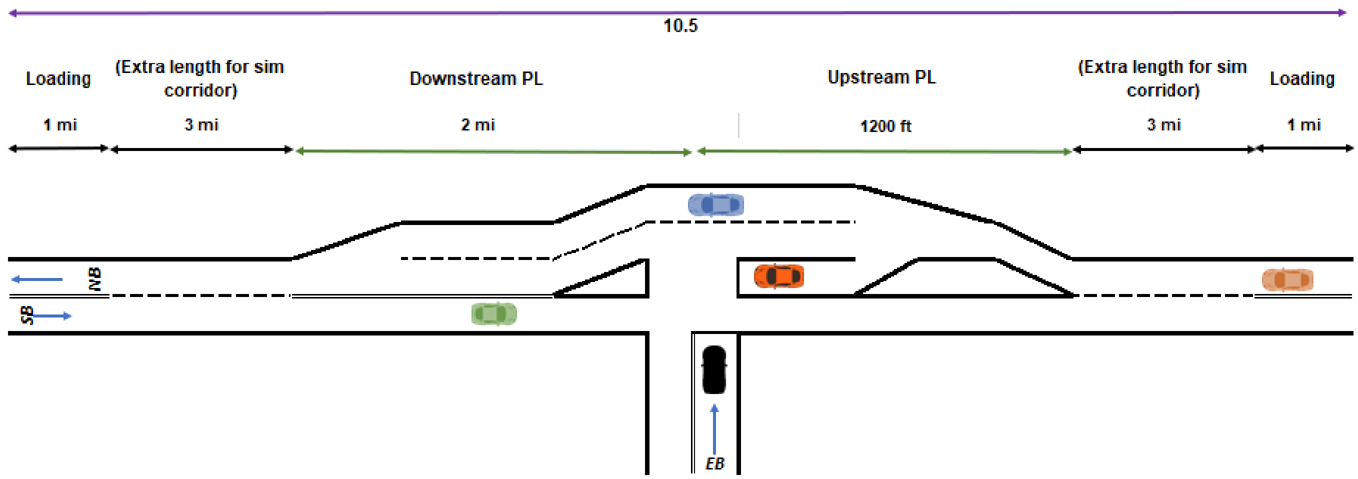


(a) Passing lane ending downstream far from the intersection (Testbed 3.2)

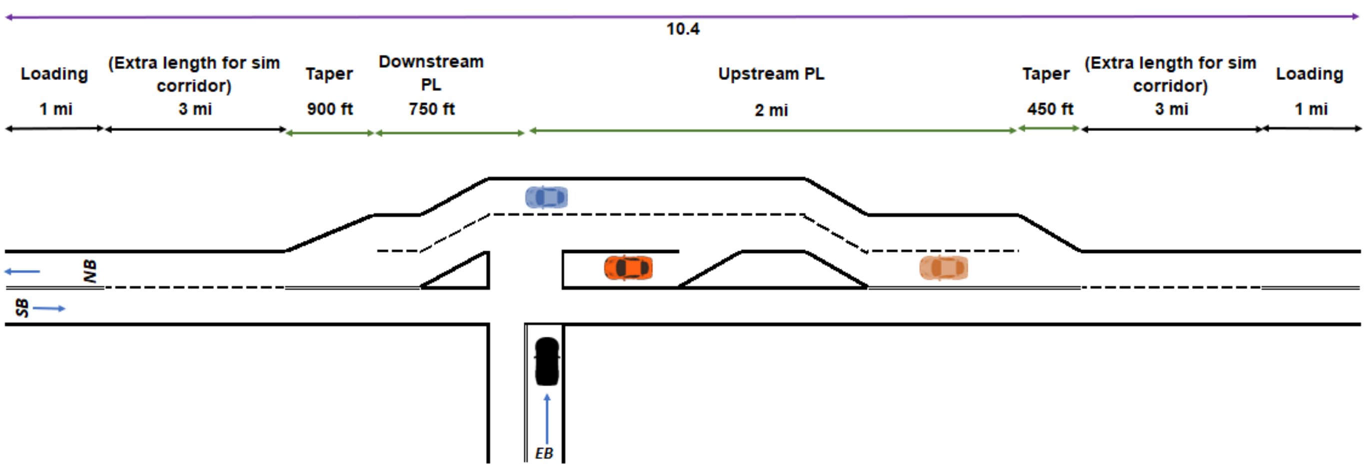


(b) Passing lane ending downstream near the intersection (Testbed 3.4)

Figure 18. Examples of 3.0 Series Testbeds—Intersections with No Left-Turn Lane within a Passing Lane with Varying Distances to the Start and End of the Passing Lane.

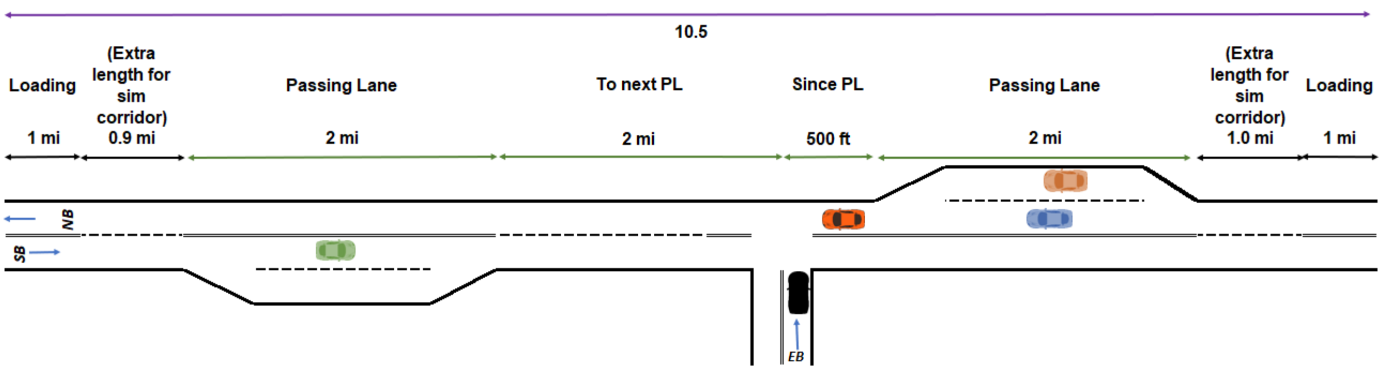


(a) Left-turn lane and passing lane ending far from the intersection (Testbed 4.2)

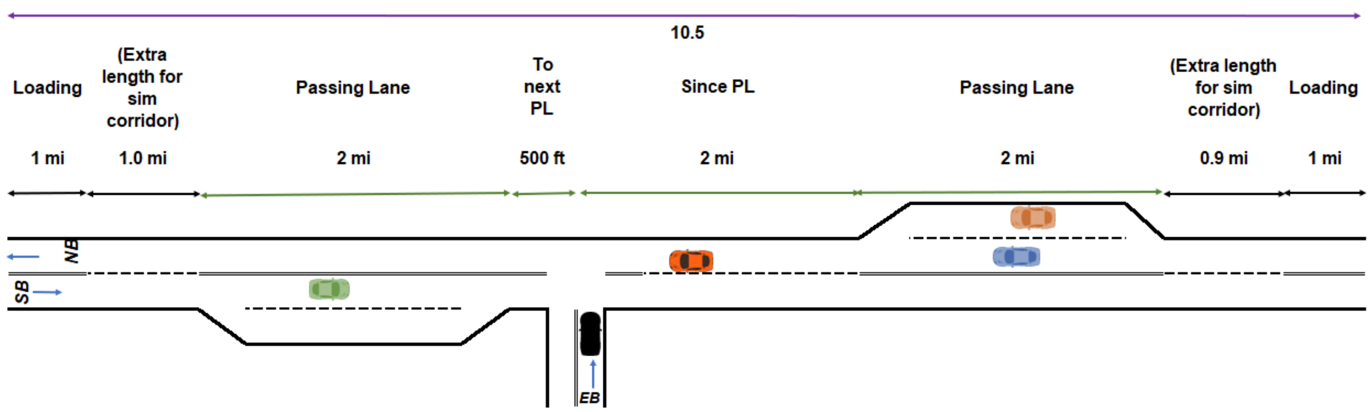


(b) Left-turn lane and passing lane ending near the intersection (Testbed 4.4)

Figure 19. Examples of 4.0 Series Testbeds—Intersections with a Left-Turn Lane within a Passing Lane with Varying Distances to the Start and End of the Passing Lane.



(a) Passing lane ending near the intersection (Testbed 5.2)



(b) Passing lane ending far from the intersection (Testbed 5.4)

Figure 20. Examples of 5.0 Series Testbeds—Passing Lane Starting and Ending at Varying Distances to the Intersection.

For each testbed, additional two-lane highway segments were added so that all testbeds were a consistent 10.5-mi length, resulting in the available passing zones varying from 3.6 mi to 8.0 mi. These two-lane undivided segments allowed passing in the opposite direction. The design of the passing lane configurations and the presence of the left-turn lanes allowed for evaluation of the incremental changes. For instance, the presence of a left-turn lane is the only difference between Testbed 1.1 and 2.1 (Figure 17), as well as the only difference between Testbed 3.2 and 4.2 (Figure 18 and Figure 19). Thus, the influence of the left-turn lane on delays can be compared for these two pairs of scenarios. The initial simulation efforts included 17 testbeds, and upon review of those findings, the research team decided to add a few more testbeds to investigate the impacts of when the passing lane is within a half-mile of the intersection (see Testbeds 3.6, 4.6, 5.6, and 5.7).

The traffic characteristics that varied in the simulation models included major-road daily volume, minor-road daily volume, percent trucks, and percent left turns from the major road onto the minor road. Specifically, the initial analysis involved the following:

- Truck percentages (10, 22, and 35).
- Left-turn percentages (5 and 10).
- Major approach ADT levels (10,000 vpd, 15,000 vpd, and 20,000 vpd).
- Minor approach ADT levels (500 vpd, 1,000 vpd, 1,500 vpd, and 2,000 vpd).

The major- and minor-road volume levels were selected with consideration of a typical peak-hour factor (10 percent), 50/50 directional split, and minor-road volumes that would satisfy the peak-hour traffic signal warrant. The truck percentage was based on the review of statewide truck percentages on rural two-lane highways, which ranged between 10 percent and 35 percent with an average of 22 percent.

The combinations for the initial efforts resulted in 72 scenarios for each testbed, or a total of 1,224 simulation scenarios. When evaluating the findings from the 1,224 scenarios, the research team decided to add a second round to investigate additional passing lane characteristics. Preliminary findings of the 35 percent trucks and 20,000 ADT showed extreme results. Therefore, the team adjusted the traffic characteristics to include three major-road ADT levels (10,000 vpd, 16,000 vpd, and 18,000 vpd) and two truck percentages (10 and 22).

Passing on the Shoulder Representation in VISSIM

On rural two-lane highways (Figure 17[a]), drivers will use the opposing lane to pass slower vehicles. When the slower or stopped vehicle is attempting to turn left at a driveway or intersection, drivers may use the shoulder to pass the vehicle blocking the travel lane. This study modified the simulation scenarios to reflect the passing-on-the-shoulder behavior to limit the overestimation of delay at a rural intersection. The research team used a partial vehicle route and formulaic relative flows tool in VISSIM to encode the shoulder use logic. A partial vehicle route enables the modeler to define a point on the through route of the simulated intersection where vehicles will decide whether to use the travel lane or travel on the shoulder for the segment around the intersection. The default route was the travel lane, and the formula used several factors to determine if the vehicle would take the shoulder route past the intersection. The formula employed four conditions to determine if the vehicle would use the shoulder route. First, the lead vehicle had to be on a route to turn at the intersection. Second, the vehicle using the shoulder had to be a passenger car, so trucks would never use the shoulder in the simulation. Third, the passenger car had to slow by at least 10 mph at the decision point. This rule was designed to prevent vehicles that could use the travel lane without slowing in response to the turning vehicle from using the shoulder. The final rule used a random number for the desired speed to ensure that only 80 percent of simulation drivers would consider using the shoulder. This rule existed because the research team expected that some travelers would not consider using the shoulder to pass the turning vehicle. The desired speed random number was used because the research team assumed this value to be correlated with aggressive drivers, so the 20 percent of vehicles that were least aggressive were the vehicles that would not consider the shoulder for passing. The vehicles that used the shoulder changed lanes to the shoulder 200 ft prior to the intersection, which was consistent with data collected from the field in this research effort. The decision point was placed 100 ft upstream of the location of the lane-change maneuver.

Scenarios and Simulation Runs

The combination of major- and minor-road ADT, percent left turns, and truck percentage resulted in 72 scenarios for each testbed included in the initial efforts and 48 scenarios for each testbed included in the second effort. Five simulation runs were performed for each combination with different random seeds. The simulation reflected the period of 2 p.m. to 6 p.m., with a 300-second warm-up period.

From the simulation output, the research team extracted the delay (seconds per vehicle) for cars and trucks on the corridor and at the intersection, along with the number of vehicles that passed through the corridor during the simulation. The number of vehicles recorded during the simulation could be less than the assumed major or minor volume values because of the lack of gaps for left-turning vehicles or the inability of some vehicles to pass slower-moving vehicles. The seconds of delay per vehicle was converted to represent the amount of delay that would be present in a typical day by multiplying seconds of delay per vehicle and number of vehicles. The intersection influence zone was defined as 1000 ft upstream and downstream of the T-intersection.

MODEL RESULTS AND DISCUSSION

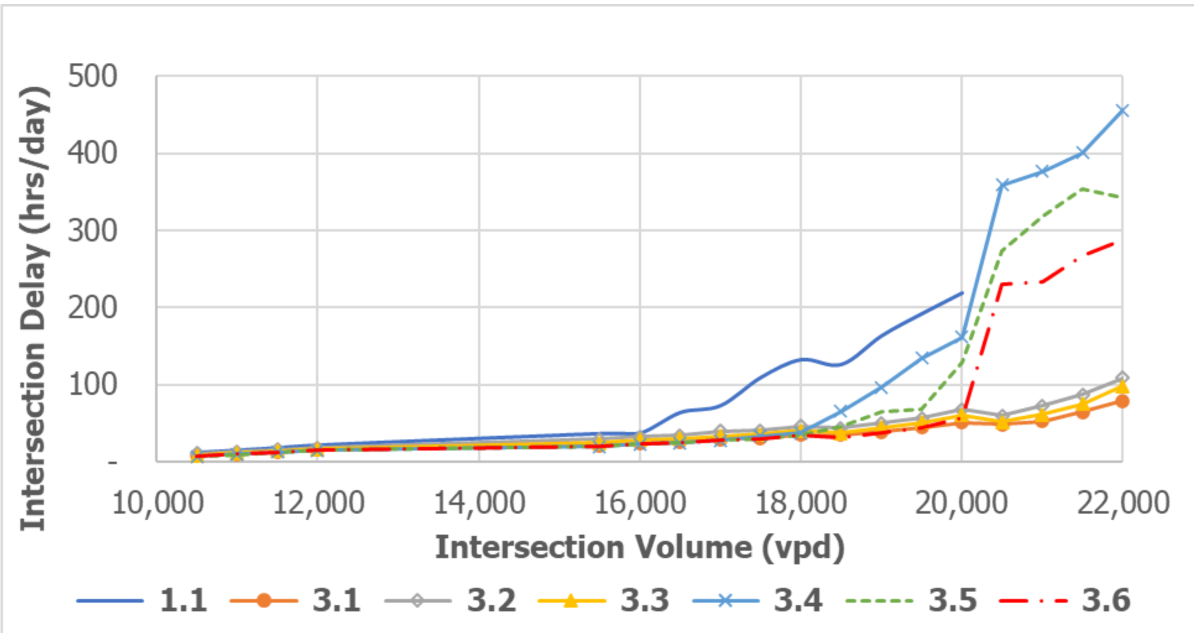
The simulation analysis focused on determining the operational impact of where a passing lane began or ended with respect to an unsignalized T-intersection along with adding a left-turn lane at the intersection. The key performance metric used to describe the performance along the corridor and at the intersection was total vehicle delay.

The initial efforts were to review the findings by key variable. Previous experience has demonstrated that changes to delay are minimal in non-congested conditions, and the interest of this project was at near-congested to congested conditions. The 35 percent trucks condition had an overwhelming impact on operations for many of the testbeds, so those delays were removed from the average calculations.

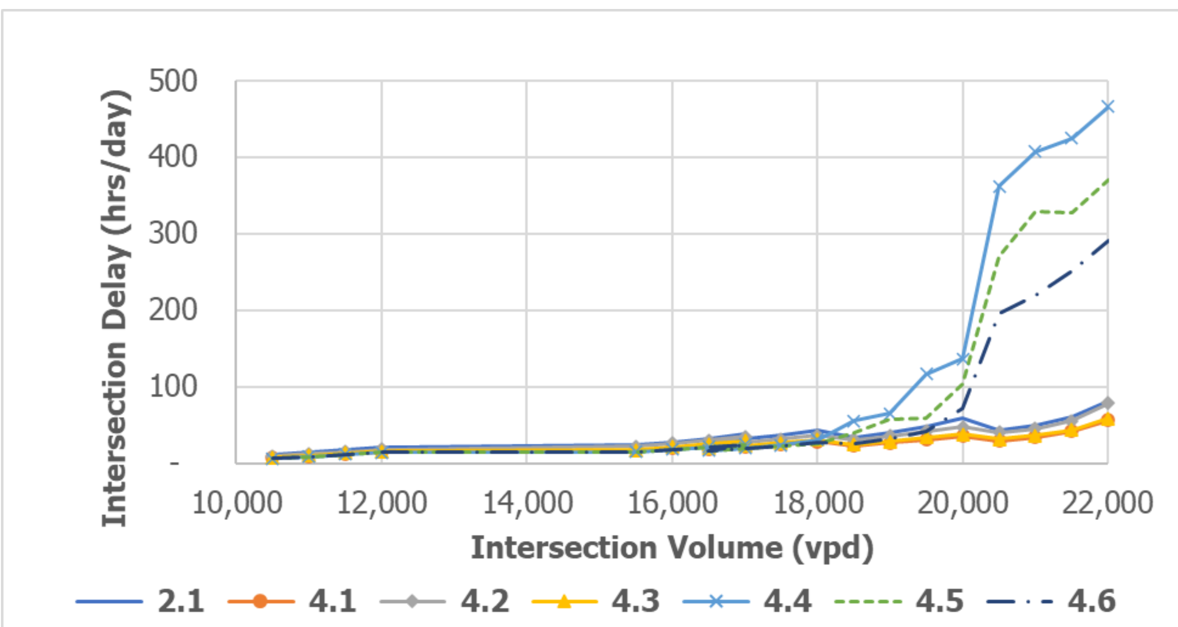
As expected, the addition of the left-turn lane (going from Testbed 1.1 to Testbed 2.1) decreased the amount of intersection delay estimated by the model.

Intersection Delay When a Passing Lane Is Added

Intersection delay from Testbed 1.1 (see Figure 17[a] for example) along with the Testbed 3.0 series (see Figure 18 for examples) was used to evaluate the impacts of the starting and ending points of a passing lane near an intersection without a left-turn lane. An overview of those results is shown in Figure 21(a). When the unsignalized intersection was within a passing lane and the passing lane ended within 1500 ft (Testbed 3.4 or 3.5), the delay at the intersection increased notably for intersection volumes greater than 17,000 vpd. When the passing lane ended within about 2640 ft of the intersection, the delay at the intersection increased notably for intersection volumes greater than 20,000. For scenarios below those volume ranges, the operational benefit of including a passing lane can be seen in Figure 21(a), with the delay curve for Testbed 1.1 being above the other testbed curves. For Testbeds 3.1 to 3.3, even with increased delay as the intersection volume increased, the intersection delay remained below the anticipated intersection delay if the passing lane was not present. Figure 21(a) shows the averaged results for the range of left-turn and for 10 and 22 percent trucks to provide a simpler graph for interpretation. The results for the various combinations were reviewed, and similar patterns for the various testbeds were identified—when a passing lane ended within 1500 ft of the intersection, and in higher-volume range within 2640 ft of the intersection, intersection delay was higher than if the passing lane was not present.



(a) No left-turn lane present at intersection



(b) Left-turn lane present at intersection

Figure 21. Intersection Delay Comparison for Scenarios with and without a Left-Turn Lane.

A similar comparison was done for when the unsignalized intersection had a left-turn lane present, as shown in Figure 21(b). The graph shows that the Testbed 4.4 to 4.6 scenarios were associated with much higher intersection delay. These testbeds represent the conditions when the passing lane was ending within a half-mile downstream of the T-intersection. A review of the simulation confirmed that having the merging of the through lanes and drivers attempting to turn left out of the minor road within 750 ft, 1500 ft, or 2640 ft should be avoided when the daily volume at the intersection is about 18,000 vpd and greater.

Intersection Delay When Both Passing Lane and Left-Turn Lane Are Added

The impact of adding both a passing lane and left-turn lane at the intersection was evaluated using simulation results from Testbeds 1.1, 2.1, 3.1, and 4.1 (Testbed 3.1 and 4.1 were used as the representatives of the Testbed 3 and 4 series). Testbed 2.1 had a left-turn lane only, Testbed 3.1 had a passing lane only, and Testbed 4.1 had both a passing lane and left-turn lane. Figure 22 shows the large benefit when either a left-turn lane only or a passing lane only was present at the intersection (i.e., the change from Testbed 1.1 to 2.1 or 1.1 to 3.1). Such configuration change was observed to reduce the intersection delay from about 220 hours per day to 60 hours per day for the largest intersection volume. Similar to the previous scenarios, significant benefits from the added lanes were observed when intersection volume was greater than 16,000 vpd. Conversely, the addition of a left-turn lane when the intersection was already within a passing lane (change from Testbed 3.1 to 4.1) had minimal incremental benefits. This observation was based on operations only. The safety trade-offs for having or not having a left-turn lane were not addressed in the simulation study.

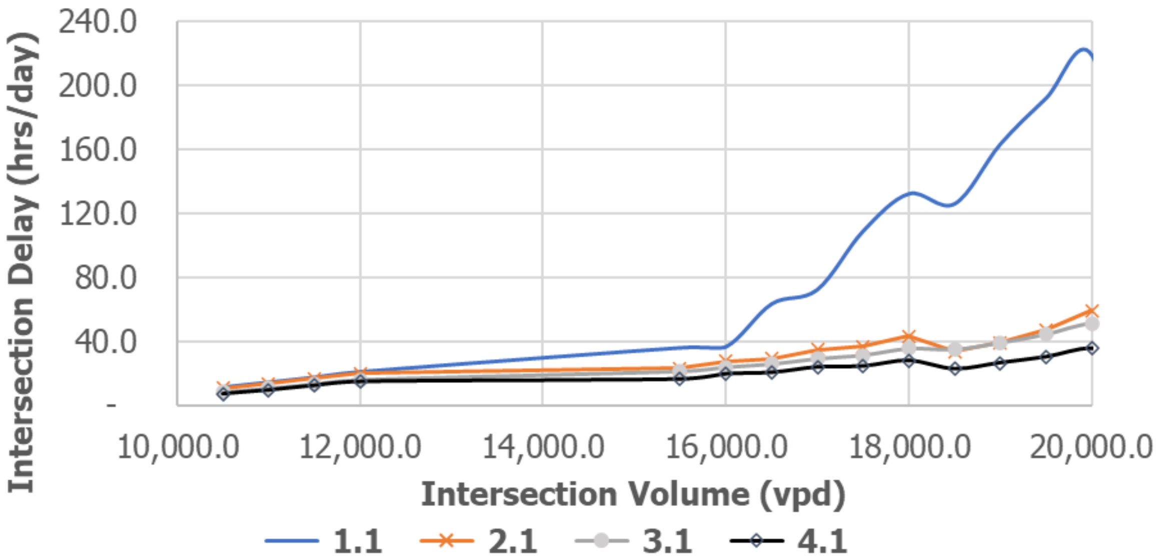
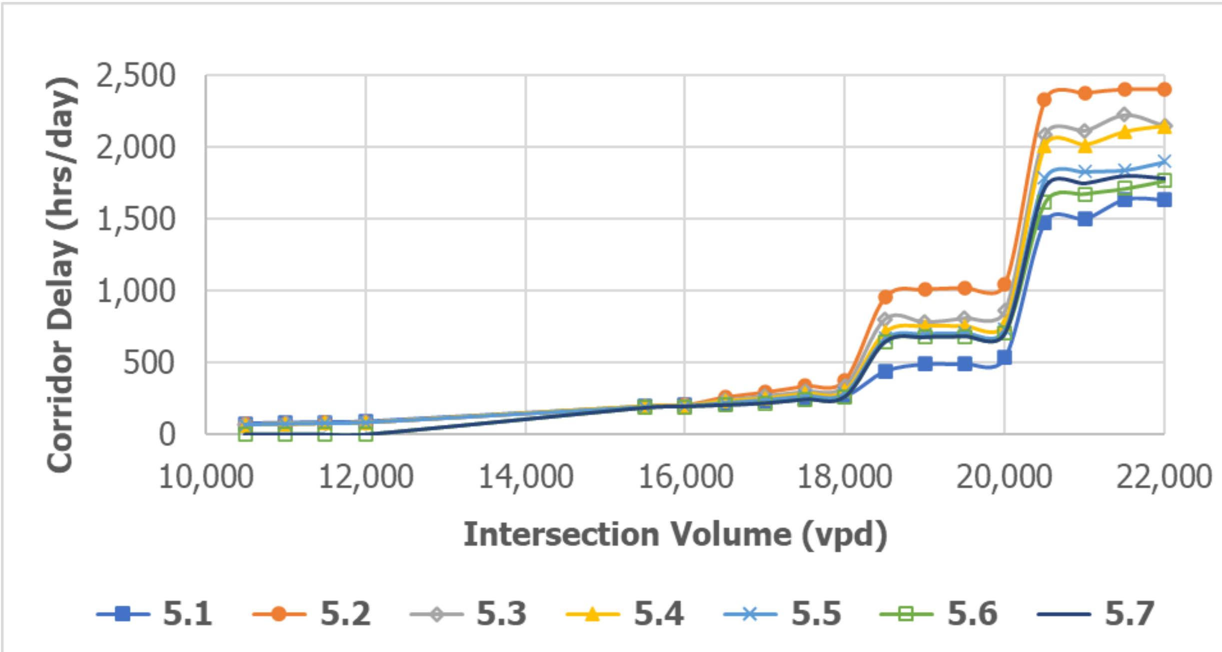


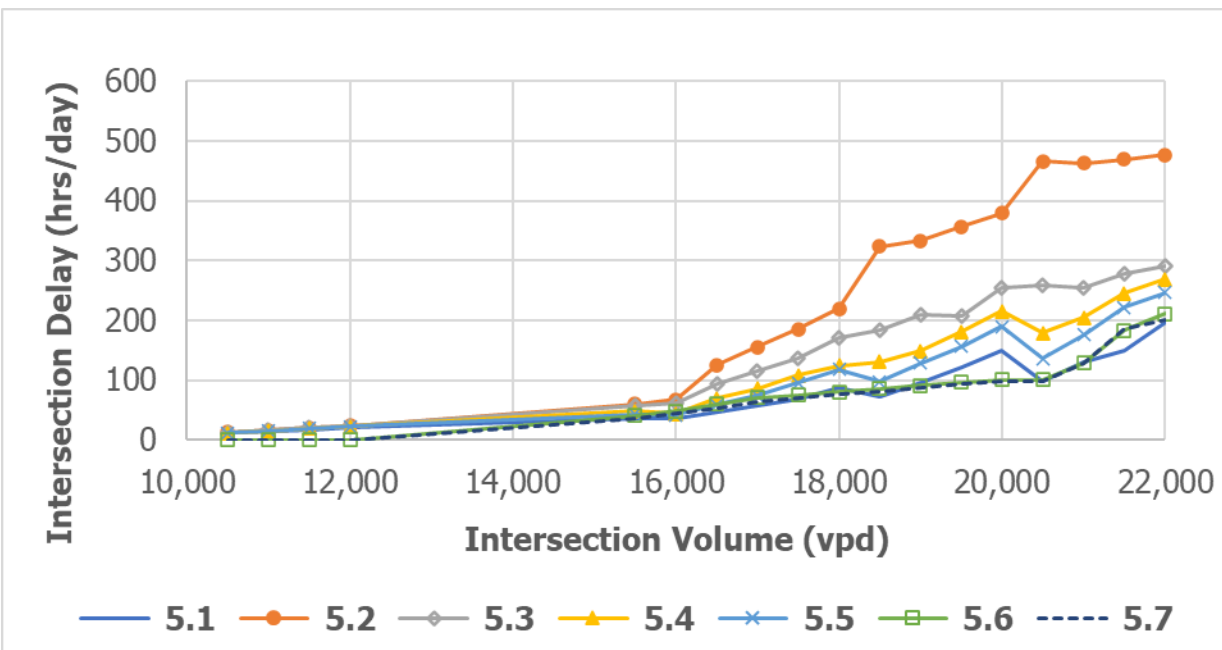
Figure 22. Intersection Delay for an Unsignalized Intersection with and without Passing Lane and Left-Turn Lane.

Intersection Delay and Corridor Delay for Multiple Passing Lanes near the Intersection

The testbeds that explored the impacts of having the intersection located between two passing lanes considered the distance between the intersection and the ending or starting points of the passing lanes for various distances (see Figure 20). As shown in Figure 23(a), the changes in corridor delay started to be noticeable when the major-road volume exceeded 18,000 vpd. When an intersection was present between two passing lanes, locating the intersection about equal distance from each passing lane resulted in a lower corridor delay. The highest corridor delay was shown for the scenario when the passing lane ended within 500 ft of the intersection. Figure 23(b) shows the intersection (rather than the corridor) delay for the same testbeds, with similar results as the corridor delay. The smallest delays were when the intersection was located at an equal distance between the two passing lanes (Testbed 5.1), while the greatest delay was for the testbed with the passing lane that ended within 500 ft of the intersection (Testbed 5.2).



(a) Corridor delay for Super 2 with unsignalized intersection



(b) Intersection delay within a Super 2

Figure 23. Intersection and Corridor Delay for an Unsignalized Intersection within a Super 2 Corridor with Different Beginning and Ending Distances for the Passing Lanes.

CHAPTER 5: BENEFIT-COST ANALYSIS

A BCA compares the amount of benefits for a project to the cost of the project over a specified amount of time. The BCR is simply the total benefits derived from the project divided by the total cost of the project, where a BCR greater than 1.0 means the benefits of the project outweigh the costs. For this TxDOT project, the BCA compared two scenarios: the no-build (or base) scenario represented by Testbed 1.1 and the build (or project) scenario represented by all other testbeds considered in this project. The BCA assumed a 20-year operating period, a 10-mi project length, and a 2-year construction period. All costs were converted to 2018 dollars to be consistent with default factors, and researchers considered both undiscounted and a 3 percent discount rate.

BENEFIT-COST ANALYSIS

The following outputs from the simulation were used as inputs to the BCA model for each comparison:

- ADT on the major highway and on the minor intersection approach.
- Percent left-turn vehicles.
- Percent trucks.
- Total number of vehicles.
- Total delay in hours for passenger vehicles.
- Total delay in hours for trucks.

Researchers estimated costs for the following benefits:

- Vehicle operating cost savings.
- Business and personal time cost savings.
- Safety benefits.
- Environmental benefits.
- Construction Costs.

The following sections describe the values used in the BCA.

Vehicle Operating Cost Savings

Vehicle operating costs include fuel, purchase payments, insurance premiums, tires, maintenance, and repairs. The net change in vehicle operating costs is the change in operating costs from the project to the base scenario. Vehicle operating cost is the cost per hour of operating a passenger vehicle or commercial truck. The base operating cost includes maintenance, tires, mileage-based depreciation, and insurance.

The hourly fuel operating cost is also calculated using fuel prices per gallon obtained from the Energy Information Administration (EIA) and a vehicle-gallons-consumed-per-hour factor obtained from the Transportation Economic Development Impact System (TREDIS):

- Base operating cost (truck and passenger) = (hours of delay × vehicle operating cost per hour).
- Fuel operating cost (truck and passenger) = (hours of delay × gallons per hour) × fuel cost per gallon.

Default time value factors from the U.S. Department of Transportation (USDOT) BCA Guidance (*52*) used to calculate these benefits are listed in Table 19.

Table 19. Time Values.

Time/Value Factors	Value
Crew cost factor ($/hr per crew member, 2018$)	$29.50
Passenger cost factor ($/hr per occupant, 2018$)	$16.60
Crew per truck	1.00
Passengers per vehicle	1.67

Source: USDOT (*52*).

Business and Personal Time Cost Savings

Business time cost savings are the business cost of labor for professional drivers and paid crew. Personal time cost savings are the valuation of the average passenger's time. The value of time savings is the crew cost for trucks and the personal time cost for passenger vehicles saved due to a reduction in delay between the project scenario and the base scenario. Time savings are calculated by multiplying the number of crew members or passengers per vehicle by the crew—or passenger-cost-per-hour factor for each crew member or passenger—and then multiplying by the hours of delay in each scenario:

- Business time cost = (number of crew per truck × crew cost per hour per crew member) × truck hours of delay.
- Personal time cost = (passengers per vehicle × passenger cost per hour per passenger) × passenger vehicle-hours of delay.

USDOT-recommended values were used for crew and personal cost factors as well as the number of crew members or passengers per vehicle. Default factors used to calculate these benefits are listed in Table 20.

Table 20. Sources for Per-Vehicle Cost Calculations.

Per-Vehicle Cost Factors	Value	Source/Notes
Environmental cost $/hr (truck), 2018$	$0.48	Based on TREDIS emission rates and USDOT BCA guidance emission costs (*52*)
Environmental cost $/hr (passenger), 2018$	$0.07	
Vehicle operating cost ($/hr) (passenger), 2018$	$4.92	*AAA's Your Driving Cost* publication; includes maintenance, repair, and tires (@ 55 mph)
Vehicle operating cost ($/hr) (truck), 2018$	$24.11	American Transportation Research Institute hourly cost value less fuel costs, driver wages, and driver benefits for 2018 (2019 publication)
Truck gallons per hour	9.84	TREDIS; assumes average speed of 50 mph
Passenger gallons per hour	1.83	TREDIS; assumes average speed of 35 mph
Truck $ per gallon, 2018$	$2.81	EIA's Petroleum & Other Liquids: https://www.eia.gov/dnav/pet/pet_pri_gnd_a_epd2d_pte_dpgal_a.htm
Passenger $ per gallon, 2018$	$2.35	EIA's Petroleum & Other Liquids: https://www.eia.gov/dnav/pet/pet_pri_gnd_dcus_r30_a.htm
Fuel cost/hr (truck), 2018$	$27.65	Brewer et al. (*5*)
Fuel cost/hr (passenger), 2018$	$4.30	Brewer et al. (*5*)

Safety Benefits

Safety benefits result from the reduction in the number of predicted annual crashes from the base scenario to the project scenario. First, the vehicle miles traveled (VMT) was estimated annually using the selected ADT and the estimated 10-mi project length. A 2 percent annual growth rate was applied for the 20-year operational period. Fatalities and injuries for the base case were determined using the HSM (*7*) prediction equation for two-lane rural highways. A CMF was applied to determine the reduction in fatalities and injuries between the two scenarios. The CMFs used are provided in Table 21. The number of reduced fatalities and injuries was multiplied by the associated cost to determine the total safety cost reduction (see Table 22 for the assumed crash costs).

Table 21. Sources for Crash Modification Factor Calculations.

Description	CMF	Source
Install left-turn lane on one major-road approach	0.56	*Highway Safety Manual* and Crash Modification Factors Clearinghouse: http://www.cmfclearinghouse.org/index.cfm Harwood et al. (*21*)
Install passing lane	0.67	Crash Modification Factors Clearinghouse: http://www.cmfclearinghouse.org/index.cfm Bagdade et al. (*53*)
Install Super 2 highway (two passing lanes)	0.58	Crash Modification Factors Clearinghouse: http://www.cmfclearinghouse.org/index.cfm Park et al. (*29*)
Covert injuries/ fatalities to crashes	NA	*FHWA Crash Costs for Highway Safety Analysis:* https://safety.fhwa.dot.gov/hsip/docs/fhwasa17071.pdf

Table 22. Sources for Safety Cost Calculations.

Safety Costs	Value	Source/Notes
Injury crash, 2018$	$250,600	USDOT BCA Guidance, January 2020
Fatal crash, 2018$	$10,636,600	
Fatalities per 100 million VMT (2017)	1.16	NHTSA's Traffic Safety Facts Annual Report Tables: https://cdan.nhtsa.gov/tsftables/tsfar.htm#
Injured persons per 100 million VMT (national rate 2017)	85	NHTSA's Traffic Safety Facts Annual Report Tables: https://cdan.nhtsa.gov/tsftables/tsfar.htm#

Note: NHTSA = National Highway Traffic Safety Administration.

Environmental Benefits

Environmental factors include the cost savings of air pollution and greenhouse gases per vehicle-hour of travel. The net change in environmental costs is the change in environmental costs from the project to the base scenario. This cost includes volatile organic compounds (VOCs), nitrogen oxides (NO_x), particulate matter (PM), sulfur dioxide (SO_2), and carbon dioxide (CO_2). Emission rates were obtained from TREDIS and are shown in Table 23. The cost per ton of each of these emission types was obtained from USDOT, and the assumed values are shown in Table 24. The environmental cost per hour for trucks and passenger vehicles was calculated by multiplying each hourly emission rate by the emission cost and then summing each type of emission cost per hour to calculate a total environmental cost per hour. This value was then multiplied by the base case and project scenario hours of delay:

- Environmental cost per hour = hourly emission rate × emission cost.
- Environmental cost = hours of delay × environmental cost per hour.

Table 23. Sources for Emission Rate Calculations.

Emissions Rates	Value	Source/Notes
VOC (Truck)	0.00000184	Passenger cars and light trucks, medium-duty trucks, heavy-duty trucks, and buses are based on the U.S. Department of Energy's (DOE's) AFLEET 2018 model. AFLEET 2018 provides state-specific emissions rates that are collapsed to national rates using registration data for each state as reported in Federal Highway Statistics 2017. AFLEET values are based on the most recent version of the U.S. Environmental Protection Agency's MOVES and analysis prepared by DOE. For cars and light trucks, fleet composition and emissions are assessed using survival rates and mileage-based exposure factors used by NHTSA in rulemaking documents and sales volumes from *Ward's Automotive Handbook*. For medium-duty trucks, the average model year (MY) 2018 vehicle is assessed based on its expected emissions after 5 years of use. For heavy-duty trucks and buses, the average MY 2018 vehicle is assessed based on its expected emissions after 10 years of use. These time frames represent roughly the average age of vehicles in these classes.
NO_x (Truck)	0.00000417	
SO_x (Truck)	0.00000004	
PM (Truck)	0.00000114	
VOC (Passenger)	0.00000112	
NO_x (Passenger)	0.00000093	
SO_x (Passenger)	0.00000000	
PM (Passenger)	0.00000016	

Table 24. Emission Costs.

Emission Costs	2018$
VOC	$2,100
NO_x	$8,600
SO_x	$50,100
PM	$387,300
CO_2	$43
Environmental cost/hr (truck)	$0.48
Environmental cost/hr (passenger)	$0.07

Source: USDOT (*52*).

Construction Costs

The construction costs reflected construction costs to install a left-turn lane, a passing lane, or two passing lanes within the 10-mi corridor. The construction costs reflected costs reported in the TxDOT 0-6997 report (*5*). In that project, researchers requested data from several TxDOT districts regarding recently completed construction projects that included left-turn lane or passing lane installations. Table 25 lists the assumed construction costs considered in this BCA.

Table 25. Construction Costs for Each Testbed.

Description	Relevant Test Beds	Construction Cost, 2018$
Base conditions (no passing lane and no left-turn lane at intersection)	1.1	$0
Add left-turn lane at unsignalized intersection	2.1	$350,000
Add Super 2 for the 10-mi corridor (one passing lane)	3.1, 3.2, 3.3, 3.4, 3.5, 3.6	$1,045,822
Add both Super 2 and LTL (one passing lane, one left-turn lane)	4.1, 4.2, 4.3, 4.4, 4.5, 4.6	$1,395,822
Add Super 2 for the 10-mi corridor (two passing lanes)	5.1, 5.2, 5.3, 5.4, 5.5, 5.6, 5.7	$2,091,645

FINDINGS

For the range of ADTs considered, the addition of a left-turn lane or passing lanes almost always resulted in a positive BCR. A negative BCR indicates that the treatment would result in fewer benefits compared to the baseline. The few comparisons with a negative BCR were when the corridor was congested and only occurred when the assumed major-road ADT was 20,000 and the percent of trucks was 22 or 35 percent. In all other comparisons, the BCR was much greater than 1.

Table 26 provides a sample of the BCR results. The sample includes both 10,000 and 15,000 major-road ADT, and 5 and 10 percent left-turn vehicles and 10 and 22 percent trucks. When adding a left-turn lane to the rural intersection (i.e., Testbed 2.1 compared to Testbed 1.1) with 10,000 major-road ADT and 1000 on the minor road with 5 percent left turns and 10 percent trucks, the treatment results in a BCR of 21.2. A BCR of 21.2 says that the benefit of the treatment is 21.2 times the cost of the treatment.

Table 26. Sample of BCA Results.

Major-Road ADT	10,000	10,000	10,000	10,000	15,000	15,000	15,000	15,000
Minor-Road ADT	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**
Left Turn %	**5%**	**5%**	**10%**	**10%**	**5%**	**5%**	**10%**	**10%**
Truck %	**10%**	**22%**	**10%**	**22%**	**10%**	**22%**	**10%**	**22%**
Testbed Comparison	**Benefit-Cost Ratio, 3% Discount**							
1.1 to 2.1	21.2	21.8	22.3	24.4	32.2	37.9	39.7	38.1
1.1 to 3.1	59.8	60.2	59.9	60.8	90.5	93.1	92.2	91.9
1.1 to 3.2	59.2	59.5	59.3	60.0	88.5	90.5	90.3	88.6
1.1 to 3.3	59.6	60.0	59.7	60.4	89.8	91.9	91.1	89.8
1.1 to 3.4	59.9	60.3	60.1	61.0	91.1	93.2	92.4	91.5
1.1 to 3.5	60.0	60.4	60.1	61.0	91.1	93.2	92.5	91.3
1.1 to 3.6	59.9	60.3	60.1	60.9	91.0	93.2	92.4	91.3
1.1 to 4.1	50.3	50.6	50.5	51.1	75.8	77.8	77.4	77.4
1.1 to 4.2	49.8	50.0	50.1	50.6	74.7	76.3	76.4	76.0
1.1 to 4.3	50.2	50.4	50.3	50.9	75.5	77.4	77.2	77.0
1.1 to 4.4	50.4	50.6	50.7	51.4	76.2	78.0	77.8	77.7
1.1 to 4.5	50.5	50.7	50.7	51.3	76.3	78.3	78.0	78.0
1.1 to 4.6	50.5	50.8	50.6	51.4	76.2	78.2	77.8	77.8
1.1 to 5.1	38.8	38.7	38.8	38.9	58.1	58.5	57.8	58.5
1.1 to 5.2	38.6	38.3	38.6	38.4	56.2	51.2	55.8	51.2
1.1 to 5.3	38.6	38.3	38.6	38.4	56.0	52.3	56.3	52.3
1.1 to 5.4	38.7	38.6	38.7	38.6	57.5	57.1	56.5	57.1
1.1 to 5.5	38.7	38.6	38.6	38.5	57.4	57.5	57.1	57.5
1.1 to 5.6	21.2	21.6	21.3	22.0	58.0	57.3	57.3	52.8
1.1 to 5.7	32.3	32.8	32.5	33.2	58.2	57.5	57.6	54.1

CHAPTER 6: GUIDELINES

INTRODUCTION

Texas has many two-lane rural roadways, which typically serve lower volumes of traffic. As volumes increase, treatments are needed to improve capacity and promote safety. Relatively lower-cost improvements include the provision of passing lanes, turn lanes, and localized alignment improvements. Passing lanes are one of the most effective methods of improving the level of service on a two-lane roadway because they increase passing opportunities and provide smoother traffic operations with fewer vehicle-vehicle conflicts (*54*). Passing lanes allow motorists the opportunity to pass slower vehicles safely and easily, improving traffic flow at a much lower cost than a traditional expansion to four lanes. Additionally, safety evaluations have shown that passing lanes and short four-lane sections reduce crash rates below the levels found on conventional two-lane highways (*55*). When the area of concern is an unsignalized intersection, the addition of a left-turn lane has been shown to be highly effective to improve both operations and safety (*14*, *15*).

The use of passing lanes on rural two-lane highway corridors is known in Texas as a Super 2 design. Passing lanes are a common treatment on two-lane roadways to improve overall traffic operations by breaking up traffic platoons and reducing delays caused by inadequate passing opportunities over substantial lengths of roadway. Passing lanes on a two-lane roadway are often much more cost-effective in providing passing opportunities than continuous four-lane sections because locations with high construction costs (e.g., major earthwork, expensive structures) can be avoided (*56*). Judicious use of Super 2 corridors on rural two-lane highways can increase capacity, reduce delay, increase average speeds of through vehicles, and reduce crashes (*2, 3, 5*).

While the Super 2 concept is used across the state to provide passing opportunities and increase capacity on rural two-lane roads, passing lanes are a treatment for through vehicles and do not address the needs of turning vehicles. Current guidance in the TxDOT RDM (*1*) provides the following discussion:

> When evaluating the termination of a passing lane at an intersection, consideration should be given to traffic operations turning and weaving movements, and intersection geometrics. If closure of the passing lane at the intersection would result in significant

operational lane weaving, then consideration should be given to extending the passing lane beyond the intersection.

Where an access point does exist within a passing lane, a left-turn maneuver involves multiple variables. Perhaps primary among those variables is whether the left-turning vehicle is turning from a through lane, a passing lane, or a left-turn lane, and how the immediate cross-section is designed to accommodate the left-turning vehicle, the through vehicle, the passing vehicle, and the opposing vehicle. As a result, there are numerous at-grade access points on rural high-speed roads that introduce potential operational delays and safety concerns, which may be compounded if passing lanes are added in locations with generators of large volumes of turning traffic.

PURPOSE

The purpose of these guidelines is to provide a single source of information for practitioners to use when considering whether to install turn lanes at major volume generators along a Super 2 corridor or other high-speed, high-volume, two-lane road, and, upon making the decision to install, determining the most appropriate design for that location. References to research and guidance documents are provided for the practitioner to obtain more detailed information.

DESCRIPTION OF RURAL TWO-LANE HIGHWAYS WITH TURN LANES

This section describes characteristics of rural two-lane highways and features related to provision for turning at intersections and driveways on those highways.

Characteristics of Rural Two-Lane Highways

Rural highways can have cross-sections that range from two-lane roadways to multilane, divided, controlled-access highways. Two-lane highways provide the majority of centerline miles in rural areas, and while they serve lower volumes than multilane highways, they are still intended to facilitate high-speed, longer-distance travel to provide connections between towns and urban centers.

The RDM (*1*) provides general geometric characteristics for rural two-lane highways in Chapter 3, Section 4. Among those characteristics are the following key features for arterials:

- Minimum design speed of 60 to 70 mph.
- Minimum lane width of 12 ft, with minimum shoulder width increasing from 4 to 10 ft as future ADT increases from 400 to 2000.
- Curvature, grade, SSD, and passing sight distance commensurate with other rural roadways with similar speeds.

The AASHTO *Green Book* (*24*) states that the choice of an appropriate design level of service is made by the road agency. It further advises that, for acceptable degrees of congestion, rural arterials in level and rolling terrain should generally be designed for level of service B. Changes in horizontal and/or vertical alignment should be sufficiently gradual to avoid surprising the driver. Minimum radii should be used sparingly, and short horizontal curves should be avoided.

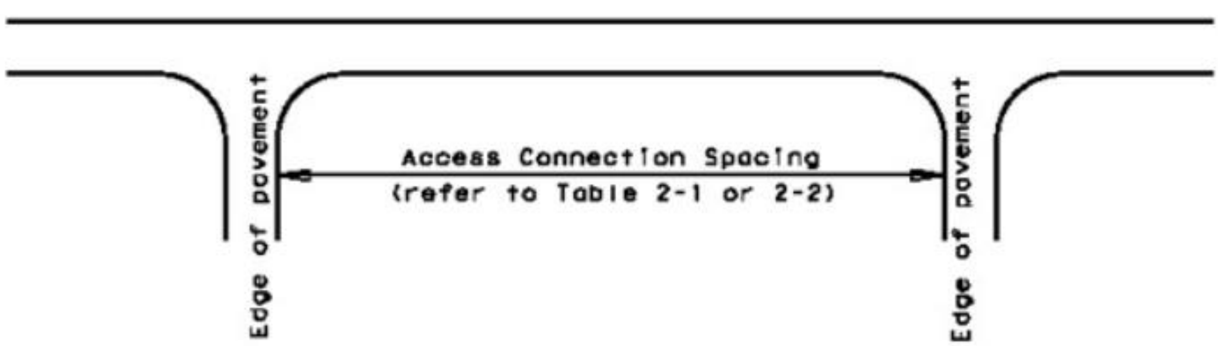
Alignment and profiles of two-lane, two-way rural highways should provide sections for passing at suitable intervals. The horizontal and vertical alignment on level or rolling terrain should provide adequate passing distance over a large proportion of the highway. Where it is not practical to achieve sufficient passing sight distance in the alignment, or where traffic volumes reduce the number of available passing opportunities, passing lanes should be considered as a treatment to obtain the desired level of service.

The RDM (*1*), in Chapter 4, Section 6, describes a Super 2 highway as one in which a periodic passing lane is added to a two-lane rural highway to allow passing of slower vehicles and dispersal of traffic platoons. Passing lanes are provided in both directions along a given corridor, and considerations for adding Super 2 passing lanes include a significant amount of slow-moving traffic; limited sight distance for passing; and/or existing traffic volume exceeding the two-lane highway capacity, creating the need for vehicles to pass on a more frequent basis. As noted in Chapter 1, the RDM mentions that the Super 2 designer should consider the trade-offs of terminating a passing lane at an intersection compared to extending the lane through the intersection.

Access Management Principles

The AASHTO *Green Book* (*24*) refers to NCHRP Report 348 (*12*) to describe access management as providing or managing access to land development while simultaneously preserving traffic flow (e.g., capacity, speed, low crash frequency and severity) on the surrounding road system. Certain highways (e.g., freeways) are access controlled and have few access points. For rural two-lane highways, access is allowed more frequently, both for driveways and intersecting roadways.

The TxDOT *Access Management Manual* (AMM) (*57*) describes benefits of access management that include reductions in crash frequency and severity, improvements in travel time and congestion, and lower construction costs. The TxDOT AMM provides standards for number, location, and spacing of access connections for a variety of roadway types. The distance between access connections is measured along the edge of the traveled way from the closest edge of pavement of the first access connection to the closest edge of pavement of the second access connection, as shown in Figure 24. For all state highway system routes that are not new highways on new alignments, freeway main lanes, or frontage roads (including existing rural two-lane highways), the standard minimum distance between access points is 425 ft when the posted speed limit is 50 mph or higher.



Source: TxDOT AMM (*57*), Figure 2-1.

Figure 24. Access Connection Spacing Diagram.

The TxDOT AMM states that arterials provide a level of mobility second only to freeways and are thus intended to carry substantial amounts of traffic over relatively long distances and at relatively high speeds. While variances can be sought to the minimum 425-ft distance, those should be the exception. As a result, the broad application is that direct property access to arterials may be provided, but it must be carefully managed to preserve arterial mobility and avoid creating unsafe and congested traffic operations. Collectors and local streets provide increasing levels of access, and the emphasis on mobility is correspondingly reduced.

Because Super 2 highways emphasize long-distance connections to travel from city to city, they generally coincide with rural arterials, with a higher emphasis on mobility than access; however, driveways and intersections can be commonly found on Super 2 sections. While traffic volumes at those access points may be low, the turning movements into and out of those access points can affect operations and safety on the Super 2, which may mean that those intersections are suitable for consideration for additional treatments, such as auxiliary lanes, to facilitate safer and more efficient operations for both turning and through traffic.

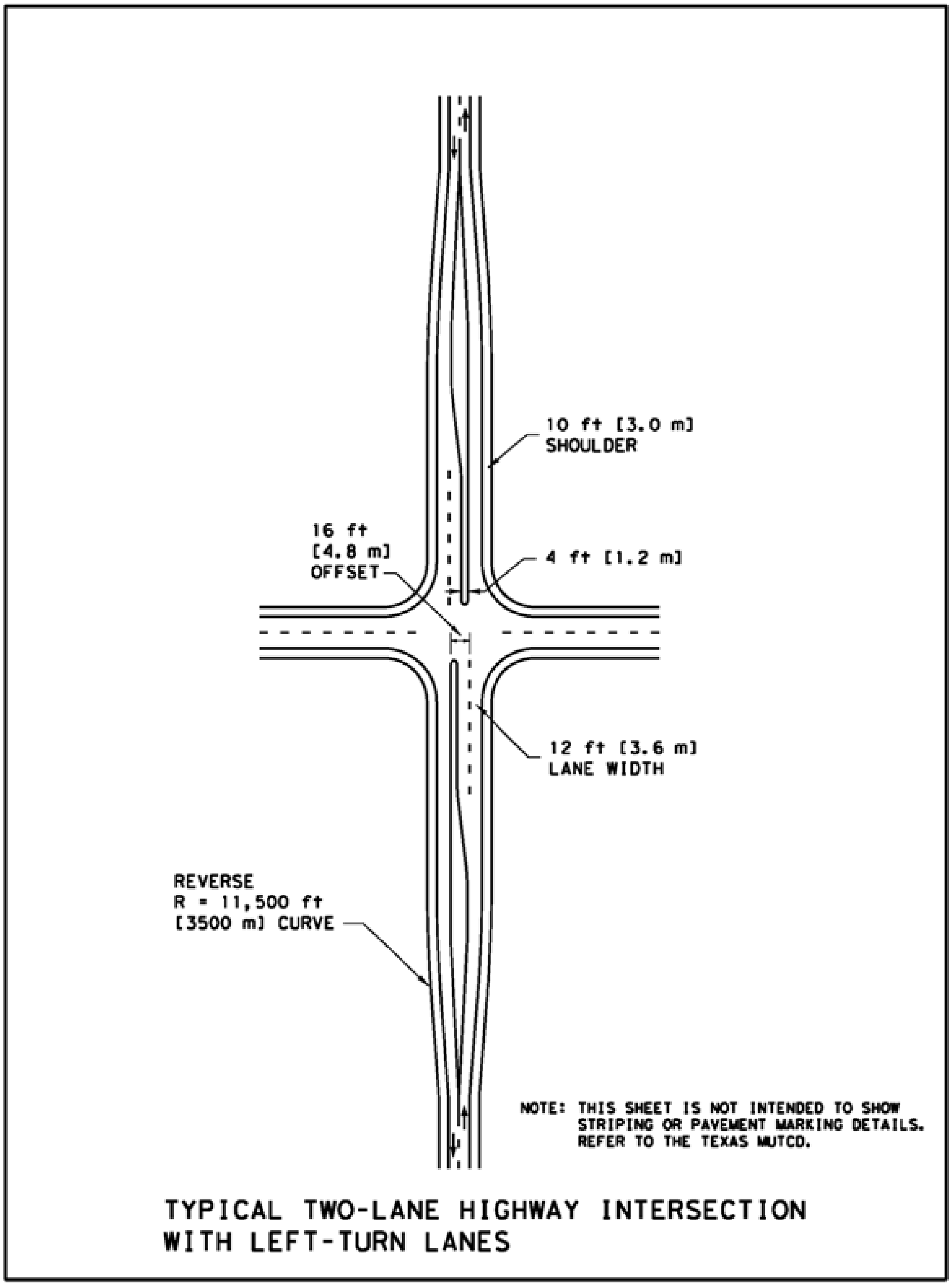
Intersection Auxiliary Lanes

The TxDOT AMM (*57*) describes an auxiliary lane as a lane "striped for use as an acceleration lane, or deceleration lane, right-turn lane, or left-turn lane, but not for through traffic use." Speaking specifically of left-turn lanes, the AMM states that left turns may pose challenges at driveways and street intersections; they may increase conflicts, delays, and crashes and often complicate traffic signal timing, particularly at major highway intersections. To mitigate those effects, the AMM and the RDM describe the use of left-turn lanes at these access points. Figure 25 shows typical geometry for a rural two-lane highway with left-turn bays at a crossroad intersection.

For undivided roads (i.e., those with non-traversable medians), the AMM refers to the RDM for thresholds for considering a left-turn deceleration lane. Table 3-11 in the RDM (*1*) provides guidelines for considering left-turn deceleration lanes based on speed and volume; that table is reproduced below as Table 27. The RDM adds that, where used, left-turn lanes should be delineated with striping and pavement markers or jiggle bars. It advises that passing should be restricted in advance of the intersection, and horizontal alignment shifts of the approaching travel lanes should be gradual.

The AASHTO *Green Book* (*24*) provides additional guidance for intersection auxiliary lanes in Section 9.7, stating that left-turning traffic should be removed from the through lanes whenever practical in designing an intersection. It adds that left-turn lanes should ideally be provided at driveways and street intersections along major arterial and collector roads wherever left turns are permitted. Left-turn facilities should be established on roadways where traffic volumes are high enough or crash histories are sufficient to warrant them, and they are often needed to provide adequate service levels for the intersections and various turning movements.

Regardless of the treatment, consideration of traffic demand, delay savings, crash reduction, and construction costs are all key factors in determining whether to install a left-turn lane or a bypass lane. Research on left-turn accommodations at unsignalized intersections (*13*, *14*) produced warrants for the installation of left-turn lanes and bypass lanes that account for those factors. Those warrants, which have been incorporated as guidelines in the AASHTO *Green Book* (*24*), are shown in Table 28 and Figure 26; while they do not represent requirements for left-turn lanes, they do indicate conditions where a left-turn lane should be provided, with consideration of site-specific conditions as discussed in Chapter 3.



Source: RDM (*1*), Figure 3-6.

Figure 25. Example of Left-Turn Lanes at an Intersection on a Two-Lane Highway.

Table 27. Guide for Left-Turn Lanes on Two-Lane Highways.

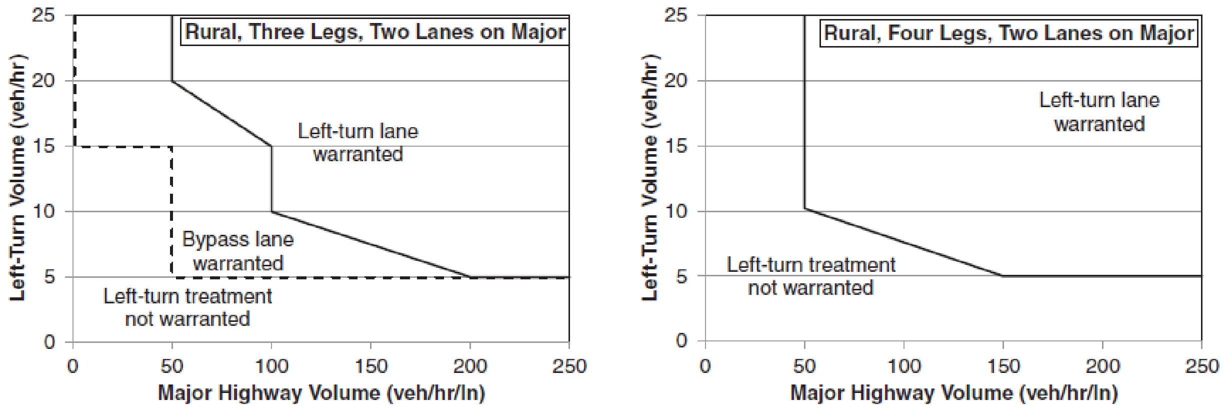
Opposing Volume (vph)	Advancing Volume (vph)			
	5% Left Turns	10% Left Turns	20% Left Turns	30% Left Turns
40 mph [60 km/h] Design Speed				
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
50 mph [80 km/h] Design Speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
60 mph [100 km/h] Design Speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

Source: RDM (*1*), Table 3-11.

Table 28. Suggested Left-Turn Treatment Warrants Based on Results from Benefit-Cost Evaluations for Rural Two-Lane Highways, Tabular.

Left-Turn Lane Peak-Hour Volume (veh/hr)	Three-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Bypass Lane	Three-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane	Four-Leg Intersection, Major Two-Lane Highway Peak-Hour Volume (veh/hr/ln) That Warrants a Left-Turn Lane
5	50	200	150
10	50	100	50
15	<50	100	50
20	<50	50	<50
25	<50	50	<50
30	<50	50	<50
35	<50	50	<50
40	<50	50	<50
45	<50	50	<50
50 or more	<50	50	<50

Source: Fitzpatrick et al. (*13*, *14*).



Source: Fitzpatrick et al. (*13, 14*).

Figure 26. Suggested Left-Turn Treatment Warrants Based on Results from Benefit-Cost Evaluations for Intersections on Rural Two-Lane Highways, Graphical.

CONSIDERATIONS FOR TURN LANES WITHIN A PASSING LANE OR WITHIN A SUPER 2 CORRIDOR

Turn lanes are commonly used at access points to provide additional benefits in safety and operations for both turning and through vehicles. Existing policy documents describe broad guidelines for their use, and warrants for installing left-turn lanes are provided in Chapter 2. However, there are additional considerations that should also be included in the process for the installation of a turn lane at an intersection that is located within the bounds of a passing lane.

Considerations for Providing and Designing Turn Lanes

The AASHTO *Green Book* (24) advises that deceleration lanes are advantageous on higher-speed roads because in the absence of such a lane, the driver of a vehicle leaving the highway has no choice but to slow down in the through-traffic lane. Similarly, if there is not a suitable gap in oncoming traffic when the turning vehicle arrives at the intersection, the turning vehicle must remain stopped in the through lane until a gap is available if left-turn storage is not provided. This can increase the chances of rear-end collisions if following drivers fail to slow down accordingly, and it can increase the chances of left-turn crashes if turning drivers become impatient and/or uncomfortable waiting on a suitable gap, particularly on roads with higher operating speeds and/or higher volumes.

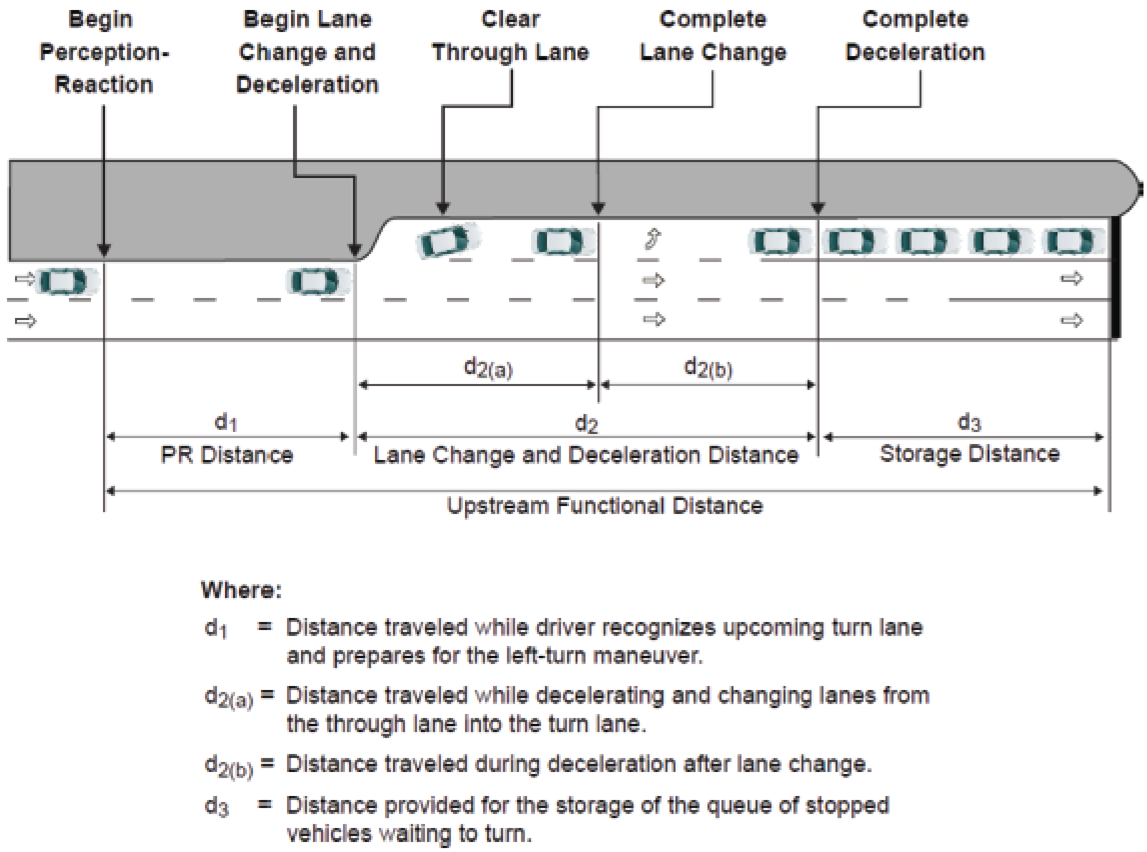
Local conditions and right-of-way costs often influence the characteristics of an intersection. For example, sight distance may affect the choice of traffic control and the decision to include a turn lane, even when traffic densities are less than those ordinarily considered appropriate for those features. Although the RDM (*1*) states that left-turn lanes on two-lane highways at intersecting crossroads generally are not economically justified, it notes that they may be justified based on reduced crash costs for certain moderate- or high-volume two-lane highways with heavy left-turn movements. The alignment and grade of the intersecting roads and the angle of intersection may make it advisable to channelize or use auxiliary pavement areas, regardless of the traffic densities. In general, level of service, functional classification, physical conditions, and available right-of-way are all considered when choosing the features of an intersection.

When a left-turn lane is provided, the design should consider the upstream functional area of the intersection in relation to the necessary length. Recent research (*13, 14*) and the AASHTO *Green Book* (*24*) offer guidance on providing a left-turn lane design that includes length for perception-reaction (PR) distance, lane-change and deceleration distance, and queue storage distance, as shown in Figure 27. Desirably, the total physical length of the auxiliary lane should be the sum of the length for the three components of lane-change, deceleration, and storage distances, though a common practice is to accept a moderate amount of deceleration within the through lanes and to consider the taper length as a part of the deceleration length.

The PR distance (d_1) in Figure 27 represents the distance traveled while a driver recognizes the upcoming turn lane and prepares for the left-turn maneuver. It increases with PR time and speed. A value of 2.5 seconds is often used for PR time in rural settings.

Provision for deceleration clear of the through-traffic lanes is a desirable objective and should be incorporated into design whenever practical; however, on many facilities, it is not practical to provide the full length of the auxiliary lane for deceleration due to constraints such as restricted right-of-way, distance available between adjacent intersections, and extreme storage needs. In such cases, at least part of the deceleration by drivers needs to be accomplished before entering the auxiliary lane. Guidance in the AASHTO *Green Book* (*24*) and NCHRP Reports 745 and 780 (*13, 14*) advises that the following distances for changing lanes and decelerating (values for d_2 in Figure 27) should be provided when possible:

- 600 ft for a speed of 60 mph.
- 700 ft for a speed of 65 mph.
- 815 ft for a speed of 70 mph.



Source: Fitzpatrick et al. (*14*).

Figure 27. Functional Area Upstream of an Intersection Illustrating Components of Deceleration Lane Length.

The above values are similar to those found in the RDM (*1*) for the deceleration lengths of left-turn lanes on multilane rural highways; the RDM also provides for a taper length of 150 ft in addition to the deceleration length for speeds of 60 mph and higher. Designs should have a deceleration length that is sufficient to allow drivers to not decelerate more than 10 mph within the through lane to accommodate the design. Additional detail is provided in the AASHTO *Green Book* and other research (*13, 14, 24*).

The AASHTO *Green Book* (*24*) provides guidance for storage length as well, based on volumes of turning and opposing traffic. The equations from the supporting research (*14*) result in a minimum storage distance of 50 ft for urban and suburban streets and for left-turn volumes of 40 or fewer vehicles per hour, though a 100-ft length is recommended for high-speed and rural locations.

The RDM (*1*) in its guidance on two-lane rural highways in Chapter 3, Section 4, states that horizontal alignment shifts of the approaching travel lanes should be gradual. The guidance in Chapter 3, Section 5, calls for a taper length of 150 ft when adding a left-turn lane on a multilane rural highway, which provides for a taper rate between 12.5:1 and 15:1 for lane widths of 10 to 12 ft.

Additional Considerations for Turn Lanes at Intersections within Passing Lanes

In addition to the previously mentioned design principles, a turn lane for an intersection within a passing lane requires consideration of other factors related to design and operations for turning traffic and through traffic.

Adding the Turn Lane

The image in Figure 27 illustrates a turn lane on a multilane divided highway in which the turn lane can be added in the median, widening the paved surface to the left of the through lanes in that direction of travel. For two-lane highways, roadway widening typically takes place to the right, and the through lane is shifted to the right to provide sufficient width for a turn lane to be added between the opposing through lanes. For intersections within passing lanes, this introduces an additional consideration for the lane addition taper, depending on where the passing lane is added relative to the intersection.

If the turn lane is well within the limits of the passing lane, then adding a turn lane is simply a matter of introducing the lane addition taper to widen the roadway and then providing the turn-lane taper for turning vehicles to move out of the passing lane into the turn lane (see Figure 28).

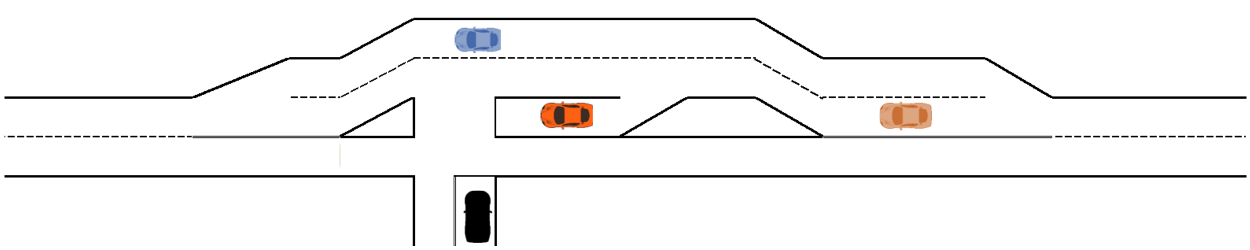


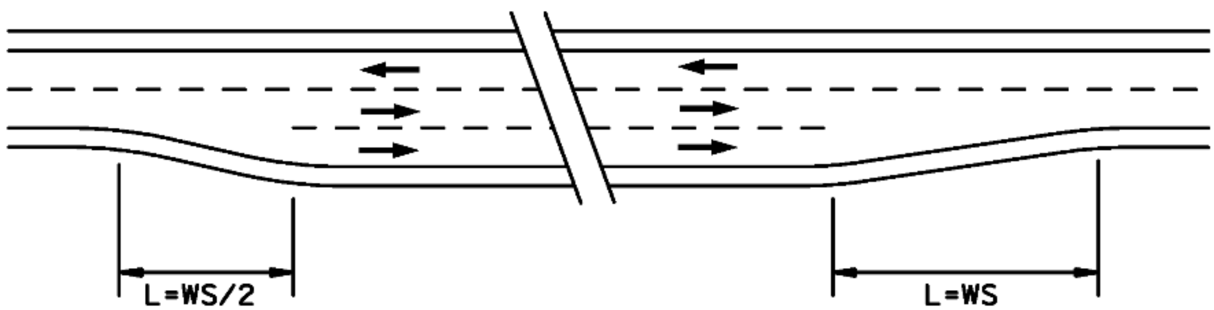
Figure 28. Example of Adding a Left-Turn Lane within the Limits of a Passing Lane.

If passing lanes are present within the corridor, the passing lanes should end more than 1000 ft from the intersection to avoid introducing additional delays at the intersection.

If the turn lane and passing lane are added in proximity to each other, then the taper length must be sufficient to add the turn lane as well as consider the taper length of adding the passing lane. The AASHTO *Green Book* (*24*) advises that long tapers approximate the path drivers follow when entering an auxiliary lane from a high-speed through lane; however, exceptionally long tapers can encourage drivers to drift into the deceleration lane—especially when the taper is on a horizontal curve. In addition, long tapers may constrain the lateral movement of a driver desiring to enter the auxiliary lanes. The RDM (*1*) describes a lane addition taper of L=WS/2 for adding a passing lane, shown in Figure 29, where:

- L = Length of taper (ft).
- W = Lane width (ft).
- S = Posted speed (mph).

For a 12-ft lane and a posted speed limit of 60 mph, the resulting lane addition taper is 360 ft. For 70 mph, the corresponding taper length is 420 ft.



Source: RDM (*1*), Figure 4-1.

Figure 29. Opening and Closing a Passing Lane.

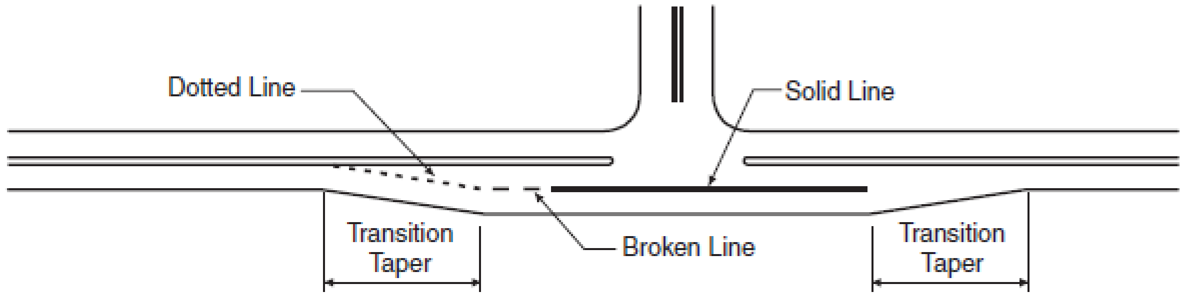
Ending the Turn Lane

Similar to the taper considerations for adding a turn lane, the design must consider the proximity of the downstream end of the passing lane to the intersection. The roadway must be narrowed downstream of the intersection to remove the extra width for the turn lane; the lane-drop taper for the turn lane should be completed prior to the beginning of the lane-drop taper for the passing lane. The RDM (*1*) describes a lane-drop taper of L=WS for closing a passing lane, shown in Figure 29; that taper length is 720 ft for a 12-ft lane at 60 mph and 840 ft for a 12-ft lane at 70 mph.

Results from a simulation in the 0-7044 project indicated that if the passing lane is present at the intersection, it should end more than 1500 ft downstream of the intersection to avoid introducing intersection delay associated with the interactions of the minor-road vehicles turning on the major road and those vehicles merging at the end of the passing lane. This distance and the interaction of merging vehicles should be considered in the location of the lane-drop taper for the passing lane.

Bypass Lanes/Shoulder Use in Direction of Left Turn

An alternative treatment to a typical turn lane is a bypass lane; this may be considered at three-leg rural intersections where installation of a typical left-turn lane is not practical. Figure 30 (*13*) shows an example of a bypass lane, which has direct entry into the turn lane. This alignment allows the through driver to change lanes to avoid the left-turning vehicle and continue through the intersection. This alignment may be used where right-of-way is constrained but a left-turn lane is warranted. The bypass lane alignment has no shadowing for the left-turn lane; thus, the taper length and lane-change distance in Figure 27 is not necessary, which shortens the length of additional right-of-way needed. It is still necessary, however, to provide a transition taper to guide through traffic around the left-turn lane. The length of the transition taper for a bypass lane (also referred to as the deceleration transition) is typically shorter than that used for a shadowed or partially shadowed lane when through traffic is shifted to the right; a useful length is the L=WS/2 distance shown in Figure 29 as the transition taper for adding a passing lane. Depending on the configuration of the intersection, it is also necessary for a bypass lane to have another transition taper (i.e., the acceleration transition) as the through traffic returns to its original alignment. This transition is commonly the same length as the transition taper on the approach to the intersection (as shown in Figure 30). Some states do not allow informal passing on the right or driving on the shoulder; constructing the additional width for through vehicles provides a legal means of passing slowed or stopped left-turning vehicles.



Source: Fitzpatrick et al. (*13*).

Figure 30. Example of Bypass Lane with Markings.

Constructing the additional width for both through vehicles and turning vehicles to have a lane provides a well-defined means for through vehicles to pass slowed or stopped left-turning vehicles. The use of a shoulder for this purpose should cease with the addition of the turn lane (or bypass lane, as applicable). Even with a turn lane or bypass lane present, sufficient shoulder width should still be provided as a refuge for disabled vehicles in the area of the intersection. Without a turn lane or bypass lane, it is not uncommon for a proportion of through drivers at such intersections in Texas to use the shoulder as a bypass around turning vehicles that have slowed or stopped to prepare for a left turn; analysis of field data from the 0-7044 study sites showed 30 to 40 percent of through drivers behind a left-turning vehicle used the shoulder for this purpose. Thus, it is prudent to provide a shoulder in the area of such intersections regardless of whether a turn lane or bypass lane is present.

Sight Distance to Turn from Minor Road

Sight distance is provided at intersections to allow drivers to recognize the presence or approach of potentially conflicting vehicles. Guidelines for the length of a turn lane incorporate the principles of SSD for vehicles on the major road to come to a complete stop when preparing to turn onto the minor road. For minor-road approaches to non-signalized intersections, sight distance along the major road should be sufficient to allow the minor-road driver to anticipate and avoid potential collisions. The RDM (*1*) states that the operator of a vehicle approaching an intersection should have an unobstructed view of the entire intersection and an adequate view of the intersecting highway to permit control of the vehicle to avoid a collision, and it refers designers to the AASHTO *Green Book* (*24*) for the criteria for selecting intersection sight distance (ISD). The *Green Book* states that if the available sight distance for an entering or crossing vehicle is at least equal to the SSD for the major road, then drivers have sufficient sight distance to anticipate and avoid collisions; however, in some cases, a major-road vehicle may need to slow or stop to accommodate the maneuver of a minor-road vehicle. Thus, the *Green Book* states that using ISD that exceeds SSD is desirable for major roads.

In particular, adding another lane (or lanes) to an intersection for passing and/or turning can affect the sight distance needs for vehicles turning from the minor road, particularly when those lanes are occupied. Thus, a sight distance review, and appropriate mitigation, should be part of a project that adds a turn lane to an intersection. Guidance in the AASHTO *Green Book* (*24*) advises that the sight distance needed for a particular intersection approach is directly related to the type of traffic control, the type of maneuvers allowed, the vehicle speeds, and the resultant distances traveled during PR and braking time. This guidance is supported by the concept of using clear sight triangles for both approach and departure, with recommended dimensions of those triangles based on intersection control. For a typical rural intersection that does not have all-way stop control, the recommended sight triangle dimensions would be based on Case B of the *Green Book*'s sight distance traffic control categories. Case B broadly refers to intersections with stop control on the minor road, and it is subdivided into three maneuvers: left turn from the minor road (Case B1), right turn from the minor road (Case B2), and crossing maneuver from the minor road (Case B3). Case B3 does not exist for a T-intersection, but Cases B1 and B2 apply for both three-leg and four-leg intersections.

The practitioner should consult the AASHTO *Green Book* for full details on developing the appropriate sight triangles and determining the corresponding ISD (including tables of recommended ISD values); however, a summary of the guidance is provided here. The *Green Book* bases its recommended Case B1 ISD values on a time gap of 7.5 seconds for a stopped passenger car to turn left onto a two-lane undivided highway. The time gap is to be converted to a distance based on the design speed of the major road using the following formula:

$$\text{Case B1 ISD} = 1.47\ V_{major}\ t_g \qquad (5)$$

Where:

V_{major} = design speed of major road (mph)
t_g = time gap for minor-road vehicle to enter major road (7.5 seconds)

Thus, for a major road with a design speed of 60 mph, the Case B1 ISD would be 661.5 ft (or rounded to 665 ft for design purposes). The Case B1 time gap value is adjusted for multilane major roads or medians by adding 0.5 seconds for each additional 12-ft lane (or median equivalent) that a passenger car needs to cross to make a left turn. The Case B1 time gap is also adjusted for large vehicles by adding 2.0 seconds for a single-unit truck (t_g = 9.5 seconds) and another 2.0 seconds for a combination truck (t_g = 11.5 seconds), plus adding 0.7 seconds (instead of 0.5 seconds) for each additional 12-ft lane to be crossed.

ISDs for Cases B2 and B3 are determined in the same manner as in Case B1, but the time gap threshold is 1.0 second shorter (i.e., t_g = 6.5 seconds for a passenger car). As a result, the Case B1 value for ISD will typically govern an intersection unless turning restrictions or other unique characteristics are present. Practitioners should review the detailed guidance in the AASHTO *Green Book* to determine the recommended ISD value for a specific intersection.

Sight Distance to Turn from Major Road

Drivers turning left from the major road onto a minor road also need sufficient sight distance to determine when it is appropriate to turn across the lane or lanes used by opposing traffic. Such sight distance should be based on the characteristics of a stopped vehicle, which will accommodate the needs of vehicles that do not need to stop before turning left. The appropriate sight distance along the major road to accommodate left turns is the distance traveled at the design speed of the major road during the time gap needed by the left-turning vehicle. The AASHTO *Green Book* (*24*) describes this as Case F ISD and uses a time gap of 5.5 seconds for passenger cars that are stopped and waiting to turn left from a two-lane undivided highway. The sight distance value is then calculated using the same equation described for Case B1 ISD, substituting the value of 5.5 seconds for t_g in the equation. Thus, for a 60-mph major road, the Case F ISD would be 485.1 ft (or rounded to 490 ft for design purposes). The Case F time gap value is adjusted for multilane major roads or medians by adding 0.5 seconds for each additional 12-ft lane (or median equivalent) that a passenger car needs to cross to make a left turn. The Case F time gap is also adjusted for large vehicles by adding 1.0 second for a single-unit truck (t_g = 6.5 seconds) and another 1.0 second for a combination truck (t_g = 7.5 seconds), plus adding 0.7 seconds (instead of 0.5 seconds) for each additional 12-ft lane to be crossed.

Because Case F ISD is less than Case B1 ISD, if SSD is provided continuously along the major road, and if Case B1 ISD is provided for the minor-road approach to the intersection, the available sight distance will generally be adequate for Case F as well; however, Case F should be checked for intersections located on or near horizontal or vertical curves. Additional details can be found in the AASHTO *Green Book* (*24*).

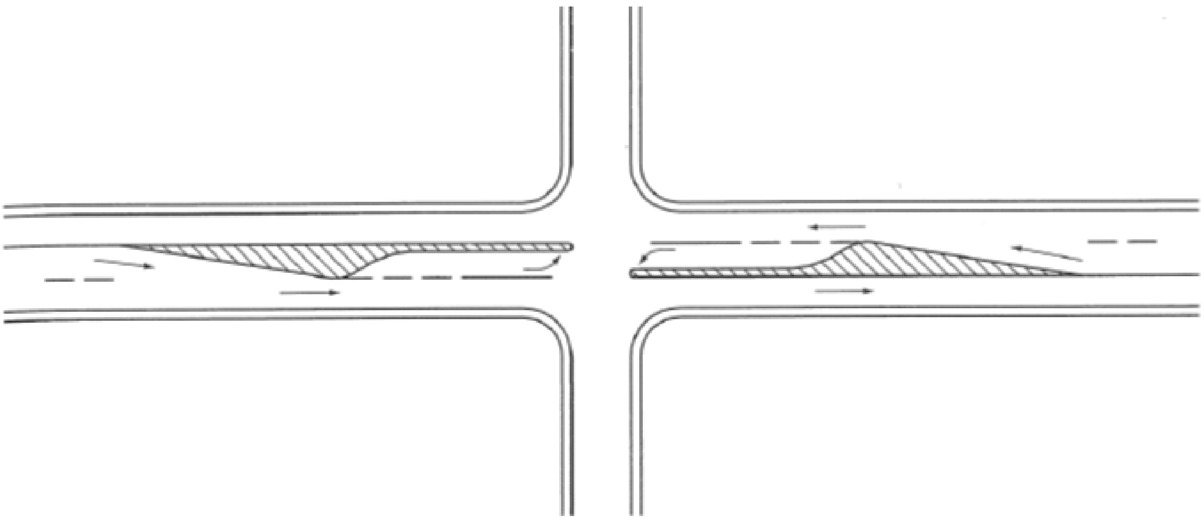
Analysis of field data from the rural two-lane study sites on the 0-7044 project found that the critical gap for turning left from the major road was 5.8 seconds, which is similar to critical gap values in the literature for rural two-lane highways (*47*) and is consistent with the AASHTO *Green Book* guidance for ISD.

Design Guidance from Other States

Most states do not currently provide specific guidelines for the design and implementation of passing lanes, and of those that do, most do not provide guidance on passing lanes in the vicinity of intersections beyond a general discouragement of having access points within passing lanes. Notable guidance that does exist in other states on designing passing lanes around intersections is provided as follows:

- Iowa DOT (*58*) recommends avoiding placing passing lanes in areas with a paved intersection. If it cannot be avoided, an offset right-turn lane should be provided in addition to the passing lane to prevent turning traffic from shadowing traffic in the through/passing lanes and affecting the ISD for vehicles on the minor road. Iowa DOT guidance also recommends providing a left-turn lane with storage length and deceleration length in the area of the passing lane to avoid affecting passing maneuvers in the through lane. If a passing lane is only provided in one direction of travel at the intersection, turn-lane warrants should be used to determine if turn lanes should be provided in the direction of travel without the passing lane. Iowa DOT states that placing passing lanes through intersections with unpaved minor roads is acceptable and, in some instances, will be necessary due to the passing lane length.
- Kentucky Transportation Cabinet (*59*) recommends that passing lane sections be designed to eliminate left turns within the first 1000 ft of the start of the passing lane because this area is the most prone to higher speeds and overtaking maneuvers. On highways where access is controlled by permit, Kentucky recommends that major intersections or access points be placed in the transition zones between passing lanes to minimize the volume of turning traffic in locations where passing is encouraged and so that auxiliary turn lanes may be readily accommodated (see Figure 31). Where

the presence of higher-volume intersections and driveways cannot be avoided, Kentucky recommends considering special provisions for turning vehicles, such as exclusive left-turn lanes.



Source: Kentucky Transportation Cabinet (*59*), Figure 2.

Figure 31. Passing Lane Transitions at Major Intersections.

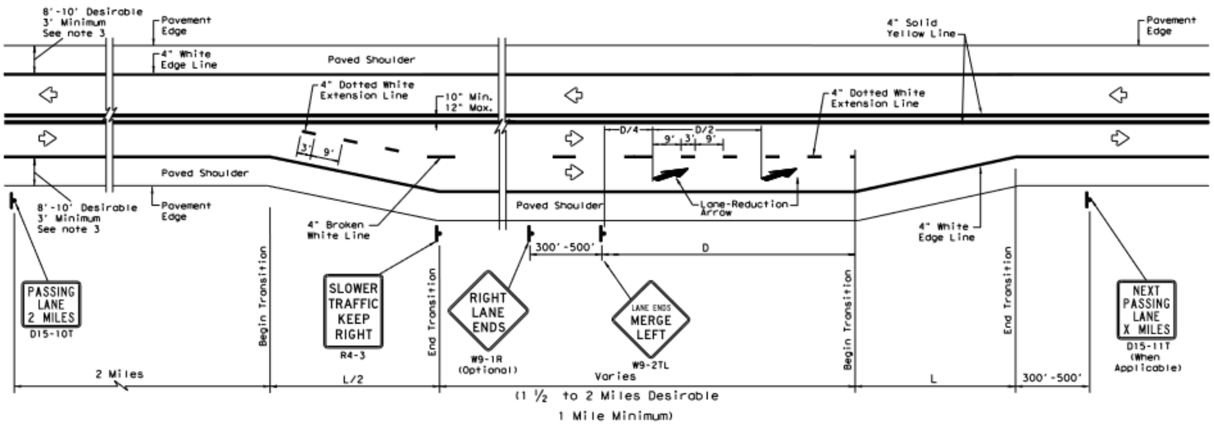
- Missouri DOT (*60*) states that the location of major intersections and high-volume driveways is to be considered in selecting passing lane locations to minimize the volume of turning movements on a road section where passing is encouraged. Missouri DOT advises that low-volume intersections and driveways do not usually create problems in passing lanes, but where the presence of higher-volume intersections and driveways cannot be avoided, special provisions for turning vehicles, such as exclusive left-turn lanes, are to be considered. Exclusive right- and left-turn lanes in passing lane sections are to be considered at any location where an exclusive turn lane would be considered on a conventional two-lane highway and opportunities to make a left turn within the first 1000 ft of a passing lane are undesirable. Some strategies mentioned to address the turning movement include exclusive left-turn lane, right-in/right-out access, and beginning the passing lane after the entrance. Missouri guidance is based on a research report from Potts and Harwood (*61*) from a study conducted for Missouri DOT.
- South Carolina DOT (*62*) advises designers to locate major intersections in the transition area between opposing passing lanes and provide striping for left-turn lanes

at the intersection, as applicable. Low-volume intersections and most driveways may be accommodated within the passing lane sections.

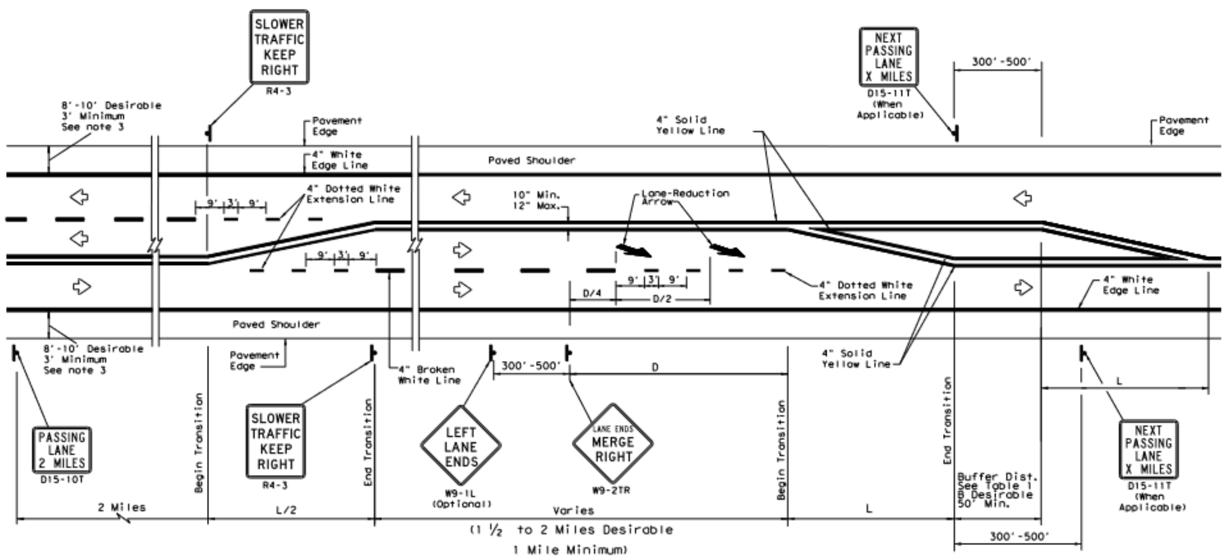
- Wisconsin DOT (*63*) states that passing lanes should be constructed in segments of highways that have minimal numbers of entrances and preferably no side roads. When selecting sites for passing lane facilities, side roads with 500 ADT and over should be avoided. Driveways and field entrances should be avoided in merge taper areas, extending from the W4-2R signs (lane-reduction transitions) to the ends of the tapers, or 1,200 ft. No driveways or intersections should be located closer than 500 ft from the end of the downstream taper. Designers should consider relocating field entrances and driveways in the merge areas. Commercial driveways may be more problematic than side roads, depending on peak-hour usage and traffic mix.

Guidance for Traffic Control Devices

Current TxDOT Traffic Standards (*64*) provide details of signs and markings to be used for left-turn lanes and Super 2 passing lanes. A pair of traffic standards sheets, TS2(PL-1) and TS2(PL-2), contain details for Super 2 passing lanes. The first sheet focuses on separated and alternating passing lanes (see Figure 32), while the second sheet describes similar details for side-by-side passing lanes. Each example provides details on the appropriate taper lengths to add and remove the passing lane, along with pavement markings that coincide with those lanes. As shown in Figure 32, dotted white extension lines are provided at the lane addition taper to encourage drivers to travel in the right-hand lane. Lane-reduction arrows are provided near the beginning of the lane-reduction taper to alert drivers that they are approaching the transition and merge area. Roadside signs support those markings, with “SLOWER TRAFFIC KEEP RIGHT” posted at the beginning of the passing lane and “LANE ENDS” and merge signs posted near the end of the passing lane.



(a) Separated Passing Lanes



(b) Alternating Passing Lanes

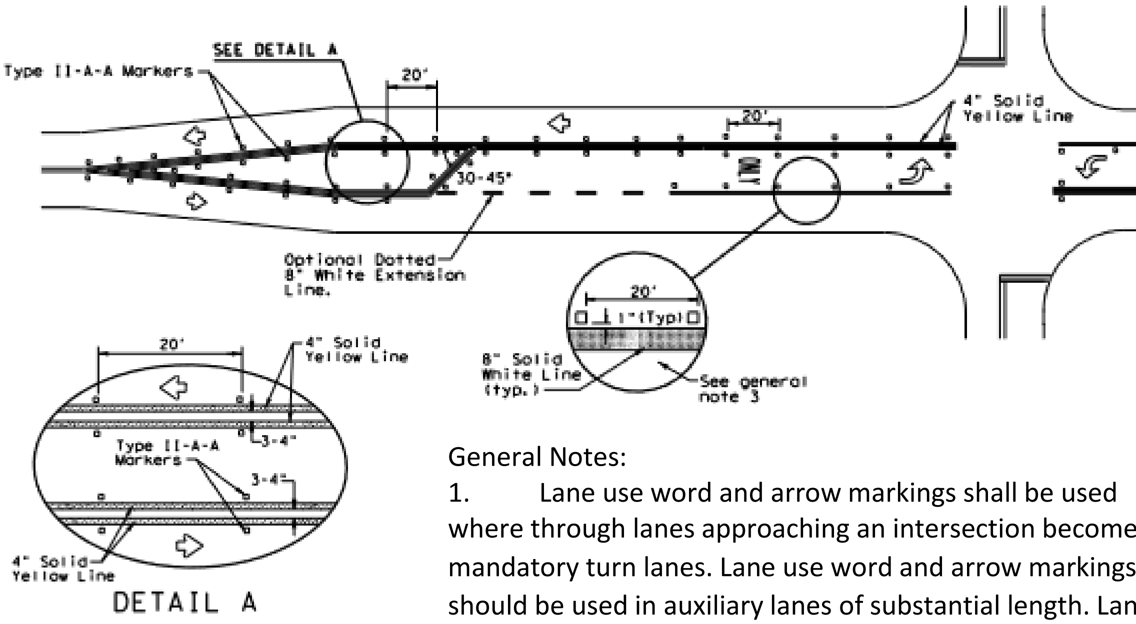
Source: Traffic Standard TS2[PL-1]-18.

Figure 32. Signs and Markings for Texas Super 2 Passing Lanes.

Traffic Standard PM(3) contains typical pavement markings for many turn-lane treatments, including rural left-turn bays. The portion of PM(3) that describes a typical two-lane highway intersection with left-turn bays is shown in Source: Traffic Standard PM[3]-20

Figure 33. That diagram shows the taper area for widening the roadway in both directions of travel to create width for the turn lane in the center. A specific taper rate or length is not provided in PM(3), nor is a recommended length for the turn-lane dimensions; instead, General Note 4 of PM(3) indicates that length of turn bays, including taper, deceleration, and storage lengths, should be as shown on the plans or as directed by the engineer. Recommendations on turn-lane lengths, based on the AASHTO *Green Book* and the TxDOT RDM, can be found in an earlier section of these guidelines. A dotted white extension line is allowed to further describe

the upstream end of the turn lane and encourage through drivers to remain in the right lane, while text and arrows are installed at the downstream end to describe it as a left-turn lane.



General Notes:

1. Lane use word and arrow markings shall be used where through lanes approaching an intersection become mandatory turn lanes. Lane use word and arrow markings should be used in auxiliary lanes of substantial length. Lane use arrow markings or word and arrow markings may be used in other lanes and turn bays for emphasis. Details for words and arrows are as shown in the Standard Highway Sign Designs for Texas.
2. When lane-use words and arrow markings are used, wo sets of arrows should be used if the length of the bay is greater than 180 feet. When a single lane use arrow or word and arrow marking is used for a short turn lane, it should be located at or near the upstream end of the full-width turn lane.
3. Use raised pavement marker Type I-C with undivided highways, flush medians and two-way left-turn lanes. Use raised pavement marker Type II-C-R with divided highways and raised medians.
4. Length of turn bays, including taper, deceleration, and storage lengths shall be as shown on the plans or as directed by the engineer.

Source: Traffic Standard PM[3]-20

Figure 33. Markings for Typical Two-Lane Highway Intersection with Left-Turn Bays.

Guidance for Lighting

The TxDOT *Highway Illumination Manual* (*65*) provides guidance on the use of highway illumination for the state highway system. The manual states in Chapter 2, Section 3, that safety lighting may be at any highway intersection or other decision-making point or points of nighttime hazard. Safety lighting may be used to the extent necessary to provide for safety enhancement and the orderly movement of traffic. Safety lighting may consist of complete intersection lighting, partial intersection lighting, or spot lighting, depending on the need of the site and warranting conditions. Complete intersection lighting covers the limits of the intersection, while partial intersection lighting covers specific features such as speed-change lanes. Spot lighting usually consists of one to five units intended to illuminate a nighttime hazard, such as sections with complex geometry or raised channelization. In general, for non-freeway locations, lighting may be considered for those locations where the relevant governmental agencies agree that lighting would contribute substantially to the safety, efficiency, and comfort of vehicular and pedestrian traffic. The manual provides recommendations for illumination levels for intersections and safety lighting in Chapter 6, Section 2; the designer is encouraged to refer to that manual for full details of appropriate illumination as well as construction and maintenance guidelines.

CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS

OVERVIEW

Researchers reviewed existing guidance and findings from literature related to operations and safety at intersections within or near passing lanes on rural two-lane highways. Using those findings as a guide, they collected and analyzed field data and crash data, conducted a simulation analysis, and completed a benefit-cost analysis to develop recommendations for guidelines and future research.

FIELD SITE FINDINGS

In-field observation data provide researchers an appreciation of actual interactions occurring at a site; however, the rarity of interactions of interest may limit the ability to understand how varying conditions can affect operations. Traffic simulation modeling can provide the means to gain that understanding when collecting sufficient in-field observations either takes a long time or is costly. The available in-field observations can be used to verify traffic simulation model findings and can be used to adjust input parameters for the simulation. The findings from the move-onto-the-shoulder analysis are important because that driving behavior may be unique for Texas. The speed distribution findings helped to generate the speed profile used in the simulation analysis. Key findings from the field site data are as follows:.

- Speed distribution curves revealed that the 85th percentile speed for passenger cars varied from 72 mph to 81 mph, while that of heavy vehicles varied from 68 mph to 80 mph.
- In the move-onto-the-shoulder analysis, looking at the distance at which through vehicles behind the left-turn vehicle move onto the shoulder, this study found that the 85th percentile of that distance varied from 80 ft to about 230 ft, with an average of 210 ft. Approximately 85 percent of the drivers who moved onto the shoulder did so at a distance of 210 ft or less.
- Focusing on the interactions when a single through vehicle arrives behind a left-turn vehicle and needs to either stop, slow, or move onto the shoulder, this study found that about 40 percent of the drivers moved onto the shoulder, with the other 60 percent being able to slow sufficiently before arriving at the cleared intersection. For

the available data, only one through vehicle arriving behind a left-turn vehicle stopped.

- The research team found that the critical gap for the study sites was 5.8 seconds, which is similar to the value reported in the literature for rural two-lane highways.
- The research team conducted a statistical evaluation on intersection and intersection-related crashes at 283 intersections located within Super 2 corridors. These rural intersections all had stop control on the minor legs and included both three-leg and four-leg intersections. The findings from the evaluation and from the literature illustrate that the presence of either a left-turn lane or a passing lane is associated with fewer crashes at the intersection.

SIMULATION FINDINGS

The findings from the simulation analysis revealed that the addition of either a passing lane or left-turn lane reduces intersection delay. The results for the various combinations were reviewed, and similar patterns for the various testbeds were identified—when a passing lane ends within 1500 ft of the intersection, and in higher-volume range within 2640 ft of the intersection, intersection delay is higher than if the passing lane is not present. A review of the simulation confirmed that having the merging of the through lanes and drivers attempting to turn left out of the minor road within 750 ft, 1500 ft, or 2640 ft should be avoided at higher volumes.

BENEFIT-COST ANALYSIS FINDINGS

In the BCA, for the range of ADTs considered, the addition of a left-turn lane or passing lanes almost always resulted in a positive BCR, indicating that the treatment provided a benefit over no treatment. The few comparisons with a negative BCR were when the corridor was congested and only occurred when the assumed major-road ADT was 20,000 and the percent of trucks was 22 or 35 percent. In all other comparisons, the BCR was much greater than 1.0.

RECOMMENDATIONS FOR FUTURE RESEARCH

This study had a few challenges that can be considered in future studies. The first challenge was to determine the appropriate sites for data collection. Most of the potential identified sites had limited numbers of left-turn vehicles and through vehicles, which made it difficult to obtain a sufficient sample size for analysis. This condition also limits the ability to study the issue using crashes. This condition is not unexpected given that intersections with large numbers of left-turn vehicles or through vehicles would be considered for intersection improvements. Obtaining a desired number of observations may require multiple days or perhaps even weeks of video data collection. Another approach for collecting this type of information is through the use of a driving simulator. Within a driving simulator, researchers can program other vehicles to appear at critical times to place the participant in a situation of having to decide whether to move onto the shoulder or to slow and possibly need to stop to avoid the left-turn vehicle. A driving simulator could also provide insights into driver gap acceptance and determine how many vehicles a driver is willing to pass on the shoulder.

The preference is to have field sites that are relatively level and straight to minimize the potential effects of roadway geometry on the driver's decision to move or not move onto the shoulder. For this study, one of the intersections that satisfied other criteria was within a horizontal curve. Another challenge was determining the appropriate distance upstream of the intersection for equipment placement to capture images of all vehicles that moved onto the shoulder prior to the vehicle leaving the travel lane. For similar future studies, the site should be observed prior to installing the equipment to determine the optimal equipment placement.

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