1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-11/0-6135-1		

4. Title and Subtitle
OPERATIONS AND SAFETY OF SUPER 2 CORRIDORS WITH HIGHER VOLUMES

5. Report Date
February 2011
Published: June 2011

6. Performing Organization Code

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8. Performing Organization Report No.
Report 0-6135-1

9. Performing Organization Name and Address
Texas Transportation Institute
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10. Work Unit No. (TRAIS)

11. Contract or Grant No.
Project 0-6135

12. Sponsoring Agency Name and Address
Texas Department of Transportation
Research and Technology Implementation Office
P.O. Box 5080
Austin, Texas 78763-5080

13. Type of Report and Period Covered
Technical Report:
September 2008–February 2011

14. Sponsoring Agency Code

15. Supplementary Notes
Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration.
Project Title: Super 2 Design for Higher Traffic Volumes
URL: http://tti.tamu.edu/documents/0-6135-1.pdf

16. Abstract
As traffic volumes increase, in both urban and rural areas, the demand on the highway network also increases. Specifically, as rural traffic volumes rise in Texas, the pressure on the state's network of two-lane highways rises accordingly. Previous research in Texas demonstrated that periodic passing lanes can improve operations on two-lane highways with average daily traffic (ADT) lower than 5000; these "Super 2" highways can provide many of the benefits of a four-lane alignment at a lower cost. This project expands on that research to develop design guidelines for passing lanes on two-lane highways with higher volumes, investigating the effects of volume, terrain, and heavy vehicles on traffic flow and safety. This report discusses findings from field observations and crash analysis of existing Super 2 highway corridors in Texas and computer modeling of traffic conditions on a simulated Super 2 corridor. Results indicate that passing lanes provide added benefit at higher traffic volumes, reducing crashes, delay, and percent time spent following. Empirical Bayes analysis of crash data reveals a 35 percent reduction in expected non-intersection injury crashes. Simulation results indicate that most passing activity takes place within the first mile of the passing lane, so providing additional passing lanes can offer greater benefit than providing longer passing lanes. Whether adding new passing lanes or adding length to existing lanes, the incremental benefit diminishes as additional length is provided and the highway more closely resembles a four-lane alignment. The simulation study also showed that the effects of ADT on operations were more substantial than the effects of terrain or truck percentage for the study corridor.

17. Key Words
Super 2, Two-Lane Rural Highway, Passing Lane, Safety, Empirical Bayes, Simulation, TWOPAS, IHSDM

18. Distribution Statement
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National Technical Information Service
Springfield, Virginia 22161
http://www.ntis.gov

19. Security Classif.(of this report)	20. Security Classif.(of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	208	

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Report 0-6135-1
Project 0-6135
Project Title: Super 2 Design for Higher Traffic Volumes

Performed in cooperation with the
Texas Department of Transportation
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Federal Highway Administration

February 2011
Published: June 2011

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DISCLAIMER

ACKNOWLEDGMENTS

This research was conducted in cooperation with the Texas Department of Transportation and the Federal Highway Administration. The research team gratefully acknowledges the contributions of the TxDOT Project Monitoring Committee:

- Mark Wooldridge, P.E., Wharton Area Engineer and Project Director.
- David Harper, P.E., Lubbock District.
- Russel Lenz, P.E., Corpus Christi District/Construction Division.
- Rory Meza, P.E., Design Division.
- Chris Reed, P.E., Childress District.
- Roy Wright, P.E., Abilene District.
- Wade Odell, P.E., Research and Technology Implementation Office.
- Loretta Brown, Research and Technology Implementation Office.
- Frank Espinosa, Research and Technology Implementation Office.

The authors also thank the following TTI staff members for their contributions to the project:

- Gary Barricklow, Research Technician.
- Srinivas Geedipally, Engineering Research Associate.
- James Campbell, Student Worker IV.

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CHAPTER 1
INTRODUCTION

BACKGROUND

As vehicular traffic in rural areas continues to rise, state departments of transportation are looking for ways to accommodate that traffic, even as demands on their budgets also increase. Specifically, as rural traffic volumes rise, often approaching the limits of capacity for two-lane highways, the pressure on those highways rises accordingly, with corresponding effects on congestion, air quality, and safety. High proportions of heavy vehicles compound the problem, contributing to a decrease in safety as impatient drivers attempt to pass slower vehicles in no-passing zones or pass trucks despite having diminished sight distance beyond such vehicles.

Traditionally, roadway agencies expand a two-lane highway to four lanes when certain criteria are met, such as average daily traffic (ADT), peak volumes, prevailing speeds, and/or crash history. As more rural highways approach conditions that meet these criteria, agencies are looking for alternatives to full four-lane expansion to provide a measure of operational benefits at lower cost. Previous research in Texas (*1*) demonstrated that periodic passing lanes can improve operations on two-lane highway corridors with low to moderate volumes (e.g., average daily traffic at or below 5000 vehicles per day); called "Super 2" highways in Texas, these improved corridors can provide many of the benefits of a four-lane alignment at lower cost.

As traffic volumes increase on the state's two-lane roads, along with the volumes of heavy vehicles, the effects of limited passing sight distance are magnified. This results in more locations where Super 2 highways may be effective. As a result, providing longer passing lanes and/or providing passing lanes with greater frequency may be justified.

PURPOSE OF THE PROJECT

Project 0-6135 expands on previous research (*1*) to develop design guidelines for passing lanes on two-lane highways with higher volumes. Recommended guidelines will address geometric design criteria; placement, length, and spacing of passing lanes; and appropriate transitions at either end of passing lanes. Tasks within the project focus on the state of the practice within Texas and in other states, collecting appropriate traffic and site characteristics data from existing Super 2 corridors in Texas, selecting and applying an appropriate computer model to evaluate combinations of traffic and site characteristics on a simulated Super 2 corridor, and analyzing crash history on existing Super 2 corridors.

ORGANIZATION OF THIS REPORT

This report consists of 10 chapters and two appendices. In addition to this introductory chapter, the report contains the following material:

- Chapter 2 summarizes relevant findings from recent research on Super 2 highways and passing lanes in Texas and in other states and countries.
- Chapter 3 provides a comparative summary of other states' practices on the design and implementation of passing lanes and Super 2 corridors.
- Chapter 4 documents researchers' activities in evaluating the state of the practice on Super 2 highways in Texas.
- Chapter 5 describes the research team's analysis of crash data on existing Super 2 corridors in Texas.
- Chapter 6 discusses the relative features of commonly used computer simulation models and the support for selecting the model used in this project.
- Chapter 7 documents the procedures for collecting field data on two Super 2 corridors, as well as key findings from analysis of that data.
- Chapter 8 describes the process used to calibrate the simulation model using the collected field data.
- Chapter 9 provides a description of the simulation modeling activities and the key results and findings from that analysis.
- Chapter 10 summarizes the researchers' findings and conclusions, and it provides recommendations for future action, including revisions to the *Roadway Design Manual*.
- Appendices A and B provide detailed results of all of the simulation scenarios that were evaluated and summarized in Chapter 9.

CHAPTER 2
REVIEW OF RECENT LITERATURE

Since the completion of TxDOT Project 0-4064, there have been a number of research studies that investigated various aspects of passing lanes and Super 2 highways, in addition to other studies pre-dating Project 0-4064. A selection of those studies and a summary of current guidance in Texas are presented in this chapter.

EXISTING CRITERIA

The Texas *Roadway Design Manual* (RDM) contains the current description of and guidance for the use of passing lanes on two-lane highways, designated Super 2 highways. Passing lane length and spacing are the critical elements to Super 2 highways, as the lanes must have sufficient length to allow drivers to complete the passing maneuver and they must be properly spaced to provide adequate passing opportunities. The October 2006 *Roadway Design Manual* (*2*), which was in effect at the beginning of this research project and governed the design of existing Super 2 installations, provided guidance on passing lane length and spacing based primarily on the ADT of the roadway, as shown in Table 4-6 of that document, reproduced here as Table 1.

Table 1. Super 2 Passing Lane Length and Spacing by ADT (*2*).

Two-Way ADT (vpd)	Recommended Passing Lane Length (mi)	Recommended Distance Between Passing Lanes (mi)
< 2000	1.0	5–9
2001–5000	1.5–2.0	4–9
> 5000	Conversion to four-lane highway should be considered	

The design criteria for passing lane sections are the same as the 3R design guidelines for other rural two-lane highways. These guidelines are also based on ADT, as shown in Table 2.

Table 2. 3R Design Guidelines for Rural Two-Lane Highways, US Customary Units (1).

Design Element [a]	Current Average Daily Traffic		
	0–400	400–1500	1500 or more
Design Speed [b]	30 mph	30 mph	40 mph
Shoulder Width (ft)	0	1	3
Lane Width (ft)	10	11	11
Surfaced Roadway (ft)	20	24	28
Turn Lane Width (ft) [c]	10	10	10
Horizontal Clearance (ft)	7	7	16
Bridges [d]: Width to be retained (ft)	20	24	24 [e]

NOTES:

[a] These values are intended for use on rehabilitation projects. However, the designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area, or to provide operational improvements at specific locations.

[b] Considerations in selecting design speeds for the project should include the roadway alignment characteristics as discussed in this chapter.

[c] For two-way left turn lanes, 11 ft–14 ft usual.

[d] Where structures are to be modified, bridges should meet approach roadway width as a minimum. (Approach roadway width is the total width of the lanes and shoulders.) Greater bridge widths may be appropriate if the rehabilitation project increases roadway life significantly or if higher design values are selected for the remainder of the project. Existing structure widths less than those shown may be retained if the total lane width is not reduced across or in the vicinity of the structure.

[e] For current ADT exceeding 2000, minimum width of bridge to be retained is 28 ft (8.4 m).

The RDM adds that passing lanes should be located to best fit existing terrain and field conditions: "Uphill grades are preferred sites over downhill grades. Passing lanes on significant uphill grades should extend beyond the crest of the hill. Passing lane sections and transitions should be placed to avoid major intersections. If present, minor intersections that do not require deceleration lanes should be located near the midpoint of passing lane sections and also avoid transition areas to the extent practical." Other than these general statements, the current guidelines do not account for effects of terrain, and they do not include adjustments for substantial proportions of heavy vehicles.

The RDM also advises that providing a passing lane section downstream of a traffic signal for platoons exiting an urbanized area is particularly beneficial in dispersing the platoons and improving operations in rural areas.

CURRENT PRACTICE IN TEXAS

The October 2006 RDM says the purpose of Super 2 highways is to allow the passing of slower vehicles and the dispersal of traffic platoons, with the caveat that they should only be considered in rural areas. The Manual also addresses installations that approach four-lane alignments, saying that "significant lengths or segments of passing lanes are not encouraged. If traffic volumes are such that significant lengths or segments of passing lanes are necessary, then construction of another category of roadway should be considered."

However, the manual adds that "a passing lane is appropriate for areas where passing sight distances are limited. The location of the proposed lane addition should offer adequate sight distances and lane taper. The location selection should also consider the presence of intersections and high volume driveways in order to minimize the volume of turning movements on a roadway section where passing is being encouraged."

Roadway characteristics from the 2004 TxDOT RHiNo database indicate that there are nearly 4250 centerline miles of rural two-lane highway with ADT above 5000 (see Table 3). As traffic volumes increase on the state's two-lane roads, along with the volumes of heavy vehicles, the effects of limited passing sight distance are magnified, creating more locations where Super 2 highways can be effective. As a result, opportunities increase for longer passing lanes occurring at shorter spacing.

In 2005, the San Antonio District requested a review (*3*) of its two-lane highways for the purpose of creating a prioritized Super 2 master plan that would support the conversion of various sections of those highways into Super 2 sections. Many of these sections had ADTs well in excess of 5000, and the suggested spacing for these passing lanes ranged from 0 to 5 miles. Anecdotal evidence suggests that other districts across the state have also been evaluating and/or installing Super 2 sections on two-lane highways that have higher ADTs or shorter spacing than those recommended in the RDM. As a result, there is a growing need to revisit the RDM guidelines, both to evaluate the performance of existing higher-volume Super 2 sections and to specify revised guidelines that allow for higher volumes, longer passing lanes, and shorter spacing where the need is justified.

Table 3. Distribution of Texas Rural Two-Lane Highway Miles by 2004 ADT.

District	ADT Range									
	0–399	400–1499	1500–1999	2000–2499	2500–4999	5000–7499	7500–9999	10000 +	All ADTs	All 5000+
ABL	1613	822	134	88	146	9	0	0	2812	9
AMA	1697	808	233	117	198	23	5	1	3082	28
ATL	283	933	217	120	316	100	22	4	1996	126
AUS	412	742	213	114	245	128	88	48	1991	265
BMT	201	541	151	142	348	176	65	61	1686	303
BWD	1237	607	141	134	188	20	1	1	2329	22
BRY	330	1148	164	179	335	220	79	32	2485	330
CHS	1523	564	97	30	10	0	0	3	2227	3
CRP	618	676	142	103	257	186	26	12	2018	223
DAL	178	576	161	105	388	118	65	133	1724	316
ELP	669	429	102	37	20	12	4	6	1278	21
FTW	305	656	162	111	388	188	82	67	1959	337
HOU	22	155	52	52	312	200	125	169	1088	494
LRD	843	432	76	73	386	15	16	7	1848	37
LBB	2256	1357	255	147	120	19	0	0	4155	19
LFK	665	963	208	152	377	71	16	12	2464	99
ODA	807	1001	172	117	66	0	0	0	2164	0
PAR	704	1065	171	154	310	177	59	18	2658	254
PHR	172	460	185	99	224	185	46	46	1417	277
SJT	1380	915	120	73	265	9	1	0	2763	10
SAT	578	1017	175	160	397	126	62	29	2545	217
TYL	358	1211	276	194	495	201	73	71	2878	344
WAC	707	915	181	144	418	176	74	60	2675	310
WFS	1038	805	134	101	218	4	0	0	2300	4
YKM	758	1058	247	201	497	154	17	24	2957	196
Total	19355	19856	4169	2947	6926	2517	926	804	57500	4246

PREVIOUS RESEARCH IN TEXAS

The previous TxDOT-sponsored research project (0-4064) produced recommendations for design guidelines to be used for future Super 2 highways in Texas (*1*). Researchers on that project collected field data at existing Super 2 sections in Minnesota and Kansas in order to gain firsthand knowledge of normal operations and to personally view installed designs with signing and marking details; they collected data on operating speeds, distribution of trucks, lane splits, and headways. The data provided them with a sample of real-world data on the passing maneuvers taking place on Super 2 sections and the conditions associated with those maneuvers. Additional field studies at passing lane transitions in Texas provided comparison data to the Minnesota and Kansas data. The research team also conducted a survey of Texas drivers to gather their input and gauge then-current attitudes toward passing behavior. In addition, researchers created a test bed scenario for microscopic simulation, evaluating operating characteristics for a variety of passing lane lengths and spacings, traffic volumes, and heavy vehicle percentages. Based on the findings from analyzing those various datasets, researchers developed recommendations for passing lane length and spacing, lane and shoulder widths, signs, and pavement markings. Those recommendations, which were the basis for the current guidelines in the Texas *Roadway Design Manual*, are shown in Table 4, Table 5, and Figure 1.

Table 4. Recommended Values of Length and Spacing by ADT and Terrain (*1*).

ADT (vpd)		Recommended Passing Lane Length (mi)	Recommended Distance between Passing Lanes (mi)
Level Terrain	Rolling Terrain		
≤ 1950	≤ 1650	0.8–1.1	9.0–11.0
2800	2350	0.8–1.1	4.0–5.0
3150	2650	1.2–1.5	3.8–4.5
3550	3000	1.5–2.0	3.5–4.0

Table 5. Recommended Values for Lane and Shoulder Widths (*1*).

Lane Width	
12 ft or Values in Table 3-8 of TxDOT's *Roadway Design Manual*	
Shoulder Width*	
Minimum (allowable only where traffic volumes are below 2000 ADT):	6 ft if rumble strips are used 4 ft if rumble strips are not used
Desirable:	Values in Table 3-8 of TxDOT's *Roadway Design Manual*
*Shoulders used in passing lane sections should be paved.	

The design elements recommended by Project 0-4064 are similar to those found in the Texas *Roadway Design Manual*, except that the RDM reduces the number of ADT categories and simplifies the length and spacing ranges. The RDM also refers the designer to existing lane and shoulder width guidelines rather than provide separate guidelines for passing lanes. The sign and marking layout, shown in Figure 1, identified two specific informational signs for the length and spacing and a skip-stripe marking to reinforce the preferred behavior that drivers should travel in the right lane except when passing.

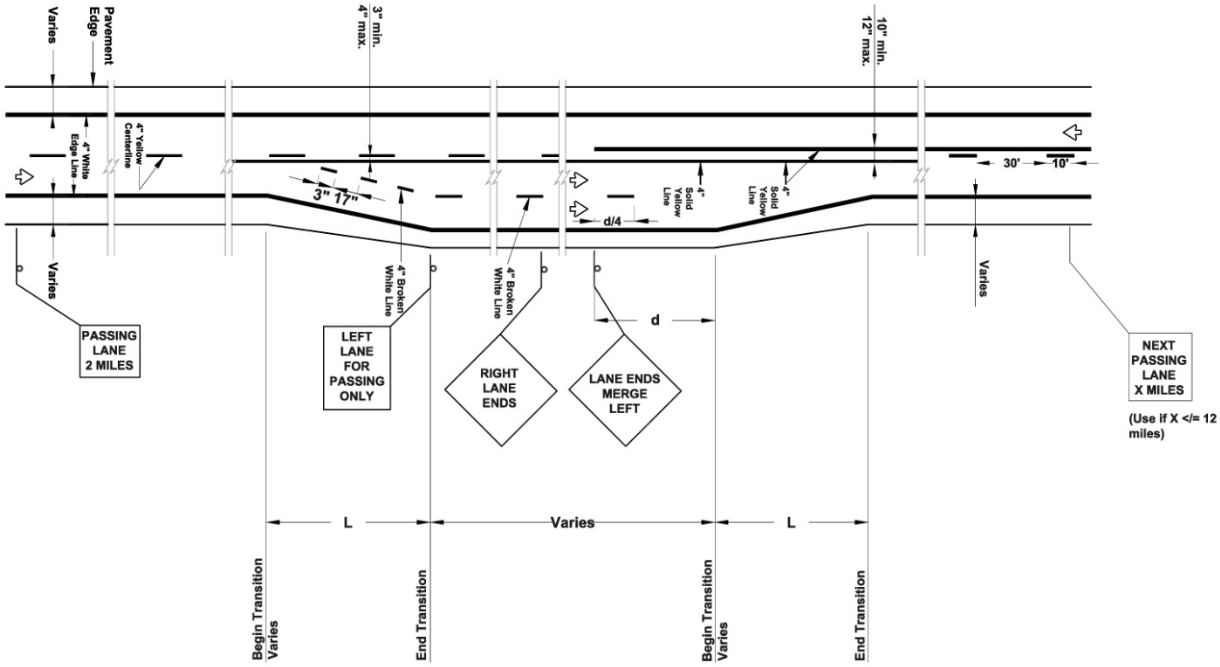


Figure 1. Super 2 Signing and Marking Layout (*1*).

TRAFFIC VOLUMES

Project 0-4064 provided recommendations for passing lanes on highways with ADT no more than 3550 vehicles per day (vpd), and current TxDOT RDM guidelines limit Super 2 recommendations to highways with less than 5000 ADT, with the advice that a four-lane cross-section should be considered for higher volumes. However, recent studies have evaluated operations on higher-volume passing lane sections in other states.

In 2006, Gattis, Bhave, and Duncan reported on a study of passing lane operations, focusing on continuously alternating passing lane sections in Arkansas (*4*). Their field study contained four sites with average flow rates between 164 and 445 vehicles per hour and maximum flow rates from 232 to 724 vehicles per hour. Their findings indicated that the passing lane sections reduced the percentage of vehicles in platoons by about 14 percent, with much of that reduction coming in the first 0.9 mile of the passing lane. They also found that passing maneuvers increased as volume increased, inferring that higher-volume roads could use longer passing lanes. A broader review of the crash data at 19 passing lane sites showed that even though the average ADT of those sites (5293 vpd) was almost three times the statewide average for rural two-lane undivided highways (1857 vpd), the crash rates at 16 of those sites were lower than the statewide average of 1.4 crashes per million vehicle miles.

Potts and Harwood conducted an evaluation of the benefits and effectiveness of passing lanes in Missouri (*5*). They analyzed three roadway sections comparing traffic operations before and after installation of passing lanes on rural two-lane highways. The three sites had ADTs ranging from 4500 to 10,600 vpd, each with truck and recreational vehicle proportions of 10 and 5 percent, respectively. Analysis of the sections showed that the level of service (LOS) improved noticeably at each site, based on the average travel speed and the percent time spent following; two of the three sites improved LOS by two letter levels. A review of crash data showed that the crash rates for two-lane highways with passing lanes were approximately 29 percent lower than the rates for traditional two-lane highways in the same districts.

PASSING LANE CONFIGURATION

A common practice in Sweden is to provide a continuous three-lane cross-section, known as a 2+1 road. While jurisdictions in other countries also use a 2+1 cross-section, Sweden is unique in that it often uses a cable barrier to separate opposing traffic. Carlsson and Bergh conducted a study for the Swedish National Road Association to evaluate operations and safety on these roadways (*6*). Swedish 2+1 roadways generally have a 13 m (42.6 ft) surface width, with two 3.75-m (12.3-ft) through lanes, one 3.5-m (11.5-ft) passing lane, and two 1.0-m (3.3-ft) shoulders. The authors made the following findings:

- LOS for normal traffic was better than expected. Speeds on 2+1 roads with cable barrier were the same or higher compared with other roadways for directional flows up to 1400 vehicles per hour.
- Emergency and tow agencies complained that their working conditions and service have deteriorated.
- Vehicles frequently struck the cable barrier, but generally avoided severe injuries. Such crashes were typically caused by skidding, flat tires, or failure to maintain control of the vehicle.
- Maintenance costs increased to about 120,000 Swedish kronor (about $15,000) per kilometer and per year, about two thirds of which was repairing the cable barrier. Work zone safety while repairing the cable was also a concern.

A recent scan tour sponsored by the National Cooperative Highway Research Program (NCHRP) looked at characteristics of 2+1 roads in several European countries to determine the potential applications of the design for use in the United States (*7*). While specifics of the designs in the respective countries varied somewhat, the authors made comparisons of some of the key design and operational criteria, which are summarized in Table 6.

Table 6. Comparison of European 2+1 Road Characteristics (*7*).

	Germany	Finland	Sweden
Critical Transition Length, m (ft)	180 (590)	500 (1,600)	300 (1,000)
Non-Critical Transition Length, m (ft)	30–50 (100–160)	50 (160)	100 (330)
Typical Passing Lane Length, km (mi)	1.0–1.4 (0.6–0.9)	1.5 (0.9)	1.0–2.0 (0.6–1.2)
Separation between Opposing Traffic, m (ft)	0.5 (1.6)	0.3 (1.0)	1.25–2.0 (4.1–6.6)
Fatal+Injury Crash Rate, per 10^6 veh-km (10^6 veh-mi)	0.16 (0.26)	0.09 (0.14)	0.50 (0.80)
Typical Volumes, veh/day	15,000–25,000	14,000–25,000	4,000–20,000
NOTE: Sweden's 2+1 roads are separated by cable barrier, and their crash rates are specifically reported as crashes per million axle pair-km.			

The NCHRP authors concluded that the benefits of 2+1 roads in Europe validated a recommendation for their use in the United States to serve as an intermediate treatment between an alignment with periodic passing lanes and a full four-lane alignment. They also recommended that 2+1 roads were most suitable for level and rolling terrain, with installations to be considered on roadways with traffic flow rates of no more than 1200 veh/hr in a single direction. The authors discouraged the use of cable barrier as a separator, and they recommended that major intersections should be located in the buffer or transition areas between opposing passing lanes, with the center lane used as a turning lane.

Mutabazi et al. conducted a study for the Kansas Department of Transportation in 1999 that examined the location and configuration of passing lanes (*8*). Looking at conflicts and simulation results, the authors concluded the following:

- Through traffic and left-turn traffic from the side road do not appear to create a high risk to the main-highway through traffic. However, left-turn traffic from the main highway appears to create the highest risk.
- Intersections located within passing lanes do not necessarily present a risk to main highway traffic. In fact, the data showed that they have significantly fewer conflicts than those located outside the passing lane section.
- The collected data could not detect any significant difference between intersections located immediately after the passing lanes and those located some distance from the passing.
- The difference between percent time delay on side-by-side and head-to-head configurations was statistically insignificant at the 95 percent confidence level; however, those two ranked better than other configurations. The difference in percent time delay among different configurations as predicted by the simulation model appeared to differ only marginally.

Based on their conclusions the authors recommended that side road intersections, especially those with high volumes, be avoided within a passing lane section, if possible. Where a low-volume side road intersection is inevitable within a passing lane, the passing lane should be located so that the intersection is as close as possible to the middle of the passing lane. Side road intersections within lane drops and lane additions should be avoided. On highways where passing restriction by roadway geometry is insignificant, passing lanes can be located in either side-by-side or adjoining (head-to-head or tail-to-tail) configuration. However, in relation to urban areas and major intersections, it is recommended that passing lanes be constructed immediately following an urban area, rather than before. Similarly, passing lanes are recommended just past a major highway intersection rather than at the approach to a major intersection. The recommendations regarding the location of passing lanes in relation to urban areas and major intersections would automatically exclude the side-by-side passing lane configuration at these passing lane locations.

PASSING LANE LENGTH

Gattis, Bhave, and Duncan reported that the greatest benefits of passing lanes in their study of continuous three-lane cross sections (see Figure 2) were observed in the first 0.9 mile (*4*). Between 0.9 and 1.9 miles, the benefits were less pronounced but were more likely to accrue as volumes increased. Where continuous three-lane cross sections with alternating passing lanes segments are present, they concluded, agencies should reexamine the need for any passing lane that continues beyond approximately 1.9 mile in length. This study suggests that a rather high volume is needed before extra length produces any notable degree of extra benefits. It may be that the other direction of travel would benefit more from an earlier termination and a switch in the direction having the additional lane for passing.

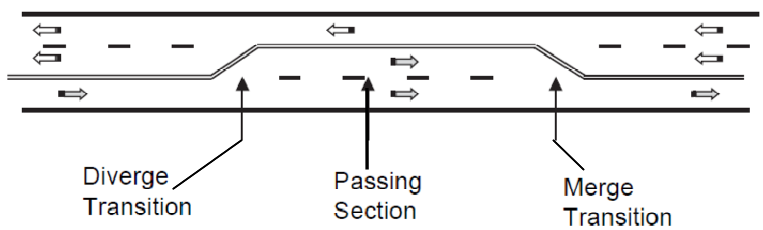


Figure 2. Schematic Example of Three-Lane Alternate Passing Design (*4*).

The lengths recommended by Gattis, Bhave, and Duncan are consistent with those used on European 2+1 roads, as shown previously in Table 6. The NCHRP study recommended that passing lane lengths on 2+1 roadways should be consistent with optimal lengths for isolated passing lanes on two-lane highways, as shown in Table 7 (*7*).

Table 7. Optimal Passing Lane Length for 2+1 Roads (*7*).

One-Way Flow Rate (veh/h)	Optimal Passing Lane Length (mi)
100	0.50
200	0.50–0.75
400	0.75–1.00
700	1.00–2.00

EVALUATION OF EFFECTIVENESS

Existing Guidance on Evaluating Super 2 Performance

The *Highway Capacity Manual* (HCM) provides guidance on the evaluation and analysis of existing passing lane sections, based on microscopic simulation, field data, and theoretical concepts (*9*). According to the HCM, the capacity of a two-lane highway is 1700 passenger cars per hour (pc/h) for each direction of travel, with a combined capacity of 3200 pc/h in both directions for extended lengths of highway. However, these theoretical capacities and the corresponding level of service are negatively affected by terrain, heavy vehicles, the peak hour factor, lane and shoulder widths, and other factors. Providing a passing lane on a two-lane highway in level or rolling terrain has a positive effect on the level of service in that direction of travel; this effect can be estimated by an operational analysis procedure.

The HCM analysis procedure provides a methodology for determining the appropriate section length for analysis, the percent time spent following, the average speed, and the level of service, among other metrics. However, the methodology is only intended for the analysis of a single passing lane section and its adjacent upstream and downstream two-lane sections. For analysis of the interactions between two or more passing lane sections (i.e., the support for appropriate passing lane spacing), the HCM recommends using simulation modeling and provides guidance on selected variables to consider in the simulation.

Measures of Effectiveness

Safety

Earlier work on the effects of passing lanes in reducing crashes indicated that there is a measurable effect to consider in the operational analysis. In data from 22 sites in four states, Harwood and St. John (*10*), found the crash rate reduction effectiveness of passing lanes to be 9 percent for all crashes and 17 percent for fatal and injury crashes. After combining the California study by Rinde (*11*) they concluded that the crash modification factor (CMF) for a conventional passing or climbing lane added in one direction of travel on a two-lane highways is 0.75 for total crashes in both directions of travel over the length of the passing lane from the upstream end of the lane addition taper to the downstream end of the lane drop taper. This CMF assumed that the passing lane is operationally warranted and that the length of the passing lane is appropriate for the operational conditions on the roadway. A later study by Harwood and Hoban

(*12*) summarizes the relative crash rates for passing lane sections and short four-lane sections, expressed as ratios between the expected crash rate for each and the expected crash rate of a conventional two-lane highway (see Table 8).

Table 8. Relative Crash Rates for Improvement Alternatives (*12*).

Alternative	All crashes	Fatal and injury crashes
Conventional two-lane highway	1.00	1.00
Passing lane section	0.75	0.70
Short four-lane section	0.65	0.60

Taylor and Jain (*13*) compared the crashes on highways with and without passing lanes in Michigan. Roadways were grouped into one of three ADT levels: less than 5000, between 5000 and 10,000, and greater than 10,000. Rather than doing a before-after study, the percentage differences in crash rates per million veh-mi of travel were compared to similar sites with and without passing lanes. For all three groups, fatal, injury, and total (the sum of fatal, injury, and property damage only [PDO]), crashes on the highways with passing lanes were lower than those without passing lanes.

Mutabazi et al. (*14*) in research for the Kansas Department of Transportation evaluated the safety effect of seven passing lane sections, three on US-54 and four on US-50. They used two methods: before-after analysis and cross-sectional analysis. In the before-after analysis, the before-period frequency and after-period frequency were estimated by the average of observed frequencies for each period. The crash frequency for the after period, assuming that the improvement (i.e., provision of passing lanes) was not implemented, was predicted from the trend of the before period and then adjusted for changes in traffic volume and differences in period lengths between before and after periods. Then, the crash reduction due to the improvement was determined with the assumption that the crash frequency during the after-period (with improvements) follows a Poisson distribution. From this analysis they concluded that the data were not sufficient to detect any safety improvement due to the highway improvement project. However, in a cross-sectional analysis where highways with passing lanes were compared with comparable highways without passing lanes, the sections with passing lanes had significantly fewer crashes than the state average rural two-lane road.

In their review of the application of European 2+1 roadway designs (*7*), Potts and Harwood found that the safety experience for two-lane highways with continuously alternating passing lanes was generally comparable to U.S. experience. Germany reported that crash frequency on two-lane highways with passing lanes was 28 percent less for total crashes and 36 percent less for fatal and injury crashes than comparable two-lane highways. In Finland, fatal and injury crashes in passing lanes were reported to be 11 percent lower than on comparable two-lane roads, and in Sweden, fatal and serious injury crashes were reduced by 55 percent after passing lane installation.

Platooning/Percent Time Following

Gattis et al. studied operations on selected passing lanes in northwest Arkansas (*15*). Two of the passing lanes were shorter than 1400 ft, while the third was longer than 2500 ft. At all three sites, vehicles had traversed roadway sections with limited passing opportunities before they encountered passing sections on slight to moderate upgrades. By studying traffic at such sites, the researchers observed the behaviors of motorists who may have been restrained by slower traffic ahead, but who then encountered a relatively unconstrained environment that allowed them to pass if they became displeased or frustrated with the confinement they experienced in the traffic stream.

The number of vehicles in platoons per hour increased linearly with the total traffic volume. A regression analysis on the data yielded the following linear relationship: number of vehicles in platoons/hr = −151 + 1.22 × (total one-direction volume). The R^2 value for the regression analysis was 0.97, with the independent variable ranging from 325 to 525 vph. A slightly smaller proportion of vehicles attempted to pass on the short lanes than on the long lanes. This could have reflected driver judgment that there was insufficient distance in which to complete a pass on the short lane sections. In both data sets, passing success declined when headways were greater than 2.0 sec.

They also found that at both the short lane and the long lane sites, when headways were 3.0 sec or more and platoon speeds were 50 mph or more, 85 percent of drivers exhibited little desire to pass. This suggests that many drivers may readily tolerate a slight level of congestion or platooning on two-lane rural roads. The findings from this research support the views of those who consider the 5.0-sec headway to be excessive when defining delay on two-lane rural highways. In the authors' opinion, a combination of both headway and platoon speed might more accurately define what the motorist considers to constitute delay.

Morrall and McGuire studied the effects of implementing a series of passing lanes on the Trans-Canada Highway (TCH) through the Rocky Mountain National Parks (*16*). The passing- and climbing-lane system on the TCH in the four mountain parks consisted of 29 auxiliary lanes, providing an average spacing of 8.3 and 9.1 km (5.2 and 5.7 mi) between assured passing opportunities eastbound and westbound, respectively. They estimated that the passing-lane system in Banff National Park has extended the design life of the TCH between Sunshine and Castle Junction interchanges as a two-lane facility by approximately 15 years. The effect of the passing-lane system overall resulted in a 6 to 7 percent reduction in percent time spent following in the 500- to 700-veh/h range, thus keeping the overall percent time spent following at less than 60 percent and hence maintaining LOS C. The authors also discovered a 20 to 25 percent increase in the number of overtakings in the 500- to 700-veh/h range.

Operating Speed

Potts and Harwood conducted an evaluation of the benefits and effectiveness of passing lanes in Missouri (*5*). They analyzed three roadway sections comparing traffic operations before and after installation of passing lanes on rural two-lane highways. The three sites had ADTs ranging from 4500 to 10,600 vpd, each with truck and recreational vehicle proportions of 10 and 5 percent, respectively. Analysis of the sections showed that the level of service improved noticeably at each site, based on the average travel speed and the percent time spent following; two of the three sites improved LOS by two letter levels. A review of crash data showed that the crash rates for two-lane highways with passing lanes were approximately 29 percent lower than the rates for traditional two-lane highways in the same districts.

CHAPTER 3
PRACTICES IN OTHER STATES

This chapter contains information from the design manuals, standards, and guidelines governing the design of passing lanes on 2-lane highways. The information was obtained from the online manuals available at the respective websites of the states' departments of transportation.

SUMMARY

Researchers reviewed the available information on the website of each state's department of transportation. Table 9 contains a summary of the information obtained in the website search.

Table 9. Summary of State Manuals.

State	Summary
Arizona	• Intervals of 3 to 5 miles, alternately in the opposing directions. • Length of passing lanes should allow several vehicles in line behind a slow-moving vehicle to pass before reaching the transition to the normal section. • Passing lanes should not be longer than 2 miles and not be shorter than 1300 ft.
California	• Should not normally be constructed on tangent sections where the length of tangent equals or exceeds the passing sight distance. • Where the ADT exceeds 5000, 4-lane passing sections may be considered.
Colorado	• Minimum recommended sight distance of 1000 ft on the approach to each taper. • Location should consider intersections and high volume driveways as well as bridges and culverts. • Minimum length, excluding tapers, should be 1000 ft.
Connecticut	• No design criteria for passing lanes. • Climbing lanes should have lane width of 11 ft, shoulder width of 4 ft.
Florida	• No information found on passing lanes; climbing lanes shall follow the same criteria for normal lanes and the lane should not terminate until well after the crest of the hill.
Idaho	• Passing lanes should be considered if volumes exceed ADTs in the Design Manual. • If separate passing lanes are used, the lanes should be separated by at least 1500 ft. • Minimum length should be 0.25 mile.
Illinois	• Passing lanes may be warranted on two-lane facilities where passing opportunities are not adequate. • Typical spacing for passing lanes may range from 3 miles to 10 miles. • The optimal length of passing lanes is usually between 0.5 mile and 1 mile.
Kansas	• Passing lanes are provided, should be at regular intervals of approximately 5 miles. • The width of passing lanes should be 12 ft. • The preferred configuration is side-by-side passing lanes with one in each direction thus creating a short four-lane section. • Lengths are taken from TTI report 0-4064-1.
Louisiana	• No directives on passing lanes; passing lanes may be considered if the two-lane road does not adequately give safe passing zones.
Michigan	• Design hour volumes used to identify candidate locations. • The lane widths should be 12 ft. • The desirable minimum length is 1 mile with an upper limit of about 1.5 miles.
Minnesota	• Passing lanes should normally be constructed systematically at regular intervals. • The optimal length of a passing lane to reduce platooning is usually 0.5 to 1.0 mile long.
Montana	• Passing lanes may be determined based on an engineering study.
New Hampshire	• Passing sections should be provided as frequently as possible in keeping with the terrain.
Ohio	• If capacity is restricted below the design LOS due to the lack of sight distance, consideration should be given to providing passing lane sections.
Oregon	• Should be considered on two-lane arterials without adequate passing sight distance. • Should be considered only in areas where the roadway can be widened on both sides.
Utah	• Localized improvements that optimize existing capacity for minimal cost.
Washington	• Desirable where sufficient safe passing zones do not exist and the warrant for a climbing lane is not satisfied.
Wisconsin	• If 20-year traffic projections exceed 12,000 AADT or exceed 1400 two-way DHV, it may be appropriate to consider expanding the facility to 4-lanes.

ARIZONA

Design specifications for Arizona can be found in Section 209.2 of the Roadway Design Guide (*17*). In rolling and mountainous terrain, passing sight distance may be difficult to provide economically over a significant portion of the highway. Vertical and horizontal curves meeting the requirements of passing sight distance are often more costly than those only meeting stopping sight distance requirements. Passing lanes may be the economical solution to providing passing opportunities on such highways.

Passing lanes should be considered on two-lane highways where passing sight distance cannot be provided at frequent intervals and where passing opportunities are negated by traffic volumes in the opposing direction. As with climbing lanes, a consideration to be weighed when implementing a passing lane on a two-lane highway is that Arizona practice (Traffic Engineering PGP) restricts passing in the opposing lane of traffic. As a minimum, passing opportunities should be provided at intervals of 3 to 5 miles. At distances greater than these, drivers will tend to tire of following slow moving vehicles and may take inappropriate risks to pass.

Generally, passing lanes are provided alternately in the opposing directions (a three-lane section). Under special conditions, a four-lane (two-directional) passing section may be provided with the approval of the Assistant State Engineer, Roadway Engineering Group. In selecting either the three- or four-lane section, consideration should be given to traffic volumes, construction costs, and the frequency of passing opportunities provided.

Passing lane shoulder widths should meet the widths established for the new mainline roadway. Care should be taken to avoid intersections within the passing lane zone. If an intersection cannot be avoided, intersection sight distance should be provided within the fully developed passing lane section.

The beginning and end of passing lanes should meet the criteria for adding and dropping lanes as provided in Section 207. For adding passing lanes to existing roadways, see the design memorandum entitled "A Policy on the Design of Passing Lanes and Climbing Lanes" on the Roadway Design website. If bicyclists are utilizing the facility, a minimum shoulder width of 4 ft should be provided. The Assistant State Engineer, Roadway Engineering Group approves the use of passing lanes by his/her signature on the final scoping document.

CALIFORNIA

Design specifications for California are located in Section 204.5(3) of the *Highway Design Manual* (*18*). Climbing and passing lanes are most effective on uphill grades and curving alignment where the speed differential among vehicles is significant. Climbing and passing lanes should normally not be constructed on tangent sections where the length of tangent equals or exceeds the passing sight distance, because passing will occur at such locations without a passing lane and the double barrier stripe increases delay for opposing traffic. Where the ADT exceeds 5000, four-lane passing sections may be considered. See Index 305.1(2) for median width standards. The Headquarters Division of Traffic Operations should be consulted regarding the length of climbing and passing lanes, which will vary with the design speed of the highway, the traffic volume, and other factors.

COLORADO

Design specifications for Colorado are found in Section 3.36 of the *CDOT Design Guide* (*19*). Passing lanes can be added on two-lane highways to improve traffic operation on sections of lower capacity and on lengthy sections (6 to 60 miles) where there are inadequate passing opportunities.

The logical location for a passing lane is where passing sight distance is restricted, but adequate sight distance should be provided at both the add and drop lane tapers. A minimum sight distance of 1000 ft on the approach to each taper is recommended. The selection of the location should consider the location of intersections and high volume driveways as well as physical constraints such as bridges and culverts that could restrict provision of a continuous shoulder.

Use the following design procedure to identify the need for passing sections on two-lane highways:

1. Design horizontal and vertical alignment to provide as much of the highway as practical with passing sight distance. See Passing Sight Distance column in Table 3-1.
2. Where the design volume approaches capacity, recognize the effect of lack of passing opportunities in reducing the level of service.
3. Determine the need for climbing lanes.

4. Where the extent and frequency of passing opportunities made available by application of Criteria 1 and 3 are still too few, consider the construction of passing lane sections.

Passing lane sections should be sufficiently long to permit several vehicles in a line behind a slow moving vehicle to pass before returning to the normal cross-section of two-lane highway. The minimum length, excluding tapers, should be 1000 ft. A lane added to improve overall traffic operations should be long enough, over 0.3 mile, to provide a substantial reduction in traffic platooning.

The transition tapers at each end of the added lane section should be designed to encourage safe and efficient operation. The lane drop taper should be computed from the *Manual on Uniform Traffic Control Devices* (MUTCD) formula: L=WS where L=length in feet, W=width in feet, and S=speed in mph. The recommended length for the lane addition taper is half to two-thirds of the lane drop length. The transitions should be located where the change in width is in full view of the driveway.

CONNECTICUT

Connecticut's online manual contained no details on passing lanes, but it did provide information on climbing lanes. Design specifications for Connecticut are found in Section 2-7.03 and Section 9-2.0 of the *Highway Design Manual* (*20*).

The design criteria in Section 9-2.0 will apply to existing or proposed climbing lanes within the limits of 3R projects (see Figure 3); however, for non-freeway projects, the following criteria are acceptable:

1. Lane Width. The minimum width of the climbing lane will be 11 ft.
2. Shoulder Width. The minimum width of the shoulder adjacent to the climbing lane will be 4 ft.

Highway Type	Design				
	Begin Climbing Lane	End Climbing Lane	Taper Length (Begin/End)	Lane Width	Shoulder Width
Freeways	45 mph	50 mph	300 ft / 600 ft	12 ft	Same as preceding roadway section
Other Facilities	10 mph below design speed or 45 mph, whichever is less	10 mph below design speed or 45 mph, whichever is less	25:1 / (1)	See Chapters Four and Five	Same as preceding roadway section

(1) The taper length on other facilities for ending the climbing lane will be determined by the following taper rates:

Design Speed (mph)	End Taper Rates
20	7:1
25	10:1
30	15:1
40	25:1
45	45:1
50	50:1
55	60:1
65	65:1
70	70:1
75	75:1

Figure 3. Design Criteria for Climbing Lanes, Connecticut *Highway Design Manual* Figure 9-2D (*20*).

FLORIDA

Design specifications for Florida are found in Chapter 3 of the *Florida Greenbook* (*21*). The criteria for a climbing lane and the adjacent shoulder are the same as for any travel lane except that the climbing lane should be clearly designated by the appropriate pavement markings. Entrance to and exit from the climbing lane shall follow the same criteria as other merging traffic lanes; however, the climbing lane should not be terminated until well beyond the crest of the vertical curve. Differences in superelevation should not be sufficient to produce a change in pavement cross slope between the climbing lane and through lane in excess of 0.04 ft per foot.

IDAHO

Design specifications for Idaho are found in Section 520 of the *Design Manual* (*22*). The capacity of a two-way, two-lane highway is a function of several variable traffic characteristics such as traffic volumes, number of commercial vehicles, roadway width, and passing opportunity. As traffic volumes increase, traffic queues can develop and create vehicle delays because the opportunity to pass another vehicle is restricted. The passing problem can be alleviated and the capacity of a two-lane highway improved when passing lanes are provided.

The purpose of a passing lane is to reduce vehicle delays at bottleneck locations such as on steep upgrades and to break up traffic platoons that can also cause following vehicle delays. The normally applied passing lane concept on hills are classified as climbing lanes that accommodate slow moving commercial vehicles on grades while allowing other faster vehicles to pass. The application and design of climbing lanes are addressed in the American Association of State Highway and Transportation Officials (AASHTO) *Green Book*.

Passing lanes are also an acceptable alternative on two-lane highways in level or rolling terrain to reduce traffic queue delays and improve the roadway capacity. Passing lanes are a cost-effective approach toward providing an adequate level of service on a two-lane facility where a four-lane highway may be neither economically nor environmentally feasible.

The need for passing lanes should be based on level of service calculations in accordance with the *Highway Capacity Manual*, Chapter 8, and utilizing the traffic and roadway characteristics for the roadway segment under study. The need for passing lanes on an existing highway can be determined from a field study of traffic platooning. Spot platooning or percentage of following vehicles is defined as the percentage of vehicles with headways (time gaps) of 5 seconds or less. This measure of spot platooning provides a lower value estimate of the percentage of time delay. The field study should be made at several spot locations to determine the percent of vehicles delayed. The field study will provide the following data:

- Identification of localized sections where passing lanes would be desirable.
- Field evaluation of a longer roadway section having a minimum total section time delay, but includes an isolated section of higher vehicle time delays.
- Field evaluation of segments with longer platoons at relatively uniform high speeds where engineering judgment is needed to determine drivers' acceptance of the platoon speed and constraints to select their own desirable speed.

A rural, two-lane highway will normally accommodate the average annual daily traffic (AADT) values shown in Table 10, assuming the design hourly flow is 15 percent of AADT and there is a 50/50 directional traffic distribution. The values in Table 10 can be adjusted for uneven directional distribution of traffic, lane, and shoulder width. The values are expressed as passenger car equivalents per day; requiring that the effects of heavy vehicles, trucks, buses, and recreational vehicles in the traffic stream be converted to equivalent passenger car volumes. Table 11 shows the minimum level of service criteria for two-lane highways related to time delay.

Table 10. Idaho Service Flows on Two-Lane Highways (*22*).

RURAL, TWO LANE HIGHWAY SERVICE TRAFFIC FLOWS EXPRESSED AS AADT (passenger car equivalents per day — 50/50 directional)							
Level of Service		Percent No Passing					
		0%	20%	40%	60%	80%	100%
Level Terrain	B	5,040	4,480	3,920	3,545	3,175	2,985
	C	8,025	7,280	6,720	6,345	6,160	5,975
	D	11,945	11,575	11,200	11,015	10,825	10,640
Rolling Terrain	B	4,855	4,295	3,545	3,175	2,800	2,425
	C	7,840	7,280	6,535	5,975	5,600	5,225
	D	11,575	10,640	9,705	8,960	8,585	8,025
Mountainous Terrain	B	4,665	3,735	2,985	2,425	2,240	1,865
	C	7,280	6,160	5,225	4,295	3,735	2,985
	D	10,825	9,335	8,400	7,465	6,905	6,160

Table 11. Idaho Level of Service Criteria for Two-Lane Highways (*22*).

Level of Service	Percentage of Time Delay on General Segments
A	30% or less
B	45% or less
C	60% or less
D	75% or less
E	75% or more
F	100%

If the traffic volumes (equivalent to passenger cars/day) exceed the tabular ADTs, or if the spot time delays exceed the value for the selected level of service, then passing lanes should be considered. Any geometric improvements to the existing highway can affect field data, making the above level of service criteria erroneous.

The location and configuration of a passing lane may be influenced by the need to alleviate an operational problem, adjacent development, terrain, or other factors. The following objectives should be considered relative to location:

- Choose a location that minimizes construction costs.
- Passing lane location should appear logical to the driver, i.e., on grades or where passing sight distance is restricted.
- Location should provide adequate sight distance for entrance and termination.
- Physical constraints such as bridges, culverts, and vertical cuts or drop-offs should be avoided because of costs.
- Passing lanes can also be considered when a realignment shift is needed to provide the width in the appropriate direction.

The configuration of multiple passing lanes is shown in Figure 4, with desirable and undesirable patterns noted. If separate passing lanes are used, the lanes should be separated by at least 1500 ft to reduce any conflicts between opposing traffic flows.

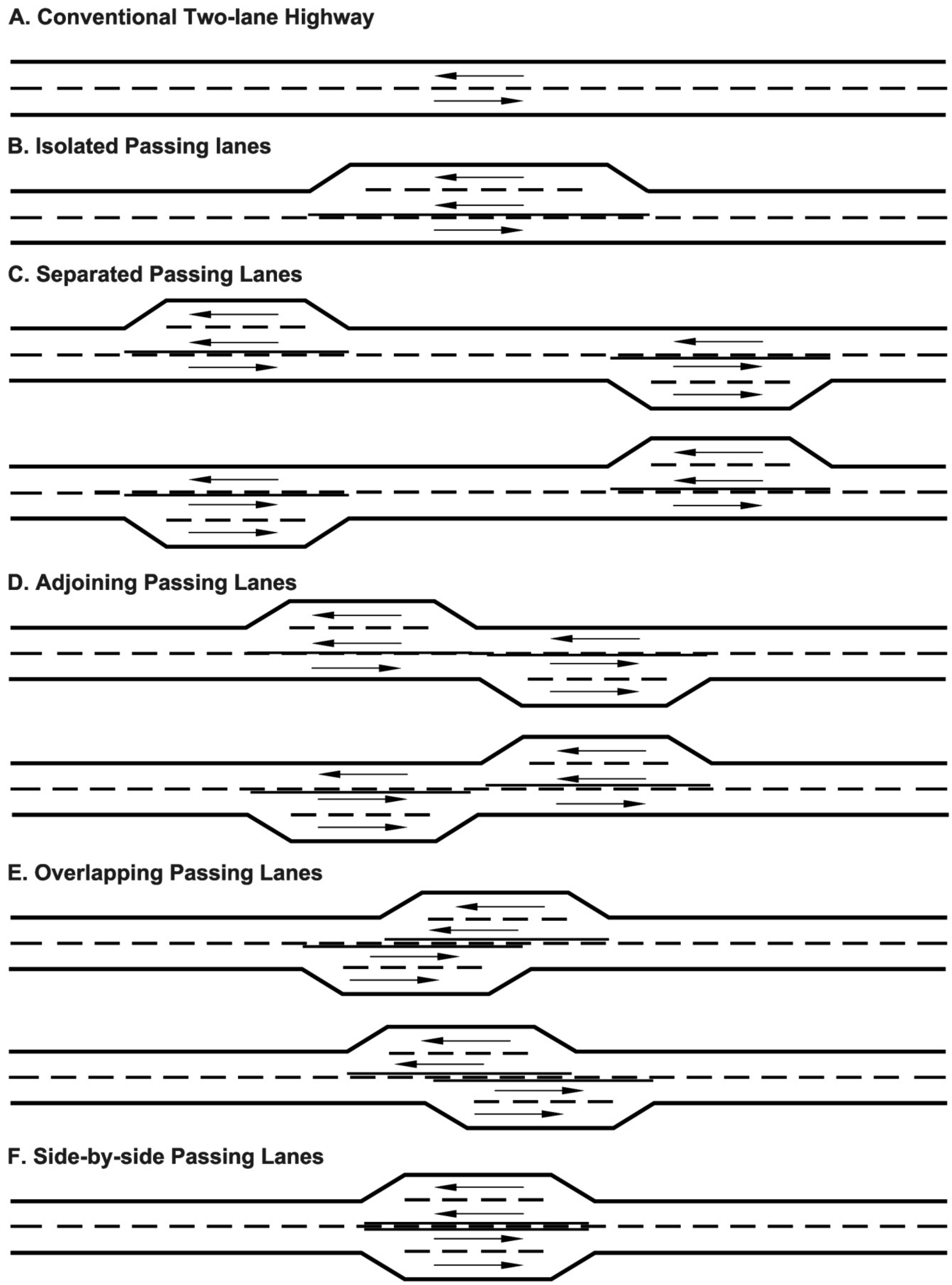


Figure 4. Idaho Alternative Passing Lane Configurations (*22*).

The minimum length of passing lanes should be 0.25 mile since anything shorter in length is not effective in reducing traffic platooning. Design lengths for passing lanes should be as shown in Table 12.

Table 12. Idaho Design Lengths for Passing Lanes (*22*).

One-Way Flow Rate (veh/hr)	Optimal Passing Lane Length (mi)
100	0.50
200	0.50–0.75
400	0.75–1.00
700	1.00–2.00

The spacing of passing lanes will depend primarily on the need to achieve satisfactory traffic operation. Normally, the operational benefits of a passing lane typically extend downstream from 3 to 8 miles. It is usually desirable to provide passing lanes at longer spacing with plans for intermediate passing lanes as the traffic volume increases. However, the spacing must be flexible to permit selection of suitable and inexpensive sites.

The geometrics of the passing lane should be similar to the adjacent two-lane highway. A minimum lane width of 12 ft is desirable with an adequate shoulder. The shoulder for the adjacent two-lane highway should be carried through the passing-lane section. The normal practice is to drop the right-hand lane, merging the traffic with the left lane (i.e., passing lane). Roadway transition length at the start and end of the passing-lane section should be in accordance with the AASHTO *Green Book*.

The pavement markings, delineations, and signing should conform to the *Manual on Uniform Traffic Control Devices*. Additionally, periodical signing along a highway segment with passing lanes to advise motorists of the distance to the passing lane is desirable. This advance signing will reduce driver impatience and reduce forced passing maneuvers.

ILLINOIS

Design specifications for Illinois are found in Chapter 47 the *Bureau of Design & Environment (BDE) Manual* (*23*). Passing lanes are defined as short added lanes that are provided in one or both directions of travel on a two-lane, two-way highway to improve passing opportunities. They present a relatively low-cost type of improvement for traffic operations by breaking up traffic platoons and reducing delay on facilities with inadequate passing opportunities.

Truck-climbing lanes are one type of passing lane used on steep grades to provide passenger cars with an opportunity to pass slow-moving vehicles. The warrant and design criteria for truck climbing lanes are discussed in Chapter 33 of the BDE Manual. Procedures for developing the climbing lane capacity analysis are also shown in Chapter 33.

Passing lanes may serve to improve safety on a segment of two-lane highway. Three-lane roadways may be considered an intermediate solution to the ultimate expansion to a four-lane highway. The various methods of providing the third lane are shown in BDE Manual Figure 47-2F.

Passing lanes other than truck-climbing lanes may be warranted on two-lane facilities where passing opportunities are not adequate. Passing lanes also may be warranted, based on an engineering study that includes judgment, operational experience, and a capacity analysis. The use of a passing lane will be determined on a case-by-case basis. For more information on passing lane warrants, see the FHWA publication *Low Cost Methods for Improving Traffic Operations on Two-Lane Roads*, Report No. FHWA-IP-87-2.

Design considerations are provided as follows:

1. Capacity Analysis. *Low Cost Methods for Improving Traffic Operations on Two-Lane Roads* presents approximate adjustments that can be made to the capacity methodology in the *Highway Capacity Manual*. These adjustments can be used to estimate the LOS benefits from adding passing lanes to two-lane facilities.
2. Spacing. When passing lanes are provided to improve the overall traffic operations over a length of roadway, they should be constructed systematically at regular intervals. Typical spacing for passing lanes may range from 3 miles to 10 miles (5 km to 15 km). Actual spacing of passing lanes will depend on the traffic volumes, right-of-way availability, and existing passing opportunities.
3. Location. When determining where to locate passing lanes, the designer should consider the following factors:
   a. Costs. Locate passing lanes to minimize costs. Rough terrain will generally increase the costs for construction of passing lanes.

b. Appearance. The passing lane location should appear logical to the driver. The value of passing lanes is more obvious to the driver at locations where passing sight distances are restricted or where opposing volumes are significant.
c. Horizontal Alignment. Avoid locating passing lanes on highway sections with low-speed horizontal curves.
d. Vertical Alignment. Where practical, construct passing lanes on a sustained upgrade. The upgrade will generally cause a greater speed differential between slow moving vehicles and passing vehicles. However, passing lanes in level terrain still should be considered where the demand for passing opportunities exceeds supply.
e. Sight Distance. Locate the passing lane where there will be adequate sight distance to both the entrance and exit tapers of the additional lane. Because of sight distance concerns, do not locate exit tapers just beyond a crest vertical curve.
f. Intersections. Use special care when designing passing lanes through intersections and high-volume commercial entrances.
g. Structures. Avoid placing passing lanes where structures (e.g., large culverts, bridges) will restrict the overall width of the traveled way, passing lane, and shoulders.
h. Alternative Configurations. See Figure 5 for various configurations of passing lanes.

4. Widths. Passing lane widths should be the same width as the adjacent travel lane width. Paved shoulder widths next to the passing lane should be a minimum of 4 ft (1.2 m).
5. Tapers. Design passing lanes by providing an additional lane to the right side of the traveled way; see BDE Manual Figure 47-2G. Develop the additional lane with an entrance taper of 25:1. For the exit taper, the most commonly used taper rate is 50:1. However, where a location warrants an extended length of taper, the following equation may be used:

$$L = WS \qquad \text{(US Customary) Equation 47-2.3}$$

$$L = 0.6WS \qquad \text{(Metric) Equation 47-2.3}$$

where: L = length of taper, ft (m)
W = width of passing lane, ft (m)
S = design speed, mph (km/h)

6. Length. The length of the passing lane will be determined by traffic volumes, length of the platoon, location of major intersections, geometrics, and distances between successive passing opportunities. The optimal length of passing lanes is usually between ½ mile and 1 mile (1 km and 1.5 km). At a minimum, passing lanes should not be less than 1000 ft (300 m) long. On the other hand, passing lane lengths greater than 1 mile (1.5 km) tend to have diminishing reductions in platooning per unit length.
7. Typical Design Layout. BDE Manual Figure 47-2G illustrates a typical design for a passing lane in one direction. Advance signing is necessary to indicate to drivers that passing opportunities exist ahead (e.g., PASSING LANE 1/2 MILES AHEAD). Coordinate the final signing and pavement marking placement with the Bureau of Operations.
8. Typical Sections. BDE Manual Figure 47-2G illustrates a cross section design for one directional passing lanes and Figure 47-2H (reproduced here as Figure 5) illustrates side-by-side passing lanes.

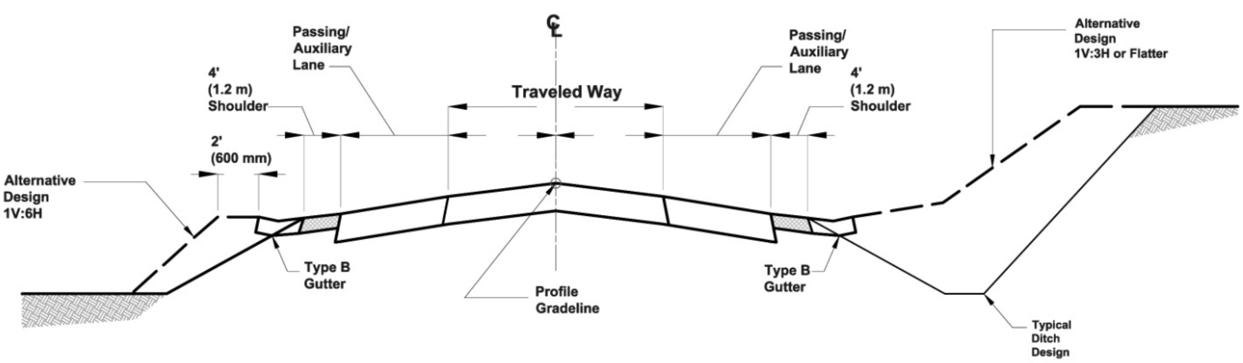


Figure 5. Illinois Typical Section for Four-Lane Passing Segment (*23*).

9. Four-Lane Sections. Short segments of a four-lane cross section, designated as side-by-side passing lanes in BDE Manual Figure 47-2F, may be constructed along a two-lane highway to break up platoons, to provide the desired frequency of safe passing zones, and to eliminate interference from low-speed vehicles. These sections may be advantageous in rolling terrain, where the alignment is winding, or where the profile includes critical grades in both directions. The decision to use a short four-lane segment, as compared to using a three-lane option, should be based on long-range planning objectives for the

facility, the availability of right-of-way, the existing cross section, topography, and the desire to reduce platooning and passing problems. Provide sufficient sight distance (e.g., 1000 ft [300 m]) in the transition area from the two-lane section to the four-lane section to allow a driver to anticipate the passing opportunity. Four-lane sections of 1 mile to 1.5 miles (1.5 km to 2.5 km) in length are usually sufficient to dissipate most queues formed by slow vehicles and terrain conditions.

KANSAS

Design specifications for Kansas are found in Chapter 7 of the *Design Manual* (*24*). A passing lane is an added lane provided in one or both directions of travel on a conventional two-lane highway to improve passing opportunities. The need for passing lanes is considered in the Planning and Program management stage. The width of passing lanes should be 12 ft (3.7 m). Shoulders on passing lanes should be 6 ft (1.8 m) in width. For additional information regarding passing lanes, refer to TTI Report 0-4064-1, "Design Guidelines for Passing Lanes on Two-Lane Roadways (Super 2)."

When passing lanes are provided, they should be constructed systematically at regular intervals of approximately 5 miles (8 km). The following factors should be considered when determining the specific location of the passing lanes:

1. Construction cost.
2. Sight distance at both the entrance transition and terminal transition tapers.
3. Major intersections and high-volume entrances locations – avoid these locations whenever possible. Where the presence of higher-volume intersections cannot be avoided, provisions for turning vehicles should be considered.
4. Avoid bridges, culverts, and other physical constraints if they restrict the provision of a continuous shoulder.
5. Driver expectation location should appear logical to the driver.
6. Centerline longitudinal grade – a relatively level section or a sustained grade are the preferred locations.
7. Existing or proposed climbing lanes – coordinate with these features.

A passing lane has an entrance transition, a passing lane section, and a terminal transition. Table 13 presents guidance for passing lane length as a function of traffic volume.

Table 13. Kansas Guidelines for Design Lengths for Passing Lanes (*24*). (Does not include the entrance and/or terminal transition lengths)

ADT (vpd)		Recommended Passing Lane length (mi)	Recommended Passing Lane length (km)
Level Terrain	Rolling Terrain		
< 1950	< 1650	0.8–1.1	1.3–1.8
2800	2350	0.8–1.1	1.3–1.8
3150	2650	1.2–1.5	1.9–2.4
3550	3000	1.5–2.0	2.4–3.2

Reference: TTI Report 4064-1, "Design Guidelines for Passing Lanes on Two-Lane Roadways (Super 2)."

The terminal transition length should be computed by the formula:

In US Customary Units:

$L = W \times S$,

where: L = Length in feet,

W = Lane Width in feet, and

S = Design Speed in mph

In Metric Units:

$L = 0.6 \times W \times S$,

where: L = Length in meters,

W = Lane Width in meters, and

S = Design Speed in km/h

The entrance transition length should be one-half to two-thirds the terminal transition length.

The preferred configuration is side-by-side passing lanes with one in each direction, thus creating a short four-lane section. Separated passing lanes may be used in certain circumstances such as when adding a passing lane to an existing highway would require the acquisition of a house. Figure 6 provides an example of a passing lane in one direction of travel, and Figure 7 shows examples of passing lane configurations.

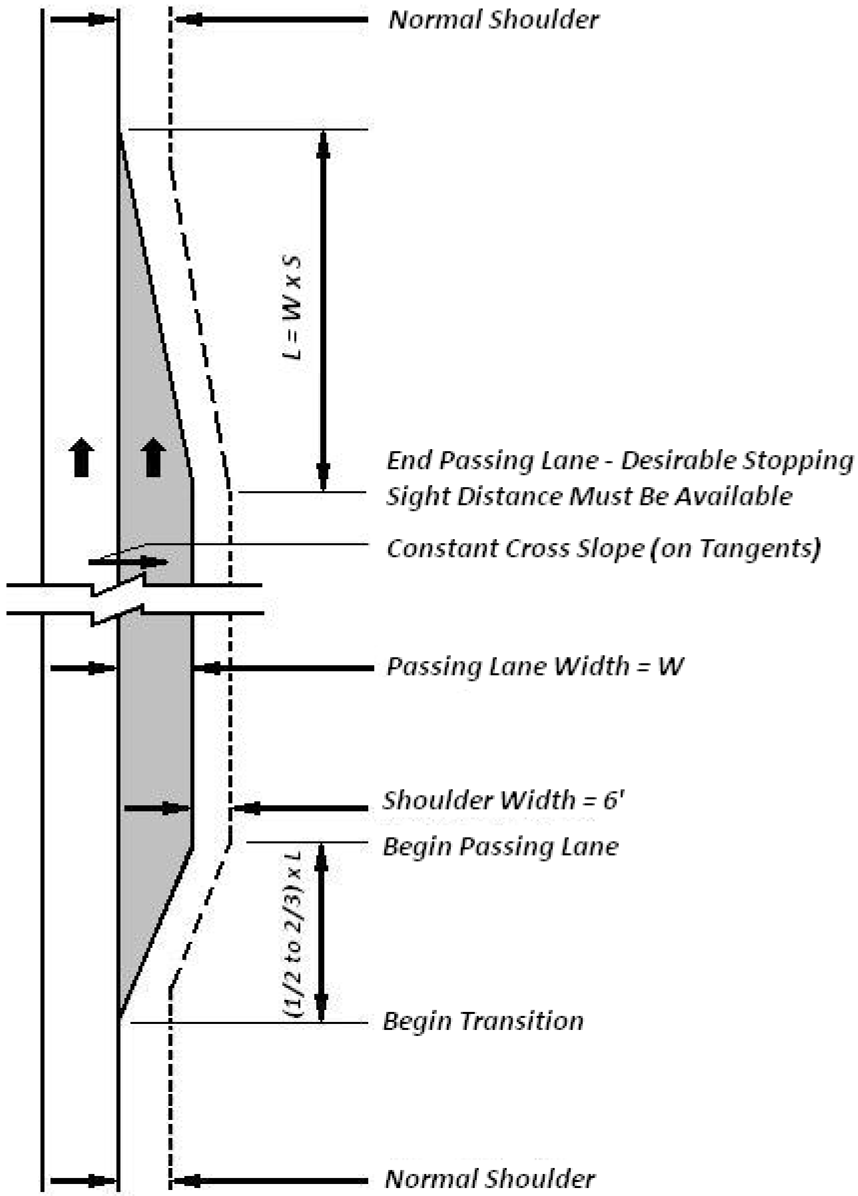


Figure 6. Kansas Example Passing Lane (adapted from *24*).

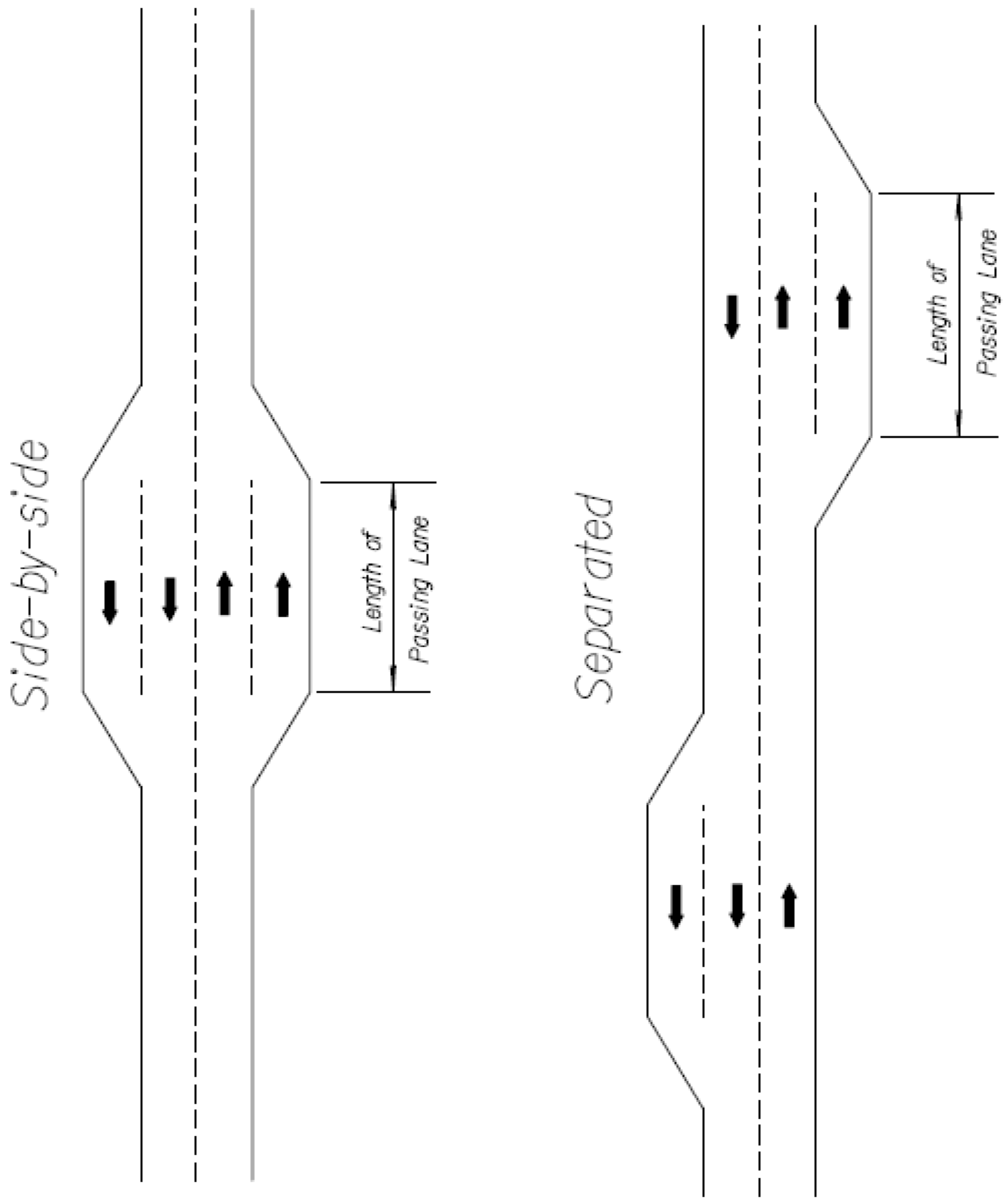


Figure 7. Kansas Passing Lane Configurations (*24*).

LOUISIANA

Design specifications for Louisiana are found in Chapter 4 of the *Roadway Design Procedures and Details* (*25*). Passing Sight Distance (PSD) is the length of roadway required for a vehicle to safely complete a normal passing maneuver. This value is not included in the Design Standards, and minimum values, as calculated using methods defined in Chapter III of the AASHTO *Green Book*, are appropriate. Lengths are calculated based on the passenger vehicle and an object height of 4.25 ft, equivalent to the height of the standard passenger vehicle.

When setting the horizontal and vertical alignment on a two-lane project, it is essential for the designer to provide as many areas as possible for safe passing maneuvers. If the horizontal and/or vertical alignments do not allow adequate length for passing, the use of passing lanes may be considered as discussed in Chapter III of the AASHTO *Green Book*.

MICHIGAN

Design specifications for Michigan are found in Chapter 3 of the *Road Design Manual* (*26*). A passing relief lane, which is either a Truck Climbing Lane (TCL) or a Passing Lane Section (PLS), is intended to reduce congestion and improve operations along two-way, two-lane, rural highways. The congestion (platoon forming) being addressed is the result of: (1) speed reduction caused by heavy vehicles on prolonged vertical grades (for TCL), and/or (2) slow-moving motorists in combination with high traffic volumes or roadway alignment limiting passing opportunities (for PLS). Platoons forming behind slow moving vehicles can be reduced or dispersed by increasing the speed or by increasing the opportunities to pass them. The conditions that cause the forming of platoons also restrict the passing opportunities needed to dissipate platoons, thereby increasing congestion.

The construction of Passing Relief Lanes (PRL) is not intended to connect existing multilane sections, but to provide a safe opportunity to pass slower vehicles. The Traffic and Safety Division should be contacted to provide assistance in project selection, location, and design based on these guidelines.

Passing Lane Sections (PLS) along two-way, two-lane rural routes are often desirable even in the absence of "critical grades" required for TCLs. PLS are particularly advantageous where passing opportunities are limited because of traffic volumes with a mix of recreational vehicles and/or roadway alignment. It is preferable to have a four-lane cross section for a PLS, but that is not always feasible because of right-of-way or environmental concerns.

Initially, design hour volumes (DHV) will be used in identifying candidate locations. Specific classification counts will be requested when required for comprehensive analysis. FHWA requests that they be advised on any Federal Aid Project in which the 30th high hour is not used as the DHV in warranting a PRL. A combination of the following should be considered in identifying the need for a PLS:

1. Combined recreational and commercial volumes exceed 5 percent of total traffic.

2. The level of service drops at least one level and is below Level B during seasonal, high directional splits.
3. The two-way DHV does not exceed 1200 vph. In situations where volumes exceed 1200 vph, other congestion mitigating measures should be investigated.

Desirably, PLS should be located in areas:

1. That can accommodate four lanes (PLS for each direction of traffic) so that the amount of three-lane sections is minimized.
2. With rolling terrain where vertical grades (even though not considered "critical grades") are present to enhance:
   a. Visibility to readily perceive both a lane addition and lane drop.
   b. Differential in speed between slow and fast traffic. This occurs on upgrade locations and produces increased passing opportunities.
   c. Slower vehicles regaining some speed before merging by continuing the PLS beyond the crest of any grade.
3. Relatively free of commercial and/or residential development (driveways) and away from major intersections.
4. Where radius of the horizontal curve is greater than or equal to 1900 ft.
5. With no restrictions in width resulting from bridges or major culverts, unless structure widening is done in conjunction with PLS construction.
6. That are farther than 750 ft from a railroad crossing.
7. Where directional spacing of approximately 5 miles can be maintained.

Design considerations for passing lane sections are described as follows:

1. The beginning and ending transition (tapers) areas of a PLS should be located where adequate decision sight distance is available in advance.
2. The added lanes should continue over the crest of any grade so that slower traffic can regain some speed before merging.
3. The beginning or approach taper should be at least 500 ft long.
4. The taper length L (feet) is approximately W×S, where W is the shift in feet and S is the posted speed in mph.

5. The lane widths on any PLS should normally be 12 ft wide.
6. PLS shoulders should be as wide as the shoulders on adjacent two-lane sections but no less than 4 ft (3 ft paved). Shoulders of 4 ft shall be limited to areas where wider shoulders are not feasible or environmental concerns prohibit wider shoulders.
7. The desirable minimum length of any PLS is 1 mile with an upper limit of about 1.5 miles.

MINNESOTA

Design specifications for Minnesota are found in Chapter 3 of the *Road Design Manual* (*27*). Many drivers are reluctant to pass a slower moving vehicle on two-lane highways unless they have sight distance of significant length. The designer should periodically provide a major passing opportunity to accommodate the conservative driver. If the roadside elements do not allow for a flatter curve, guidelines for passing lanes may be used.

The following guidelines are based primarily on the FHWA publication FHWA-87-2 "Low Cost Methods for Improving Traffic Operations on 2-lane Road," dated January 1987, and the 1990 and 1994 AASHTO, "A Policy on Geometric Design of Highways and Streets."

On two-lane highways, the passing lanes have two important functions: 1) to improve overall traffic operation by breaking up traffic platoons and 2) by reducing delays caused by inadequate passing opportunities.

The four-lane passing section is comprised of a two-lane highway with an added lane in each direction for improving passing opportunities. The three-lane passing section is comprised of a two-lane highway with an added lane in only one direction. A four-lane passing section is generally more desirable than a three-lane passing section because the three-lane passing section would normally restrict the passing opportunities in the single lane direction. If physical constraints do not allow the construction of a four-lane passing section, use two staggered three-lane passing sections (three-lane passing section in the first direction followed by a two-lane section then a second three-lane passing section in the second direction). Use advance signing to inform motorists of the upcoming passing opportunities and reduce their level of frustration and impatience.

When planning, designing, and implementing passing sections, the following six features should be considered:

1. Evaluation Methods.
2. Location.
3. Length.
4. Spacing.
5. Geometrics.
6. Signing and Marking.

Currently there are no specific warrants for passing lanes used in the United States. However, there are several methods available for assessing the effectiveness of proposed highway improvements and determining whether such improvements are warranted for a given road and traffic volume. These methods can be considered in five groups: operational criteria, level of service criteria, cost effectiveness analysis, benefit-cost analysis, and safety methods.

1. Operational criteria are direct measures of the effectiveness of a proposed improvement, such as the percent reduction in vehicle platooning, travel time, or crashes. These are important measures for evaluating alternatives and determining appropriate design characteristics.
2. The determination of need for passing improvements is usually based on a level of service analysis. The levels of service on two lane highways are defined in chapter 8 of the 1994 *Highway Capacity Manual* in terms of the percentage of time spent delayed, i.e., traveling in platoons behind other vehicles. The level of service concept provides a set of uniform operational criteria for assessing existing conditions, comparing improvement alternatives, and setting targets of operating conditions on a given highway network. The cost of achieving the target level of service should also be considered.
3. Cost-effectiveness analysis considers the cost of achieving a given level of improvement. The analysis is done by calculating a ratio, such as percent crash reduction per thousand dollars of expenditure. Such ratios can be used to compare different types of investments and to examine the incremental or marginal effects (i.e., the added benefits verses the added costs) of different designs.
4. Benefit-cost analysis provides a more accurate and detailed method for taking into account the economics of highway expenditures. This analysis provides a measurement of operational and safety improvements vs. cost.

5. Safety evaluation procedures may make use of operational, cost-effectiveness, or benefit-cost analysis. The objectives are to identify high crash locations and to estimate crash reductions that may be expected from proposed road improvements. These are generally determined from research studies.

When passing lanes are to be provided to improve overall traffic operations over the length of a highway, they should normally be constructed systematically at regular intervals. For passing improvements, the evaluation should consider traffic operation for an extended highway length, up to 50 miles or an entire major section. See Chapter 2-5.01 for the definition of a major section. The following are some factors that should be considered in choosing locations for the passing lanes:

1. The passing lane location should appear logical to the driver. The value of the passing lanes is more obvious where passing sight distance is restricted rather than on long tangent sections, which already provide passing opportunities.
2. Highway sections with low speed horizontal curves should be avoided.
3. Passing lanes are also effective in level terrain where the demand for passing opportunities exceeds supply.
4. Safe and effective passing lane operations require adequate sight distance on the approach to both the lane addition and lane drop tapers. A minimum sight distance of 1000 ft on the approach to each taper is required.
5. Comparative construction costs should be considered when selecting the location of a passing lane.
6. Other physical constraints, such as bridges and culverts, should be avoided if they restrict the provisions of a continuous shoulder.
7. The passing section shall be located where a minimal number of entrances are present. On the lane drop side, entrances are prohibited in the area of a lane drop transition and 170 ft beyond unless approved by the Geometrics Engineer. On the two lane side, entrances are undesirable over the same distance. See figure 3 4.05A.
8. Public road intersections are undesirable anywhere within the passing lane section. If a public road intersection cannot be avoided, it should have a very low ADT and good sight distance. Exclusive left turn lanes should be considered. Public road intersections are

extremely undesirable near the end or beginning of a passing lane section. If such an intersection cannot be avoided near the end (beginning) of a passing lane section, the passing lane should be extended a minimum of 900 ft past (prior to) the intersection.

9. Districts are strongly encouraged to acquire access control throughout the passing lane section to prevent new entrances from being built.
10. Contact your District Traffic Engineer for signing requirements.

To improve overall traffic operations on a two-lane, two-way highway, the passing lane should be long enough to provide a substantial reduction in traffic platooning. The passing lane length, as used here, does not include the lane addition and lane drop transitions. The optimal length of a passing lane to reduce platooning is usually 0.5 to 1.0 mile long. As the length increases above 1.0 mile, passing lanes generally decline in cost-effectiveness per unit length and provide diminishing reductions in the platooning of vehicles.

The length of passing lane sections (excluding lane addition and lane drop tapers) should be based on the highest existing daily flow rate (vehicles per hour in one direction). Table 14 shows the length guidelines.

Table 14. Minnesota Guidelines for Lengths of Passing Lanes (*27*).

Daily Flow Rate (vph)	Passing Lane Length (mi)
100	0.5
200	0.5–0.75
400	0.75–1.0
700 or higher	1.0

Spacing of passing lanes will depend primarily on the magnitude of improvements needed to achieve satisfactory traffic operations. The operational benefits of a passing lane typically carry over in reduced traffic platooning for 3 to 9 miles downstream, depending on traffic volumes and passing opportunities. Advance signing, up to 6 miles before the start of a passing lane section, should be provided to minimize driver frustration and risky passing maneuvers. On a highway that needs only a moderate improvement in passing opportunities, a good strategy may be to construct passing lanes initially at fairly large spacings. Where the need for improved passing opportunities is greater or grows with increasing traffic volumes, more passing lanes may be added.

The geometric design of the passing lanes considers the width of the lanes and shoulders, the addition of a lane, and the dropping of lane tapers, see *RDM* Figure 3-4.05A. The passing lane width shall be 12 ft. The desirable shoulder width is 10 ft (desirable) with a minimum of 6 ft. When a composite shoulder is used on the highway, the passing section should also use a composite shoulder. An example would be a highway that has a 10 ft composite shoulder comprised of 2 ft bituminous and 8.0 ft gravel (desirable), or 2 ft bituminous and 4.0 ft gravel (minimum). A 10 ft bituminous shoulder shall be used in the lane drop area and 500 ft (desirable) beyond to provide a recovery area for drivers who may encounter a conflict. A 1:25 taper transition should be used from the 10 ft shoulder to the normal shoulder.

Lane addition and lane drop tapers are to be carefully designed. Inadequate sight distance on lane addition and lane drop tapers can cause erratic, unsafe behavior of vehicles, and poor utilization of the passing lane. The lane addition taper should be designed at 1:50 rate, and the lane drop taper, at the downstream end of a passing lane, should be designed at 1:60 rate.

Passing lanes are much more effective if the majority of drivers enter the right lane at the lane addition transition and use the left lane for passing slower vehicles. Therefore, the geometric design of the lane addition transition should encourage drivers to enter the right lane. Signing and markings will also provide guidance for drivers to enter the right lane. For concrete pavements, the longitudinal joint should guide traffic into the right lane at the lane addition area. At the drop lane area, the right lane should be tapered out, and the inside lane longitudinal joints should proceed straight ahead. For construction details, see the CADD Directory.

The signing and marking criteria for passing lanes is discussed in the Minnesota Manual on Uniform Traffic Control Devices (MN MUTCD) and Mn/DOT's Traffic Engineering Manual.

MONTANA

Design specifications for Montana are found in Chapter 8 of the *Road Design Manual* (*28*). Passing lanes are defined as a short added lane provided in one or both directions of travel on a two-lane, two-way highway to improve passing opportunities. They may present a relatively low-cost improvement for traffic operations by breaking up traffic platoons and reducing delay on facilities with inadequate passing opportunities. Truck-climbing lanes are one type of passing lane used on steep grades to provide passenger cars with an opportunity to pass slow-moving trucks. The criteria for and design of truck-climbing lanes are discussed in Chapters 26 and 30 of the *Traffic Engineering Manual*.

Passing lanes other than truck-climbing lanes may be necessary on two-lane facilities where the desired level of service cannot be obtained. Passing lanes also may be determined to be necessary based on an engineering study that includes judgment, operational experience, and a capacity analysis. The use of a passing lane will be determined on a case-by-case basis. The Traffic Engineering Section is responsible for conducting the study to justify the need for passing lanes. For more information on passing lane guidance, see the FHWA publication *Low Cost Methods for Improving Traffic Operations on Two-Lane Roads*, Report No. FHWA-IP-87-2. The Report discusses the following for passing lanes:

1. Their location and configuration.
2. Their length and spacing.
3. Geometrics.
4. Signing and pavement marking.
5. Operational and safety effectiveness.

The Report also presents approximate adjustments that may be made to the highway capacity methodology in Chapter Eight of the *Highway Capacity Manual* to estimate the level-of-service benefits from adding passing lanes to two-way facilities.

NEW HAMPSHIRE

Design specifications for New Hampshire are found in Chapter 4 of the *Highway Design Manual* (*29*). Passing sections are used to provide opportunities to pass slower moving traffic on two-lane highways. Passing sections should be provided as frequently as possible in keeping with the terrain.

The extent of restrictive sight distance has a considerable effect on the design capacity of a two-lane highway. Sight distances to the road surface in the range of 450–600 m at frequent intervals are considered essential if the gaps in the traffic stream created by slow-moving vehicles are to be filled and a more desirable operating speed maintained. This measurement criterion is selected for the purpose of evaluating design capacity on two-lane highways. Both horizontal and vertical sight distance should be determined and the restricted portion expressed as a percentage of total length of highway for evaluation purposes. Refer to the *Green Book* for passing sight distance criteria.

OHIO

Design specifications for Ohio are located in Section 200 of *Roadway Standards* (*30*). If the available passing sight distance restricts the capacity from meeting the design level of service, adjustments should be made to the profile to increase the available passing sight distance. If, after making all feasible adjustments to the profile, capacity is still restricted below the design level of service due to the lack of sufficient passing sight distance, consideration should be given to providing passing lane sections or constructing a divided multi-lane facility.

OREGON

Design specifications for Oregon are located in Chapter 5 of the *Roadway Engineering Manual* (*31*). Passing lane specifications are in Chapter 5.10.2. Passing lanes should be considered on two-lane arterials where it is not practical to achieve adequate passing sight distance or where increased traffic volumes have an adverse impact on the desired LOS. Ideally, passing lanes should be considered only in areas where the roadway can be widened on both sides to provide simultaneous passing opportunities for both directions.

The standard travel lane for a passing lane section is 12 ft. The desirable shoulder width should be 6 ft with a minimum of 4 ft. If the roadway has substantial bike use, consult the ODOT Bicycle-Pedestrian Program Manager for input on shoulder width. The minimum median width in a passing lane section (three or four lanes) shall be 2 ft.

If at all possible, passing lanes should be located where there are no approaches (driveways or intersections). If there are existing approaches, the type of approach is critical. Consideration of closing the approach should be given. It may be possible to allow a passing lane where there are single residential approaches or possible forest service type roads, but the approach to public/county roads and approaches that serve multiple trip generation opportunities are not favorable in a passing lane section. Other poor locations for passing lanes include those that require ending the passing lane at the crest of a hill or on a curve, or where there is potential for left turns at the end of the passing lane.

Passing lanes should be clearly identified to prevent motorists from thinking they are entering a four-lane section of roadway. The minimum length of a passing lane should be 1250 ft, plus tapers. The taper section at the end of a climbing lane should be computed by the following formula: L=WS (L=Length in ft, W=Width in ft, S=Speed in mph). The recommended length for the lane addition taper is half to two-thirds of the lane drop length. Optimum passing length is 1.25 miles. It is very important to have passing lanes long enough to allow the passing of vehicles but not too long as to make the added passing lane seem like an additional travel lane. The Transportation Planning Analysis Unit (TPAU) should be contacted to determine the appropriate length of passing lane.

Design considerations for providing passing lanes on two-lane highways are as follows:

1. Horizontal and vertical alignment should be designed to provide as much length as feasible with sight distance for safe passing.
2. To maximize safe operations, drivers should be able to clearly recognize both lane additions and lane drops.
3. For volumes approaching design capacity, the effect of lack of passing lanes in reducing capacity should be considered.
4. Where the traffic is slowed or capacity reduced because of trucks climbing long grades, construction of climbing lanes should be considered.
5. Where the passing opportunities provided by application of Items 1 and 4 are still inadequate, the construction of a four-lane highway should be considered. Inability to economically justify climbing lanes or multilane may require that the roadway be designed for the minimum acceptable level of service.
6. Consider providing extensions to the passing lane section to allow slower vehicles the opportunity to attain free-flow speed prior to merging. This reduces the speed differential between vehicles at the merge, improving safety and operations.

UTAH

Design specifications for Utah are located in Chapter 7 of the *Roadway Design Manual of Instruction* (*32*). Passing lanes are a safety measure because they help reduce the number of collisions caused by unsafe passing choices of impatient drivers on rural two-lane highways.

They are localized improvements that optimize existing capacity for minimal cost. They also increase capacity, especially on rural highways.

WASHINGTON

Design specifications for Washington are located in Division 10 of the May 2008 *Design Manual* (*33*). Passing lanes are desirable where a sufficient number and length of safe passing zones do not exist and the speed reduction warrant for a climbing lane is not satisfied. Figure 8 may be used to determine whether a passing lane is recommended.

When a passing lane is justified, design it in accordance with Figure 9. Make the lane long enough to permit several vehicles to pass. Passing lanes longer than 2 miles can cause the driver to lose the sense that the highway is basically a two-lane facility. Where practicable, locate passing lanes on an upgrade to increase their efficiency. Passing lanes are preferably four-lane sections; however, a three-lane section may be used. When a three-lane section is used, alternate the direction of the passing lane at short intervals to ensure passing opportunities for both directions and to discourage illegal actions of frustrated drivers.

Make the passing lane width equal to the adjoining through lane and at the same cross slope. Full-width shoulders for the highway class are preferred; however, with justification, the shoulders may be reduced to 4 ft. Provide adequate signing and delineation to identify the presence of an auxiliary lane.

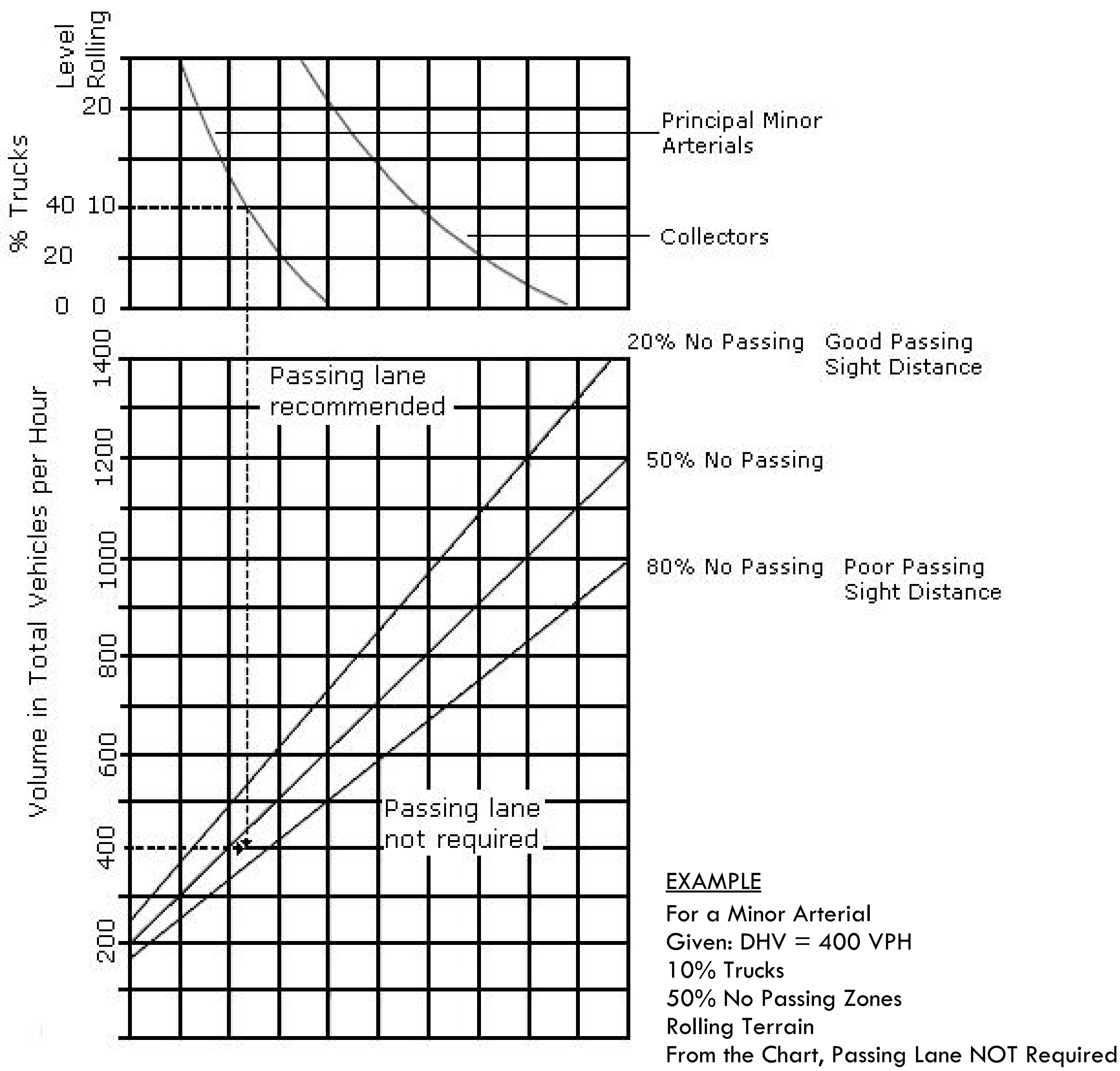


Figure 8. Washington Warrant for Passing Lanes (adapted from *33*).

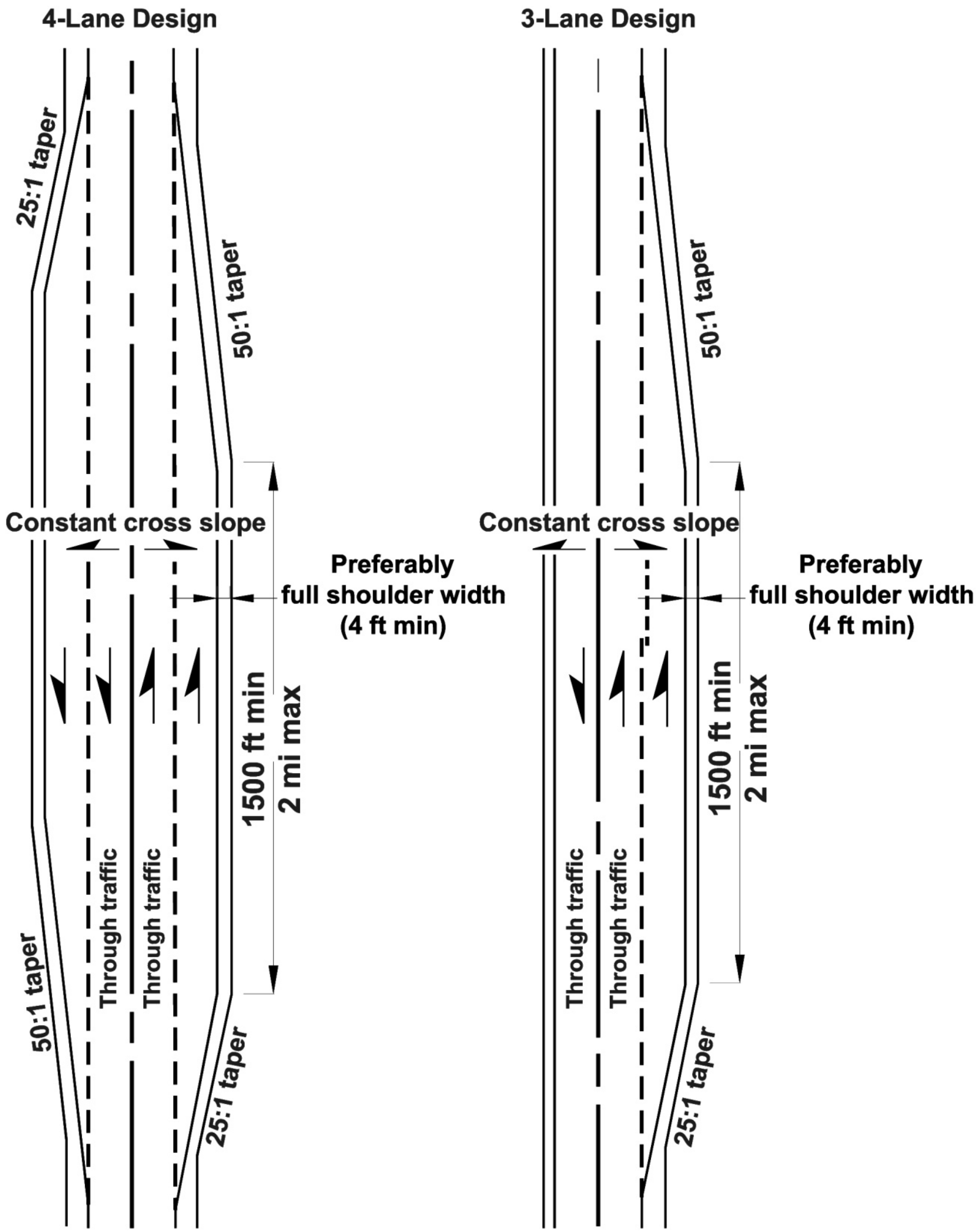


Figure 9. Washington Auxiliary Passing Lane Configurations (*33*).

WISCONSIN

Design specifications for Wisconsin are located in Chapter 11 of the *Facilities Development Guide* (*34*). A passing lane is an auxiliary lane constructed alongside a two-way, two-lane rural highway to provide the desired frequency of safe passing zones. Passing lanes are particularly advantageous where passing opportunities are limited because of traffic volumes, roadway alignment, or a high proportion of slower vehicles. Passing lanes differ from truck climbing lanes in that passing lanes are provided regardless of topography.

Passing lane areas should be access-controlled early in the process to protect the corridor from potential conflicts. Corridor lengths of 15 to 50 miles (24–80 km) are appropriate for planning and design purposes. Designers must also consider logical termini and abutting projects, such as Corridors 2020. Some sections of the corridor may not warrant passing lanes at the same time or with the same urgency as others; however, the entire corridor should be reviewed as a whole.

The general guidelines for selecting appropriate locations for passing lane segments are given below:

1. Passing lanes should be constructed in segments of highway that have a minimal number of entrances and preferably no side roads. For some passing lane segments it may be necessary to include side roads. When selecting a site for a passing lane facility, avoid side roads with 500 ADT and over. Driveways and field entrances should be avoided in the merge taper area on either side of the highway. The merge area extends from the W4-2R sign (lane reduction transition) to the end of the taper, or 1200 ft (366 m). No driveways or intersections should be located closer than 500 ft (152 m) from the end of the downstream taper. Designers should consider relocating field entrances and driveways in the merge area. A commercial driveway may be more problematic than a side road, depending on peak hour usage and traffic mix.
2. A widened segment of roadway, with protected left turn lanes, may be constructed in a passing lane section to provide for the left turning traffic when left turn volumes are significant. In those limited areas where four-lane undivided passing lane sections are required, crossing intersections are not permitted and tee intersections are not desirable.
3. If the comparative cost for construction of passing lanes in rolling and level terrain is nearly the same, it may be desirable to construct them in the rolling terrain at locations

where passing sight distance is unavailable, leaving flat sections for normal passing during the off peak periods. Avoid passing lanes on horizontal curves greater than 3 degrees, if possible.

Determine the current and design year (projected 20-year) Average Annual Daily Traffic and two-way Design Hour Volume. Use the 100th highest hour (K100) when determining the DHV. On most rural two-way highways the DHV ranges from 10 percent to 15 percent of the AADT. Recreational routes, however, can have a significantly higher percentage of traffic in the DHV. Districts should consult with their Systems Planning and Operations section to get site specific hourly counts for recreational routes (including weekends) in order to gain a more realistic understanding of the situation. Generally, if the 20-year traffic projections exceed 12,000 AADT or exceed 1400 two-way DHV it may be appropriate to consider expanding the facility to four lanes. The district will consider the priority and funding of all projects, and then determine whether passing lanes or other treatment is most appropriate.

When the 20-year projected, two-way DHV falls between 200 and 1400 use the nomograph provided in the Facilities Development Guide and the DHV from the Traffic Forecast to see if passing lanes should be considered further. Note this nomograph is from the Washington State DOT design manual so "rolling" implies a high degree of elevation variation. Higher priority highways will generally have design year AADT > 3500 and <12,000; two-way DHV greater than 400 and less than 1400; passing opportunity less than 61 percent; trucks and RVs greater than 4 percent.

1. Passing lane width is normally 12 ft (3.6 m) for new construction, reconstruction, and 3R projects.
2. Shoulders should be full width, similar to the adjacent two-lane highway section, for the classification and ADT of the facility. Shoulders should be paved similar to the adjacent two-lane facility. Designers may consider providing less than standard shoulder width in certain areas where excessive cuts and fills would substantially increase the construction cost. In such cases the designer must request an exception to design standards.
3. Minimize the occurrence of four-lane sections of undivided highways (overlapping passing lane areas).

4. It is important, where possible, to provide advancing traffic with the experience of the passing lane prior to seeing it in the opposing lane.

The clear zone on newly constructed passing lane sections, independent of project type, shall be computed from the outermost lane, outside edge of traveled way. On new construction and reconstruction projects the clear zone shall meet new construction standards. On reconditioning projects the desirable clear zone adjacent to new passing lanes is the new construction standard. The minimum clear zone on reconditioning projects is the greater of the built clear zone distance from previous construction or the 3R clear zone requirement. Justification for not meeting/exceeding the desirable new construction standard shall be stated in the DSR. Resurfacing and pavement replacement projects will typically not include the construction of new passing lanes. The optimal passing lane length, excluding tapers, is provided in Table 15 and is based on design year two-way DHV.

Table 15. Wisconsin Guidelines for Lengths of Passing Lanes (*34*).

Two-Way Total DHV	Length of Passing Lane (mi)
Less than 600	0.5–1.0
600–1000	0.75–1.5
1000–1400	1.0–2.0

Provide 3- to 8-mile (5 to 13 km) spacing between passing lanes in the same direction of traffic. This spacing depends on traffic volumes and passing opportunities outside of the actual passing lane location. The spacing must be flexible to permit selection of suitable and inexpensive passing lane locations.

Other design and operational considerations are provided as follows:

1. Passing lane approach and merge taper lengths should be 700 ft (213 m).
2. Passing lanes should be designed with good visibility at the end of merge taper. Do not end a merge taper at or near the crest of a hill. The end of the taper should be physically visible from the W4-2R sign (lane reduction transition).

3. Access is undesirable on either side of the highway in merge taper areas. Do not end merge tapers immediately prior to an intersection. Provide a minimum of 500 ft (152 m) of space downstream from the end of the taper to the nearest access point.
4. Signals downstream from passing lanes should be at least 1 mile (1.6 km) from the closest merging taper end.
5. A merge taper shoulder may include rumble strips and/or raised pavement markers.

Drivers may not know if the extra lane they encounter is a passing lane or a truck climbing lane. For driver expectancy and design consistency similar signing and pavement marking standards should apply where practical.

Provide diagonal skip-dash pavement marking at the entrance taper to guide traffic to the right when the shoulder width and construction is the same as the adjacent two-lane facility. Do not install the skip-dash pavement marking when the shoulder width is less than standard for the facility.

Allow passing by opposing lane traffic if passing sight distance is available. This is allowed in accordance with the *MUTCD* and *Highway Capacity Manual*. Studies have found no adverse problems with this procedure. Districts should consider side roads, commercial driveways, or other situations when it may be desirable to provide a double yellow at the center line.

CHAPTER 4
TXDOT STATE-OF-THE-PRACTICE

This chapter contains responses and findings from a questionnaire designed to collect information from the TxDOT districts and areas on existing Super 2 locations and policies on new installations. Questionnaire responses provided a sense of TxDOT's current state-of-the-practice and facilitated finding potential study sites at which to collect field data.

RESPONSES TO QUESTIONNAIRE

In order to determine the locations of Super 2 Highways constructed in Texas, a questionnaire was developed and distributed to TxDOT Area Engineers, who are generally responsible for overseeing a two- to four-county region. The questionnaire was designed to ask about the presence and location of passing lanes in each area, along with brief insights as to the reason(s) the passing lanes were installed. On December 12, 2008, the questionnaire was sent out to each Area Engineer through the Project Director, with a response deadline of January 2, 2009; respondents were asked to answer as many as six questions. Out of the 110 TxDOT areas, 21 surveys were returned. The questions and respective responses are described in the remainder of this section.

Question 1

The first question in the questionnaire was worded as follows:

Are there passing lane sections on two-lane highways currently located within your area? If yes, please continue with Question 2. If no, your response is complete; thank you for your time!

As shown in Figure 10, of the 21 responses received, nine said that there was at least one Super 2 section in their Area. Several of these nine responses described multiple highways in their area, each of which was reviewed for inclusion in Task 3 field studies. Six responses initially indicated that they had passing lanes, but a follow-up question confirmed that they were referring to climbing lanes. The remaining six respondents indicated that there were no passing lanes in their areas.

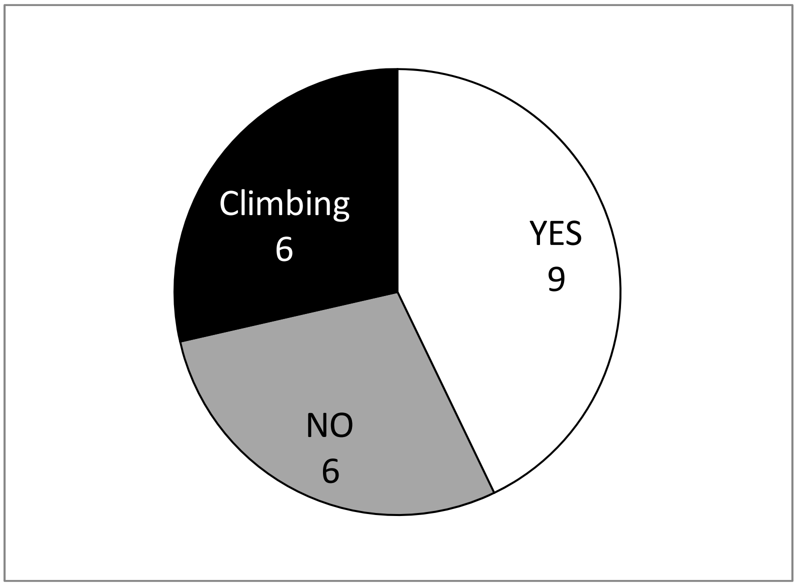


Figure 10. Responses to Question 1.

To illustrate the geographical distribution of the responses, Figure 11 contains a Texas county map that shows the counties covered by the responses received. The counties shaded blue represent affirmative answers to Question 1, red-shaded counties represent negative answers, and gray counties are in TxDOT Areas that contain climbing lanes. White-shaded counties indicate that no response was received for those counties.

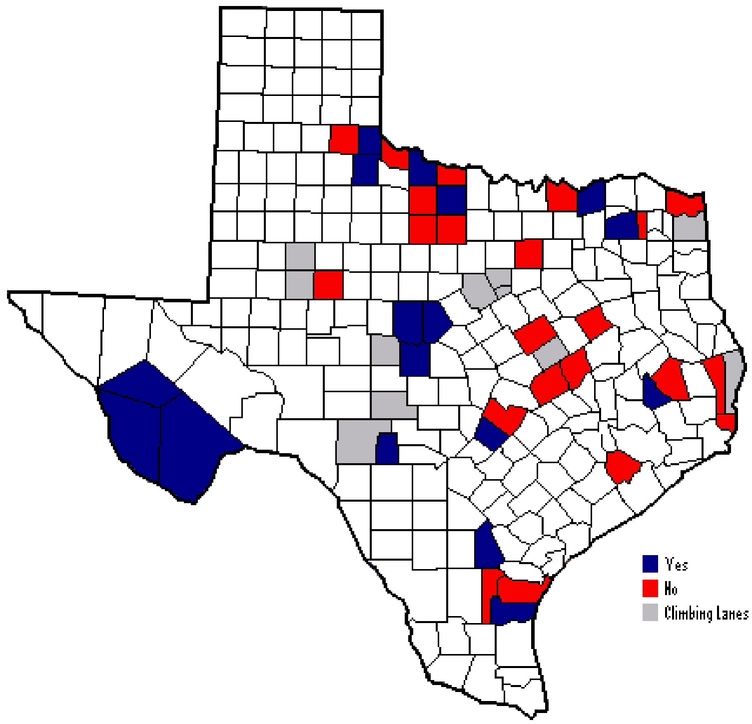


Figure 11. Counties Represented in Responses to Questionnaire.

Question 2

Discussion of Questions 2 through 6 is limited to the nine respondents who gave affirmative responses to Question 1. Question 2 asked:

Where are your current passing lane sections located? (Please list by Highway, County, and Limits [e.g., highway intersections or Control & Section with Beginning and Ending Milepoints].)

Responses to Question 2 varied in their scope and detail. Some responses provided the location of a single passing lane, while others described multiple Super 2 corridors. Location data were given by Reference Marker, by county line and city limit, and by highway intersection. Table 16 provides a summary of the responses to Question 2, listing the locations of the passing lanes and corridors provided. A map showing the approximate locations of Super 2 highways is shown in Figure 12.

Table 16. Locations of Passing Lanes Provided in Response to Question 2.

Highway	County	Corridor Limits	Approximate Length of Corridor (mi)
SH 121	FANNIN	Bonham City Limits to Collin County Line	17.7
US-62/83	CHILDRESS/COTTLE	US-62/70 junction to US-62/83 split	46.8
US-281	ARCHER	1 mi north of FM 172 to 4 mi south of FM 1954	8.5
FM 12	HAYS	RM 452+00.157 to 454+00.773	2.6
SH 11	HOPKINS	SH 19 to Commerce City Limits	16.5
US-59	LIVE OAK	I-37 to Bee County Line	7.7
SH 285	KLEBERG	US-77 to Brooks County Line	11.9
SH 206	COLEMAN	RM 330-1.077 to 338+1.624	10.7
SH 153	COLEMAN	RM 354+0.254 to 358-1.336	2.4
US-67	COLEMAN	RM 598-1.118 to 610+0.723	13.8
SH 279	BROWN	RM 320+2260’ to 336+2260’	16.0
US-377	BROWN	RM 438-970’ to 436+3625’	1.5
FM 45	BROWN	RM 346+4290’ to 356-5204’	10.2
US-87	McCULLOCH	RM 536+1330’ to 562-7562’	24.3
US-283	McCULLOCH	RM 394+5580' to 398+1209'	3.2
US-377	McCULLOCH	RM 452+10' to 452+5845'	1.1
US-190	McCULLOCH	RM 470+0' to 470+1700'	0.3
US-67	BREWSTER	RM 880+1.89 to 898+0.56	16.7
US-67/90	BREWSTER/PRESIDIO	RM 902+0.35 to 928+1.13	26.8
US-67/90	PRESIDIO	RM 950+0.83 to 952+0.63	1.8
SH 17	JEFF DAVIS	406+0.44 to 416+1.73	11.3

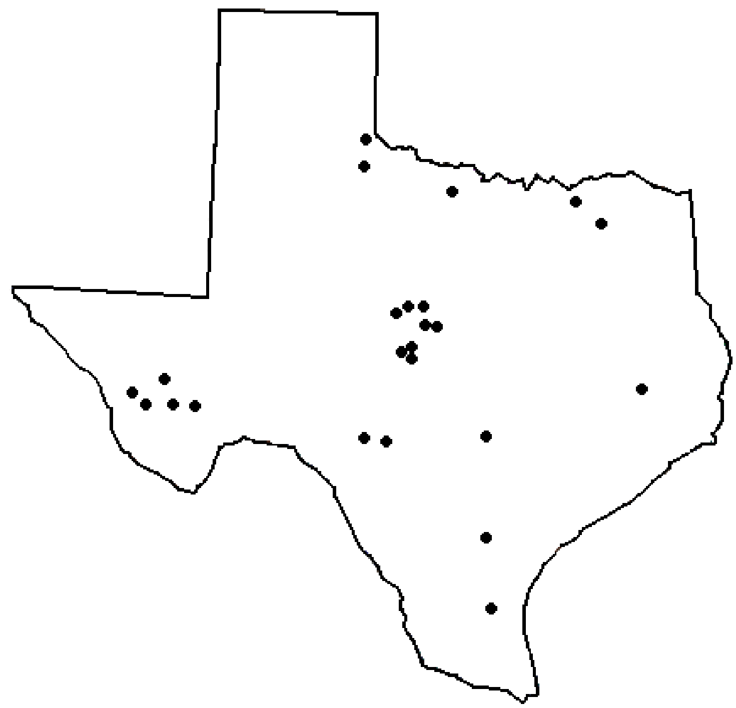


Figure 12. Approximate Location of Super 2 Highways Described in Question 2.

Question 3

Researchers wanted insight on the conditions that prompted the installation of passing lanes. Question 3 asked the respondent to indicate one or more contributors:

What conditions led to the installation of current passing lane sections in your area? (In your answer, please indicate all choices that apply.)

- *a) Large percentage of heavy vehicles*
- *b) Restricted sight distance*
- *c) High traffic volumes*
- *d) Limited passing opportunities*
- *e) Safety issue related to passing*
- *f) Other (please describe)*

As shown in Figure 13, the most common reason for installing passing lanes was the condition of limited passing opportunities. Restricted sight distance, high traffic volumes, and passing safety were each mentioned five times by the nine respondents. The two "Other" conditions were described as "limited funds" and "steep vertical grade."

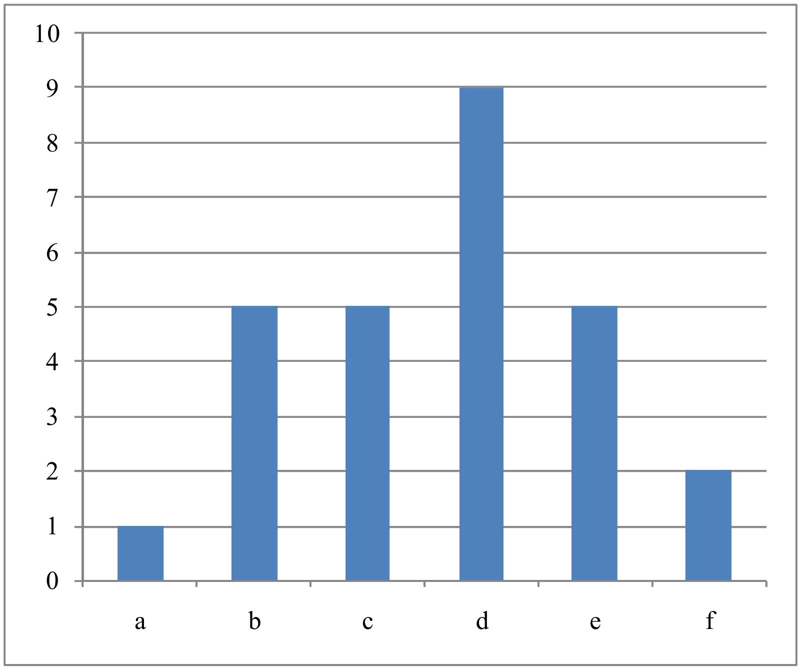


Figure 13. Responses to Question 3.

Question 4

Researchers also wanted to learn how the length and spacing of the passing lanes were determined. Question 4 asked for those responses as follows:

What criteria were used to determine the length of the extra passing lane and the spacing between passing lanes in the same direction of travel? (In your answer, please indicate all choices that apply.)

a) ADT

b) Terrain (level, rolling, etc.)

c) Proportion of heavy vehicles

d) Available sight distance

e) TxDOT Roadway Design Manual criteria

f) Grade

g) Other (please describe)

Six of the nine respondents to Question 4 indicated that they used the TxDOT *Roadway Design Manual* to determine passing lane length and spacing. Three respondents each said that the terrain and available sight distance affected length and spacing, two referred to grade, and one said that ADT was a factor. In addition, there were five respondents who said that there were other criteria that played a role in determining the final values for length and spacing; one said that the Super 2 project was constrained by bridges and grade, while the other four did not elaborate. The distribution of responses can be seen in Figure 14.

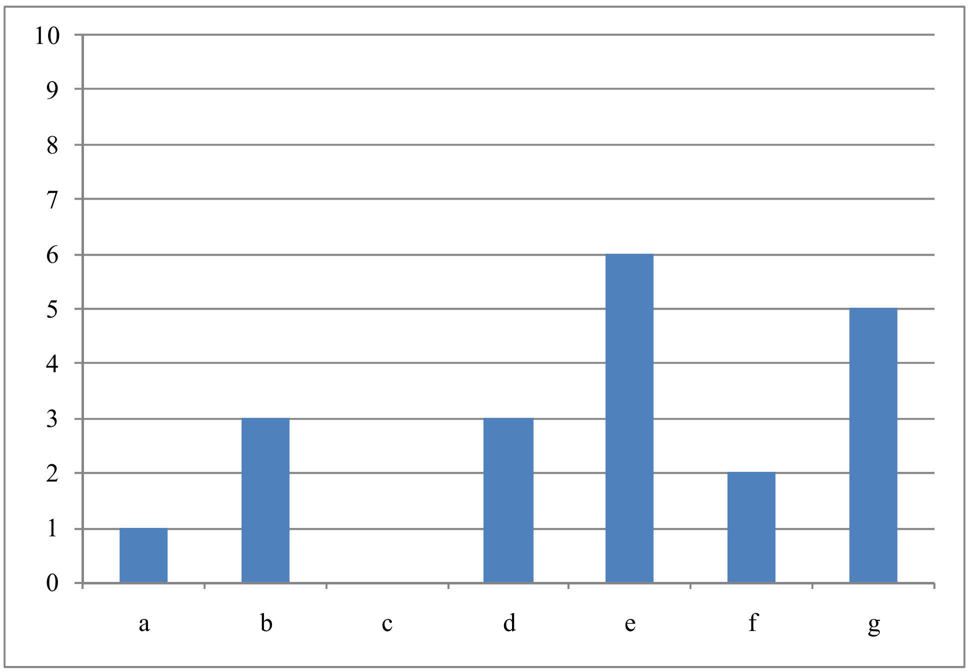


Figure 14. Responses to Question 4.

Question 5

The purpose of Question 5 was to determine whether any previous studies had investigated the benefits of passing lanes that are currently installed. Question 5 was worded as follows:

Have there been any studies or evaluations to determine the effectiveness of passing lane sections in your area? If "yes," what measures of effectiveness were used?

Of the nine responses to this question, only one provided an affirmative answer, indicating that the corridor had been studied by TTI on a previous project. Two other responses, though negative on the completion of a study, indicated that public response had been positive or that the Super 2 section had been very effective.

Question 6

The final question simply asked the respondent for contact information so that researchers could follow up for more details or clarification if needed:

May a member of the research team contact you or your representative to obtain more details about these passing lane sections? If so, please provide the appropriate name and phone number here.

All of the respondents provided at least one name and telephone number or e-mail address for future correspondence. Many Area Engineers identified themselves as the appropriate contact, while others included maintenance supervisors or other area personnel. The contact information will be kept on file if needed for further follow-up questions or other information related to conducting field studies on the corridors described.

The responses from the survey have identified a number of locations for potential use in Task 3 field studies. The research team will use the information in Task 2 to search for available crash data and further identify roadway characteristics in preparation for study site identification in Task 3.

CHAPTER 5
REVIEW AND ANALYSIS OF CRASH DATA

INTRODUCTION

The safety task focused on the review of the crash data at recently installed Super 2 highways, which provided an insight into the safety benefits of adding Super 2 sections to existing rural two-lane highways.

METHODOLOGY REVIEW

A review of the findings of previous research studies is provided in Chapter 2. That review focused on the analyses of crash data and comparison of crash and injury rates with other types of roadways. Overall most studies indicated that there is a measurable safety effect when the passing lanes were provided. However, the magnitude of safety improvements due to the installation of passing lanes differed greatly across the studies depending on the data and methodologies they used. This section provides a summary of relevant methodologies that have recently been used.

Two widely used methods for evaluating safety effectiveness of a countermeasure are before-after and cross-sectional study methods. The former is the more widely used approach when the effectiveness is developed in the form of a crash reduction factor (CRF) or a crash modification factor (*35*). The studies cited in the previous section utilized at least one of these two methods to evaluate the safety improvements due to passing lane installation. The concepts and their strengths and weakness of the two methods are well described in Shen and Gan (*35*), and they are briefly reviewed here.

The cross-sectional approach focuses on the difference in safety between treated and untreated locations without taking into account the actual changes in safety over time. For this approach, regression methods are usually used to estimate the expected frequencies from a large sample of roadway segments whose design attributes vary systematically. The expected crash frequency of a group of locations with a treatment is compared to the expected crash frequency of a group of locations with similar characteristics, but which do not have the treatment. While this approach is advantageous in that the regression models can be used in sensitivity analysis of alternative highway improvements, it cannot take into account the effects of factors that are not included in the model. One application example applied to passing lanes can be found in Potts and Harwood (*5*).

In contrast, the before-after study focuses on the changes in safety over time by investigating those locations where a given improvement has been applied within the period of analysis (*36*). While this approach is more rigorous than the cross-sectional approach, it requires a database of geometric and crash information for a large number of conversions in which many different sets of before-conditions were converted to many different sets of after-conditions. Crashes that occurred during the construction period are not included in the analysis. In this approach, the safety effect of a countermeasure is determined by the difference in the expected number of crashes occurring before the improvement and the actual number of crashes occurring after the improvement. However, there are many factors other than a treatment that can affect the safety of a location within the analysis period. Therefore, how these factors are accounted for in the analysis places the analysis into one of three types of before-after study methods: simple (or naïve) before-after study, comparison group method, and the empirical Bayes (EB) method.

The basic assumption of the simple before-after study is that if nothing has changed, the number of crashes that occurred before improvement is a good estimate of what would have occurred during the after period without improvement. In reality, however, many things can change from the before to after period, for example, traffic volume or weather conditions. Therefore, the simple before-after study cannot distinguish between the effect of the treatment and the effect of such external causal factors that may have changed from the before period to after period. This approach also suffers from other important problems such as regression-to-the-mean, crash migration, and maturation. Detailed explanations about these factors can be found in Shen and Gan (*35*). Because of these factors, the results from this simple approach are often biased and tend to overestimate the true effectiveness of a countermeasure (*37*, *38*).

To overcome some of the aforementioned problems the comparison group method uses a group of control sites selected as being similar enough to the treated sites in traffic volume and geographic characteristics. Two assumptions underlying this approach are (*37*): (a) the factors that affected safety have changed in the same way from before the improvement to after the improvement for both the treatment and the comparison groups, and (b) the changes in the various factors influence the safety of the treatment and the comparison groups in the same manner. The results from this approach are considered more accurate and reliable than the simple before-after study because it can account for the external causal factors and maturation problems. However, the results are greatly dependent on the availability of comparison sites and the similarity between the comparison and the treated sites. While this approach can improve the weakness of the simple method by carefully selecting the comparison groups, it is still subject to the regression-to-the-mean bias because it predicts the expected number of target crashes of a site based on the before-period crash number only.

The empirical Bayes (EB) method has been developed, particularly, to adjust for the regression-to-the-mean bias. The key element in EB method is to predict what would have been the expected frequency of target crashes in the after period for each treated site had the treatment not been applied (*37*). The EB method is superior to other methods in that it predicts the expected number of target crashes of a site based on two pieces of information: (a) actual number of crashes at treated sites during the before period, and (b) crashes at reference sites with similar geometric characteristics. This prediction is compared with the actual number of crashes after treatment. A detailed discussion on how the EB method addresses the regression-to-the-mean bias and its relevance to a before-after study is provided in Hauer (*37*). The results greatly depend on the accuracy of the safety performance function (SPF) for reference sites that match the characteristics of the treated sites. For developing the SPF for the reference group, the negative binomial regression model is usually adopted. While the EB method is believed to be the best among others, it is not without limitations. Shen and Gan (*35*) identified five issues that need further research in the future:

- The appropriate length of the before and the after analysis periods.
- Better guidelines for selection of the reference group.
- The size of a reference group that will sufficiently adjust for the regression-to-the-mean bias.
- The number of sites that can adequately measure a treatment effect at the treated sites.
- Proper statistics for providing sufficient information on which to base an evaluation of the quality of the study and the conclusions drawn.

Despite the usefulness of the EB method, no studies have been found that have applied this approach to evaluate the effectiveness of passing lanes. This may be partly because it is more difficult to implement by requiring more extensive roadway inventory and crash history data for both treated and untreated sites.

DATA COLLECTION ACTIVITIES

This section describes the data collection activities undertaken to assemble a database suitable for evaluating the safety effects of passing lanes by means of the before-after study with the empirical Bayes method.

Identifying the Locations of Passing Lane Segments

In order to determine the locations of Super 2 Highways constructed in Texas, a questionnaire was distributed to TxDOT Area Engineers. Detailed responses from this questionnaire are presented in Chapter 4, leading to the identification of potential study sites in five districts (Paris, Childress, Corpus Christi, Austin, and Wichita Falls). In addition to collecting data from the questionnaire, the information on the passing lane segments on US-183 in the Yoakum District was provided and segments on SH 30 in the Bryan District were identified. Depending on the availability of location information (i.e., control/section number, reference marker information, mile point limits, or city boundary limits), each passing lane segment was located using the aerial photographs available through Google Earth®.

Table 17 lists the Super 2 locations in Texas identified by the questionnaire and other sources of information. Passing lane types were determined by the configurations given in Figure 15 (*5*). All identified types fall into three categories: alternating, separated, and side-by-side passing lanes.

Table 17. Identified Passing Lanes.

District	County	Highway	Control-Section	Number of Passing Lane Segments	Passing Lane Mileage (mile)	Passing Lane Type (see Figure 15)	2007 AADT[a] (veh/day)
Paris	Fannin	SH 121	549-1	4	6.18	Alternating	6500~6900
		SH 121	549-2	6	7.39	Alternating	6300~7700
Childress	Cottle	US-62/83	32-3	5	7.35	Alternating/ Separated	1200
		US-62/83	32-2	4	5.64	Separated	1750
	Childress	US-62/83	32-1	4	5.88	Separated/ Alternating	1800
		US-62/83	31-6	4	7.84	Alternating	1800
		US-62/83	31-5	3	5.91	Alternating	1800
Corpus Christi	Live Oak	US-59	447-1	3	4.68	Separated	4700
	Bee	US-59	447-2	1	0.62	Separated	4600
Austin	Hays	RM 12	683-3	1	1.01	Side-by-side	4800
	Caldwell	US-183	153-1	1	0.94	Separated	5400
Wichita Falls	Wilbarger	US-283	124-2	10	16.45	Alternating	1850
Yoakum	Gonzales	US-183	153-2	4	7.08	Separated	5000
Bryan	Grimes	SH 30	212-4	4	6.76	Alternating/ Separated	4300
Total				**68**	**106.09**	**-**	**-**

NOTE: [a] AADTs were obtained from the TxDOT Statewide Planning Map, located at http://www.dot.state.tx.us/apps/statewide_mapping/StatewidePlanningMap.html

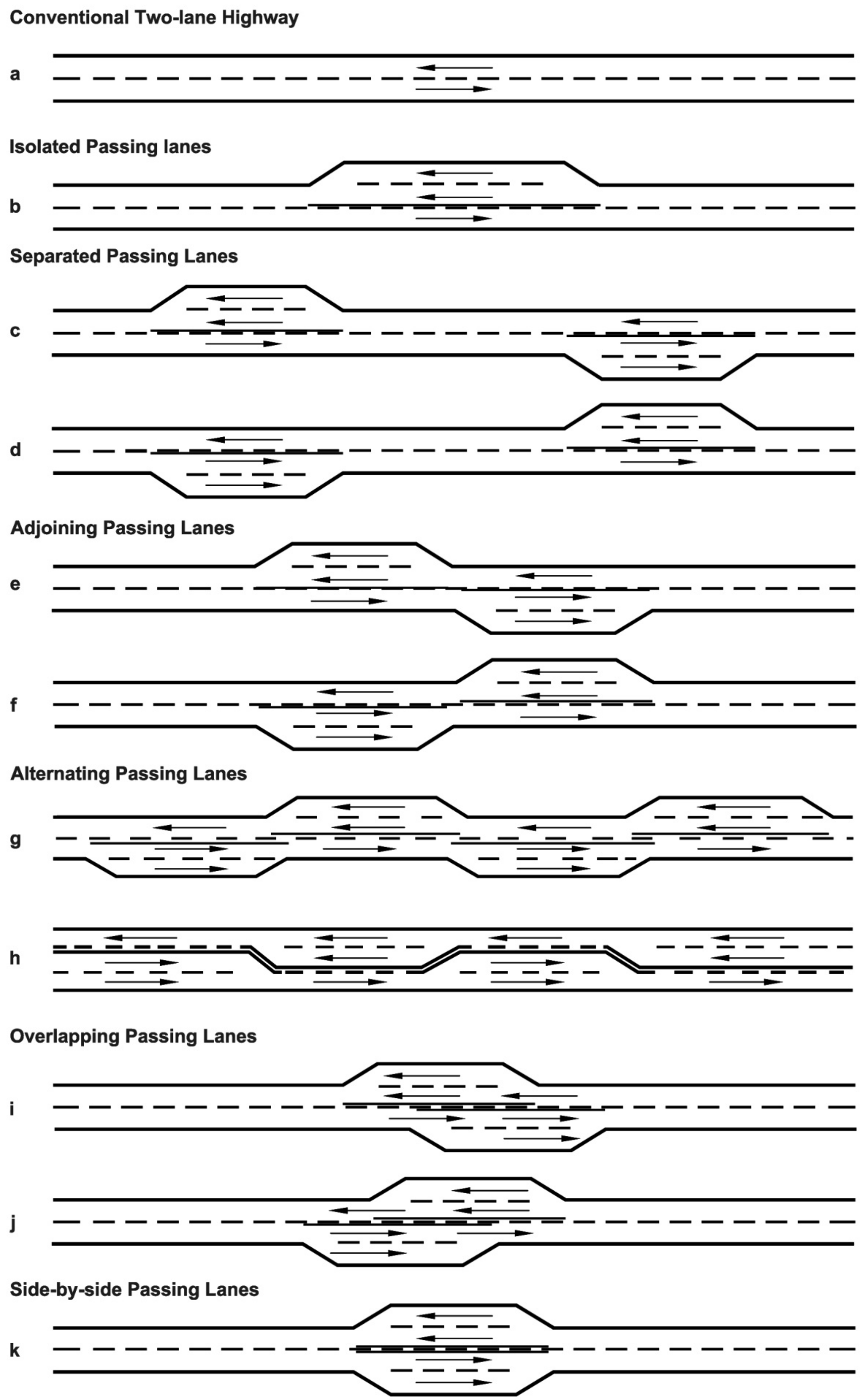


Figure 15. Alternative Configurations for Passing Lanes (*5*).

Extracting Crash Data on Passing Lane Segments

While the information provided by the responses to questionnaire was enough to identify the general location of passing lanes, more detailed and accurate location information was needed for extracting the crash data on passing lane segments. The beginning and ending milepoints information was crucial. In order to obtain the beginning and ending milepoints of a passing lane segment, the distance from the nearby intersection or county line was measured using Google Earth. Since the quality of aerial images in Google Earth is often poor in rural areas, the Street View tool provided in Google Earth or Google Maps was used to confirm the existence of a passing lane. Initially each passing lane segment was divided into three areas (see Figure 16) and crash data were collected separately for each area (i.e., beginning transition area, full-width area, and ending transition area). However, those areas were later combined for analysis because of the small number of crashes in each area. This is consistent with the findings of Harwood and St. John (*10*), who found no marked safety problem in the lane addition or lane drop transition areas after conducting field studies of traffic conflicts and erratic maneuvers.

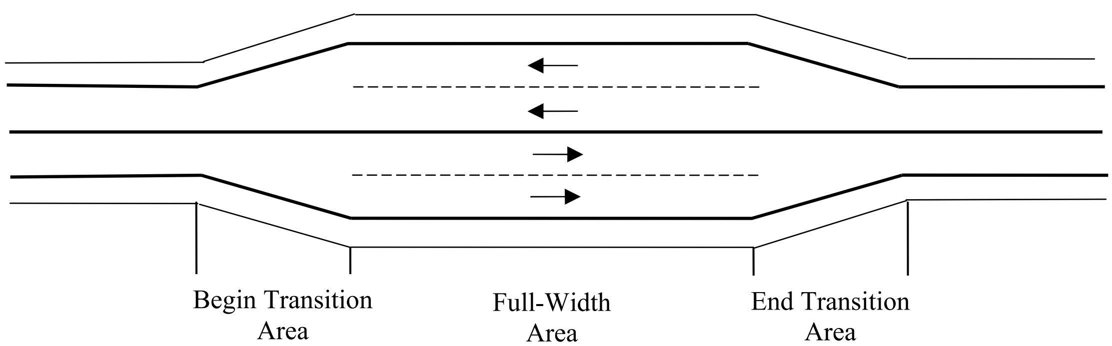


Figure 16. Individual Passing Lane Segment.

After determining the correct beginning and ending milepoints for all segment pieces, it was possible to extract crashes within each passing lane segment by matching that information with those in Texas crash database. Currently two TxDOT crash databases are available. TxDOT made major revisions to crash codes and data for 2003 and beyond include those codes. The former codes are present for 2001 and earlier data. For this study the 1997–2001 (5 years) and 2003–2009 (7 years) data were used. Thus, a total of 12 years of crashes were considered.

Two categories of crashes were developed for the evaluation: segment-only crashes (KABC) and segment-and-intersection crashes (KABC). "Segment crashes" include both driveway and non-intersection crashes, while "intersection crashes" include both intersection and intersection-related crashes. The acronyms KABC and KABCO indicate the crash injury severity types, representing fatal (K), incapacitating injury (A), non-incapacitating injury (B), minor injury (C), and property damage only (O), respectively.

ANALYSIS METHOD

The empirical Bayes method was used to evaluate the safety effectiveness of providing passing lanes. The procedures for using the before-after study with EB method are described in the following.

Step 1. *Define the Reference Group*

Since the final outcome about the safety-effectiveness of passing lanes can be different depending on the chosen reference group, four potential reference groups were considered in this study. The basic restrictions below were imposed on all four reference groups considered. Those restrictions were selected after a careful examination of the geometric characteristics for the treated sites before conversion.

- Basic restrictions:
  - Record_type = 1: (represents mainlanes).
  - District_ID = (24, 1, 25, 16, 14, 13, 17, 3): (represent those districts identified in Table 17).
  - Rural_urban_code = 1: (represents rural area).
  - Number of lanes = 2 (two-lane highway).
  - Length_of_section > 0.1 mile.
  - Highway_system = ('US,' 'SH').
  - Highway_status = 6: (represents the highways open to traffic with all data input).
  - Median_type = 0: (represents no median).
  - Control_section_number: do not include those where the passing lanes were identified in Table 17.

The restriction on the segment length (0.1 mile) was introduced based on prior research by Hauer (*39*) that noted the negative binomial model is unduly influenced by very short segments. Along with the basic restrictions, other restrictions were considered with respect to location (county), AADT, and shoulder width in order to let the potential reference groups be as similar to the before condition of treated sites as possible. Those restrictions include:

- Location restrictions:
  - County_number: include those counties where the passing lanes were identified in Table 17.
- AADT restriction:
  - 500 < ADT_current < 10,000.
- Shoulder width restriction:
  - (Shoulder_width_left = Shoulder_width_right) and (Shoulder_width ≤ 10)

Table 18 shows the summary of potential reference groups defined by using different restrictions. The number of segments and total mileage is based on 12 years of data (1997 to 2001 and 2003 to 2009) extracted from TxDOT roadway inventory databases (i.e., RHiNo). As shown in the table, Reference Group 1 is the most focused reference group; this is the reference group selected and will be discussed during the remainder of the analyses.

Table 18. Summary of Potential Reference Groups.

Reference Group Number	Meets Location Restriction?	Meets AADT Restriction?	Meets Shoulder Width Restriction?	Number of Segments	Total Mileage
1	Yes	Yes	Yes	8,139	9,734
2	Yes	No	No	8,274	11,150
3	No	Yes	Yes	40,850	46,036
4	No	No	No	48,237	57,458

Step 2. *Develop Safety Performance Functions (SPFs) for Each Reference Group*

Negative binomial regression (NB) models were used to develop a safety performance function for the reference group. An important characteristic associated with the development of NB models is the choice of the functional form linking crashes to the covariates. The functional form used in this study is as follows:

$$E(\kappa_{i,y}) = L_i \cdot F_{i,y}^{\alpha} \cdot \exp(\mathbf{x}_{i,y}^{T} \cdot \boldsymbol{\beta} + \gamma_y \cdot D_y) \quad (1)$$

where

$\kappa_{i,y}$ = Expected number of crashes at site *i* in year *y* (crashes/year).

$E(\kappa_{i,y})$ = Mean of the κ 's in year *y* in the reference group for site *i*.

L_i = Segment length of site *i* (mile).

$F_{i,y}$ = Traffic flow (AADT) at site *i* in year *y* (veh/day).

$\mathbf{x}_{i,y}$ = A set of explanatory variables at site *i* in year *y*.

D_y = Yearly or database dummy variable.

α, $\boldsymbol{\beta}$, and γ_y: The coefficients to be estimated.

$i = 1, \cdots, N$.

$y = 1, \cdots, Y, Y+1, \cdots Y+Z$.

Y represents the last year before the treatment, and Z is the number of years after the treatment for which we wish to predict. A yearly or database dummy variable was introduced in the model. It accounts for the yearly changes in the expected number of crashes over the study period due to the factors not represented by the explanatory variables. The crash occurrence trends by year shown in Figure 17 suggest the justification of using the yearly dummy variables or database dummy variables. Particularly for the KABC categories (segment-only crashes and segment-and-intersection crashes), the number of crashes tends to decrease within each database and the 2003–2009 database shows a lower number of crashes compared to the 1997–2001 database.

A comparison of the two crash databases shows that the segment-only (KABC) and segment-and-intersection (KABC) crashes in the 2003–2009 database are often lower than those in the 1997–2001 database. For the segment-and-intersection crashes (KABCO), however, the crash totals are typically higher in the 2003–2009 database. This increase is attributable to the change in definition of a reportable PDO collision. In the earlier period (1997–2001), a reportable PDO crash only included crashes when a vehicle was towed away from the site. In the after period (2003–2009), a reportable PDO crash included damages to a vehicle that were estimated to be at least $1,000.

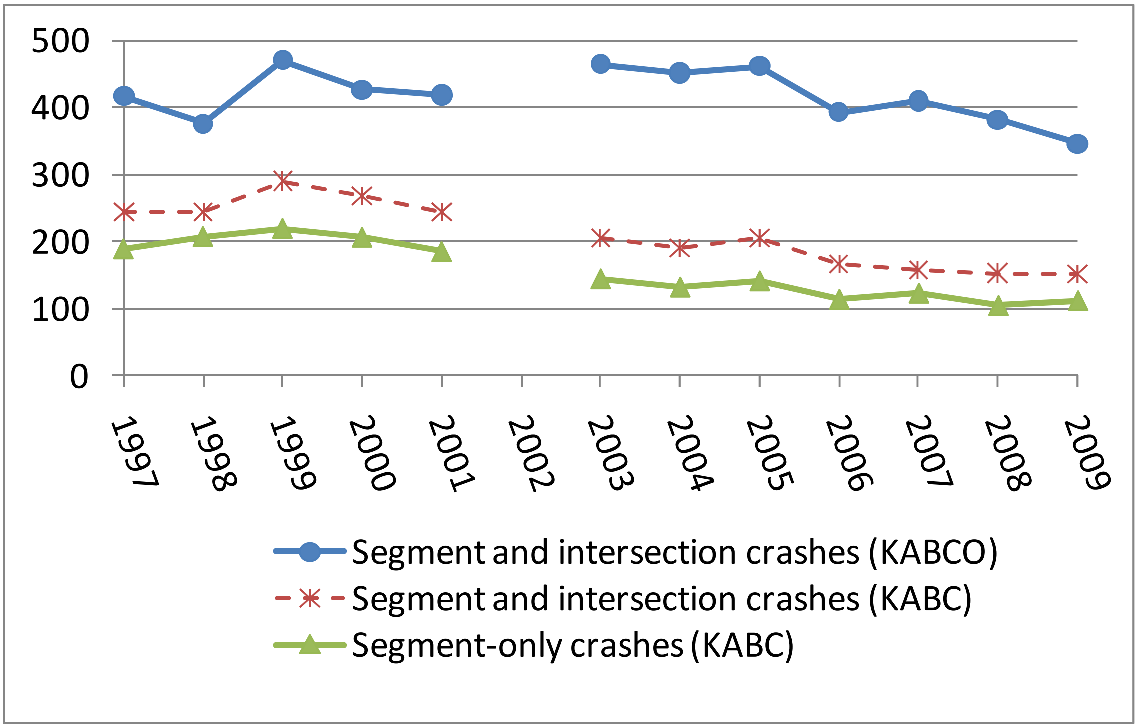


Figure 17. Crash Occurrence Trends by Year (Reference Group 1).

The NB model is estimated from data spanning the entire before and after periods on sites representing the reference group. Twelve years of crash data and roadway inventory data (1997 to 2001 and 2003 to 2009) were used for modeling. The final output in this step is $\hat{E}(\kappa_{i,1}), \cdots, \hat{E}(\kappa_{i,Y}), \hat{E}(\kappa_{i,Y+1}), \cdots, \hat{E}(\kappa_{i,Y+Z})$ over the entire study period.

Step 3. *Compute the Yearly Correction Factors,* $\hat{C}_{i,y}$

$$\hat{C}_{i,y} = \frac{\hat{E}(\kappa_{i,y})}{\hat{E}(\kappa_{i,1})} \tag{2}$$

Since we have chosen year 1997 as the first year, $\hat{E}(\kappa_{i,1})$ is the mean of the κ s in year 1997 in reference group for site *i*. This makes $\hat{C}_{i,1} = 1$ for year 1997. Therefore, the final output in this step is $\hat{C}_{i,1}, \cdots, \hat{C}_{i,Y}, \hat{C}_{i,Y+1}, \cdots, \hat{C}_{i,Y+Z}$ over the entire study period.

Step 4. *Compute the EB Estimates and Their Variances for Before Period*

$$\hat{\kappa}_{i,1} = \frac{\hat{\phi} + \sum_{y=1}^{Y} K_{i,y}}{\frac{\hat{\phi}}{\hat{E}(\kappa_{i,1})} + \sum_{y=1}^{Y} \hat{C}_{i,y}} \text{ and } Var(\hat{\kappa}_{i,1}) = \frac{\hat{\kappa}_{i,1}}{\frac{\hat{\phi}}{\hat{E}(\kappa_{i,1})} + \sum_{y=1}^{Y} \hat{C}_{i,y}} \quad \text{for } y = 1 \text{ (year 1997)}$$

$$\hat{\kappa}_{i,y} = \hat{C}_{i,y}\hat{\kappa}_{i,1} \text{ and } Var(\hat{\kappa}_{i,y}) = \hat{C}_{i,y}^2 Var(\hat{\kappa}_{i,1}) \text{ for } y = 2, \cdots, Y \tag{3}$$

where

$K_{i,y}$ = The actual number of crashes at site *i* in year *y* ($y = 1, \cdots, Y$) .

$\hat{\phi}$ = The estimate of over-dispersion parameter from the NB model in Step 2.

As shown in the equations above, the EB estimates ($\hat{\kappa}_{i,y}$) are the results of the joint use of two kinds of information ($K_{i,y}$): those contained in actual crash numbers and those contained in the roadway characteristics of the site and the corresponding reference group ($\hat{E}(\kappa_{i,y})$). The final outputs in this step are $\hat{\kappa}_{i,1}, \cdots, \hat{\kappa}_{i,Y}$ and $Var(\hat{\kappa}_{i,1}), \cdots, Var(\hat{\kappa}_{i,Y})$.

Step 5. *Predict the Expected Number of Crashes and Variances for After Period*

$$\hat{\kappa}_{i,y} = \hat{C}_{i,y}\hat{\kappa}_{i,1} \text{ and } Var(\hat{\kappa}_{i,y}) = \hat{C}_{i,y}^2 Var(\hat{\kappa}_{i,1}) \text{ for } y = Y+1, \cdots, Y+Z \tag{4}$$

The $\hat{C}_{i,Y+1}, \cdots, \hat{C}_{i,Y+Z}$ are available from Step 3, and $\hat{\kappa}_{i,1}$ and $Var(\hat{\kappa}_{i,1})$ were computed in Step 4. The final outcomes in this step are $\hat{\kappa}_i = \sum_{y=Y+1}^{Y+Z} \hat{\kappa}_{i,y}$ and $Var(\hat{\kappa}_i) = \sum_{y=Y+1}^{Y+Z} Var(\hat{\kappa}_{i,y})$. They represent the expected number of after-period crashes and their variances for site *i* had the treatment not been implemented at the treated site.

Step 6. *Compute the Sum of the Predicted Crashes over All Treated Sites and Its Variance*

$$\hat{\kappa} = \sum_{i=1}^{N} \hat{\kappa}_i \text{ and } Var(\hat{\kappa}) = \sum_{i=1}^{N} Var(\hat{\kappa}_i) \tag{5}$$

where N is the total number of sites in the treatment group, and $\hat{\kappa}$ is the expected after-period crashes at all treated sites had there been no treatment.

Step 7. *Compute the Sum of the Actual Crashes over All Treated Sites*

$$K = \sum_{i=1}^{N} K_i \tag{5}$$

where K_i is the total crash counts during the after period at site *i*.

Step 8. *Compute the Unbiased Estimate of Safety-Effectiveness of the Treatment and Its Variance*

$$\hat{\theta} = \frac{K}{\hat{\kappa}\left(1 + \frac{Var(\hat{\kappa})}{\hat{\kappa}^2}\right)} \tag{6}$$

The percent change in the number of target crashes due to the treatment is calculated by $100(1-\hat{\theta})$ %. If $\hat{\theta}$ is less than 1, then the treatment has a positive safety effect.

The estimated variance and standard error of the estimated safety-effectiveness are given by:

$$Var(\hat{\theta}) = \hat{\theta}^2 \frac{\left(1/K + Var(\hat{\kappa})/\hat{\kappa}^2\right)}{\left(1 + Var(\hat{\kappa})/\hat{\kappa}^2\right)^2} \tag{7}$$

$$s.e.(\hat{\theta}) = \sqrt{Var(\hat{\theta})} \tag{8}$$

The approximate 95 percent confidence interval for θ is given by adding and subtracting $1.96 s.e.(\hat{\theta})$ from $\hat{\theta}$. If the confidence interval contains the value 1, then no significant effect has been observed.

ANALYSIS RESULTS

SPF Models

The negative binomial regression models were developed for the safety performance functions of four potential reference groups based on the functional form given in Equation 1. For explanatory variables, in addition to the flow variable, shoulder width was considered for inclusion in the model. For yearly factor variables, three alternatives were examined, that is, 11 yearly factor dummy variables using 1997 as a base year, two database dummy variables using the 1997–2001 crash database as a base group, and no yearly dummy variables. The SAS software (*40*) was used to estimate the parameters. For model comparison, the Pearson chi-square (X^2), AIC (Akaike information criterion), and BIC (Bayesian information criterion) were used to assist in selecting the best model. Table 19 and Table 20 show the parameter estimation results along with various goodness-of-fit measures for Reference Group 1 for segment only and segment and intersection crashes, respectively.

Table 19. SPFs for Segment-Only Crashes (KABC).

	Reference Group 1		
Parameters	**Estimate**	**S.E.**	**Pr>Chi Sq.**
Intercept	−8.3880	0.2733	<0.0001
Log(AADT)	0.9472	0.0350	<0.0001
Shoulder_width	−0.0460	0.0076	<0.0001
YR1997–YR2001	0	0	-
YR2003–YR2009[a]	−0.3866	0.0587	<0.0001
Dispersion ($1/\phi$)	0.4051	0.0603	<0.0001
Goodness-of-fit Measures	**Value**		**Value/DF**
Deviance	4166.2		0.5121
Pearson X^2	8694.0		1.0687
Log Likelihood	−3360.9		
AIC	7739.9		
BIC	7774.9		

NOTE: [a] indicates the database dummy variable: YR2003-YR2009 = 1 if years are from 2003 to 2009; otherwise, YR2003-YR2009 = 0.

Table 20. SPFs for Segment-and-Intersection Crashes (KABC).

	Reference Group 1		
Parameters	**Estimate**	**S.E.**	**Pr>Chi Sq.**
Intercept	−9.5949	0.2687	<0.0001
Log(AADT)	1.1374	0.0345	<0.0001
Shoulder_width	−0.0362	0.0073	<0.0001
YR1997–YR2001	0	0	-
YR2003–YR2009 [a]	−0.3514	0.0563	<0.0001
Dispersion ($1/\phi$)	0.7241	0.0663	<0.0001
Goodness-of-fit Measures	**Value**		**Value/DF**
Deviance	4845.6		0.5956
Pearson X^2	13063.0		1.6058
Log Likelihood	−4083.7		
AIC	9852.8		
BIC	9887.8		

NOTE: [a] indicates the database dummy variable: YR2003-YR2009 = 1 if years are from 2003 to 2009, otherwise YR2003-YR2009 = 0.

The traffic flow and shoulder width were found to be statistically significant for all SPFs considered in this study. The negative signs for shoulder width indicate that crashes decrease as the shoulder width increases, which is a desirable finding. Regarding the estimates of yearly factor variables, using two database dummy variables (indicated as YR1997–YR2001 and YR2003–YR2009) resulted in the best models for both segment-only crashes (KABC) and segment-and-intersection crashes (KABC). The negative sign for the YR2003–YR2009 variable in the KABC models indicates that, assuming all else remains unchanged, the expected number of crashes for years 2003 through 2009 is less than that for years 1997 through 2001.

Results of EB Analysis

In order to carry out the before-after study with the EB method, it is necessary to know the construction period for passing lane installation. The research team contacted the questionnaire respondents to confirm the beginning and end of the construction period in which the passing lanes were installed. Table 21 shows the construction periods that were available at the time of analysis. Since the crash data up to 2009 were available, only Super 2 sections on SH 121 (Paris), SH 30 (Bryan), US-183 (Austin, Yoakum), and US-283 (Wichita Falls) could be considered for the analysis.

Table 21. Construction Period of Passing Lanes.

District	County	Highway	Control-Section	Begin Construction	End Construction
Paris	Fannin	SH 121	549-1	4/17/2002	2/11/2004
		SH 121	549-2	4/1/2002	11/27/2003
Bryan	Grimes	SH 30	212-4	1/10/2005	8/4/2006
Austin[a]	Caldwell	US-183	153-1	3/7/2007	5/30/2008
Yoakum	Gonzales	US-183	153-2		
Wichita Falls	Wilbarger	US-283	124-2	11/18/2005	5/27/2008
Corpus Christi[b]	Live Oak	US-59	447-1	1/25/2007	1/26/2009
	Bee	US-59	447-2		

NOTE:
[a] This site was not included in the EB analysis because of short length.
[b] This site was not included in the EB analysis because sufficient post-construction crash data were not available.

A Super 2 highway project is usually implemented by constructing more than one individual passing lane segment within a corridor. Therefore, the safety effectiveness of a Super 2 highway project can be better appreciated by a corridor-based analysis rather than a segment-based analysis because the safety effect of the passing lane extends beyond the physical boundaries of the passing lanes section (*14*).

In determining the analysis corridor, the corridor length was adjusted to include all passing lanes located within the corridor using the TxDOT roadway inventory database. The total length of analysis corridors was about 53 centerline-miles. The analysis corridor generally consists of several roadway segments as defined in the TxDOT database, which do not necessarily overlap with the passing lanes, and the traffic flow and other variables may change across those segments. Thus, the representative values for the flow and shoulder width for a particular year were obtained by taking the weighted average with respect to the individual segment length.

Results for the EB analysis of the five study corridors for KABC segment-only crashes, excluding non-injury crashes and intersection crashes are shown in Table 22 (Paris and Bryan) and Table 23 (Yoakum and Wichita Falls). The results for KABC segment and intersection crashes are shown in Table 24 for Paris and Bryan and Table 25 for Yoakum and Wichita Falls. Key variables in the analysis are presented by study period (i.e., before and after construction) and subdivided by calendar year. A summary of results for the before and after periods is provided in Table 26. With the exception of Control Section 549-1 in the Paris District, the number of actual segment crashes in the after period for each corridor was lower than the number of expected crashes estimated by the EB analysis.

In the estimation of overall changes in crashes shown in Table 26, the EB estimates ($\hat{\kappa}_i$) were summed over all corridors and compared with the sum of the actual crashes during the after period for each corridor. The results indicate that the number of actual crashes was 35 percent lower than could have been expected if no passing lanes were installed on the study corridors. For segment and intersection crashes the reduction was 42 percent. These findings are statistically significant above the 95 percent confidence level, which indicates that the reduction in crashes can be attributed to the Super 2 treatments with a high degree of certainty.

Table 22. Result of Empirical Bayes Analysis for Paris and Bryan – Segment Crashes (KABC).

Year	1997	1998	1999	2000	2001	2003	2004	2005	2006	2007	2008	2009
Site	**Paris (Corridor 1), SH 121 (549-01)**											
Dates	Before: 1/1/1997 to 4/17/2002 (12/31/2001)						After: 2/11/2004 to 12/31/2009					
Days	365	365	365	365	365		322	365	365	365	365	365
Crashes	4	2	5	2	1		2	3	2	0	3	4
AADT	5076	5394	5294	6559	6506		6354	7031	6041	6628	6500	6500
Length (mi)	6.81	6.81	6.81	6.81	6.81		6.81	6.81	6.76	6.73	6.73	6.73
Shoulder (ft)	9.22	9.22	9.22	9.22	9.22		9.22	9.22	9.22	9.22	9.22	9.22
E(k_y)	3.28	3.47	3.41	4.18	4.15		2.43	3.03	2.61	2.83	2.78	2.78
$\hat{\kappa}_i$	2.58	2.73	2.68	3.28	3.26		1.91	2.38	2.05	2.23	2.19	2.19
Var(EB)	0.40	0.45	0.44	0.65	0.64		0.22	0.34	0.26	0.30	0.29	0.29
Site	**Paris (Corridor 2), SH 121 (549-02)**											
Dates	Before: 1/1/1997 to 3/31/2002 (12/31/2001)					After: 11/27/2003 to 12/31/2009						
Days	365	365	365	365	365	33	365	365	365	365	365	365
Crashes	3	7	10	2	3	0	7	2	0	3	3	1
AADT	4948	5169	5558	6412	7008	6012	6851	6775	6521	6952	6800	6800
Length (mi)	9.34	9.34	9.34	9.34	9.34	9.34	9.34	9.34	9.34	9.34	9.34	9.34
Shoulder (ft)	9.56	9.56	9.56	9.56	9.56	9.56	9.56	9.56	9.56	9.56	9.56	9.56
E(k_y)	4.32	4.51	4.83	5.53	6.01	0.32	4.00	3.95	3.81	4.05	3.97	3.97
$\hat{\kappa}_i$	4.29	4.47	4.79	5.49	5.97	0.32	3.97	3.93	3.79	4.02	3.94	3.94
Var(EB)	0.67	0.73	0.84	1.10	1.30	0.00	0.57	0.56	0.52	0.59	0.57	0.57
Site	**Bryan, SH 30 (212-4)**											
Dates	Before: 1/1/1997 to 1/9/2005								After: 8/5/2006-12/31/2009			
Days	365	365	365	365	365	365	365	9	147	365	365	365
Crashes	7	2	5	8	0	5	5	0	0	1	2	1
AADT	2700	3300	3100	3300	3600	3700	3900	3960	4300	4300	4100	4100
Length (mi)	7.52	7.52	7.52	7.52	7.52	7.52	7.52	7.52	7.52	7.52	7.52	7.52
Shoulder (ft)	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
E(k_y)	2.99	3.62	3.41	3.62	3.93	2.74	2.88	0.07	1.27	3.16	3.02	3.02
$\hat{\kappa}_i$	4.01	4.85	4.57	4.85	5.26	3.67	3.86	0.10	1.70	4.23	4.04	4.04
Var(EB)	0.47	0.68	0.61	0.68	0.80	0.39	0.43	0.00	0.08	0.52	0.47	0.47

Table 23. Result of Empirical Bayes Analysis for Yoakum and Wichita Falls – Segment Crashes (KABC).

Year	1997	1998	1999	2000	2001	2003	2004	2005	2006	2007	2008	2009
Site	**Yoakum, US-183 (153-02)**											
Dates	Before: 1/1/1997 to 3/6/2007										After: 5/30/2008-12/31/2009	
Days	365	365	365	365	365	365	365	365	365	65	214	365
Crashes	6	10	9	12	11	3	1	3	3	0	1	4
AADT	4684	4631	4777	5230	5128	5160	5211	5323	5183	5137	5216	5216
Length (mi)	11.6	11.6	11.6	11.6	11.6	11.6	11.6	11.6	11.6	11.6	11.6	11.6
Shoulder (ft)	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00
E(k_y)	5.49	5.43	5.59	6.09	5.98	4.08	4.12	4.21	4.10	0.72	2.42	4.13
$\hat{\kappa}_i$	6.87	6.80	7.00	7.63	7.49	5.12	5.16	5.27	5.14	0.91	3.03	5.17
Var(EB)	0.78	0.76	0.81	0.96	0.93	0.43	0.44	0.46	0.44	0.01	0.15	0.44
Site	**Wichita Falls, US-283 (124-02)**											
Dates	Before: 1/1/1997 to 11/17/2005								Construction		After: 5/27/2008-12/31/2009	
Days	365	365	365	365	365	365	365	321			217	365
Crashes	2	2	2	4	2	0	5	2			0	1
AADT	1655	1868	1666	1968	1944	2089	2069	2103			1900	1900
Length (mi)	17.2	17.2	17.2	17.2	17.2	17.2	17.2	17.2			17.2	17.2
Shoulder (ft)	0.00	0.00	0.00	0.00	9.00	9.00	9.00	9.00			7.00	7.00
E(k_y)	4.39	4.92	4.42	5.17	3.38	2.46	2.44	2.18			1.46	2.46
$\hat{\kappa}_i$	2.96	3.32	2.98	3.49	2.28	1.66	1.64	1.47			0.99	1.66
Var(EB)	0.41	0.51	0.41	0.57	0.24	0.13	0.13	0.10			0.05	0.13

Table 24. Result of Empirical Bayes Analysis for Paris and Bryan – Segment and Intersection Crashes (KABC).

Year	1997	1998	1999	2000	2001	2003	2004	2005	2006	2007	2008	2009
Site	**Paris (Corridor 1), SH 121 (549-01)**											
Dates	Before: 1/1/1997 to 4/17/2002 (12/31/2001)						After: 2/11/2004 to 12/31/2009					
Days	365	365	365	365	365		322	365	365	365	365	365
Crashes	7	4	10	7	6		5	10	6	10	9	8
AADT	5076	5394	5294	6559	6506		6354	7031	6041	6628	6500	6500
Length (mi)	6.81	6.81	6.81	6.81	6.81		6.81	6.81	6.76	6.73	6.73	6.73
Shoulder (ft)	9.22	9.22	9.22	9.22	9.22		9.22	9.22	9.22	9.22	9.22	9.22
E(k_y)	5.44	5.83	5.71	7.28	7.21		4.36	5.55	4.64	5.12	5.01	5.01
$\hat{\kappa}_i$	3.54	3.79	3.71	4.74	4.70		2.84	3.61	3.02	3.34	3.26	3.26
Var(EB)	0.59	0.67	0.65	1.05	1.03		0.38	0.61	0.43	0.52	0.50	0.50
Site	**Paris (Corridor 2), SH 121 (549-02)**											
Dates	Before: 1/1/1997 to 3/31/2002 (12/31/2001)					After: 11/27/2003 to 12/31/2009						
Days	365	365	365	365	365	33	365	365	365	365	365	365
Crashes	5	9	12	8	11	0	14	5	3	5	6	4
AADT	4948	5169	5558	6412	7008	6012	6851	6775	6521	6952	6800	6800
Length (mi)	9.34	9.34	9.34	9.34	9.34	9.34	9.34	9.34	9.34	9.34	9.34	9.34
Shoulder (ft)	9.56	9.56	9.56	9.56	9.56	9.56	9.56	9.56	9.56	9.56	9.56	9.56
E(k_y)	7.16	7.53	8.17	9.62	10.64	0.57	7.30	7.20	6.90	7.42	7.23	7.23
$\hat{\kappa}_i$	5.05	5.31	5.76	6.78	7.50	0.40	5.15	5.08	4.86	5.23	5.10	5.10
Var(EB)	0.81	0.90	1.06	1.47	1.79	0.01	0.84	0.82	0.75	0.87	0.83	0.83
Site	**Bryan, SH 30 (212-4)**											
Dates	Before: 1/1/1997 to 1/9/2005								After: 8/5/2006-12/31/2009			
Days	365	365	365	365	365	365	365	9	147	365	365	365
Crashes	10	7	11	17	8	7	11	0	3	4	7	7
AADT	2700	3300	3100	3300	3600	3700	3900	3960	4300	4300	4100	4100
Length (mi)	7.52	7.52	7.52	7.52	7.52	7.52	7.52	7.52	7.52	7.52	7.52	7.52
Shoulder (ft)	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
E(k_y)	4.04	5.07	4.72	5.07	5.60	4.06	4.31	0.11	1.94	4.82	4.57	4.57
$\hat{\kappa}_i$	4.16	5.22	4.86	5.22	5.76	4.18	4.44	0.11	2.00	4.96	4.70	4.70
Var(EB)	0.49	0.77	0.67	0.77	0.94	0.49	0.56	0.00	0.11	0.70	0.62	0.62

Table 25. Result of Empirical Bayes Analysis for Yoakum and Wichita Falls – Segment and Intersection Crashes (KABC).

Year	1997	1998	1999	2000	2001	2003	2004	2005	2006	2007	2008	2009
Site	**Yoakum, US-183 (153-02)**											
Dates	Before: 1/1/1997 to 3/6/2007										After: 5/30/2008-12/31/2009	
Days	365	365	365	365	365	365	365	365	365	65	214	365
Crashes	15	20	23	14	20	7	3	4	8	2	1	11
AADT	4684	4631	4777	5230	5128	5160	5211	5323	5183	5137	5216	5216
Length (mi)	11.6	11.6	11.6	11.6	11.6	11.6	11.6	11.6	11.6	11.6	11.6	11.6
Shoulder (ft)	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00	8.00
E(k_y)	8.86	8.74	9.06	10.0	9.82	6.96	7.04	7.21	6.99	1.23	4.13	7.04
$\hat{\kappa}_i$	7.60	7.51	7.78	8.62	8.43	5.97	6.04	6.19	6.00	1.06	3.55	6.05
Var(EB)	0.87	0.85	0.91	1.12	1.07	0.54	0.55	0.58	0.54	0.02	0.19	0.55
Site	**Wichita Falls, US-283 (124-02)**											
Dates	Before: 1/1/1997 to 11/17/2005								Construction		After: 5/27/2008-12/31/2009	
Days	365	365	365	365	365	365	365	321			217	365
Crashes	5	5	4	6	5	7	12	11			3	8
AADT	1655	1868	1666	1968	1944	2089	2069	2103			1900	1900
Length (mi)	17.2	17.2	17.2	17.2	17.2	17.2	17.2	17.2			17.2	17.2
Shoulder (ft)	0.00	0.00	0.00	0.00	9.00	9.00	9.00	9.00			7.00	7.00
E(k_y)	5.38	6.17	5.42	6.55	4.66	3.56	3.52	3.15			2.04	3.44
$\hat{\kappa}_i$	3.02	3.47	3.05	3.68	2.62	2.00	1.98	1.77			1.15	1.93
Var(EB)	0.41	0.54	0.41	0.61	0.31	0.18	0.18	0.14			0.06	0.17

Table 26. Empirical Bayes Results for all Study Sites.

	Segment Crashes		Segment and Intersection Crashes	
Period	**Before**	**After**	**Before**	**After**
Site	**Paris (Corridor 1), SH 121 (549-01)**			
Number of days	1825	2147	1825	2147
Actual number of crashes during period	14	14	20	18
Expected number of crashes without treatment	14.5	12.9	31.47	29.69
Site	**Paris (Corridor 2), SH 121 (549-02)**			
Number of days	1825	2223	1825	2223
Actual number of crashes during period	25	16	30	18
Expected number of crashes without treatment	25.0	23.9	43.11	43.85
Site	**Bryan, SH 30 (212-4)**			
Number of days	2564	1242	2564	1242
Actual number of crashes during period	32	4	34	4
Expected number of crashes without treatment	31.2	14.0	32.980	15.896
Site	**Yoakum, US-183 (153-02)**			
Number of days	3350	579	3350	579
Actual number of crashes during period	58	5	65	5
Expected number of crashes without treatment	57.4	8.2	75.94	11.17
Site	**Wichita Falls, US-283 (124-02)**			
Number of days	2876	582	2876	582
Actual number of crashes during period	19	1	21	1
Expected number of crashes without treatment	19.8	2.6	38.41	5.48
Empirical Bayes Results				
Number of after crashes		40		46
Expected number of crashes during after period had passing lanes not be installed ($\hat{\kappa}_i$)		61.73		79.29
Variance		7.41		10.91
Estimated index of effectiveness		0.65		0.58
Standard error		0.11		0.09
95% Confidence interval (lower limit)		0.439		0.406
95% Confidence interval (upper limit)		0.854		0.753
Statistical significance		99.9%		100%
Percent reduction in the number of crashes		**35%**		**42%**

SUMMARY OF CRASH DATA ANALYSIS

The objective of this task was to evaluate the safety effectiveness of installing Super 2 highways. The literature review on this topic suggested that there is a measurable safety effect when Super 2 sections are provided, although the magnitude of safety improvement due to the passing lane installation greatly differed depending on the data and methodologies adopted. The empirical Bayes method was developed to address regression-to-the-mean bias in observational before-after studies, and it was selected for use in this TxDOT study.

Based on the responses from the questionnaire and other sources, potential study sites in eight districts (El Paso, Paris, Childress, Corpus Christi, Austin, Wichita Falls, Yoakum, and Bryan) were identified. Within these districts, four reference groups were considered by imposing various restrictions, and negative binomial regression models were used to develop the safety performance functions for each reference group. As a result of this process, the most restricted group (Reference Group 1) was selected for the final analysis. Researchers reviewed and analyzed 12 years (1997–2001 and 2003–2009) of roadway inventory and crash history data. The crash data were divided into two categories for a review of SPF models: segment-only crashes (KABC) and segment-and-intersection crashes (KABC). Researchers conducted an EB analysis on the crash data for five corridors on SH 121 (Paris District), SH 30 (Bryan District), US-183 (Yoakum District), and US-283 (Wichita Falls District). The total length of the five corridors is about 53 centerline-miles.

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

CHAPTER 6
COMPARISON OF COMPUTER SIMULATION MODELS

BACKGROUND

The analytical approach for this research included using simulation software tools for creating a broad range of cases wherein analysts varied the length, frequency, and spacing of passing lanes along two-lane roadways in background environments where the terrain, traffic volume, and traffic composition were also varied. The research team was then tasked with interpreting the output from the software tools in order to make informed judgments on recommended passing lane design for high-volume, two-lane roadway conditions—including passing lane length, spacing and frequency—to best accommodate a given traffic stream and roadway environment.

SIMULATION TOOLS

Modern traffic analysis software comes in a wide variety of forms. Macroscopic tools—those that look at the big picture and mathematically represent traffic flow—are primarily used in transportation planning. Mesoscopic tools, which are often applied during analysis of traffic routing through portions of an urban network, are more detailed than macroscopic models but retain a focus at the sub-regional or corridor level. Microscopic tools are the types of tools that were employed in this research as they model down to the vehicle and driver level and account for interactions between vehicles/drivers in the network as well as the influences of traffic controls and roadway features on system vehicles.

Modern microscopic traffic simulation tools tend to have similar features and environments, regardless of whether they originated in the public domain or as a product of a private company. These tools operate in Microsoft® Windows®-based personal computer environments and have graphical user interfaces. They also feature productivity enhancements that enable analysts to "draw" a network over a digital aerial photograph and in some cases automatically create intersections where roadways cross. Most programs, including CORSIM (*41*), TransModeler® (*42*), AIMSUN® (*43*), and Paramics® (*44*) operate with an inherent link and node structure that uses links to represent roadways and nodes to represent intersections. VISSIM® (*45*), a popular simulation package developed in Germany, varies in this respect in that it uses links to represent roadways but link connectors to form roadway connections at intersections. All of these programs produce output results in both report and visual form, where visualization includes animation of vehicle flow over an aerial photograph or schematic of the network. Some programs even provide animation viewing capabilities in three dimensions.

All of the simulation tools briefly described above can be coded to model a two-lane roadway with varying quantities and classes of traffic. Each can even be coded to include passing lanes of a specific design at a given longitudinal frequency along the roadway. However, none of the tools includes an option for having traffic in one direction cross over the centerline to perform a passing maneuver utilizing the roadway lane in the opposing direction. Coding techniques can be used to create "dummy" links within the model that can replicate the behavior of a pass using the opposing lane, and logic external to the model can be applied to ensure drivers correctly assess passing opportunities before accessing the opposing lane; but analysts cannot use these tools to directly model two-lane roadway operations where passing is allowed.

One microsimulation tool that is different than the previously-mentioned tools is the Traffic Analysis Module (TAM) of the Federal Highway Administration's Interactive Highway Safety Design Model (IHSDM) (*46*). One component of a series of semi-automated design analysis tools created to improve roadway design consistency and safety, the TAM is actually a previously-developed program, known as TWOPAS (*47*), that is used to determine the expected operational performance of a proposed two-lane roadway design. TWOPAS itself was first developed by the Midwest Research Institute for FHWA in the mid-1970s and was adapted and improved over time. Prior to incorporation into IHSDM, TWOPAS was integrated with an improved user interface known as UCBRURAL (*48*) and upgraded to perform the two-lane roadway analysis necessary to obtain the empirical results contained in the 2000 edition of the Highway Capacity Manual (HCM) (*9*). While it lacks the graphical user interface of modern simulation tools, TWOPAS does allow the user to provide varying traffic volumes and classifications, different terrain, and a wide range of pavement marking options for allowing or disallowing passing. It also allows and provides results for the direct simulation of passing in both passing lane sections and along two-lane roadway sections where passing is permitted.

PERFORMANCE MEASURES

The HCM establishes performance requirements for the level of service on two-lane highways. Where motorists expect higher-speed performance—a condition known as a Class I two-lane roadway—performance is gauged both in terms of percent time following another vehicle and average travel speed. On recreational routes or on two-lane roadways in rugged terrain, motorist performance expectations are less demanding and level of service is determined by the Class II criterion of percent time following. As the higher-volume two-lane roadways investigated in the current research carry a motorist performance expectation, the Class I criteria are assumed for the remainder of this report.

Of the two two-lane roadway performance criteria, average speed is much simpler to calculate, both in the field and using computer simulation tools, than percent time following. Any of the simulation programs mentioned herein include average speed as a basic output, and variations to this measure can be calculated at specific points along the roadway, for various designated portions the roadway, or for various time ranges within the overall network simulation time. Regardless of how average speed is aggregated across distance or time, it is fundamentally calculated as the distance traveled (say, for all vehicles using the network in a given hour) divided by the travel time.

Percent time following, however, cannot be computed from the basic system performance and operating condition data aggregated by almost all simulation models. Though some subjectivity is involved in defining "following," below some threshold headway value (say, five seconds) a vehicle is assumed to be influenced in its speed choice by a leading vehicle. In the field or in a simulation model, the percent time following can be estimated for a given section of roadway either by locating sensors or detectors at multiple points along the roadway and monitoring and aggregating time headway data, or by sampling vehicles that have the capability of recording and reporting the portion of their travel time over the given roadway segment that they are close enough to a leading vehicle to be considered in a following position. All of the simulation tools described herein include at least one of these capabilities, though for most models extracting the data necessary to compute percent time following would involve extracting data from hundreds or thousands of detector entry reading or vehicle trajectory files using third-party software applications. Only the TWOPAS simulation model, which was designed for two-lane roadway analysis, directly tracks the following status of all simulated vehicles in such a manner that percent time following is an output performance measure directly available from the model.

MODEL COMPARISON

Table 27 presents a side-by-side comparison of candidate microscopic simulation models that were considered for high-volume Super 2 analysis. As shown in the table, no one model fully supported all of the issues or features desired in conducting the modeling runs necessary for the project. An initial review indicated that TWOPAS, run under the IHSDM platform, was the most utilitarian tool since it provided the output performance measures necessary for the project without post-processing. However, discussion with the IHSDM development team revealed that some inconsistencies exist in input data transfer between the IHSDM "shell" program and the version of TWOPAS embedded within IHSDM. Workarounds for such issues were discussed.

In an attempt to circumvent the impacts of possible coding or data transfer inconsistencies in IHSDM, the research team obtained a copy of UCBRURAL from the Institute of Transportation Studies (ITS) at the University of California at Berkeley. Again, the intent was to access and perform analysis runs using the TWOPAS program embedded under the UCBRURAL shell program. However, it was discovered that some legacy (DOS) programs, including UCBRURAL, do not run under modern operating systems (i.e., the Windows® XP operating system). An open-source DOS emulation program (DOSBox v0.72) was downloaded from the Internet and the UCBRURAL application was found to operate successfully in this virtual DOS environment.

Early in the research project, it was anticipated that the majority of the high-volume two-lane roadway simulation work necessary to support the current research would be performed using the TWOPAS program, running as the TAM under IHSDM, which was, in fact, the case. If any compatibility or programming issues had been encountered, the research team would have used the older, UCBRURAL interface. VISSIM, TransModeler, Paramics, and AIMSUN collectively formed the group of simulation tools best able to conduct the project research behind TWOPAS. The research team relied on its greater experience with VISSIM in support of project objectives. It was determined that VISSIM would be used for visualization purposes and to confirm basic traffic flow characteristics should traffic operations or traffic behavior questions arise while using TWOPAS.

Table 27. Simulation Tool Comparison for Super 2 Modeling.

Feature	CORSIM	TransModeler	Paramics	AIMSUN	VISSIM	TWOPAS
Runs on PC	●	●	●	●	●	○
Easy-to-use Interface	●	●	●	●	●	○
Model 2-Lane Roads	●	●	●	●	●	●
Model 2-lane Passing	○	○	○	○	○	●
Model Passing Lanes	●	●	●	●	●	●
Model Terrain Impacts	○	●	●	●	●	●
Model Vehicle Classes	○	●	●	●	●	○
Output Average Speed	●	●	●	●	●	●
Output % Time Following	○	○	○	○	○	●
NOTE: ● Fully supported; ○ Partially or indirectly supported						

CHAPTER 7
COLLECTION OF FIELD DATA

As described in Chapter 4, TxDOT districts and areas were surveyed for locations where passing lanes currently exist. Sufficient detail was requested to differentiate locations where passing lanes were added to serve as climbing lanes from locations where passing lanes were added to improve operational performance by allowing more frequent passing (i.e., Super 2 design) in level or gently rolling terrain. From all locations identified, Super 2 locations in the Paris District and the Yoakum District were selected for data collection.

The goal of each data collection effort was to document driver behavior and traffic conditions at the beginning and ending of studied passing lanes and to collect real-world traffic volume, classification, speed, and headway data before, within, and beyond each passing lane. These data were used to calibrate the traffic simulation model (the Traffic Analysis Module [TAM] within the Federal Highway Administration's Interactive Highway Safety Design Model [IHSDM]) used later in the research analysis and to develop estimates of passing lane impacts across ranges of traffic volumes found along two-lane roadways in Texas.

STUDY SITES

Paris District

Passing lanes in the TxDOT Paris District were located along SH 121 between the Collin/Fannin County line and the SH 56 junction just west of the town of Bonham, Texas (Figure 18). Within these boundaries, SH 121 is a two-lane rural roadway with a 70-mph speed limit. Within and between passing lanes the roadway is striped as a no-passing zone. SH 121 intersects several Farm-to-Market roads in this area, and all are at-grade intersections with two-way stop control (with the FM roads stopping). The corridor also has crossings with two major roadways, US-69 and SH 11, both of which are grade-separated and allow SH 121 traffic to remain uninterrupted. Right lane additions are present along both the SH 121 approaches to and departures from SH 11; the right lanes act simultaneously as passing lanes farther from SH 11 and right-turn deceleration or acceleration lanes closer to the interchange.

A listing of passing lane sections along both northbound and southbound SH 121 is provided in Table 28. Each pair of passing lanes in each direction was examined for potential data collection. In making the selection of which passing sections would be studied, researchers desired to have the longest possible spacing between upstream and downstream passing lane sections in order to examine changes in speed and headway as vehicles departed a given passing lane section. Stand-alone passing lane sections were preferred over those that also served an interchange-related acceleration or deceleration function.

For northbound SH 121, the passing lane section north of the Fannin County line was selected for data collection. The spacing between the end of this passing lane section and the beginning of the next downstream passing lane north of the US-69 interchange was approximately 3 miles. For southbound SH 121, researchers chose the stand-alone passing lane section between SH 11 and US-69 for field data collection. Since no acceleration or deceleration lanes are found at US-69 for southbound traffic, the next passing lane section was located approximately 2.7 miles downstream, approaching the FM 814 intersection.

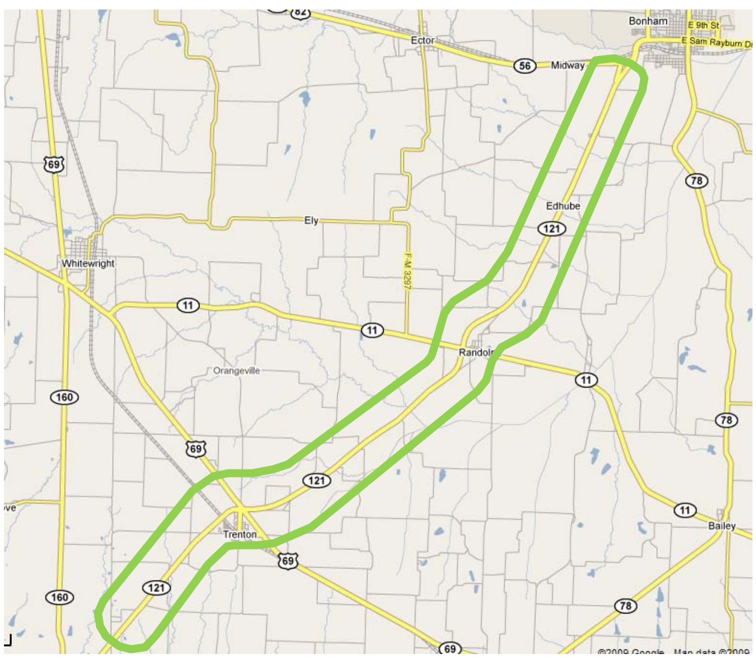


Figure 18. SH 121 Study Boundaries.
(Source of Base Map: Google® Maps, maps.google.com)

Table 28. SH 121 Passing Lane Sections.

Direction	Location	Approximate Length (mi)	Detail
Northbound	North of County Line Road	1.0	Stand-alone passing lane
	North of US-69	1.1	Also serves as acceleration lane
	Between US-69 and SH 11	0.7	Stand-alone passing lane, very close to SH 11 lane addition
	South of SH 11	0.8	Also serves as deceleration lane
	North of SH 11	1.1	Also serves as acceleration lane
	Between SH 11 and SH 56	1.9	Stand-alone passing lane, very close to added lane from SH 11
Southbound	South of SL 311	2.5	Also serves as acceleration lane
	North of SH 11	1.6	Also serves as deceleration lane
	South of SH 11	0.8	Also serves as acceleration lane
	Between SH 11 and US-69	0.7	Stand-alone passing lane
	South of FM 814	1.1	Also serves as acceleration lane

Yoakum District

Several passing lane sections are found along US-183 between Interstate 10 (I-10) and the city of Gonzales, Texas. US-183 is a two-lane roadway with a 70 mph speed limit through the rural area between I-10 and northern Gonzales, but approaching the city it expands to a four-lane facility south of Business 183. Passing is allowed between passing lane sections in locations with adequate sight and passing distance, though mildly rolling terrain and horizontal curves limit the number of locations where passing is allowed. Several minor roadway intersections are found within the study boundaries, including Park Road 11 and FM 1586. All cross-street intersections with US-183 are two-way stop controlled with cross-street traffic stopping.

Two passing lanes are found in the southbound direction along US-183 within the study boundaries. The first passing lane is located just south of I-10, and the second passing lane begins roughly 4.5 miles downstream of the end of the first passing lane. With only two passing lanes present, researchers opted to study the upstream passing lane, which was about 3.1 miles long, and the roadway segment downstream of this passing lane to the start of the second passing lane section. In the northbound direction, there is only a single passing lane within the study bounds. This passing lane begins about 5.1 miles north of Business 183 in northern Gonzales and is 2 miles long. Whereas at all other data collection sites there is a length of roadway to study between passing lane sections, in this case there is no location where a second passing lane is added approaching the I-10 interchange (to the north of the single existing passing lane section). Accordingly, the data collection procedure at this site required the setup of traffic monitoring and counting equipment upstream of, rather than downstream of, the passing lane section.

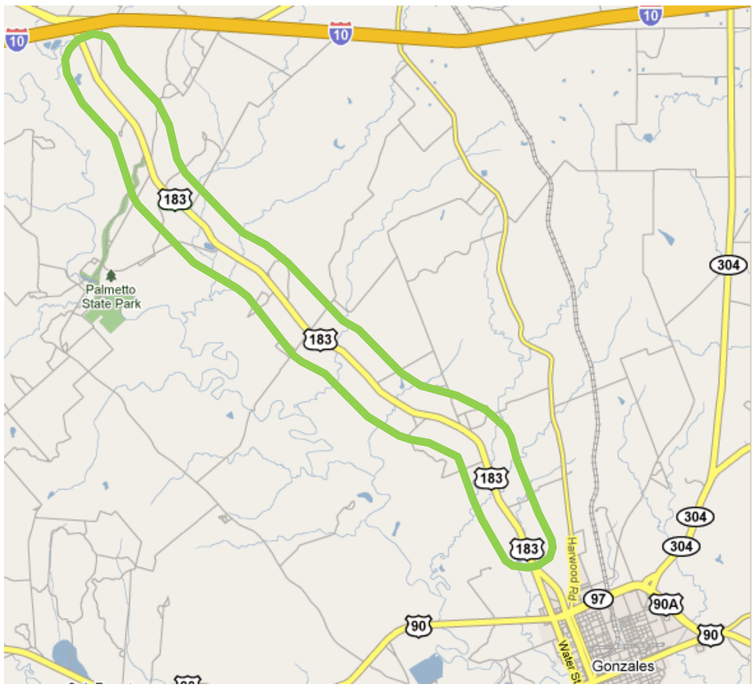


Figure 19. US-183 Study Boundaries.
(Source of Base Map: Google® Maps, maps.google.com)

DATA COLLECTION EQUIPMENT

Two types of data collection equipment were employed for collecting passing lane data along SH 121 and US-183. A video trailer with a telescoping mast was used to capture driver behavior approaching each of the four passing lane sections studied (Figure 20). A second trailer was also used to collect driver merging behavior at the downstream end of each passing lane section. Two cameras with pan, tilt, and zoom capability atop the mast allowed the field analysts to observe a field of view that included a short roadway segment preceding the passing lane, the expansion taper, and an additional distance of roughly 0.25 mile downstream at the beginning of each passing lane. At passing lane termini, a single camera was used and the field of view included the lanes approaching the reduction taper, the taper itself, and a short distance downstream. Digital video recording equipment was used to create a permanent 24-hour site visit video for passing lane beginning and ending points for each of the four passing lanes studied.

Figure 20. Video Trailer and Telescoping Mast.

The second type of field data collection equipment analysts used for the passing lane field studies was portable traffic analyzers, or "plate counters." An example of such a counter is depicted in Figure 21. The design of portable on-pavement traffic analyzers allows them to provide accurate count, speed, and vehicle classification data. The units are self-contained in an aluminum housing designed to withstand the wheel-load impact of heavy vehicles and damage from most chemicals such as oil or fuel. Technicians deploying the counters use a rugged sheet embedded with asphalt mastic to secure the sensor to the roadway surface, centered on a lane. The sensor determines vehicle count, speed, and classification data using magnetic imaging technology and is able to record speed, classification, and headway data for each individual vehicle passing over the sensor. No pneumatic tubes are required as with traditional traffic counting equipment, reducing the possibility of data loss due to equipment failure.

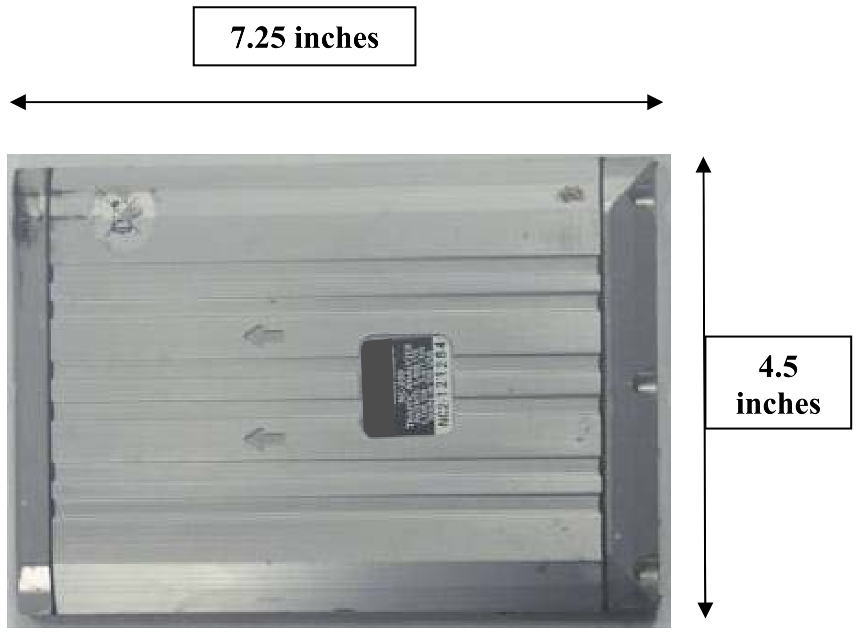


Figure 21. Portable (On-Pavement) Traffic Analyzer.

Portable traffic analyzers were deployed immediately upstream of each passing lane, in the two lanes within each passing lane section, immediately downstream of the passing lane and at evenly-spaced intervals between each passing lane and the next downstream passing lane. Five counters were available for the study of northbound SH 121, while nine counters were available for the study of southbound SH 121 and both the southbound and northbound studies along US-183.

FIELD DATA

As described previously, data on motorist behavior were collected with video at the passing lane beginning and end for each of the four studied passing lane locations. Counter data were collected for each location before, within and after each passing lane. Each type of data is described in the following sections, and summary values of each data type are presented.

Entering the Passing Lane

Data collected from video at the beginning of the passing lane for each of the four study sites included lane selection and observed passing behavior. Analysts recorded motorist lane selection by vehicle type and whether vehicles entering the passing (left) lane were initiating a passing maneuver at the upstream end of the passing lane. Hour-by-hour summaries of these data for each site are provided in Table 29 through Table 32. Readers will note that Table 29 and Table 30 are missing data for late evening and early morning hours. When video data were retrieved from DVR equipment for these time periods, the combination of low ambient light and camera light sensitivity and contrast created a situation where only the headlights of vehicles approaching the passing lane were visible. No roadway illumination was visible in the recorded video, and taillights of vehicles leaving the passing lane entrance section were not clearly visible. As a result, technicians could determine neither the lane use nor following behavior of vehicles entering the SH 121 passing lanes at these time periods during the data collection study.

Table 29. Passing Lane Entrance Data – SH 121 Northbound (7/21–22/2009).

<table>
<tr><th>Time</th><th>Count</th><th>Percent Vehicles Entering Left Lane</th><th>Percent Passing (of Total Count)</th><th>Percent Trucks</th><th>Percent Trucks Entering Right Lane</th></tr>
<tr><td>12–1 AM</td><td colspan="5" rowspan="7">Video under nighttime lighting inadequate for data reduction.</td></tr>
<tr><td>1–2</td></tr>
<tr><td>2–3</td></tr>
<tr><td>3–4</td></tr>
<tr><td>4–5</td></tr>
<tr><td>5–6</td></tr>
<tr><td>6–7</td></tr>
<tr><td>7–8</td><td>166</td><td>30.1</td><td>27.7</td><td>6.0</td><td>76.9</td></tr>
<tr><td>8–9</td><td>161</td><td>17.3</td><td>13.7</td><td>7.5</td><td>100.0</td></tr>
<tr><td>9–10</td><td>164</td><td>19.5</td><td>15.2</td><td>9.8</td><td>94.1</td></tr>
<tr><td>10–11</td><td>177</td><td>24.3</td><td>19.2</td><td>6.8</td><td>83.3</td></tr>
<tr><td>11–12</td><td>143</td><td>18.9</td><td>16.8</td><td>9.1</td><td>92.9</td></tr>
<tr><td>12–1 PM</td><td>196</td><td>19.9</td><td>17.3</td><td>5.6</td><td>100.0</td></tr>
<tr><td>1–2</td><td>175</td><td>15.4</td><td>13.1</td><td>2.9</td><td>100.0</td></tr>
<tr><td>2–3</td><td>198</td><td>18.9</td><td>18.7</td><td>5.6</td><td>84.6</td></tr>
<tr><td>3–4</td><td>318</td><td>27.0</td><td>26.1</td><td>2.8</td><td>90.0</td></tr>
<tr><td>4–5</td><td>330</td><td>23.9</td><td>23.9</td><td>2.4</td><td>100.0</td></tr>
<tr><td>5–6</td><td>391</td><td>20.5</td><td>20.2</td><td>1.3</td><td>100.0</td></tr>
<tr><td>6–7</td><td>312</td><td>24.0</td><td>22.4</td><td>1.3</td><td>100.0</td></tr>
<tr><td>7–8</td><td colspan="5" rowspan="5">Video under nighttime lighting inadequate for data reduction.</td></tr>
<tr><td>8–9</td></tr>
<tr><td>9–10</td></tr>
<tr><td>10–11</td></tr>
<tr><td>11–12</td></tr>
<tr><td>Total/Avg.</td><td>2731</td><td>22.1</td><td>20.3</td><td>4.3</td><td>92.4</td></tr>
</table>

Table 30. Passing Lane Entrance Data – SH 121 Southbound (7/22–23/2009).

Time	Count	Percent Vehicles Entering Left Lane	Percent Passing (of Total Count)	Percent Trucks	Percent Trucks Entering Right Lane
12–1 AM	Video under nighttime lighting inadequate for data reduction.				
1–2					
2–3					
3–4					
4–5					
5–6					
6–7	272	26.1	16.9	2.9	100.0
7–8	239	15.5	9.2	2.1	83.3
8–9	220	21.4	15.0	5.5	91.7
9–10	168	22.6	18.5	7.7	78.6
10–11	170	19.4	14.1	7.6	85.7
11–12	190	18.9	14.2	8.9	76.5
12–1 PM	186	19.4	12.9	6.5	78.6
1–2	184	25.5	16.8	9.2	82.4
2–3	163	22.7	11.7	2.5	75.0
3–4	182	20.9	12.1	7.7	71.4
4–5	202	22.8	15.3	5.0	70.0
5–6	186	22.0	15.6	2.2	66.7
6–7	152	18.4	10.5	0.0	n/a
7–8	111	18.0	10.8	0.9	100.0
8–9	111	13.5	6.3	2.7	100.0
9–10	Video under nighttime lighting inadequate for data reduction.				
10–11					
11–12					
Total/Avg.	2736	20.8	13.7	4.9	80.9

Table 31. Passing Lane Entrance Data – US-183 Southbound (8/10–11/2009).

Time	Count	Percent Vehicles Entering Left Lane	Percent Passing (of Total Count)	Percent Trucks	Percent Trucks Entering Right Lane
12–1 AM	35	8.6	0.0	25.7	100.0
1–2	10	0.0	0.0	20.0	100.0
2–3	13	0.0	0.0	23.1	100.0
3–4	11	0.0	0.0	54.5	100.0
4–5	17	0.0	0.0	35.3	100.0
5–6	39	10.3	5.1	30.8	84.6
6–7	121	9.9	9.1	15.7	100.0
7–8	137	14.6	11.7	8.8	100.0
8–9	171	14.6	10.5	11.7	90.5
9–10	188	19.1	18.1	12.2	100.0
10–11	167	11.4	9.6	12.6	95.5
11–12	141	13.5	12.1	12.8	100.0
12–1 PM	163	14.1	11.7	14.1	95.8
1–2	207	15.5	13.5	13.0	96.3
2–3	159	8.2	7.5	11.9	100.0
3–4	187	13.9	10.7	7.0	92.9
4–5	220	16.8	13.6	6.3	100.0
5–6	213	15.0	10.3	3.8	87.5
6–7	191	15.2	13.6	6.3	100.0
7–8	122	7.4	6.6	4.9	100.0
8–9	103	13.6	9.7	4.9	80.0
9–10	80	8.8	3.8	3.8	100.0
10–11	45	8.9	4.4	11.1	83.3
11–12	40	7.5	0.0	5.0	100.0
Total/Avg.	2780	13.2	10.6	10.4	95.3

Table 32. Passing Lane Entrance Data – US-183 Northbound (8/11–12/2009).

Time	Count	Percent Vehicles Entering Left Lane	Percent Passing (of Total Count)	Percent Trucks	Percent Trucks Entering Right Lane
12–1 AM	8	12.5	0.0	0.0	n/a
1–2	7	14.3	0.0	0.0	n/a
2–3	5	40.0	0.0	20.0	0.0
3–4	12	25.0	0.0	58.3	100.0
4–5	30	43.3	0.0	16.7	80.0
5–6	88	38.6	1.1	4.5	100.0
6–7	110	33.6	3.6	4.5	60.0
7–8	153	42.5	11.1	7.2	90.9
8–9	161	47.2	12.4	4.3	55.6
9–10	154	53.2	16.2	13.6	81.0
10–11	212	53.8	21.2	7.5	58.8
11–12	158	49.4	12.0	10.8	77.8
12–1 PM	184	54.9	13.6	11.4	86.4
1–2	214	49.1	14.0	4.7	90.0
2–3	207	50.2	7.2	6.3	69.2
3–4	209	51.7	12.0	7.2	73.3
4–5	218	52.8	17.9	7.8	70.6
5–6	191	41.9	17.3	7.3	92.9
6–7	166	45.8	12.7	7.8	50.0
7–8	112	47.3	6.3	6.3	85.7
8–9	90	46.7	11.1	7.8	50.0
9–10	85	47.1	4.7	4.7	50.0
10–11	44	56.8	0.0	9.1	25.0
11–12	26	46.2	0.0	7.7	0.0
Total/Avg.	2844	48.1	12.0	7.8	73.6

Table 29 and Table 30 present a consistent view of passing lane traffic operations along SH 121. Just over 20 percent of vehicles enter the left lane, and a large majority of the vehicles entering the left lane are passing vehicles. The percent using left lane and percent passing values are closer to equivalent for northbound SH 121, indicating slightly better left lane (for passing only) compliance, but both directions show that there is a high level of motorist understanding and compliance with the passing lane. Heavy vehicles compose less than 5 percent of the traffic stream in both directions, and trucks consistently use the right lane as they enter the passing lane section. Local (Fannin County) law enforcement was visibly present in the SH 121 corridor during the field data collection, and a brief interview with an enforcement officer confirmed that the passing lane signing (i.e., left lane for passing only) was actively enforced.

Table 31 and Table 32 show that passing lane operations along US-183 are in many ways similar to SH 121, but some significant differences were also noted. For US-183 in the southbound direction (i.e., away from I-10 and toward Gonzales), 13 percent of vehicles enter the passing lane at its beginning and 80 percent of those vehicles are preparing to pass a slower moving vehicle. Truck percentage is on the order of 10 percent, and trucks consistently use the right lane when entering the passing lane section. However, for northbound US-183 almost 50 percent of vehicles entering the passing lane do so in the left lane while only 25 percent of those vehicles are doing so to pass a slower-moving vehicle. Truck percentage in the northbound direction is just under 8 percent, and roughly 75 percent of trucks enter the right lane of the passing lane section.

While remarking the differences in passing lane operations between SH 121 and US-183, both the passing lane signing and markings and the level of enforcement are markedly different between the two corridors. The studied portion of SH 121 is striped for no passing along its entirety, with passing maneuvers only allowed in passing lane sections. US-183 is striped to allow passing in two-lane sections and for traffic in the direction opposing passing lane sections where sight distance and roadway geometry allow. Also, "left lane for passing only" signing was posted with greater frequency in the SH 121 corridor and diagonal striping across the left lane (directing vehicles to the right lane unless passing) found in the SH 121 corridor was not present at passing lanes on US-183. Finally, during the week-long data collection studies the SH 121 corridor was observed to be actively enforced while the US-183 corridor appeared to be more intermittently enforced.

Passing Lane Terminus

At the end of each passing lane, data were reduced from video to determine lane selection by vehicle classification and the presence and severity of merging conflicts between passing and passed vehicles. If analysts observed merging conflicts between vehicles at the end of the passing lane, they ranked the conflicts as none (vehicles merging had a headway less than roughly three seconds, but no merge conflict was observed), low level, which did not involve braking, medium level, which involved braking, or high level, which involved both braking and swerving to avoid collision.

Data from SH 121 northbound and southbound are found in Table 33 and Table 34, respectively, while data from US-183 southbound and northbound are found in Table 35 and Table 36. Driver behavior at the end of each passing lane is more consistent than driver behavior at the beginning of passing lanes for the sites under investigation. Drivers chose the left lane between 18 and 28 percent of the time on average, with some increases in left lane usage noted during higher-volume (peak) periods of the day. Passing percentages were slightly lower than those observed for the start of the passing lane and vary on average between 41 and 66 percent. Truck utilization of the right lane remained high, but it was also slightly lower than that observed at the start of the passing lanes. Truck utilization of the right lane, passing behavior, and the percentage of vehicles using the left lane are all likely influenced by driver reactions to the passing lane terminus.

Table 33. Passing Lane Departure Data – SH 121 Northbound (7/21–22/2009).

Time	Count	Percent Vehicles in Left Lane	Percent Passing (of Total Count)	Percent Trucks	Percent Trucks in Right Lane	Merge Conflict			
						None	Low (No Brake)	Medium (Braking)	High (Brake & Swerve)
12–1 AM	27	7.4	0.0	7.4	100.0	1	0	0	0
1–2	14	0.0	0.0	14.3	100.0	0	0	0	0
2–3	8	12.5	0.0	0.0	n/a	0	0	0	0
3–4	6	16.7	0.0	16.7	0.0	0	0	0	0
4–5	22	13.6	0.0	13.6	66.7	0	0	0	0
5–6	39	12.8	5.1	17.9	85.7	0	0	0	0
6–7	114	17.5	15.8	11.4	61.5	4	5	0	0
7–8	166	25.3	21.7	9.6	75.0	8	7	0	0
8–9	147	20.4	19.0	8.8	84.6	7	4	0	0
9–10	164	26.8	18.3	20.1	60.6	7	5	0	0
10–11	212	34.4	23.6	13.2	46.4	5	3	0	0
11–12	152	32.9	15.8	11.2	58.8	1	0	0	0
12–1 PM	198	36.4	13.6	9.6	68.4	4	2	0	0
1–2	171	17.5	7.6	5.3	88.9	0	1	0	0
2–3	186	25.3	17.2	9.7	83.3	3	0	0	0
3–4	269	29.0	16.4	4.1	90.9	6	0	6	0
4–5	321	31.5	19.6	2.8	88.9	5	1	5	0
5–6	357	29.1	20.4	1.1	50.0	5	4	3	0
6–7	306	33.7	29.1	2.0	100.0	3	1	3	0
7–8	236	30.9	24.6	2.5	83.3	3	1	0	0
8–9	130	26.2	17.7	2.3	100.0	1	0	0	0
9–10	125	16.8	16.8	1.6	100.0	0	3	0	0
10–11	64	15.6	9.4	9.4	33.3	0	0	0	0
11–12	61	29.5	0.0	21.3	7.7	0	0	0	0
Total/Avg.	3495	27.5	18.2	6.9	66.8	63	37	17	0

Table 34. Passing Lane Departure Data – SH 121 Southbound (7/22–23/2009).

Time	Count	Percent Vehicles in Left Lane	Percent Passing (of Total Count)	Percent Trucks	Percent Trucks in Right Lane	Merge Conflict			
						None	Low (No Brake)	Medium (Braking)	High (Brake & Swerve)
12–1 AM	21	38.1	4.8	9.5	50.0	1	1	0	0
1–2	9	44.4	0.0	22.2	50.0	0	0	0	0
2–3	14	35.7	0.0	35.7	40.0	0	0	0	0
3–4	21	19.0	0.0	14.3	100.0	0	0	0	0
4–5	58	41.4	10.3	3.4	50.0	3	0	0	0
5–6	178	24.2	11.2	10.7	78.9	10	7	2	0
6–7	253	37.5	26.1	6.7	58.8	14	11	5	1
7–8	256	19.5	14.5	5.9	93.3	9	1	0	0
8–9	234	31.6	18.8	8.5	70.0	12	5	0	0
9–10	169	24.3	17.2	9.5	81.3	8	7	0	0
10–11	163	28.2	16.6	12.3	70.0	10	5	4	0
11–12	197	22.3	19.3	10.2	75.0	14	7	0	1
12–1 PM	199	22.6	17.1	9.0	83.3	12	6	1	0
1–2	191	29.3	23.0	8.9	88.2	6	4	1	0
2–3	170	31.2	18.8	4.7	87.5	7	12	2	0
3–4	181	37.6	14.9	9.4	64.7	15	6	0	0
4–5	205	27.3	17.6	7.8	75.0	7	4	1	0
5–6	199	30.7	21.1	6.5	46.2	8	7	2	0
6–7	167	25.7	18.0	0.6	100.0	6	2	0	0
7–8	126	21.4	17.5	1.6	100.0	1	1	0	0
8–9	116	22.4	19.0	1.7	50.0	5	7	6	0
9–10	72	26.4	11.1	2.8	50.0	5	3	0	0
10–11	26	38.5	11.5	0.0	n/a	2	0	0	0
11–12	25	36.0	4.0	0.0	n/a	1	1	0	0
Total/Avg.	3250	28.0	17.5	7.3	72.6	156	97	24	2

Table 35. Passing Lane Departure Data – US-183 Southbound (8/10–11/2009).

Time	Count	Percent Vehicles in Left Lane	Percent Passing (of Total Count)	Percent Trucks	Percent Trucks in Right Lane	Merge Conflict			
						None	Low (No Brake)	Medium (Braking)	High (Brake & Swerve)
12–1 AM	28	10.7	0.0	32.1	100.0	0	0	0	0
1–2	10	20.0	0.0	0.0	n/a	0	0	0	0
2–3	8	0.0	0.0	12.5	100.0	0	0	0	0
3–4	5	40.0	0.0	20.0	100.0	0	0	0	0
4–5	22	27.3	9.1	13.6	66.7	2	0	0	0
5–6	33	15.2	3.0	15.2	100.0	1	0	0	0
6–7	108	20.4	9.3	16.7	100.0	8	0	1	0
7–8	126	19.0	8.7	3.2	100.0	3	6	2	0
8–9	161	15.5	5.6	9.9	93.8	8	1	0	0
9–10	166	18.7	10.2	9.6	87.5	10	5	2	0
10–11	160	23.8	10.0	7.5	91.7	12	2	1	0
11–12	127	18.1	8.7	6.3	100.0	6	2	1	0
12–1 PM	147	14.3	6.8	8.8	92.3	6	5	1	0
1–2	170	12.9	11.8	10.6	94.4	13	5	1	0
2–3	189	19.6	13.8	8.5	87.5	19	6	1	0
3–4	177	19.2	12.4	6.8	83.3	14	6	2	0
4–5	197	16.8	12.7	5.6	100.0	15	8	2	0
5–6	185	21.1	11.4	2.2	75.0	8	10	3	0
6–7	180	22.8	15.6	7.2	84.6	17	6	5	0
7–8	186	16.7	10.2	2.7	80.0	10	3	3	0
8–9	95	11.6	8.4	3.2	100.0	5	2	1	0
9–10	72	18.1	9.7	4.2	66.7	4	2	1	0
10–11	46	13.0	2.2	13.0	100.0	1	0	0	0
11–12	44	13.6	0.0	2.3	0.0	0	0	0	0
Total/Avg.	2642	18.0	10.0	7.5	90.9	162	69	27	0

Table 36. Passing Lane Departure Data – US-183 Northbound (8/11-12/2009).

Time	Count	Percent Vehicles in Left Lane	Percent Passing (of Total Count)	Percent Trucks	Percent Trucks in Right Lane	Merge Conflict			
						None	Low (No Brake)	Medium (Braking)	High (Brake & Swerve)
12–1 AM	9	11.1	0.0	0.0	n/a	0	0	0	0
1–2	7	14.3	0.0	0.0	n/a	0	0	0	0
2–3	15	13.3	0.0	6.7	0.0	0	0	0	0
3–4	12	25.0	8.3	58.3	71.4	1	0	0	0
4–5	28	14.3	3.6	17.9	100.0	1	0	0	0
5–6	48	2.1	2.1	8.3	100.0	1	0	0	0
6–7	113	15.9	4.4	8.0	66.7	3	1	0	0
7–8	152	28.9	10.5	5.9	88.9	11	4	1	0
8–9	150	23.3	7.3	4.0	66.7	8	3	0	0
9–10	153	28.1	12.4	7.2	72.7	18	1	0	0
10–11	194	28.9	12.9	5.2	80.0	16	7	2	0
11–12	158	26.6	10.1	10.1	87.5	11	4	1	0
12–1 PM	181	23.2	10.5	8.8	81.3	14	2	3	0
1–2	199	30.2	13.1	4.5	77.8	18	6	1	0
2–3	185	25.9	10.8	5.4	80.0	14	5	1	0
3–4	204	22.1	9.3	4.9	80.0	11	5	1	0
4–5*	220	21.8	10.9	7.3	100.0	12	8	4	0
5–6	188	19.7	11.2	6.9	92.3	15	5	1	0
6–7	147	27.2	11.6	3.4	40.0	10	5	1	0
7–8	103	35.0	16.5	7.8	75.0	12	3	0	0
8–9	68	30.9	7.4	17.6	66.7	5	0	0	0
9–10	68	16.2	4.4	8.8	83.3	3	0	0	0
10–11	40	22.5	2.5	7.5	100.0	1	0	0	0
11–12	22	22.7	4.5	13.6	33.3	1	0	0	0
Total/Avg.	2664	24.5	10.1	7.1	79.9	186	59	16	0

* Data partially missing; hourly values estimated from 15 minutes of available data.

The rate of merging conflicts observed in the field was consistently low across the study sites. Merging events rated as medium (i.e., involving braking) or high (i.e., involving braking and swerving) occurred with greater frequency during higher-volume peak periods, but on average the number of moderate or high merging conflict events was less than 1 per 100 daily vehicles (Table 37). The conflict rate was generally observed to increase during the highest volume hour of the day, but this trend was not consistent across all sites at all times.

Table 37. Daily and Peak Hour Passing Lane Merge Conflict Rates.

Site	Daily Volume (vpd)	Peak Volume (vph)	Daily Merge Events*	Peak Merge Events*	Daily Conflict Rate (conflicts/veh)	Peak Conflict Rate (conflicts/veh)
SH 121 NB	3495	357 (5–6 PM)	17	3	1/206 (0.004)	1/119 (0.008)
SH 121 SB	3250	256 (7–8 AM)	26	0	1/125 (0.008)	0/256 (0.000)
US-183 SB	2642	197 (4–5 PM)	27	2	1/98 (0.010)	1/99 (0.010)
US-183 NB	2664	220 (4–5 PM)	16	4	1/167 (0.006)	1/55 (0.018)

* Events included are those rated medium or high.

Passing Lane Speed and Headway Data

Field data collection with portable traffic analyzers supplied traffic count, speed, vehicle classification, and headway data for the model calibration and analysis of higher-volume two-lane roadways with passing lanes. Options exist when programming the traffic analyzers to specify certain site characteristics, such as the roadway name and speed limit, and to enter the data file name. The time frame over which data are to be collected is also pre-programmed, and the analyst selects whether data are to be "binned," or automatically averaged and categorized, or whether "sequential" data collection is desired. Technicians selected the sequential data collection option for the passing lane speed, class, and headway studies so those data for each individual vehicle passing over the traffic analyzer were collected. Data were downloaded from the traffic analyzers after each data collection trip.

The data file for each station at each study site contained a vehicle count identifier, the time the vehicle passed over the counter, and indication of speed and classification accuracy, vehicle speed, vehicle length, gap (headway) time, gap (headway) distance, and whether the analyzer estimated following vehicles were tailgating. Analyzer locations within each study site and speed and headway data at each data collection station are provided in Figure 22 through Figure 25.

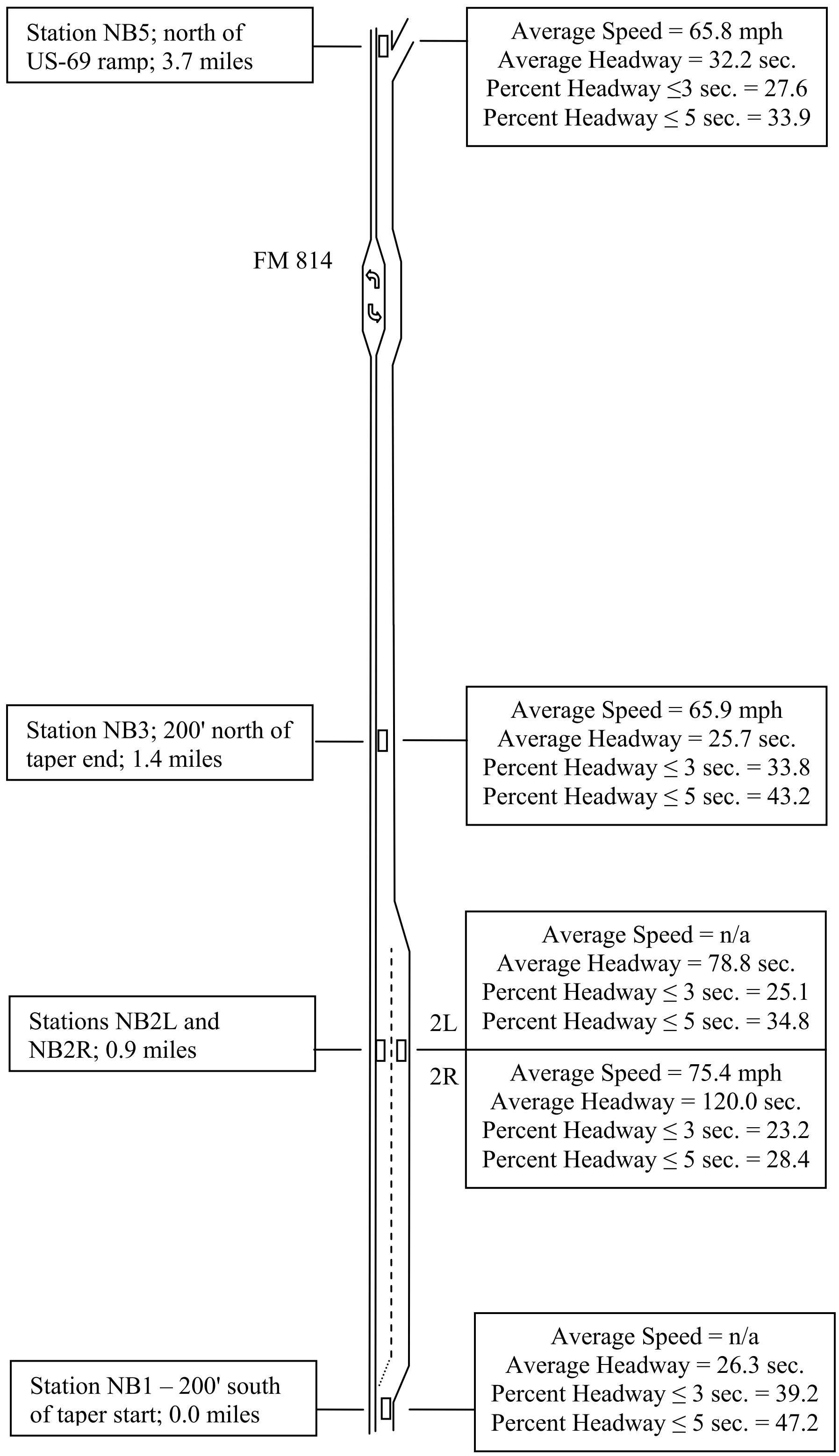


Figure 22. Speed and Headway Data for SH 121 Northbound (7/21–22/2009).

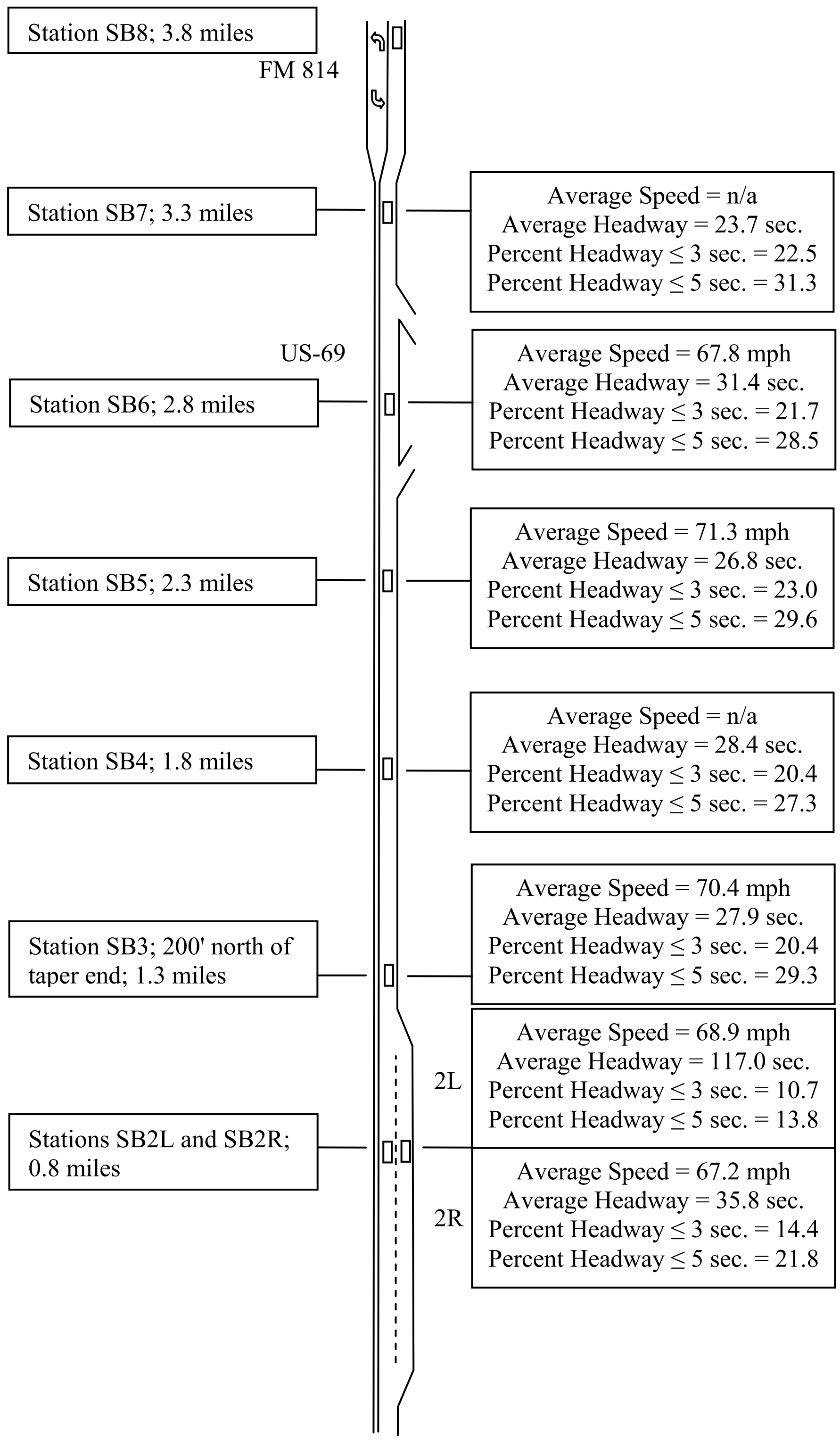


Figure 23. Speed and Headway Data for SH 121 Southbound (7/22–23/2009).

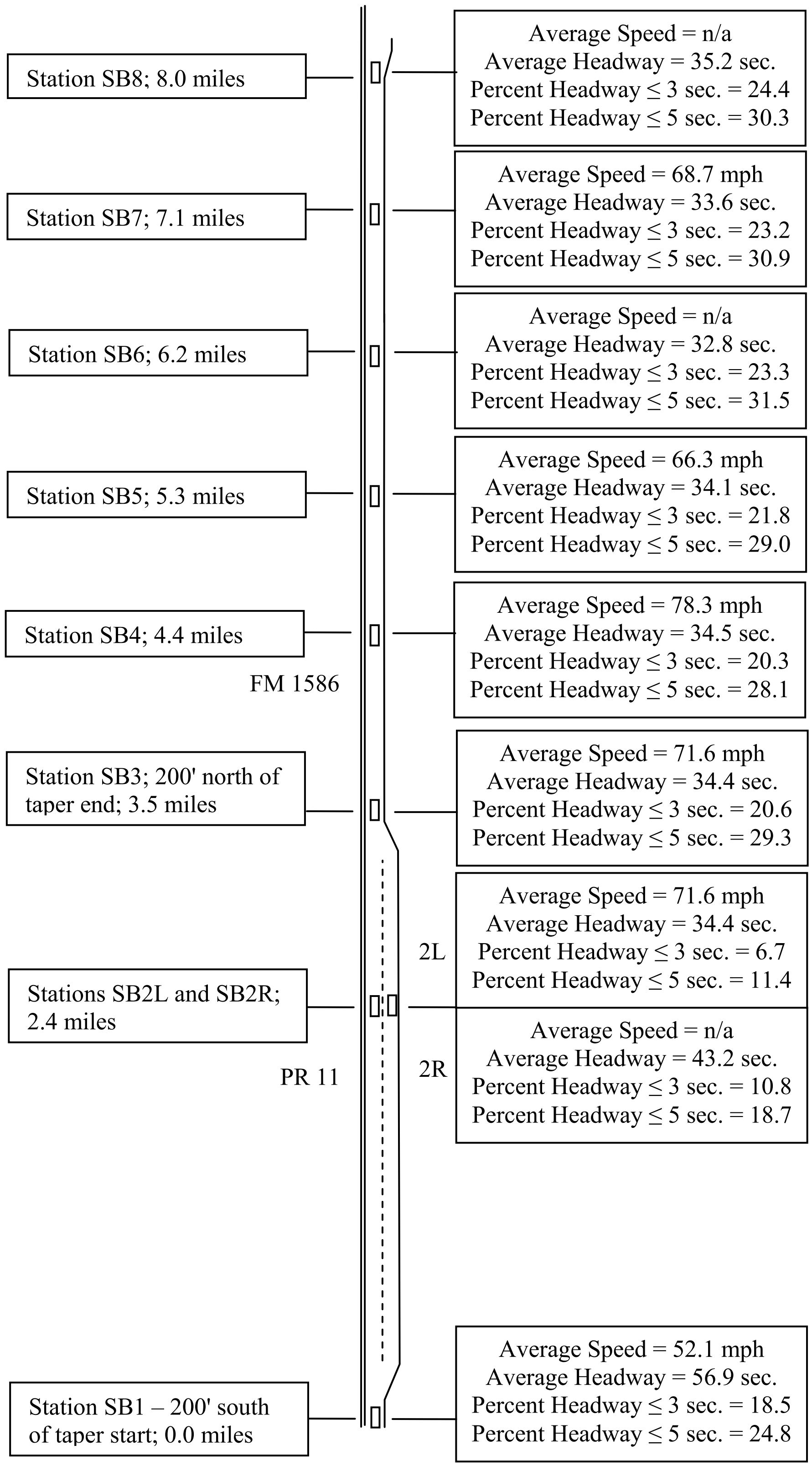


Figure 24. Speed and Headway Data for US-183 Southbound (8/10–11/2009).

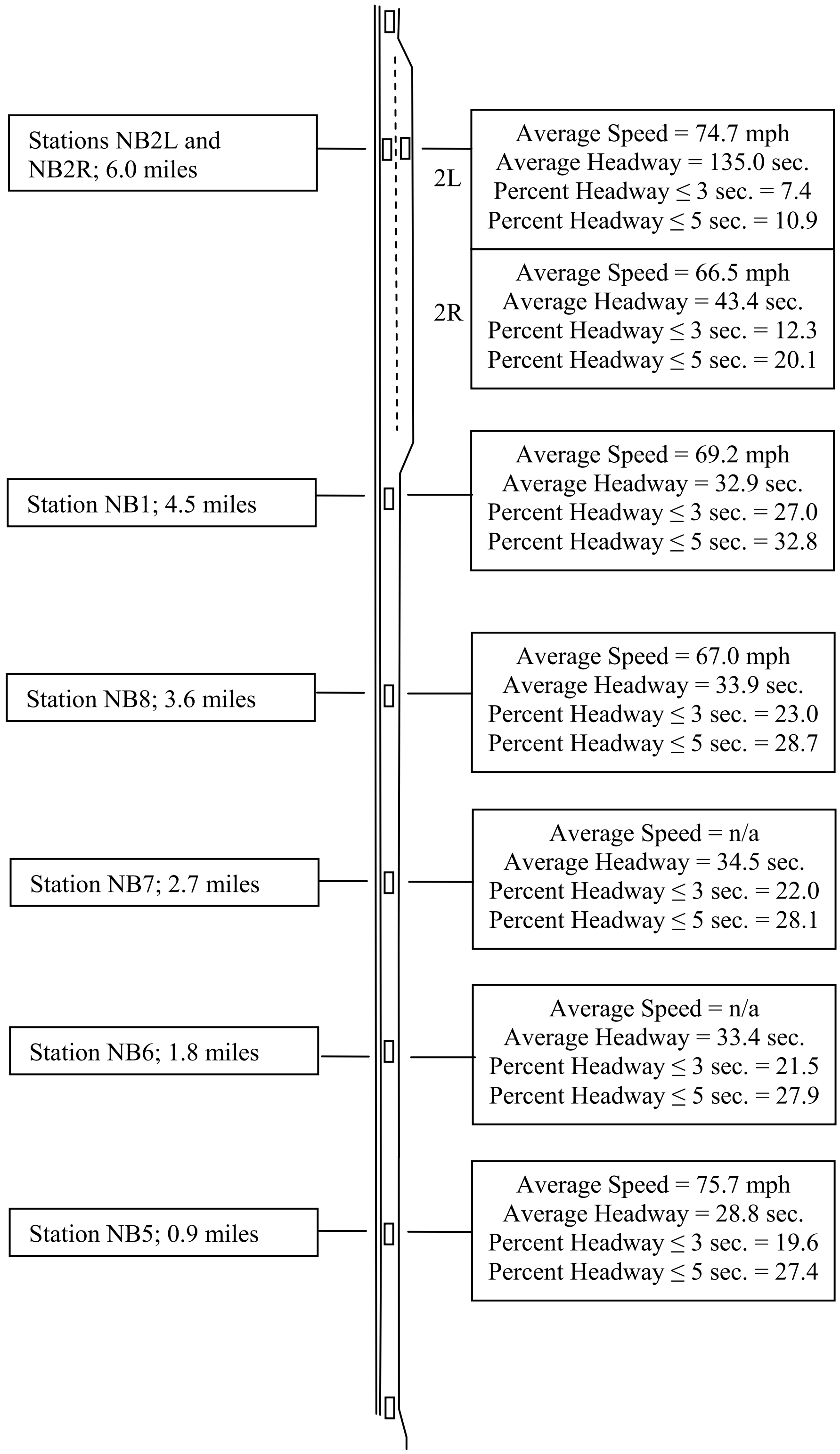


Figure 25. Speed and Headway Data for US-183 Northbound (8/11–12/2009).

As researchers processed the data from each study site station, they noticed that certain portable traffic analyzer devices had an abnormally high error rate in classifying vehicles passing over the device. Review of speed data from analyzers with a high classification error rate revealed that average speeds from these analyzers was not within a reasonable range with respect to either the roadway speed limit where the device was placed in the field or the speeds recorded at adjacent data collection stations. Speed data for the counters with a high error rate was excluded from further analysis, and resulted in the "n/a" designations for average speed shown in Figure 22 through Figure 25. Uncertainty as to the cause of speed under- or over-reporting for the analyzers and the fact that the error rate for the remaining analyzers was variable (and had an unknown impact on speed accuracy for those devices) led researchers to focus on the headway data from the analyzers for passing lane impact estimation and calibration against the model used for traffic simulation analysis of passing lanes.

Headway data for each study and each station were calculated as the arrival time difference between following vehicles over each traffic analyzer device. Where following headways are shorter, which typically include locations just before passing lanes, the distribution of headways in a headway frequency diagram is shifted toward the left (or y axis) and the proportion of headways less than 3 seconds is relatively high. Where volumes are low and following times between vehicles are greater, such as in the left lane of a passing lane section, the headway frequency distribution is "flatter" and the proportion of vehicles with short headways is low. Data from the SH 121 and US-183 field study sites consistently follow these general headway observation trends.

Headway Data for SH 121 Northbound

Data for the northbound SH 121 passing lane study is depicted in Figure 26. Upstream of the passing lane section—which is the first passing lane and passing opportunity for many miles—the headway distribution heavily favors headways of 1 and 2 seconds. In the passing lanes, however, the volume is split into the left and right lanes and the headway distribution represents longer headways present in each separate lane. The right lane, which has a much larger proportion of the volume that the left lane of the passing lane section, continues to have some shorter headways but the overall headway distribution is "rounder" than upstream of the passing lane and is shifted away from the y axis. Average headway values for each study station are provided in Figure 22. The left lane of the passing lane section shows the "flattest" distribution, emphasizing that vehicle following is minimized in this lane.

Station 3, which is immediately downstream of the passing lane section, has a headway distribution similar to Station 1. This finding would suggest that many of the passing lane benefits of increasing headway, reducing time spent following, and providing passing opportunities are minimized downstream of passing lanes for roadways with volume and geometric circumstances similar to northbound SH 121. Statistical analysis of these data in later project phases will provide an improved estimate of the impacts of passing lanes on roadway segments immediately downstream for roadways similar to SH 121.

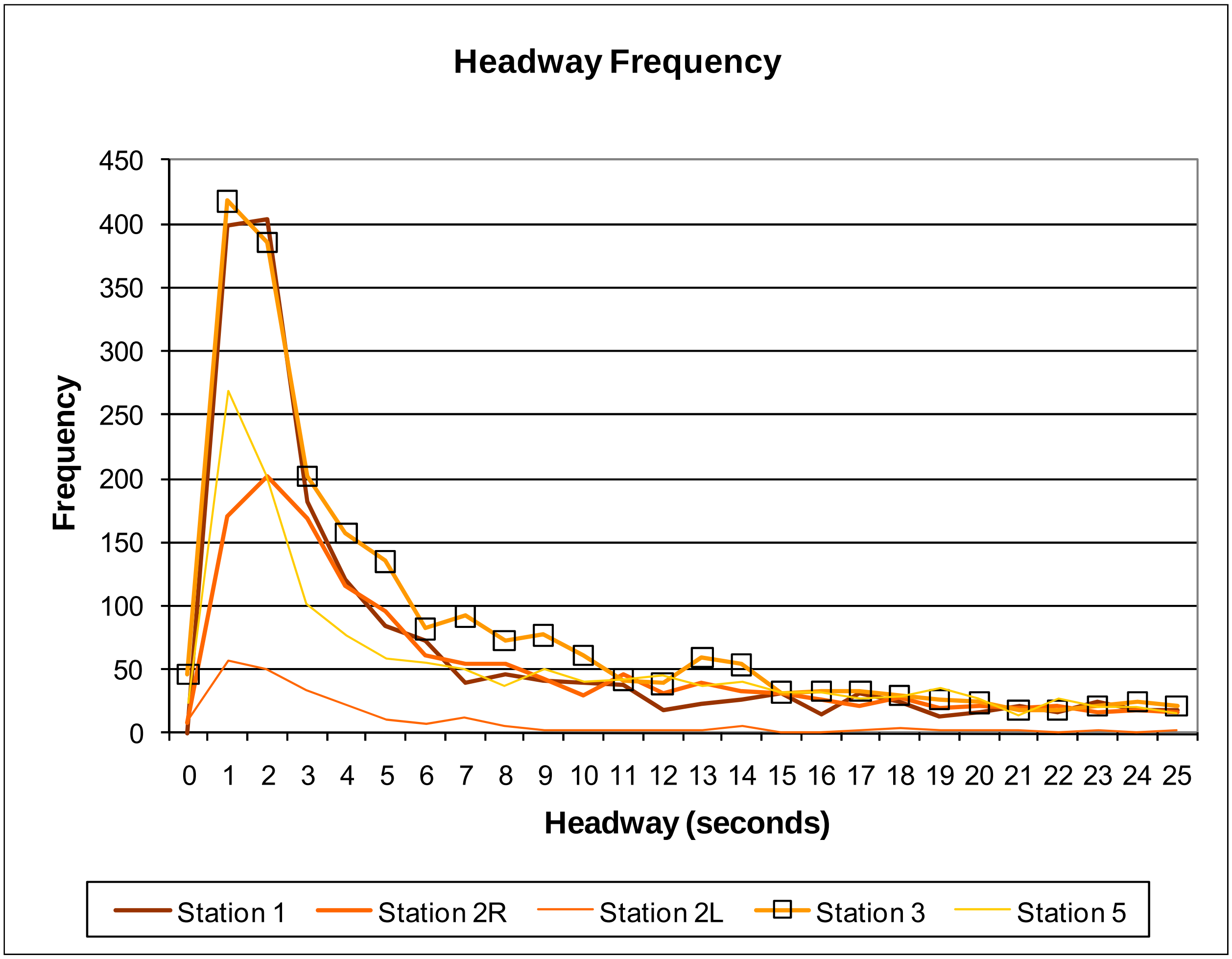


Figure 26. Headway Frequency Distribution for SH 121 Northbound.

The final headway distribution for SH 121 northbound is for Station 5, which was located just downstream of the exit ramp to US-69. The location downstream of the ramp was selected to avoid speed reduction influences of the ramp, an issue that has unfortunately been compromised by uncertainties with the speed analysis accuracies of the counters. Because vehicles exiting at the ramp are no longer in the traffic stream at Station 5, the volume and headway distribution are directly affected. Nonetheless, the headway frequency distribution shows a high proportion of vehicles with headways of 1 or 2 seconds, indicating that vehicle following remains prevalent at a location 2.3 miles downstream of the passing lane end.

Headway Data for SH 121 Southbound

Figure 27 contains headway frequency distribution data for the data collection stations along the SH 121 southbound study sites shown in Figure 23. Consistent with expected trends, Station 1 before the beginning of the passing lane section shows a headway distribution shifted toward the y axis (and a high proportion of low headway values). The studied passing lane section is approximately 1.7 miles downstream of the next upstream passing lane, which is also a right-hand acceleration lane added at the entrance ramp from SH 11 onto southbound SH 121.

Data from Stations 2R and 2L again show the change in headway distribution associated with operations within the passing lane section, where headways are significantly higher and more evenly distributed than locations either upstream or downstream of the passing lane section. Stations 3 and 4 demonstrate that headways have not yet returned to the level associated with conditions that existed before the passing lane, potentially suggesting that passing lane benefits extend beyond the physical limits of the passing lane for traffic and roadway conditions similar to those found along SH 121 southbound. The Station 5 headway distribution closely resembles conditions that existed at Station 1, indicating that passing lane benefits may not extend beyond 1 mile past the passing lane terminus under prevailing conditions.

Station 6 is located past the exit ramp to US-69, so uncertainty increases with respect to headway distribution impacts (i.e., whether changes in the distribution are caused by driver behavior over the given distance downstream of the passing lane or by vehicle departures to US-69). Station 7 is downstream of the entrance ramp for US-69 traffic merging onto southbound SH 121, and Station 8 is located downstream of the beginning of a right-turn lane added to accommodate traffic turning onto FM 814 and as the beginning point of the last passing lane along southbound SH 121.

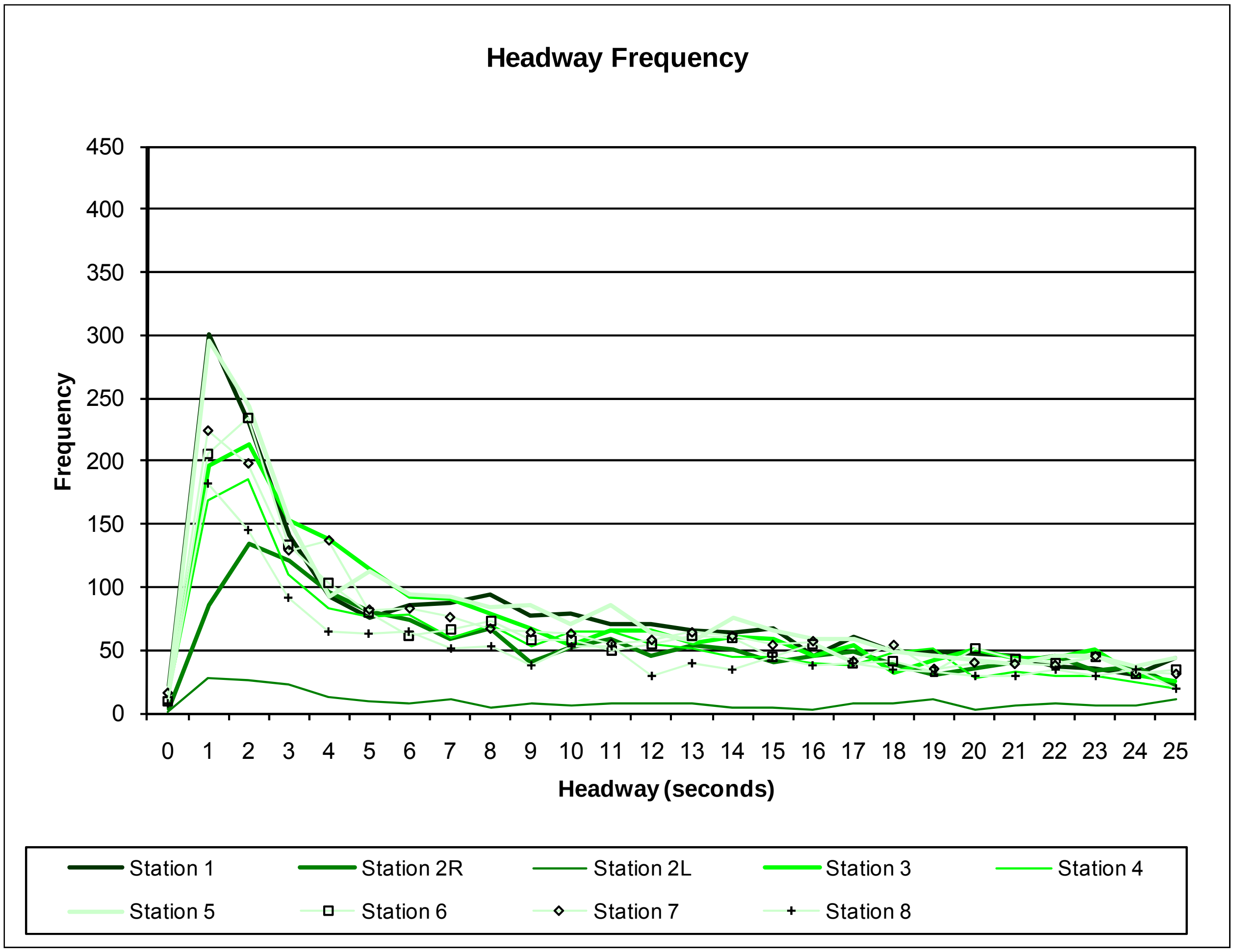


Figure 27. Headway Frequency Distribution for SH 121 Southbound.

Headway Data for US-183 Southbound

The roadway environment for US-183 in Gonzales County is significantly different than that found along SH 121 in Fannin County. High-volume intersecting roadways found along SH 121, such as US-69 and SH 11, are not found along US-183. Intersecting roadways, such as FM 1586 and Park Road 11, exist in the US-183 corridor, but their volume level and the extent of traffic interchange with US-183 were not expected to have major influences on headway distribution and following behavior in the corridor. Also, the SH 121 corridor was continuously striped for no passing while passing was allowed along US-183 where sight distance and roadway conditions permitted.

Figure 28 provides headway distribution data for US-183 southbound. Consistent with previously-described distributions, Station 1 just upstream of the passing lane section shows a headway distribution with a high proportion of short, or following, headways. However, because this station was located relatively close to I-10, drivers along US-183 southbound have not traveled a sufficient distance downstream from the interchange to sort into a free-flow headway distribution. In later analysis phases, Station 8 data may be used in place of Station 1 data as Station 8 represents a point just upstream of a passing lane section but one that is sufficiently downstream of intersections or other passing lane sections to represent driver headway and following behavior under free-flow conditions.

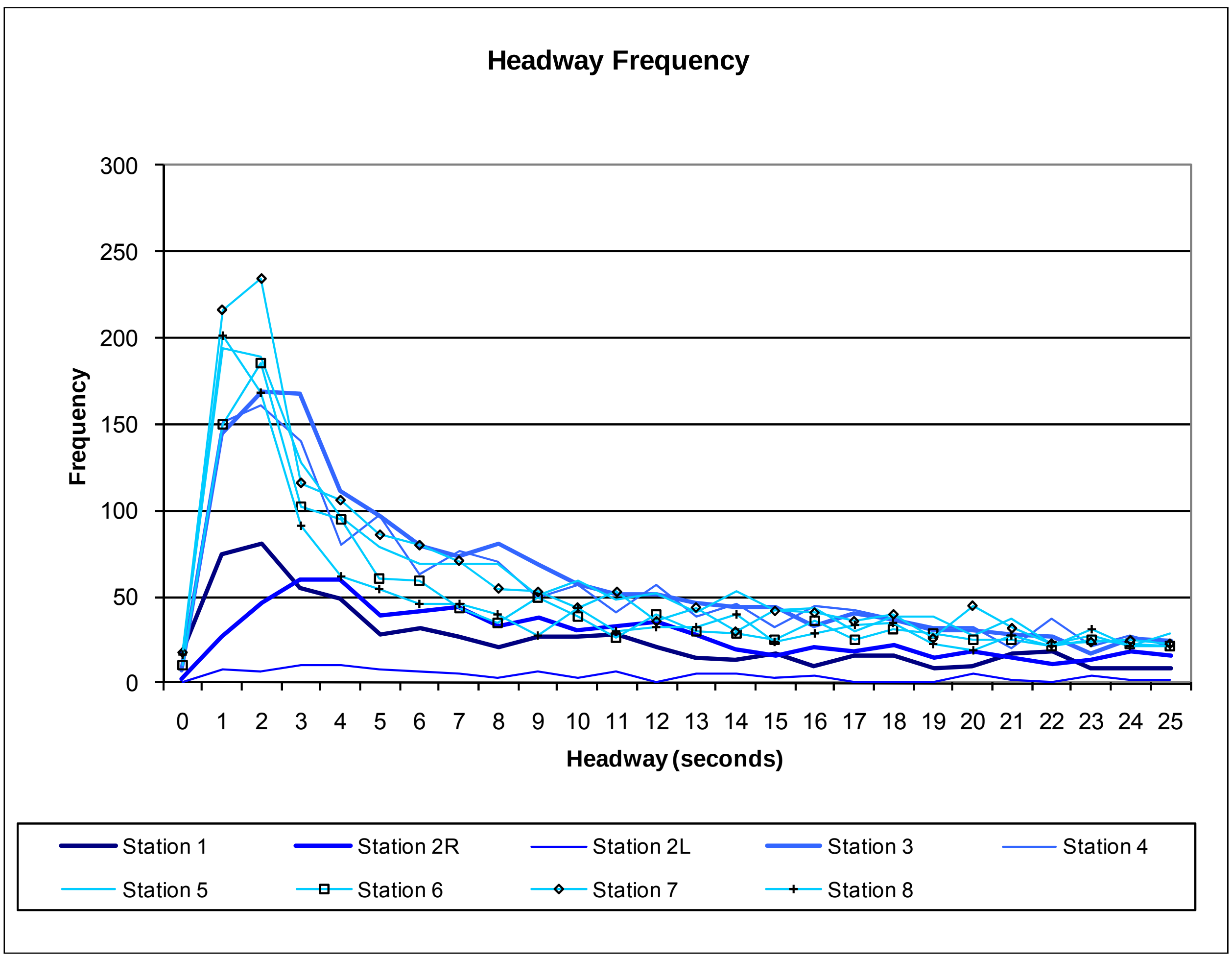


Figure 28. Headway Frequency Distribution for US-183 Southbound.

Headway distribution data for Stations 3 and 4 appear to indicate that headway (vehicle following) benefits persist downstream of the passing lane. Careful analysis of data for Stations 5, 6, and 7 will indicate the extent to which passing lane benefits exist downstream of the passing lane terminus.

Headway Data for US-183 Northbound

Site conditions displayed for US-183 in Figure 25 result in the station-wise headway distributions shown in Figure 29. The high proportion of following headways (less than 5 seconds) in the Station 1 distribution is separated across the passing lane section represented by Stations 2R and 2L, which demonstrate both longer and more evenly distributed headways. Data for Station 3 through Station 7 show the redistribution of headways over time and distance downstream of the passing lane, and Station 8 shows a headway frequency distribution very similar to Station 1.

Similar to the other study sites, statistical analysis of these distributions in later project phases has the potential to reveal the extent over which passing lane benefits extend beyond the physical limits of the passing lane addition. Subsequent analysis will account for the fact that US-183 northbound data stations were upstream of the passing lane section (i.e., data for Station 3 and Station 4 are both immediately downstream of passing lane sections).

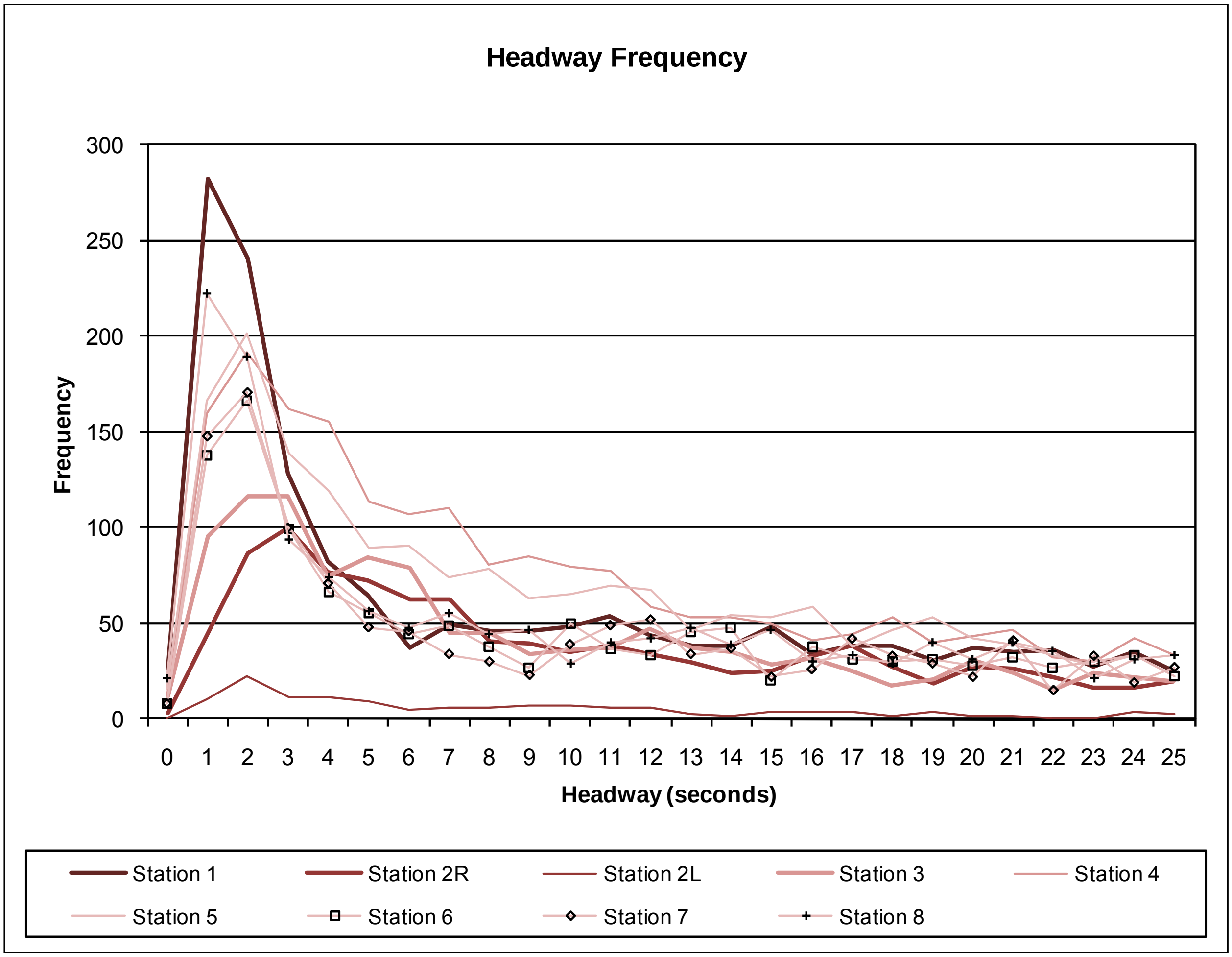
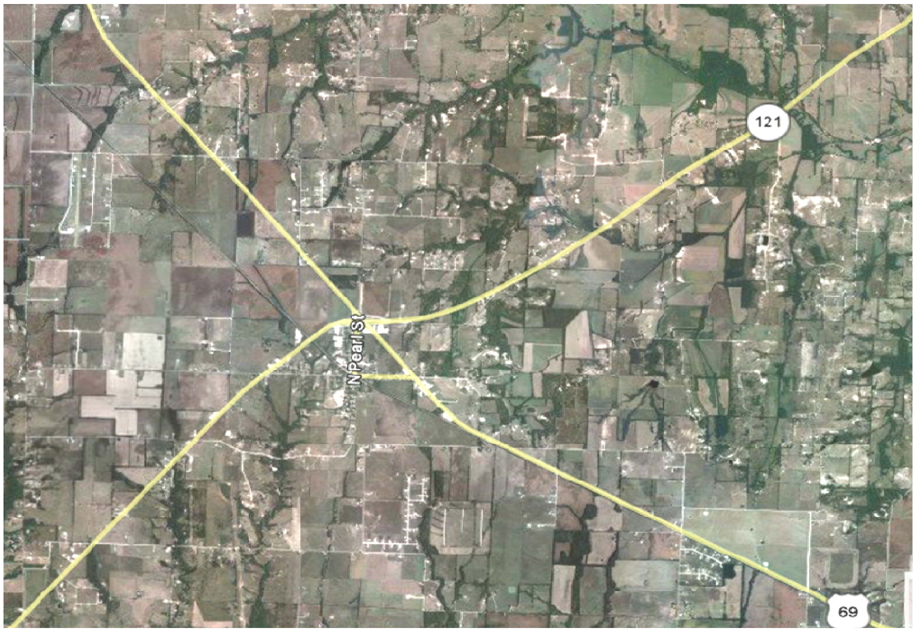


Figure 29. Headway Frequency Distribution for US-183 Northbound.

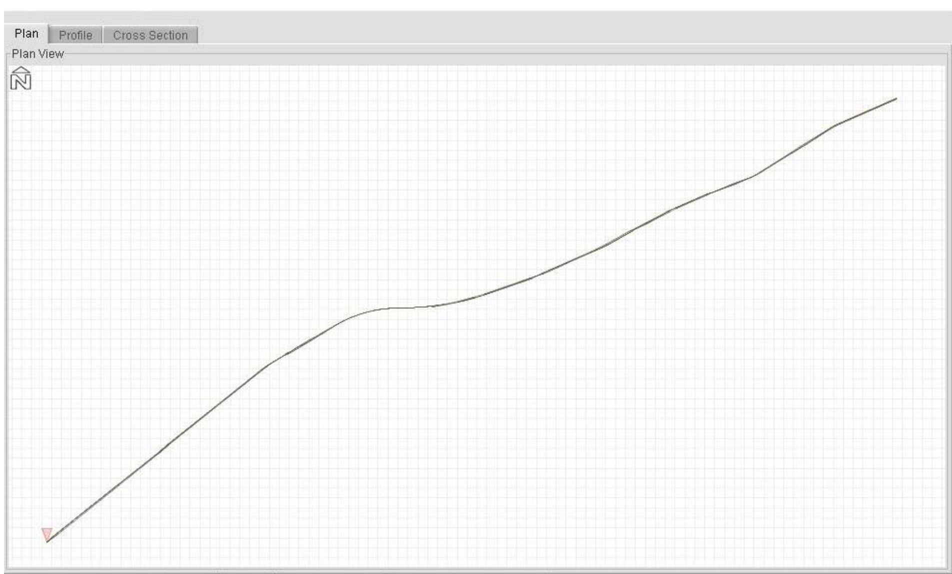
MODEL CODING AND CALIBRATION PROCESS

While the headway data collected for this project will be used to determine the impacts of passing lanes beyond their physical limits, traffic modeling will be used to analyze passing lane impacts under a variety of roadway volume and geometric conditions. Headway data collected at data collection stations along both SH 121 and US-183 will be compared with data for those same locations along modeled versions of both roadways. Using the IHSDM's TAM module, roadway alignment horizontal and vertical profile data (along with cross-sections, volumes, classification data, speed limits, etc.) are coded into the model to create a three-dimensional representation of the real-world SH 121 and US-183 corridors (SH 121 example in Figure 30).

Statistical comparison of headway distributions from the field data set and the modeled data set will be used to determine when the model is replicating real-world operations within acceptable thresholds. Calibration adjustments made during the coding and validation process will be carried into the analysis phase, where model experimentation will determine the impacts of passing lanes for roadway operations under a broader range of geometric, volume and passing lane length, and frequency conditions than would be possible if only real-world passing lane sites were studied.



a) Aerial Photo of SH 121 Study Section



b) IHSDM TAM Model of SH 121 Study Section

Figure 30. Aerial Photo and TAM Representations of the SH 121 Study Corridor.

CHAPTER 8
CALIBRATION OF SIMULATION MODEL

INTRODUCTION

Calibrating a traffic operations analysis model, such as the IHSDM's Traffic Analysis Module, is a process that involves adjusting the model's input parameters so that the outputs are representative of real-world operating conditions. In reality, a two-step process occurs. First, the model's input values and internal attributes are adjusted to match the values observed in the real world for the same phenomena. Next, the model is validated by comparing its behavior—in the form of output measures of system performance—against real-world measures of those same system measures.

Project analysts conducted calibration and validation exercises on Super 2 roadway sections along both SH 121 in the TxDOT Paris District and US-183 in the TxDOT Yoakum District. In each case, input parameters included traffic composition and headway data observed and recorded at each site. After these input values were entered within TAM files for each roadway, analysts validated, or compared, the model output with field-collected headway data within each passing lane section.

MODEL DEVELOPMENT

Data entry into the TAM component of IHSDM requires tabulated data entry of all roadway geometric details found along the real-world roadway segment being analyzed for passing lane alternatives. These data include:

- Roadway classification.
- Design and operating speeds.
- Daily and peak hour volume data.
- Beginning and ending roadway stations of the entire study segment.
- Beginning and ending station, radius, and curve direction for horizontal curves.
- Beginning and ending station and grades for vertical curves.
- Type and dimensions of shoulders.
- Type and dimensions of ditches (drainage structures).
- Pavement and shoulder surface/material.
- Lane striping details.

- Turn lane location and extent.
- Passing lane location, extent, and lane striping.
- Speed zone location and extent.
- Passing sight distance along the roadway (can be internally computed by the TAM).

Figure 31 through Figure 34 provide examples of data entry and review screens within the roadway geometric data entry component of the TAM. The majority of coding errors in horizontal geometry, vertical geometry, or cross section detail are readily identified in review screens and corrected in the data entry tables before a "run" of the program is performed. The analyst enters traffic volume and classification data for the study roadway section each time an analysis run is initiated. An example data entry screen for this stage of the data entry process is provided as Figure 35. The final step of data entry before the TAM will perform a simulation run is the specification of input speeds for each class of vehicle. Figure 36 provides a sample screen for this process.

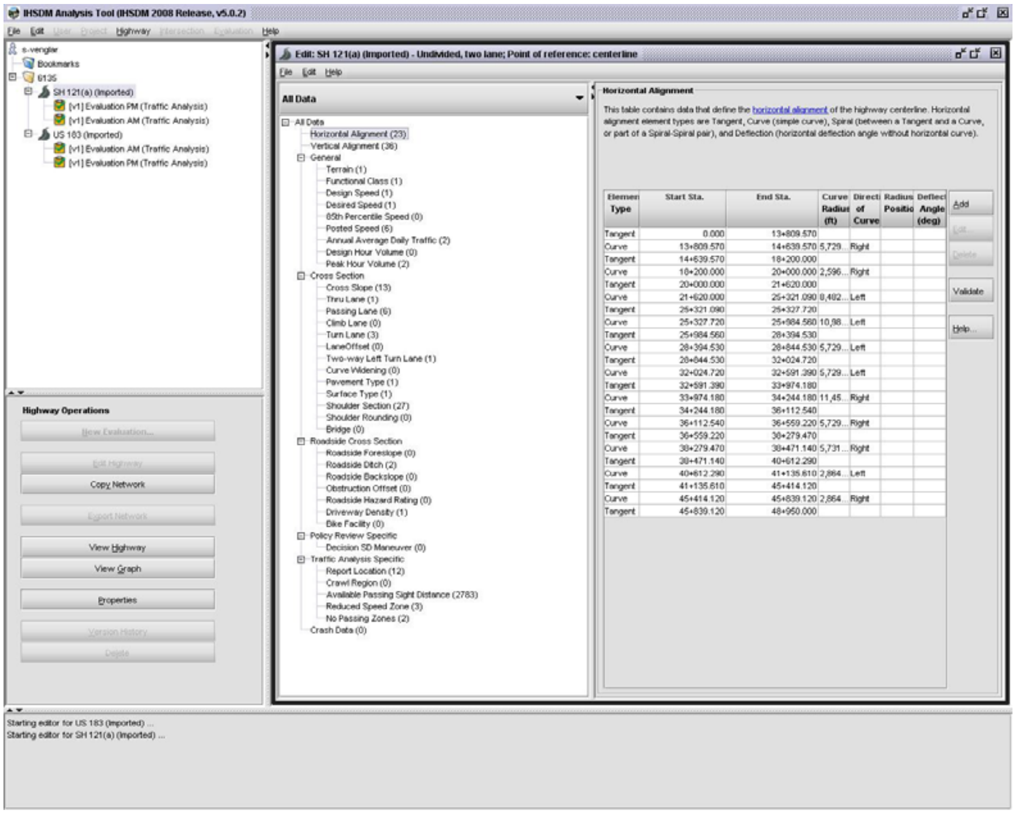


Figure 31. IHSDM TAM Data Entry – Horizontal Alignment.

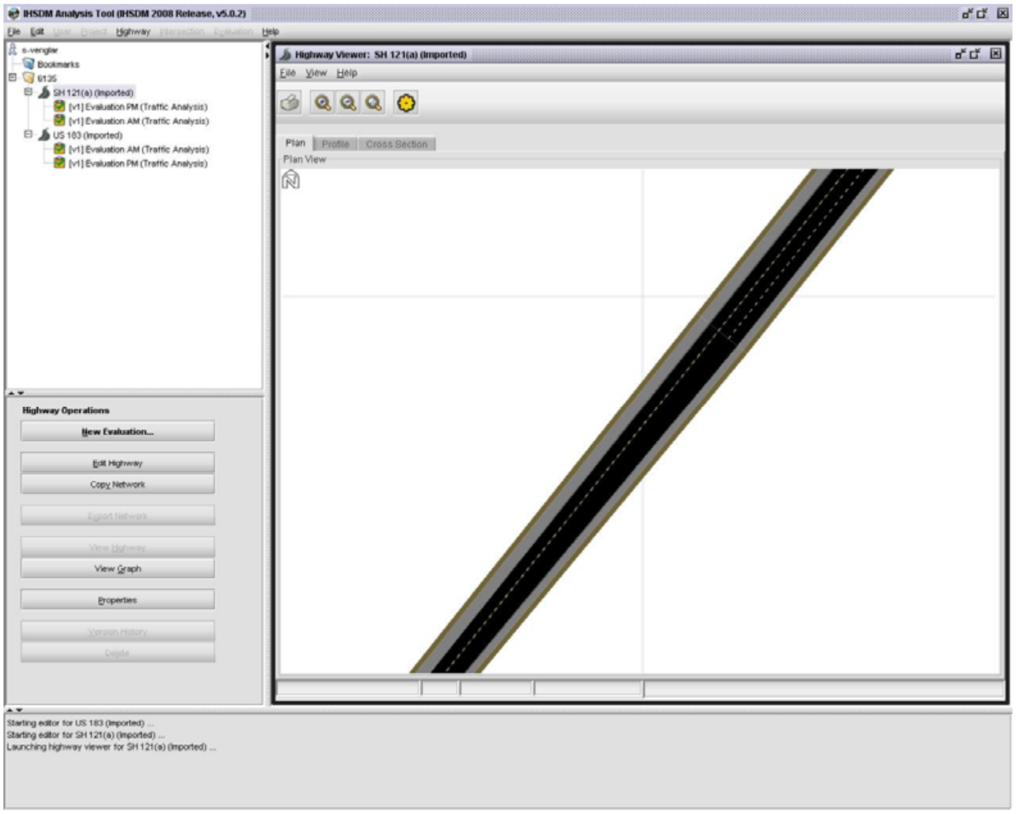


Figure 32. IHSDM TAM Data Entry – Roadway Data.

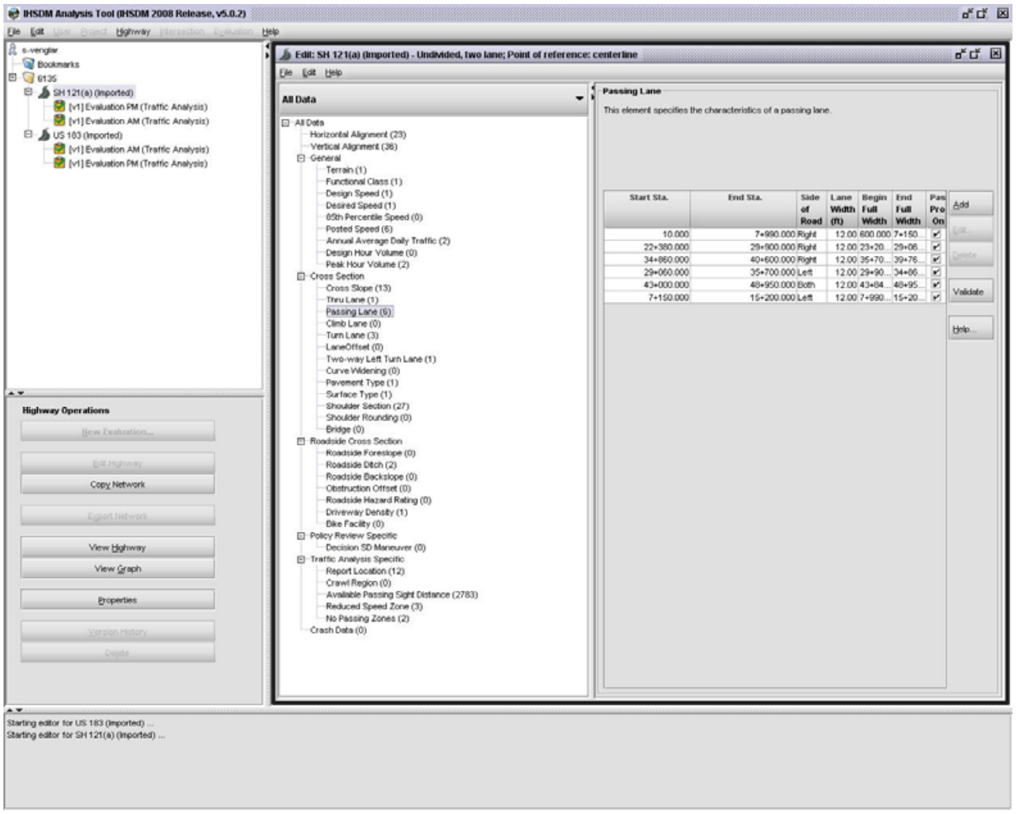


Figure 33. IHSDM TAM Data Entry – Passing Lane Details.

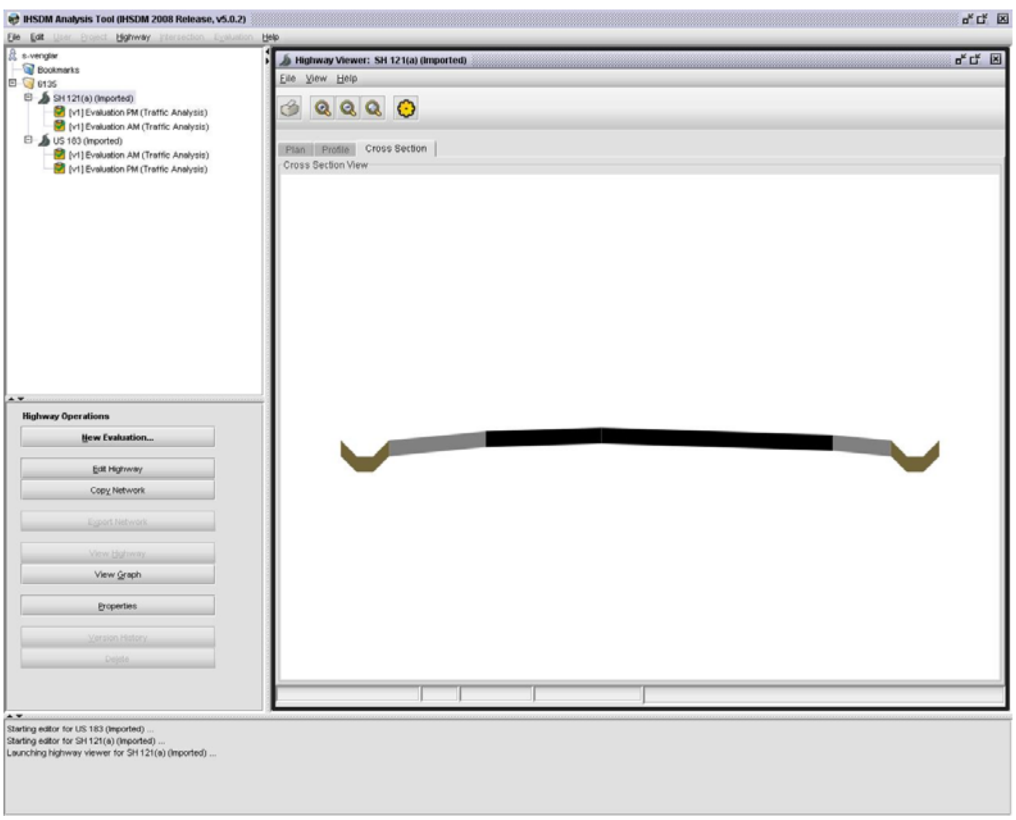


Figure 34. IHSDM TAM Data Entry – Cross Section Details.

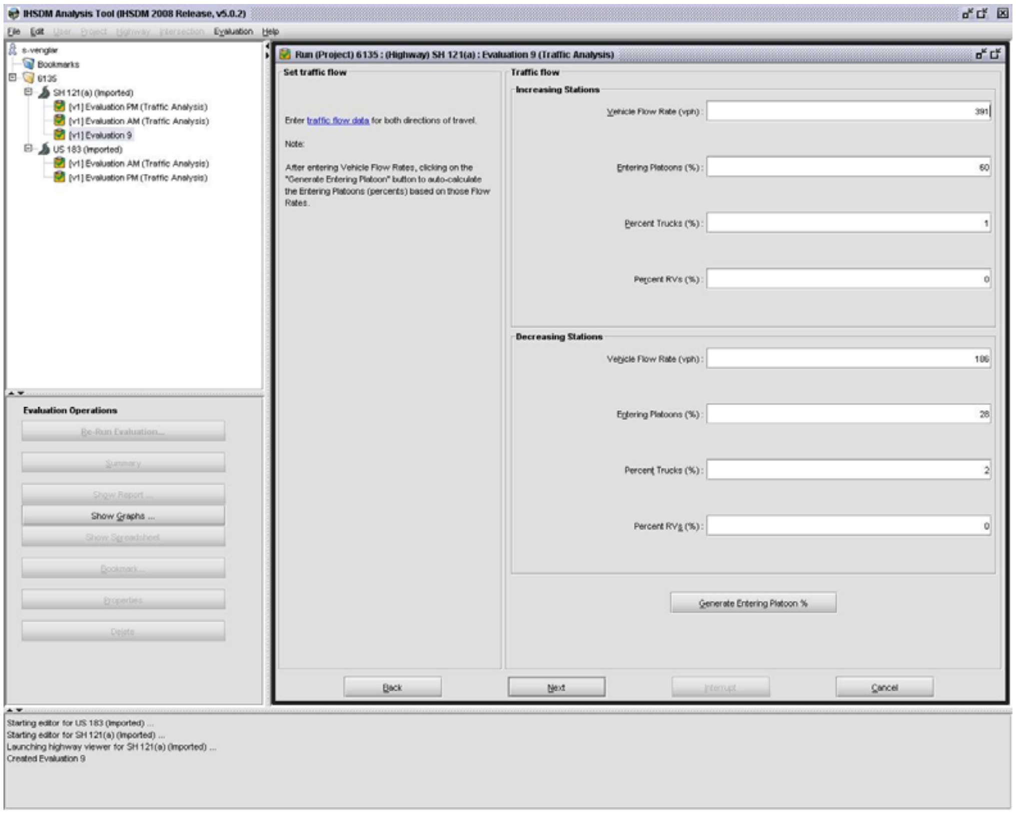


Figure 35. IHSDM TAM Data Entry – Volume and Classification Data.

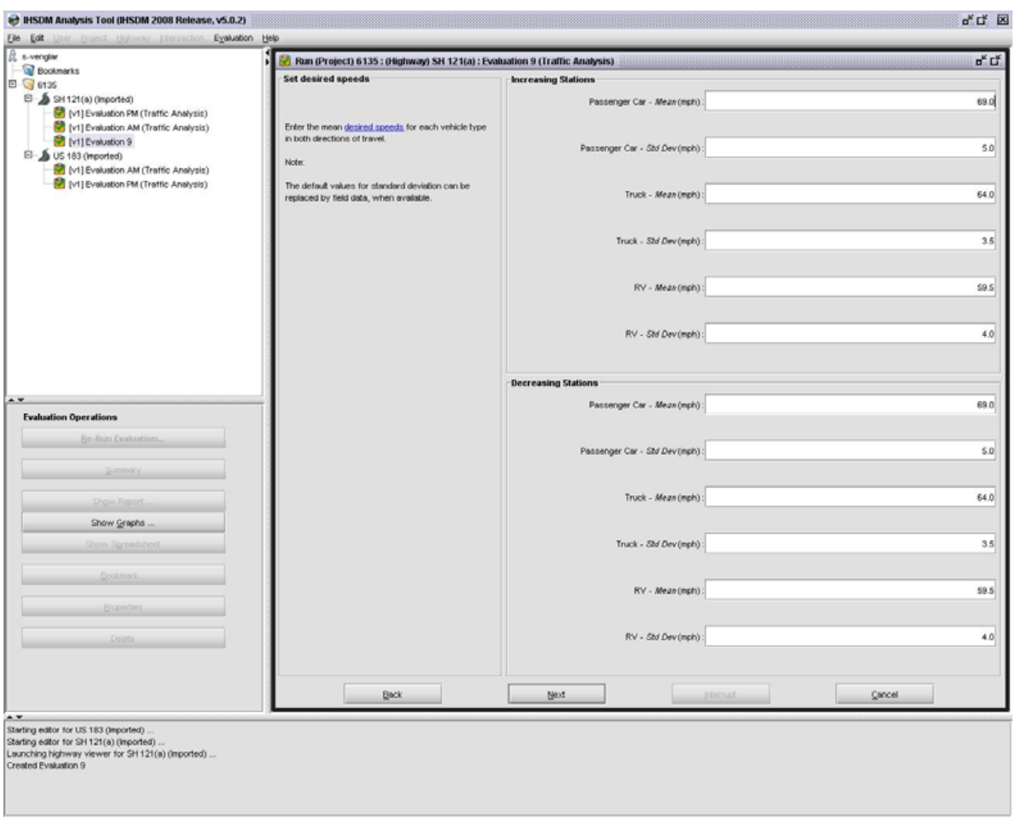


Figure 36. IHSDM TAM Data Entry – Vehicle Speed Data by Class.

CALIBRATION SETTINGS

Locations where the analyst can make calibration adjustments to the TAM are found at two different locations within the IHSDM. On the more global level, changes can be made to configuration data sets used by the different analysis tools within the IHSDM, including the TAM. An "IHSDM Administration Tool" is provided with the IHSDM software, and this tool is used to make configuration/calibration changes at the global level. Specifically for the TAM, fundamental changes can be made to general settings, driver factors, and each of three vehicle type categories (Truck, RV, and Passenger Car). Sample TAM configuration screenshots from the Administrative Tool are provided in Figure 37 and Figure 38. Aspects of the TAM that can be altered under the General tab (Figure 37) include such factors as passing sight distances, speed limit ranges, and object heights. Vehicle type changes (Figure 38) can be made to weight/horsepower ratios, vehicle length, representation of the vehicle type in each fleet category, and other factors.

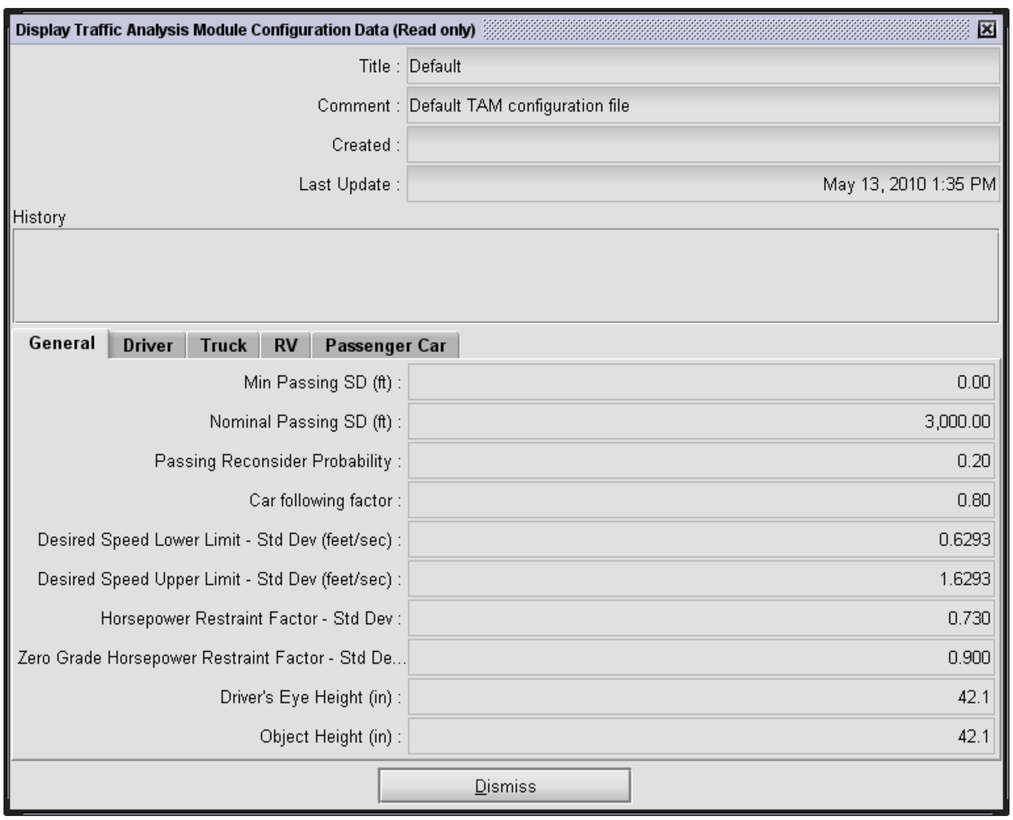


Figure 37. IHSDM Administrative Tool Settings – TAM Module, General.

Calibration settings at the global level are rarely changed due to the fact that the amount of effort required to collect sufficient performance and vehicle behavior data for such calibration is excessive. Realistically, the only time such detailed data sets are assembled is when models are initially being developed, or an entire research or model development project is conducted with the specific intent of refining the parameters used in the model. The research team concluded that these aspects of model calibration were beyond the scope of the current Super 2 research.

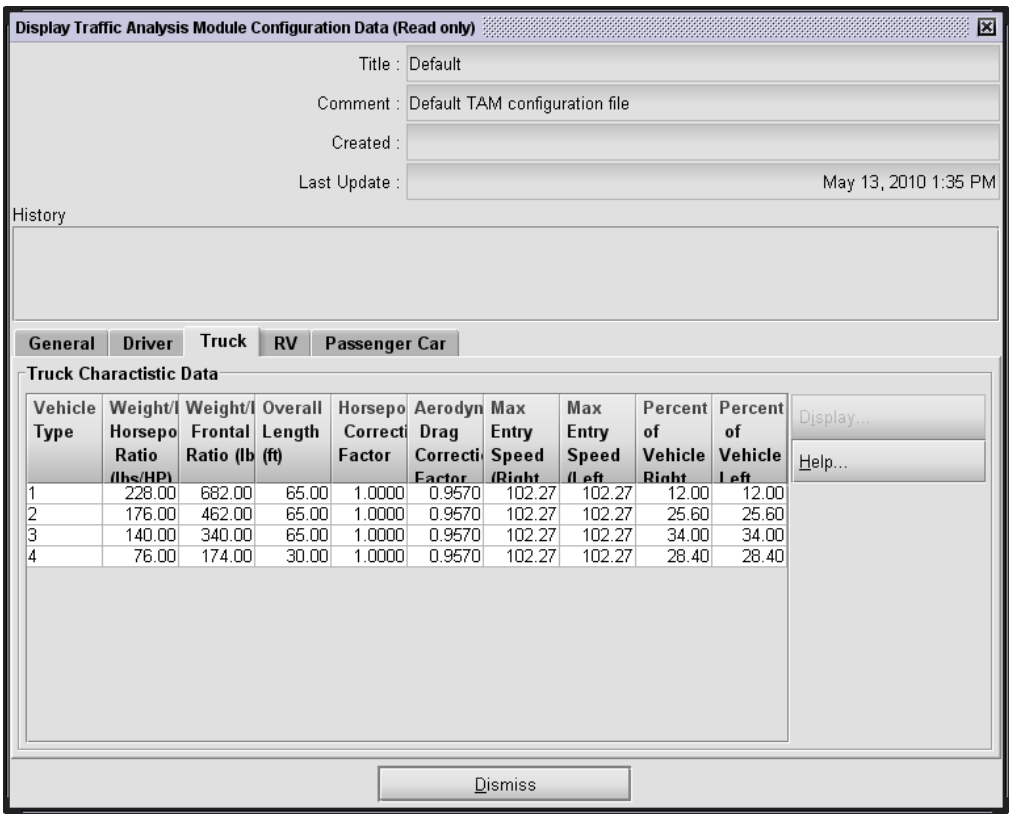


Figure 38. IHSDM Administrative Tool Settings – TAM Module, Trucks.

The calibration data used by the TAM that could be realistically gathered and utilized by the research team included the speed, headway, and classification data collected using portable traffic counters/classifiers. These data are entered into the TAM using the traffic volume entry screens activated before an analysis run is executed by the TAM. Figure 35 illustrates that in addition to vehicle volume/flow rate and classification data, there is also a data entry field for the entering platoon percentage. The IHSDM documentation indicates that "platooned" vehicles are those within 4 seconds of a leading vehicle. This entry is automatically calculated by the TAM based on flow rate data, but the default input can be over-written with field data, if available. For both the SH 121 and US-183 study sites, such data were available and entered for each analysis run. Results observation from trial runs revealed that entering the field data platoon percentage (rather than the default TAM value) resulted in more accurate headway results not only at the beginning of each roadway segment, but also along the several miles of roadway immediately downstream of the project start.

TAM speed data for each vehicle type (Figure 36) are automatically entered for each analysis run based on default averages and standard deviations stored within IHSDM. As with the platoon percentage, these data were also available from the field data collection studies for SH 121 and US-183. Trial runs of the SH 121 network using default and field-based speeds and standard deviations revealed minor differences in the results between the cases. Due to the uncertainty associated with the accuracy of the field-collected speed data and the very high standard deviations found for these data (see discussion on Passing Lane Speed and Headway Data in Technical Memorandum #3), researchers decided to use the default rather than field data for average speed and standard deviation.

RESULTS

Calibration results for SH 121 northbound are presented graphically in Figure 39 and Figure 40 for AM and PM peak hour traffic conditions, respectively. In each case, the percentage of vehicles reported by the IHSDM TAM as following vehicles (i.e., those at a headway equal to or less than 4 seconds) is bracketed by the results for field data at each data collection site and reported in each graph as percentages of vehicles in the traffic stream following at a headway of less than or equal to 3 seconds and less than or equal to 5 seconds. For SH 121 northbound, note that field data collection devices were deployed before the passing lane section began (Station "0"), in the two-lane portion of the roadway where the passing lane was present and at the end of the passing lane section where the roadway had returned to a single northbound lane.

Headway results presented in Figure 39 and Figure 40 are both internally and externally consistent. Within each figure, the following percentage is observed to be at its highest before the passing lane begins and then directly and significantly decreases starting with the passing lane section and continuing the length of the passing lane section. An increase in vehicle following is observed at the passing lane terminus; however, despite this increase the percentage of following vehicles at the end of the passing lane section is not as high as it was before the passing lane began. There exists an after-influence of the passing lane beyond its physical limit, and both the influence itself and its extent are consistent between the field data and the model results. The results are externally consistent in that a lower initial following percentage is observed for AM peak conditions (Figure 39) when compared with PM peak conditions (Figure 40) for the same segment of roadway. Analysts anticipated this result given that the flow rate in the AM peak was 166 vehicles per hour while the flow rate in the PM peak was substantially higher at 391 vehicles per hour.

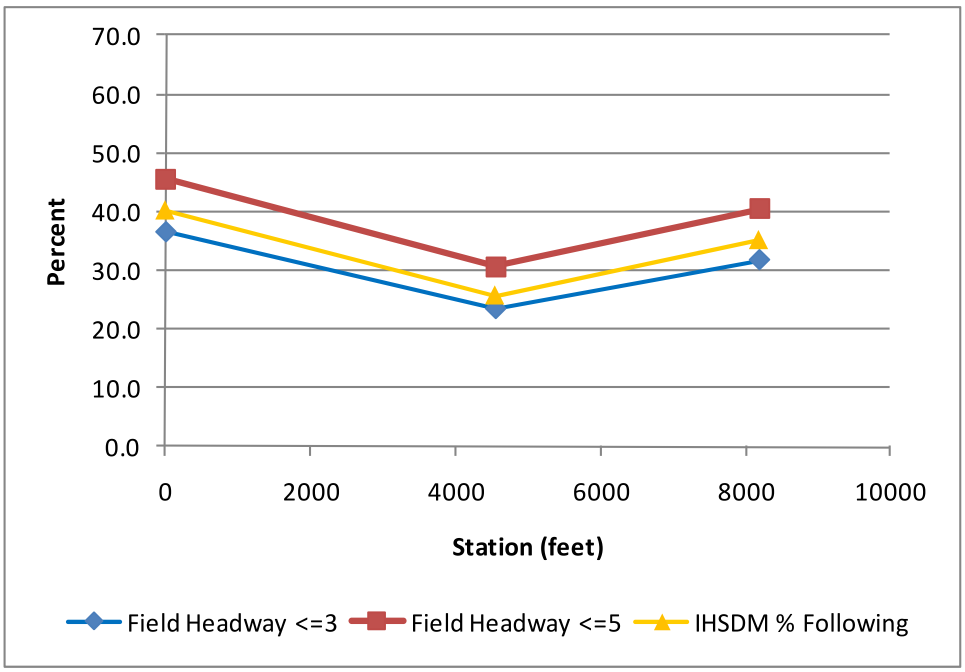


Figure 39. SH 121 Northbound, AM Peak.

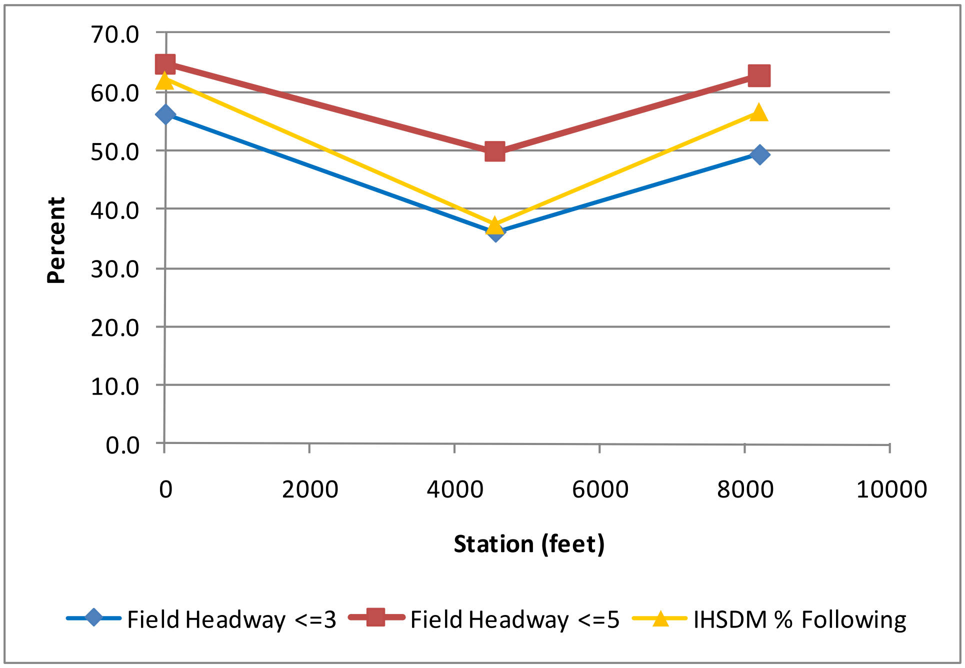


Figure 40. SH 121 Northbound, PM Peak.

Calibration results for SH 121 southbound can be found in Figure 41 and Figure 42. Since more traffic data collection devices were available for the field study of southbound SH 121 than northbound SH 121, field data and TAM results are shown not only within and proximate to the passing lane section, but also farther downstream from the passing lane section to the point where the next passing lane section begins. Note that station numbers for results appear backwards in these figures, as the station values decrease from left to right. This reporting detail arises from the fact that northbound SH 121 was considered the direction of increasing stations for data entry into the TAM model. When SH 121 southbound was modeled, it was approached from the direction of decreasing stations, resulting in the start of the passing lane section being reported at a higher station number than the end of the passing lane section.

Similar to the SH 121 northbound results, the southbound results are both internally and externally consistent. Passing lanes during both AM and PM peaks cause a reduction in following percentage, and at locations farther and farther downstream from the passing lane section the following percentage increases until the next (downstream) passing lane section is reached. AM peak conditions, where the flow rate was 272 vehicles per hour, are associated with a higher initial following percentage that PM peak conditions, where the flow rate was 202 vehicles per hour.

While the model results for SH 121 northbound were well bracketed by field data, the TAM results for SH 121 appear to slightly over-report the percentage of vehicles considered to be following a leading vehicle. These differences are likely due to the presence (in the southbound direction) of a passing lane only 1.6 miles upstream of the passing lane section selected for field data collection, causing the field data (rather than the model) to be the source of the results discrepancy. The presence of the upstream passing lane section undoubtedly had an impact of reducing the following percentage approaching the studied passing lane, creating the differences between field data and model results found in Figure 41 and Figure 42.

Despite the constraints imposed by real-world conditions, including passing lanes in close proximity to one another and intersections or interchanges found between passing lane sections, the TAM modeling results for southbound SH 121 remain within a reasonable bound of real-world field data. Further, discrepancies between field data and model results were readily explained by real-world field phenomena and influences and were not found to be unexpected results from the analysis model.

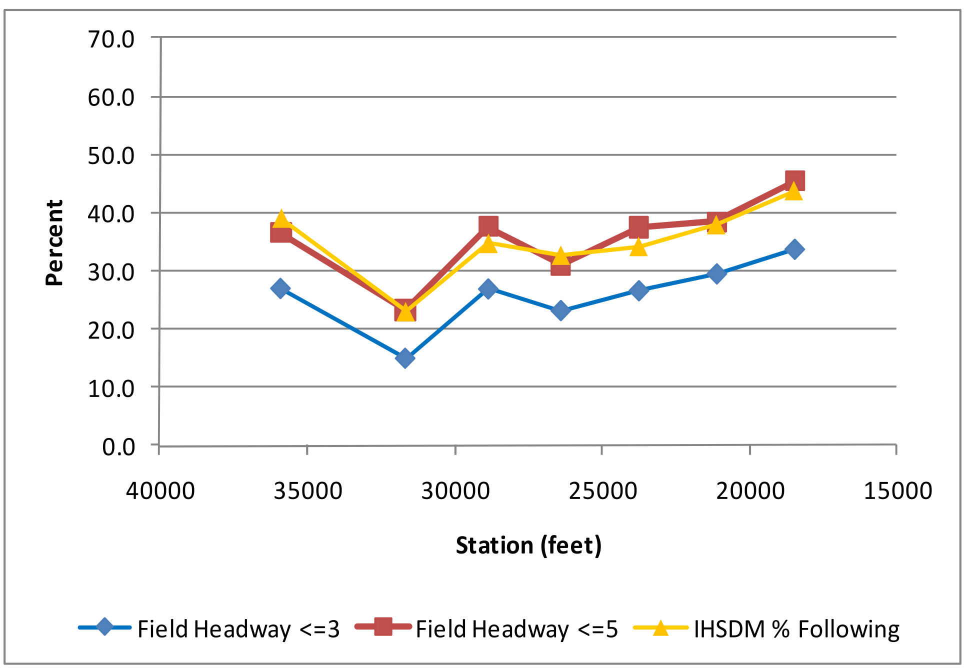


Figure 41. SH 121 Southbound, AM Peak.

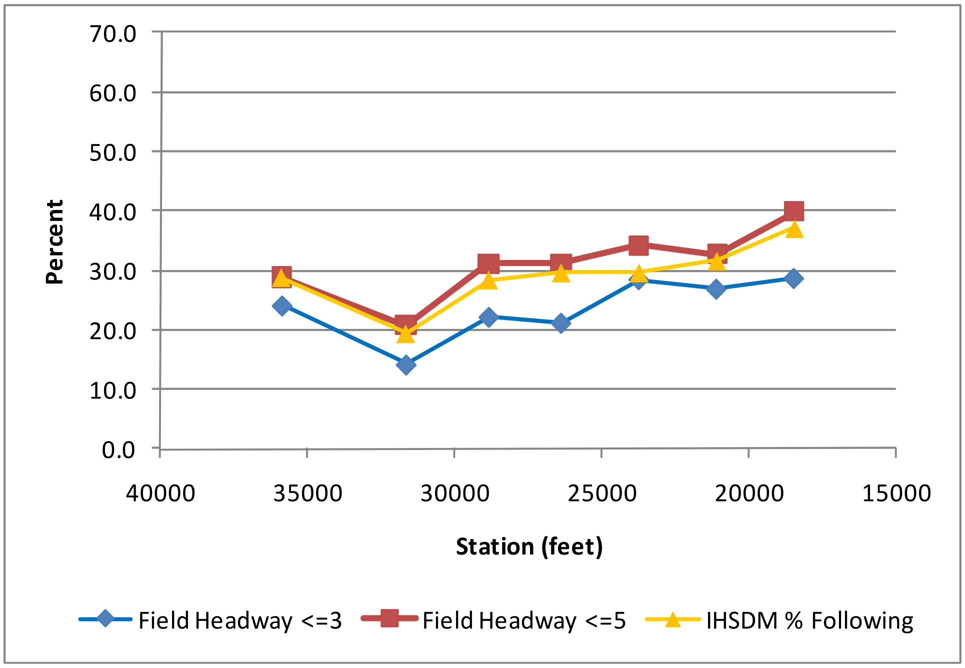


Figure 42. SH 121 Southbound, PM Peak.

While very positive calibration results were obtained from the modeling effort along SH 121, researchers did not have the same success using the TAM to analyze the US-183 study sites and field network. Results for southbound US-183 (the direction of increasing stations) can be found in Figure 43 (AM Peak) and Figure 44 (PM Peak), while results for northbound US-183 are located in Figure 45 and Figure 46.

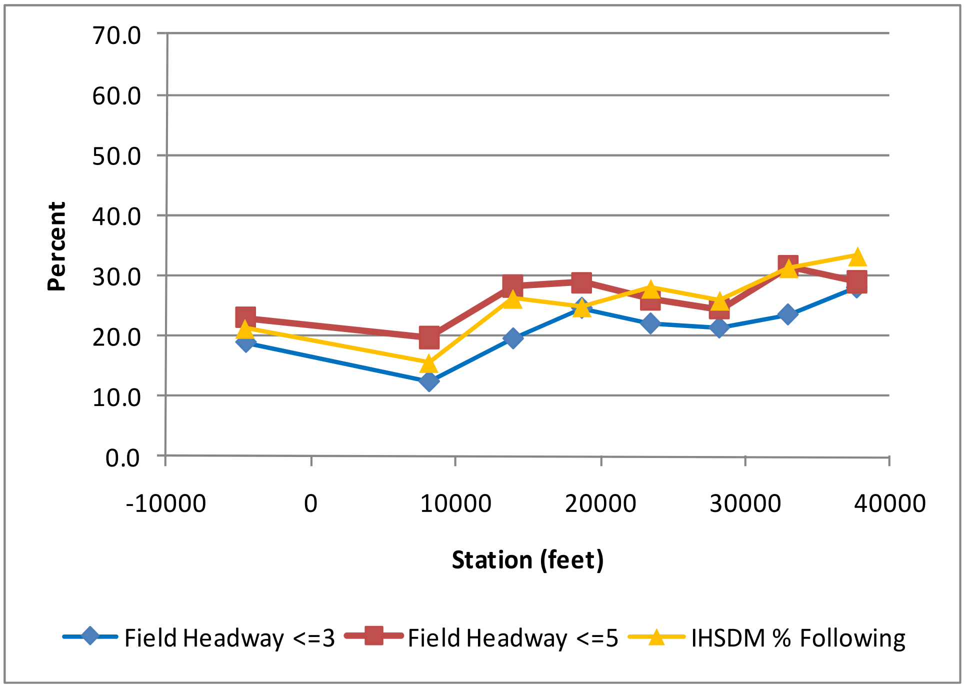


Figure 43. US-183 Southbound, AM Peak.

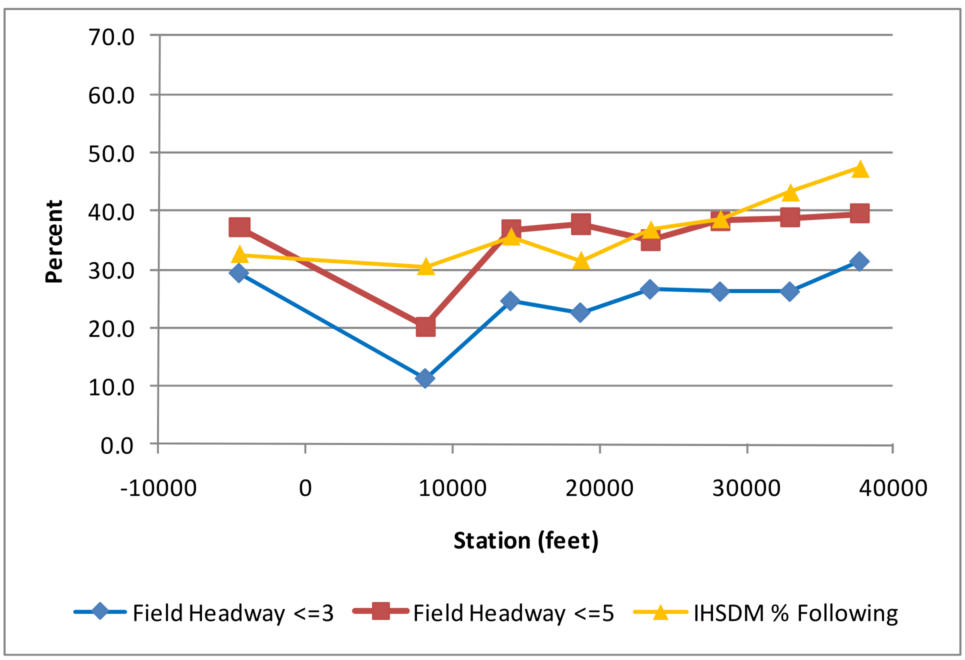


Figure 44. US-183 Southbound, PM Peak.

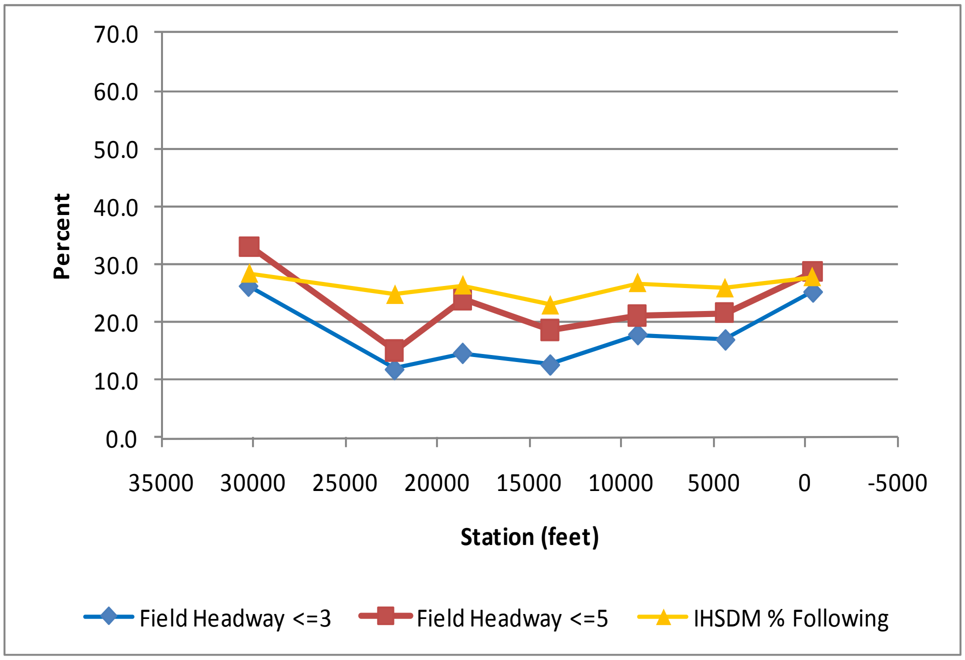


Figure 45. US-183 Northbound, AM Peak.

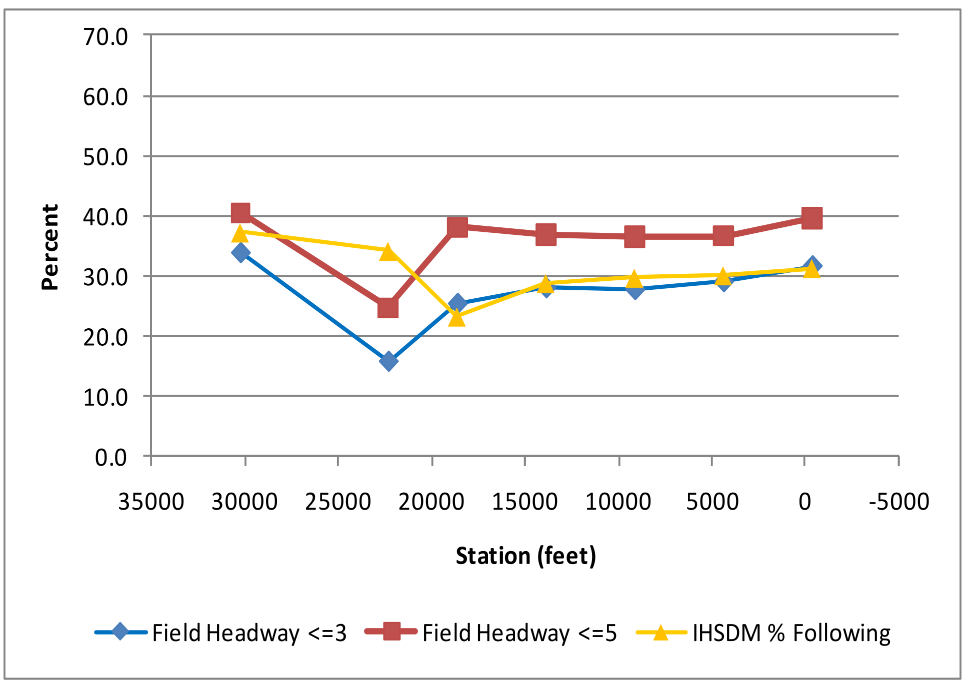


Figure 46. US-183 Northbound, PM Peak.

Review of the US-183 TAM results reveals discontinuities in the following percentage that could not be explained by features coded into the model or roadway elements in the real world that were not accounted for in model coding or were beyond the physical limits of the study section. Comparing the four figures of US-183 results, it is also apparent that the discrepancies between the real-world following percentages and the results of the model are more pronounced as traffic volumes increase; the results for the lowest volume US-183 case, shown in Figure 43, are close to the field-based boundaries for following percentage but vary significantly from the findings for the other three US-183 analysis models, which have higher peak-hour traffic volumes.

In an attempt to determine the cause of the inconsistencies in the US-183 results, all of the input parameters were rechecked and a continuity check was performed by comparing input and output traffic volumes. Analysts discovered a traffic volume inconsistency within the TAM that occurred at or around station 18,000 for which there was no explanation in either the model or in the field data entered into the input file. The volume inconsistency appeared in the output file and is presented as Figure 47, where traffic flow in vehicles per hour is reported in the fourth of five rows of results.

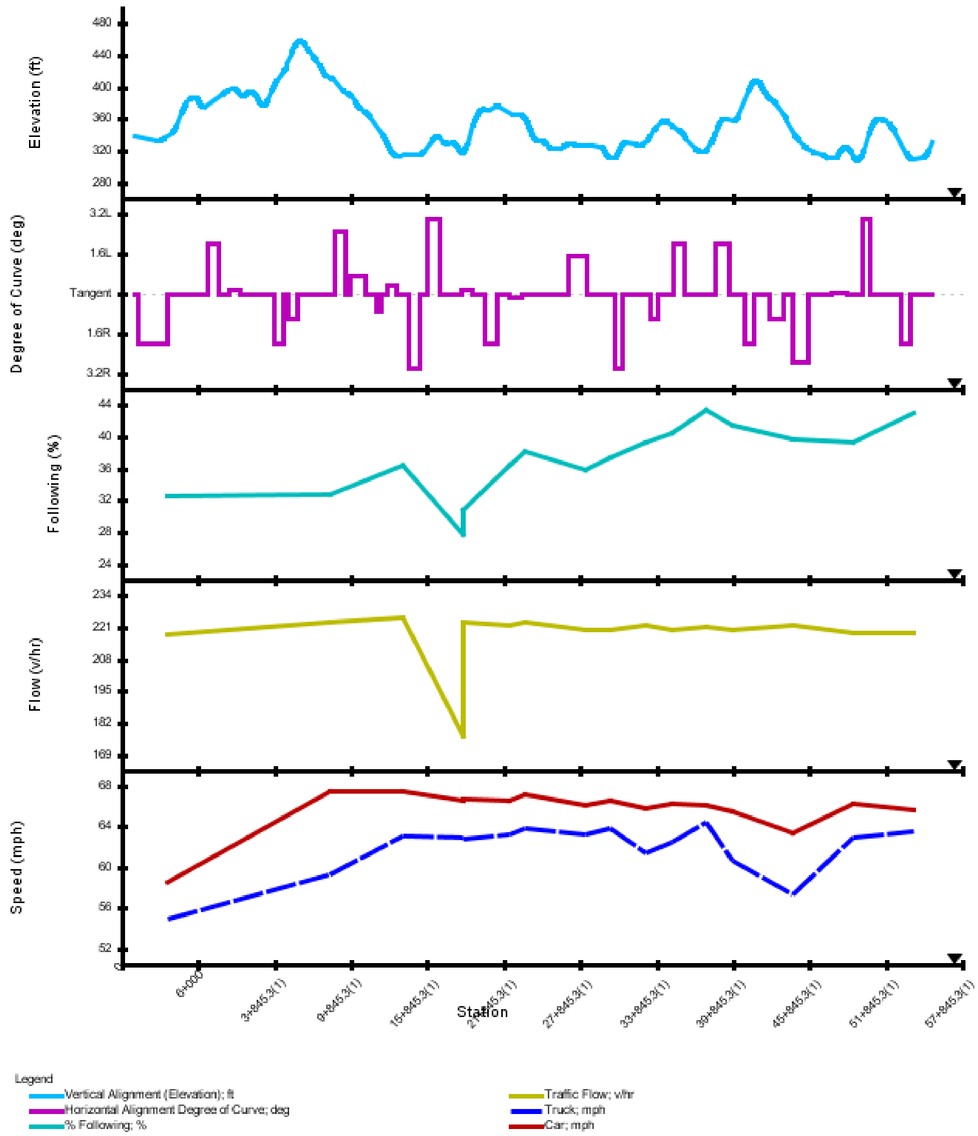


Figure 47. TAM Output for US-183 Northbound, PM Peak.

Unable to determine the cause of the volume inconsistency within the TAM user interface, the research team contacted the developer of the IHSDM in an attempt to get clarification on the cause of the problem and how it could be resolved. While the software developer was able to verify that there was a problem with the file as it was being read within the TAM module of the IHSDM, no resolution was forthcoming as to how to remove the problem or avoid it in future analysis scenarios. The model developer committed to identifying the source of the internal software problem, but was not able to do so in a time frame consistent with reporting the results of the current research.

In the absence of further clarification from the model developer, researchers relied on an internal problem assessment approach and identified any inconsistencies in analysis approach or input data between the SH 121 and US-183 models. Researchers identified the only major coding difference as a two-regime station numbering sequence that was present in the US-183 model but not in the SH 121 model. This approach was necessitated by the fact that the US-183 project crossed a county boundary at which roadway station numbering was reset to zero. By avoiding a two-regime station numbering approach in later analysis and modeling phases, researchers accurately predicted they could avoid the software internal problem that produced errata in the US-183 results, demonstrating that the IHSDM TAM is an appropriate modeling tool for future analyses of passing lanes on other Super 2 corridors.

CHAPTER 9
SIMULATION MODELING AND RESULTS

BACKGROUND

Microsimulation is a popular and effective tool in both quantifying and illustrating transportation problems and evaluating possible solutions to these problems. The more complex the situation is and the more detailed the results desired, the greater the advantage microsimulation can have compared to theoretical methods. After the model is calibrated, a simulation test bed is created and used to evaluate the simulation scenarios, defined as all traffic conditions and geometric scenarios that are of interest in developing relationships between traffic volumes, passing lane characteristics, and other key variables. The goal of the simulation modeling process in this task was to identify if and/or when a certain type of passing lane application may be more beneficial for operations.

As described in Chapter 8, analysts used detailed roadway data from the coded US-183 models (e.g., horizontal and vertical alignment, cross-section, etc.) to create the base model for evaluation through the use of the Traffic Analysis Module of the FHWA IHSDM package. Researchers then created a wide variety of models by making adjustments to the alignment in that base model to cover a range of conditions and scenarios. Adapting roadway details from the calibrated model instead of creating an artificial roadway for the simulation makes the representation as realistic as possible and improves the likelihood that the simulation results will mirror those shown on a real-world roadway. After creating the models for the various scenarios, researchers conducted multiple runs of the simulation for each scenario, providing a broader basis for producing results for analysis.

EXPERIMENTAL DESIGN

The TAM model simulates traffic operations on two-lane highways by inputting the position, speed, and acceleration of each individual vehicle along the roadway and moving those vehicles along the simulated highway in a realistic manner. The model also takes into account details such as driver preferences, vehicle size, performance characteristics, and the impact of the oncoming and same-direction vehicles that are in sight of a given vehicle at any given time. The model incorporates realistic passing and no-passing decisions along the roadway.

In order to analyze the impact of varying geometric and traffic conditions, as well as the operational improvements provided by different configurations of Super 2 passing lanes (i.e., length and spacing), it was necessary to evaluate each possible scenario in the TAM. To properly set up the simulation and achieve realistic results, a number of parameters and variables must be input or defined prior to commencing each simulation run.

General Conditions

Highway Geometry

Typically, building the roadway models in the TAM is conducted through direct inputs in the model editor. However, in this task, detailed roadway data from the previously coded US-183 models were used to create the base model for evaluation. Parameters such as grades (vertical alignment), radius, superelevation, degrees of curvature (horizontal alignment), lane width and shoulder width, and passing and climbing lane geometrics were kept as the input.

Vehicle and Driver Characteristics

The vehicle characteristics input into the models include such variables as maximum acceleration rate, maximum speed capabilities, and vehicle lengths of different vehicle types. The driver behavior parameters include driver characteristics and preferences, such as a car following factor, driver's eye height, horsepower restraint factor, and acceptance/rejection of passing opportunities. These parameters were defined in the IHSDM TAM configuration files. In this study, the IHSDM default values were used.

Speed

Different modules of the IHSDM package, such as Policy Review Module and Intersection Review Module, use different speed variables, including design speed and 85th percentile speed. Three speed variables were actually used in the TAM simulation modeling. Those variables were posted speed, reduced speed areas, and input traffic volume speed; each is defined as follows:

- Posted speed – the value posted on speed limit signs to reflect speed limit changes as vehicles travel through the network. The posted speed values are based on those obtained from the US-183 field data.

- Reduced speed area – a defined area for the purpose of reducing vehicle speeds approaching turns and horizontal and vertical curves. The reduced speed areas are placed in the model consistent with the US-183 field data.
- Input volume speeds – speed distributions for vehicles entering the network. Free-flow speed for passenger vehicles entering the network was input with a mean of 70 mph with normal distribution and a 5-mph standard deviation; trucks were assigned a 65-mph mean desired speed and 4-mph standard deviation.

Variables Related to Passing Maneuvers

Sight distance is the length of roadway that the driver of the passing vehicle must be able to see initially, in order to make a passing maneuver safely. In the IHSDM package, passing sight distance regions are generated automatically by calculating the available sight distance based on road geometry and obstruction offsets input by the user.

Passing and no-passing zones on a conventional two-lane highway are defined by user input data. No-passing zones can either be established automatically on sections with inadequate sight distance or they can be specified manually. In the simulation, drivers do not start passes in no-passing zones, and a passing maneuver will be aborted if the TAM projects that the pass will extend beyond the end of the passing zone.

Key Study Variables

Average Daily Traffic

Previous research (TxDOT Project 0-4064) demonstrated that periodic passing lanes can improve operations on two-lane highways with low to moderate volumes. The current Texas *Roadway Design Manual* contains these guidelines for highways with ADT lower than 5000. The simulation modeling in this task expanded to include higher volumes ranging up to about 15,000 ADT, which approaches the limits of capacity for a typical two-lane highway. Six volume classes were created at 3000, 5333, 7667, 10,000, 12,333, and 14,667 ADT.

The TAM simulation is conducted based on the peak hour flow rate, which is converted into ADT by using a k-factor of 10 percent and a 50 percent directional distribution. Table 38 indicates the flow rates used in the simulation.

Table 38. Classes of Input Traffic Volume.

ADT	3000	5333	7667	10,000	12,333	14,667
Directional Input Flow Rate (vph)	150	267	383	500	617	733

Terrain

Two types of terrain were considered in the evaluation. The original study section of US-183, located in an area of rolling hills, was adapted directly. A level terrain was created by setting the grades to zero, so the whole study corridor remained flat for that scenario.

Proportion of Heavy Vehicles

Heavy vehicles often have an impact on operational performance in a corridor, often manifested in a reduction in operating speeds. High proportions of heavy vehicles also contribute to a decrease in safety as impatient drivers attempt to pass slower vehicles in no-passing zones or pass trucks despite having diminished sight distance beyond such vehicles. To examine the effects of such vehicles, two heavy vehicle proportions, 10 and 20 percent, were used in the simulation scenarios.

Passing Lane Length

Passing lane length is one of two critical elements in the design of the passing lane sections on Super 2 highways. The lanes must have sufficient length to allow drivers to complete the passing maneuver, but lanes longer than a certain length may not be fully utilized. Based on the previous research, the minimum length evaluated was 1 mile (passing lanes in the previous project were 1 mile in length with 0.1-mile transitions on either end and equally distributed throughout the 10-mile corridor). In addition to that base scenario, passing lanes were analyzed with lengths of 2.0 and 3.0 miles.

Passing Lane Spacing

Passing spacing is the second critical design element. Passing lanes must be properly spaced to optimize the provision for adequate passing opportunities. Three scenarios were created for this evaluation: no passing lanes (i.e., original two-lane, two-way highway), three passing lanes (two in the direction of increasing station and one in the decreasing direction), and six passing lanes (three passing lanes in each direction). In each scenario, the passing lanes were equally distributed throughout the 10-mile corridor.

In the scenario with the longest passing lane length (3 miles) and six passing lanes (three in each direction) in the 10-mile corridor, the resultant model creates a near-equivalent to a four-lane highway through the entire 10 miles.

Simulations

Before a simulation is run, it is necessary to have a warm-up period, which is a period of a few minutes of simulation before traffic data are collected. It allows time for the model to realistically populate the road with traffic and for traffic to reach a steady state/equilibrium. After the warm-up period has concluded, the test period begins. The test period is the period of time that traffic is simulated and data are collected. The warm-up period and test period in these evaluations were 10 minutes and 60 minutes, respectively.

Multiple simulation runs were required to analyze all combinations of geometric and traffic operational data specified by the simulation scenario matrix. Due to the stochastic nature of some of the input parameters, each scenario was run three times using a different random number seed as the starting point for the variation of some of parameters. The TAM uses five 8-digit random number seeds for generating the traffic streams and other randomly occurring events.

The total number of simulations was 648: 6 ADT values × 2 terrain values × 2 heavy vehicle proportions × 3 passing lane lengths × 3 passing lane spacing values resulted in 216 scenarios, which were each run three times, producing 648 total simulation runs.

Measures of Effectiveness

Researchers extracted the relevant measures of effectiveness (MOEs) from the simulation output files. The TAM model collects and reports operational data accumulated over user specified sections. Some measures of effectiveness (e.g., average speed, percent time spent following, travel time, number of passing maneuvers, average delay) are calculated for the entire study corridor. These measures are collected and reported for each direction of travel and both directions combined. The selected decision-making MOEs are those that have been used in the past for assessing the performance of two-lane roadways, including:

- Percent time spent following – a measure used to estimate the portion of time within the segment that a vehicle would like to pass another vehicle but cannot due to limited sight distance or oncoming vehicles. Percent time spent following is the percent of travel time that vehicles were impeded by other vehicles and not traveling freely according to the TAM logic.
- Average total delay – the algebraic average total delay per vehicle. Total delay consists of two parts: geometric and traffic delays. Average total delay can be viewed as the difference between the measured travel time and the travel time on an ideal (straight and level) roadway alignment with zero traffic impedance.
- Number of passes – the total number of times vehicles overtake other vehicles during the test period. Valid passing maneuvers can occur on the two-lane section (e.g., passing in the opposing lane) or on a passing lane section.

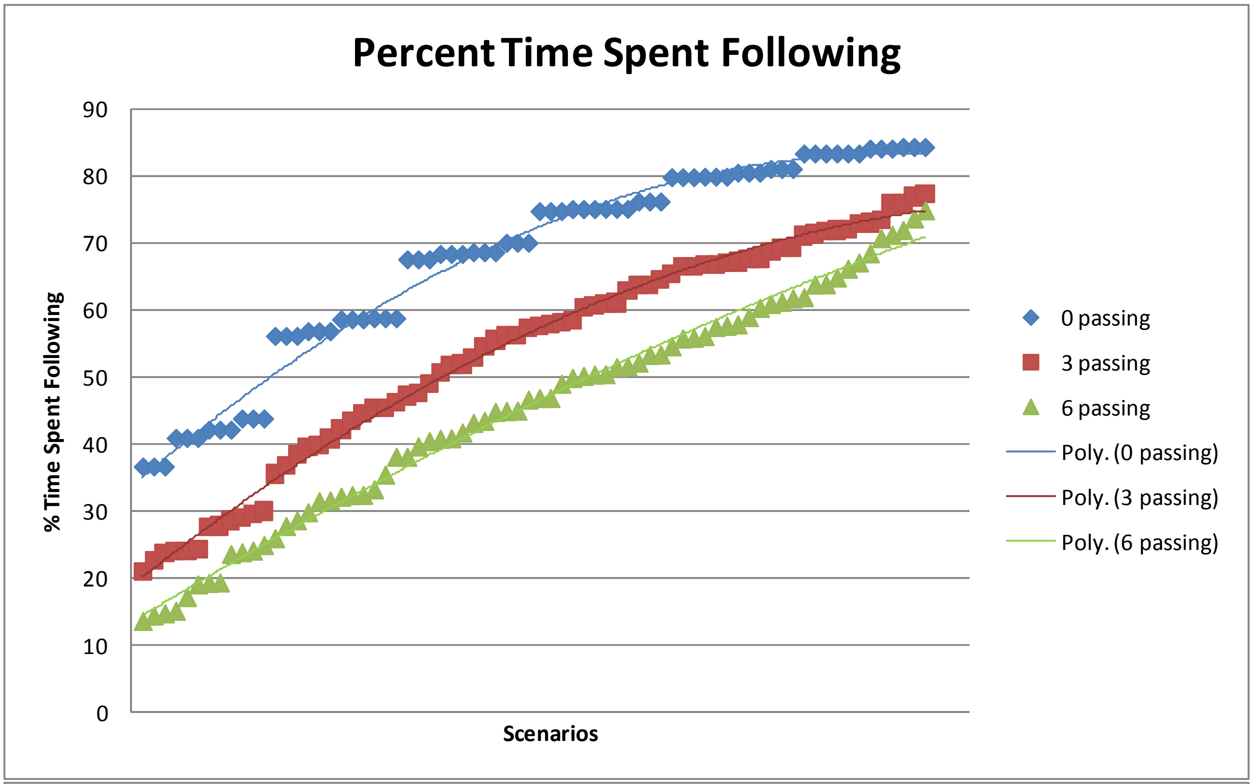
After the simulation files were processed, the model output was evaluated to determine how traffic operations within the Super 2 section were affected by length, spacing, volume, vehicle mix, grade, and other key variables.

RESULTS AND ANALYSIS

Because of the large number of simulation scenarios and the complexity of variable combinations, analysis was conducted in a variety of ways, investigating the impact of each variable on two-lane highways with a Super 2 configuration.

Comparison of Performance Based on Single Variables

A review of Figure 48 reveals the effect on operations by increasing the number of passing lanes. Two critical measures, percent time spent following and average total delay, are shown in the figure; each plotted point represents a measured value from one scenario and they are listed in increasing order of value.



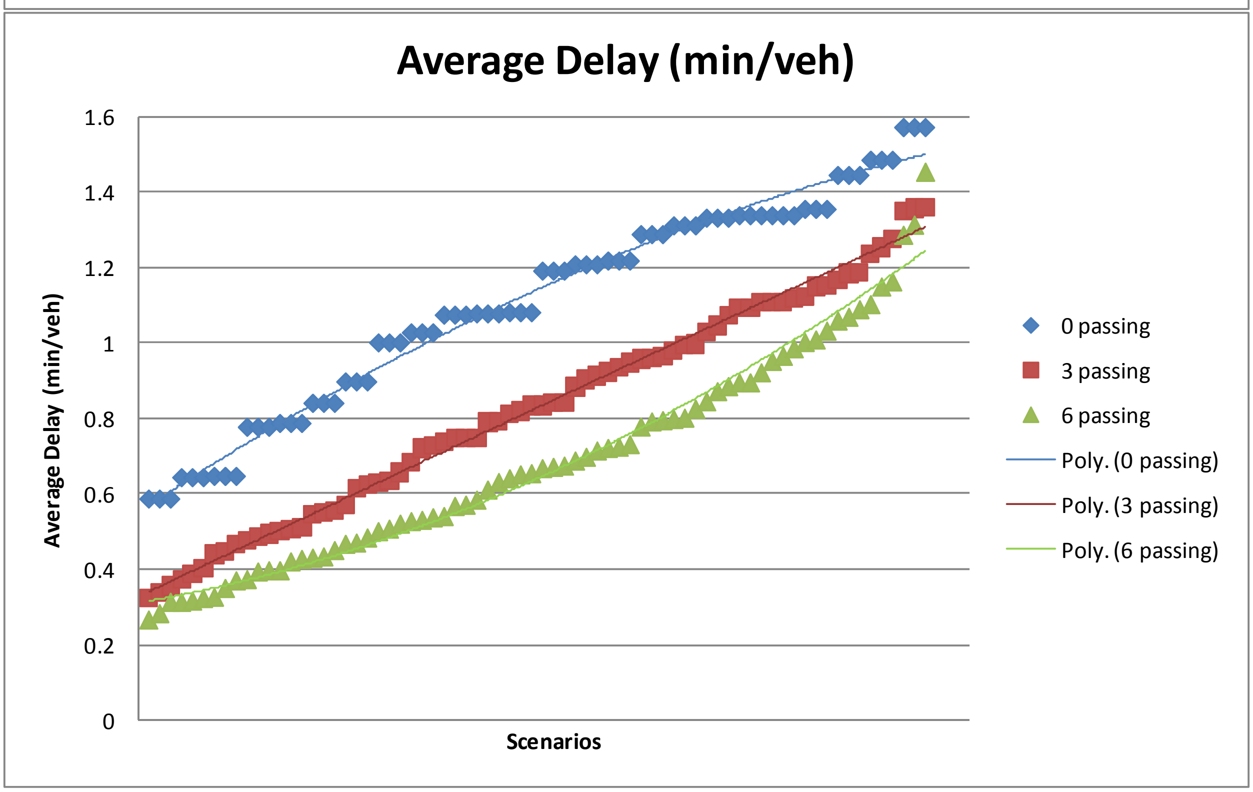
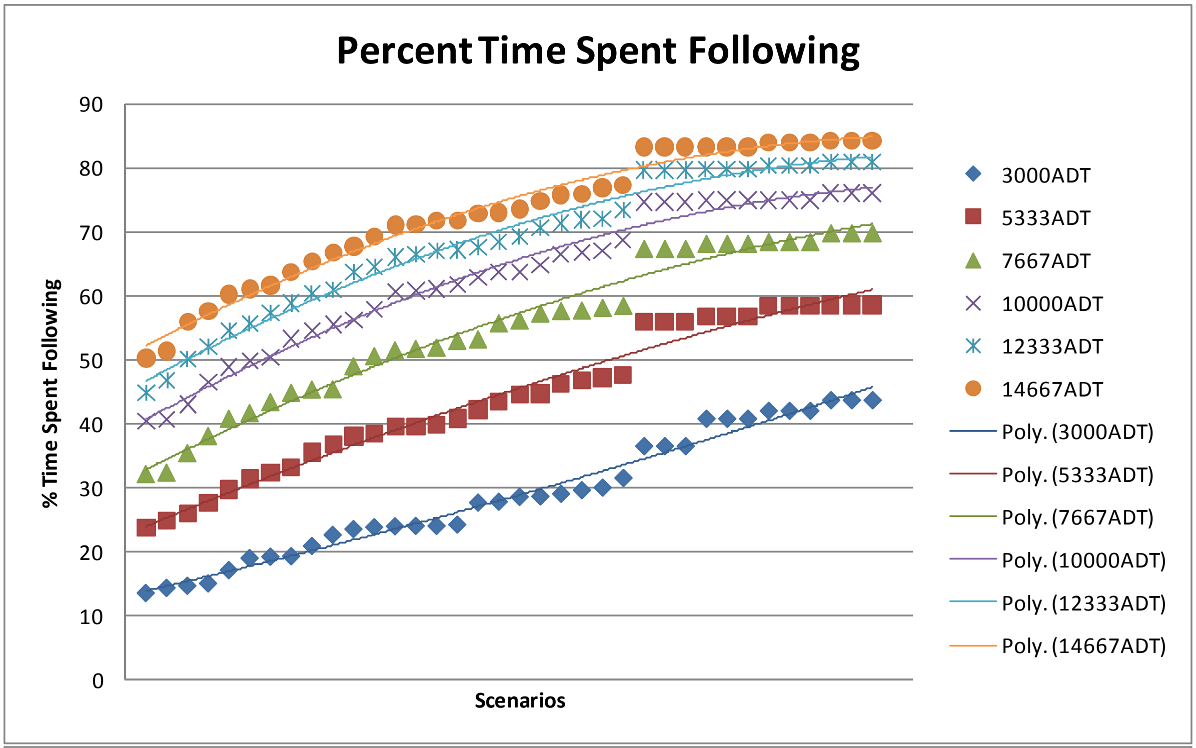


Figure 48. Performance Comparison for Different Passing Lane Spacing.

Because variables were considered in isolation, effects of other variables' attributes are not included in this figure, so it is not possible to make a direct comparison between measures from the two plots in the figure. However, the relationships represented by different series within each plot reveal the following:

- Results showed varying degrees of improvement (i.e., reduction in delay, reduction in percent time spent following) as the number of passing lanes increased.
- Results indicate that the lowest percent time spent following (percentage of vehicles with headway of 4 seconds or less) was about 40 percent when no passing lanes are provided, and that value increased to about 85 percent. The corresponding range in delay per vehicle was about 0.6 minutes and 1.6 minutes, respectively.
- The lowest value of percent time spent following was about 13 percent when six passing lanes were provided and increased to about 75 percent. Corresponding delay per vehicle was about 0.25 minute and 1.5 minutes, respectively.
- The incremental improvement achieved by adding passing lanes along the 10-mile long corridor also tends to be diminished. The benefit in having six passing lanes compared to three is not as great as the benefit in having three passing lanes compared to zero. There is still an additional benefit when the next three lanes are added, but the second three lanes are not as effective as the first three.
- In comparing the scenarios with the highest following percentage or delay for each variable, there appears to be a trend that the performances measures tend to converge as performance degrades. It suggests that under the most restrictive scenarios, such as combining high volume with short passing lane length, the incremental benefit brought by adding another passing lane is minimal after a certain threshold value.

Similar analysis figures for other variables were created as well. Figure 49 reveals the effects of ADT on operational performance. Not surprisingly, the plots reveal that the performance deteriorates as the ADT level increases. Similar to the findings from Figure 1, the separation between trendlines for different ADTs decreased as ADT increased, suggesting that with each additional vehicle the incremental impact on operational performance declines. The analysis did not investigate a scenario with a corridor at or above capacity, so there are no definitive answers as to where the upper limit of this trend might be, though the trendline for the highest ADT value appears to level off around 85 percent time spent following.



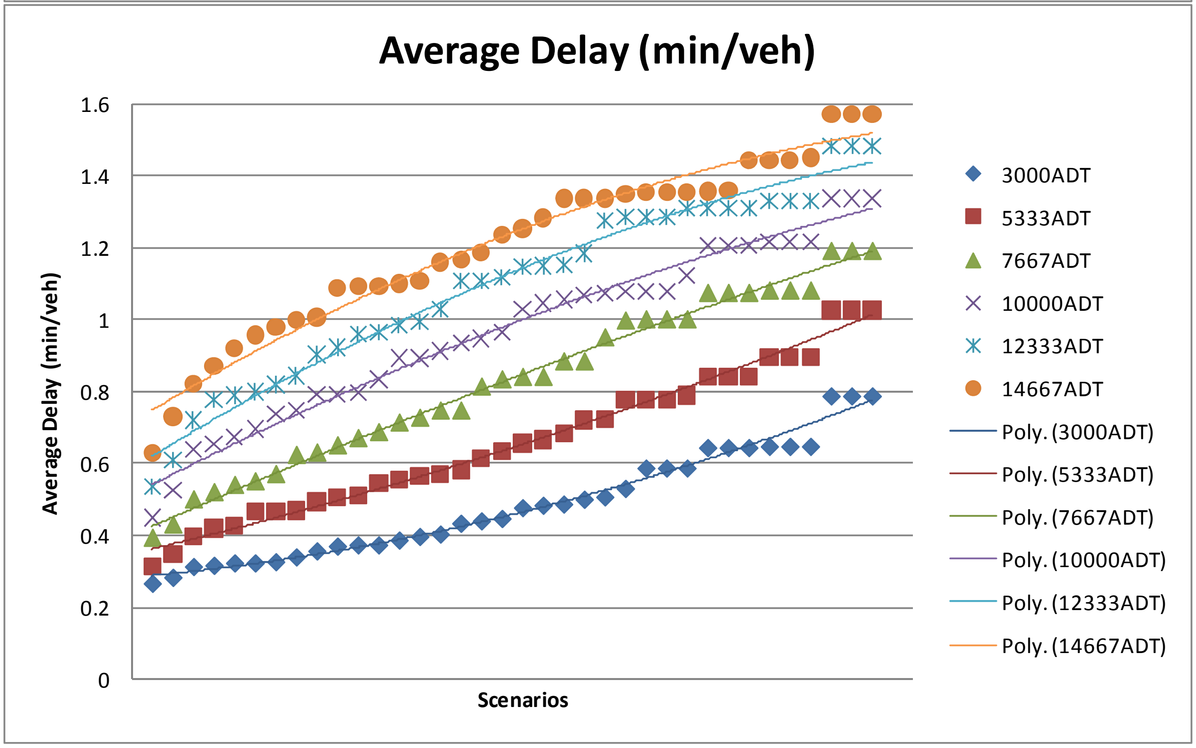


Figure 49. Performance Comparison for Different ADT Levels.

Figure 50 compares the effects of truck percentage on percent time spent following and average delay. Note that the difference between the trend lines for the two percentages is almost negligible for percent time spent following, and it is definitely minimal compared to the effects by other variables. This suggests that truck percentage may not be as critical to performance as previously thought.

Researchers also examined the effects of passing lane length. The comparison of length of passing lane reveals similar findings to the number of passing lane scenario. As shown in Figure 51, the improvement tends to be diminished by comparing the difference between the 1-mile and 2-mile scenarios and the 2-mile and 3-mile scenarios. In demanding scenarios with high volumes and/or lower numbers of passing lanes, the benefit realized by simply extending the passing lane length is reduced. The comparison of different terrain types is similar to the truck percentage scenario. Results from Figure 52 indicate that the difference between the trendlines for the two terrain types is minimal compared to the effects of other variables.

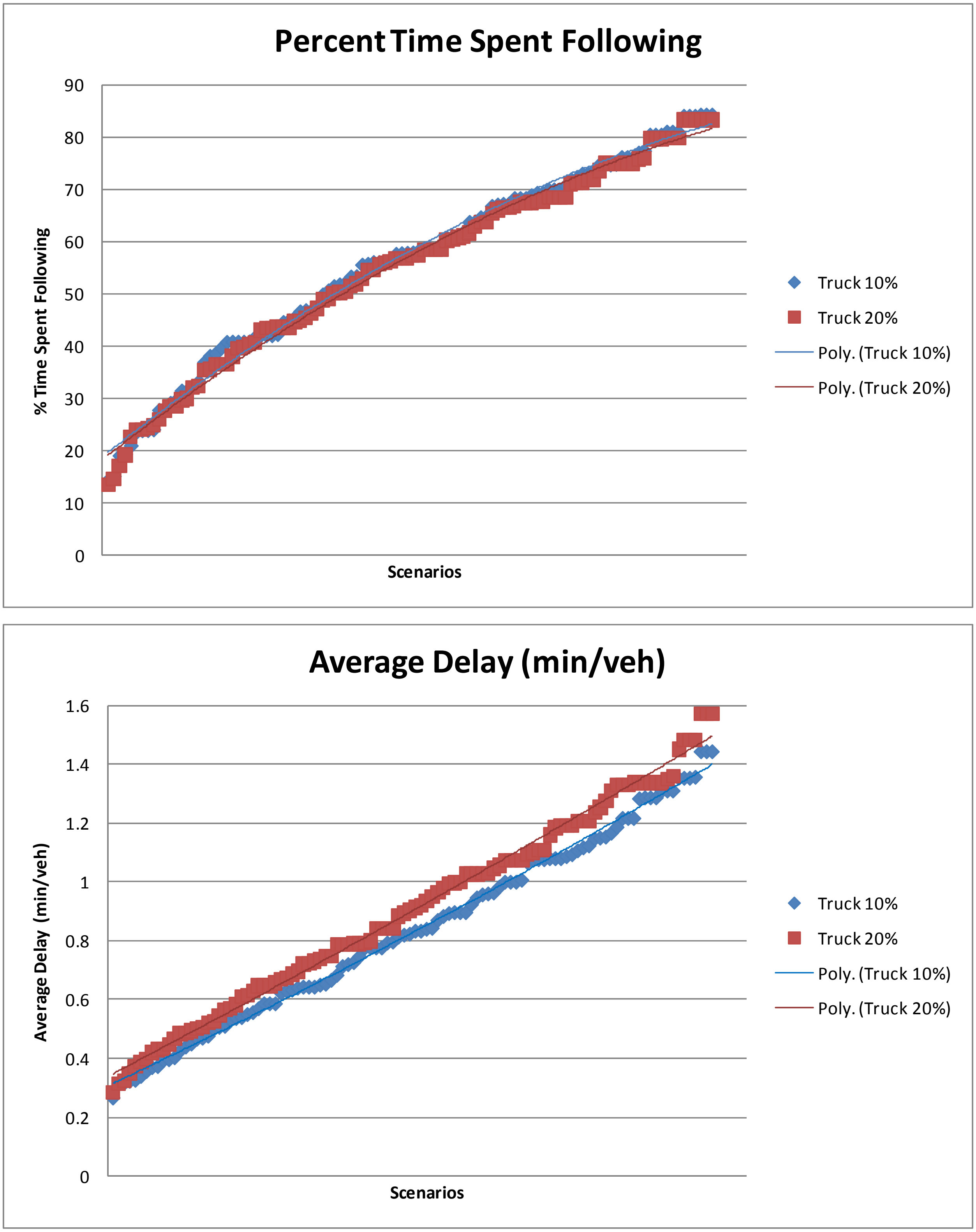
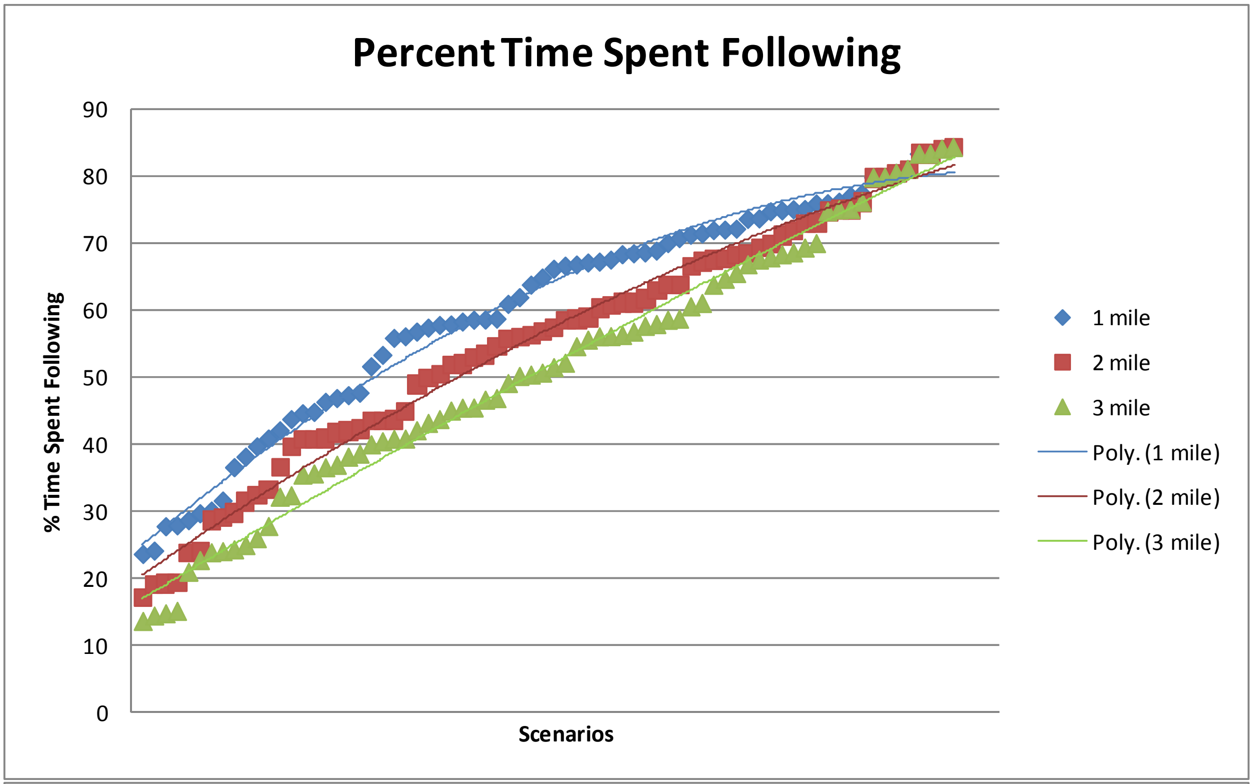


Figure 50. Performance Comparison for Different Truck Percentage.



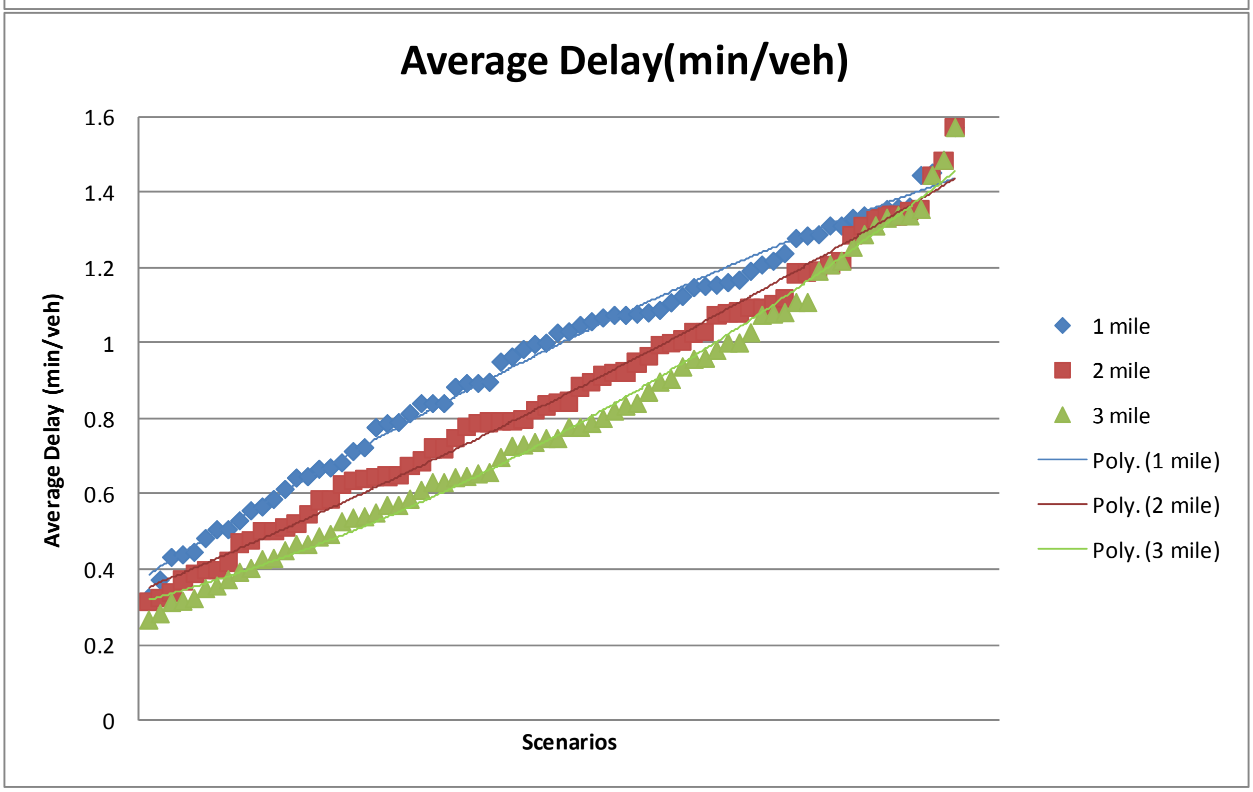
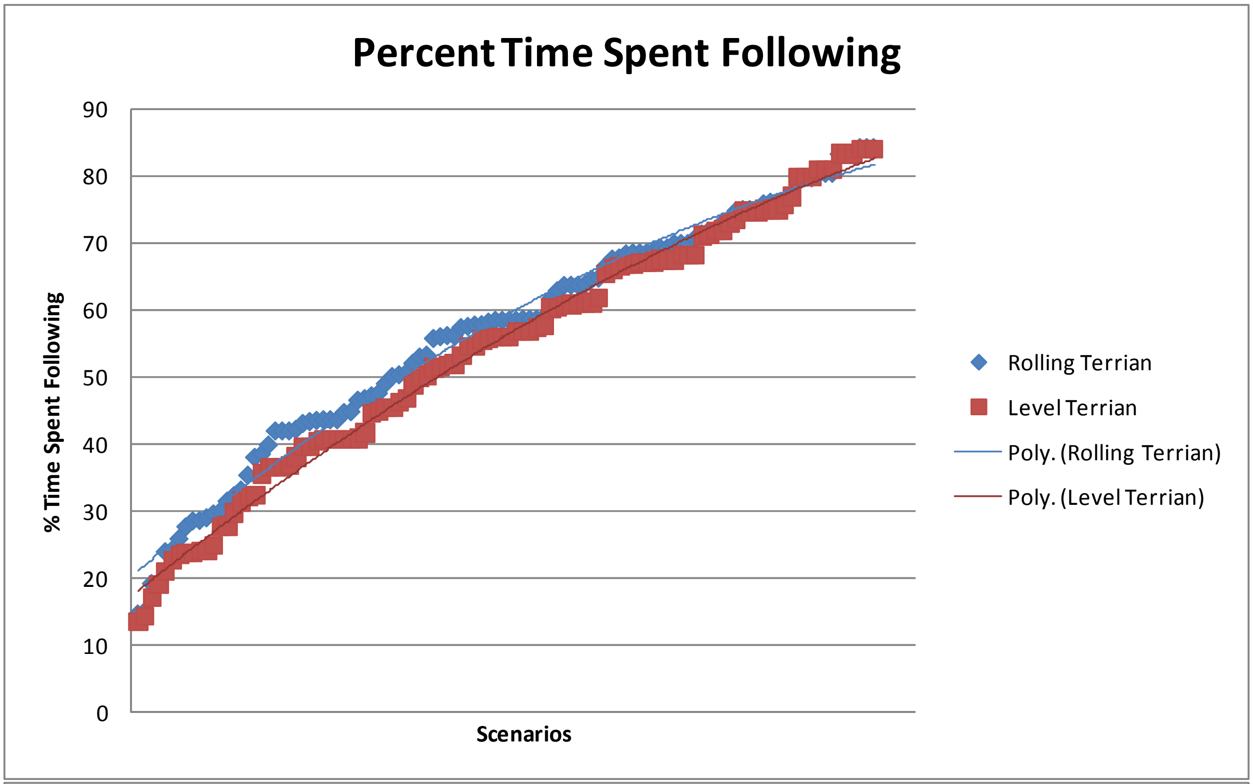


Figure 51. Performance Comparison for Different Passing Lane Lengths.



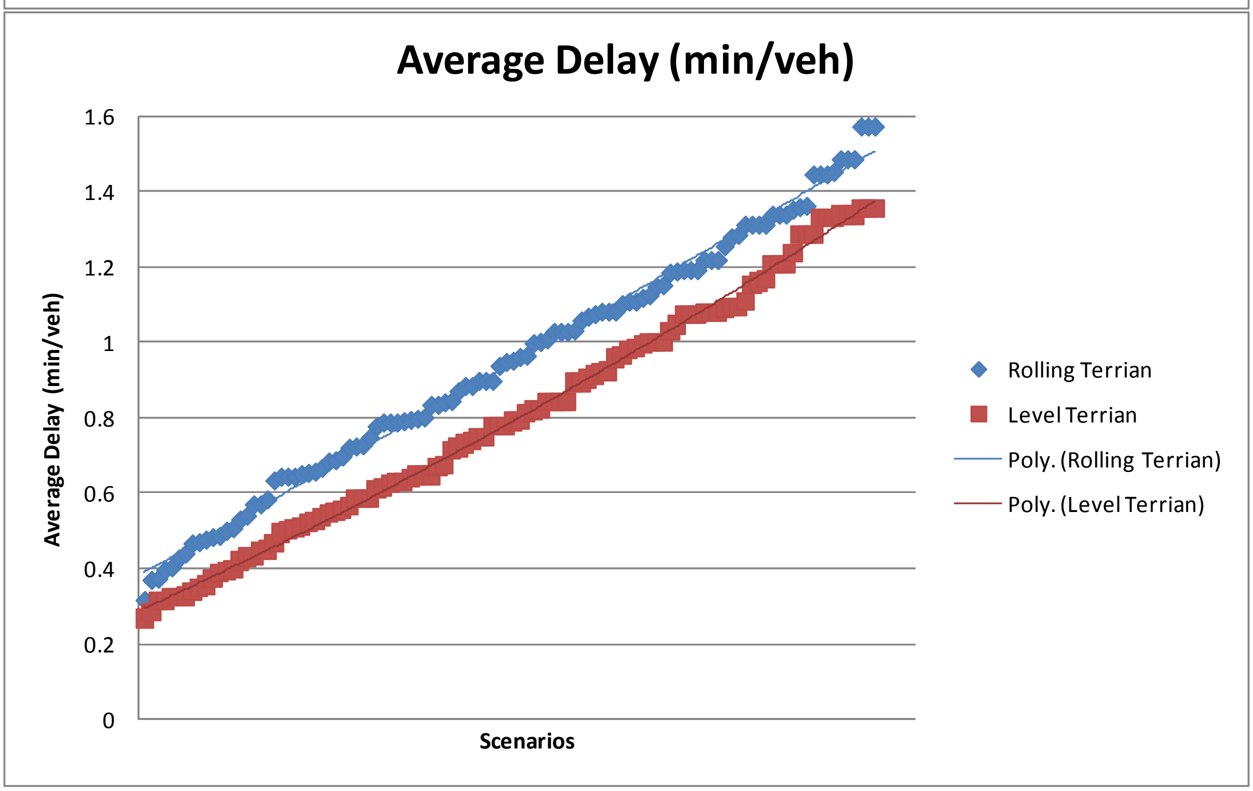


Figure 52. Performance Comparison for Different Terrain Types.

Comparison of Performance for Different Passing Lane Configurations

The summary results shown above provided a basic indication of the overall effects of each individual variable on Super 2 operations. However, it was necessary to determine operational performance of the treatment for specific traffic volume and geometric layout scenarios, with combinations of variables. As mentioned previously, passing lane spacing and passing lane length are the two critical elements of passing lane design considered in this analysis. Revealing how they affect operational performance under certain traffic and geometry conditions (e.g., varying ADT levels, truck percentage, and terrain types) is important to developing appropriate guidelines for installation of future Super 2 corridors.

Appendix A shows the complete set of results, but two examples are given in Figure 6 and Figure 7. One is for the lowest (3000 ADT) volume level with 20 percent truck percentage and level terrain; the other is for the highest (14,667 ADT) volume level with 10 percent truck percentage and rolling terrain.

The 3-D contour maps in Figure 6 and Figure 7 use the degree of slope to reveal the rate of performance improvement. Also, the transition from the cold color (blue) to warm color (dark red) reveals higher percent time spent following and delay, which can also be used as an indicator of performance. Results of these two examples show that though specific values of the measures of effectiveness are noticeably different between the ADT levels, adding new passing lanes and extending their lengths will help to improve operational performance in similar ways in both cases.

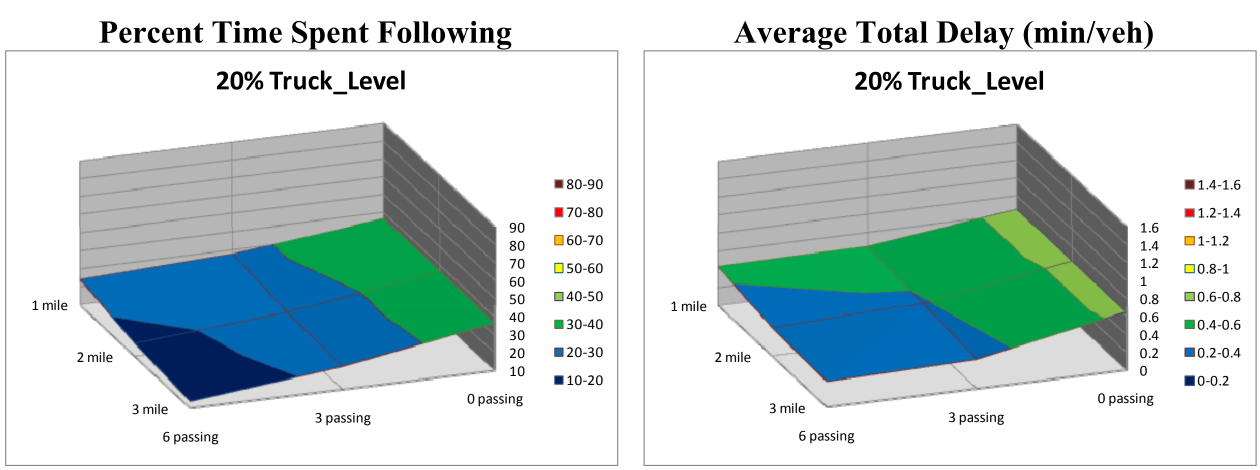


Figure 53. Performance Measures on Different Passing Lane Configurations – 3000 ADT, 20% Truck, Level Terrain.

Percent Time Spent Following | **Average Total Delay (min/veh)**

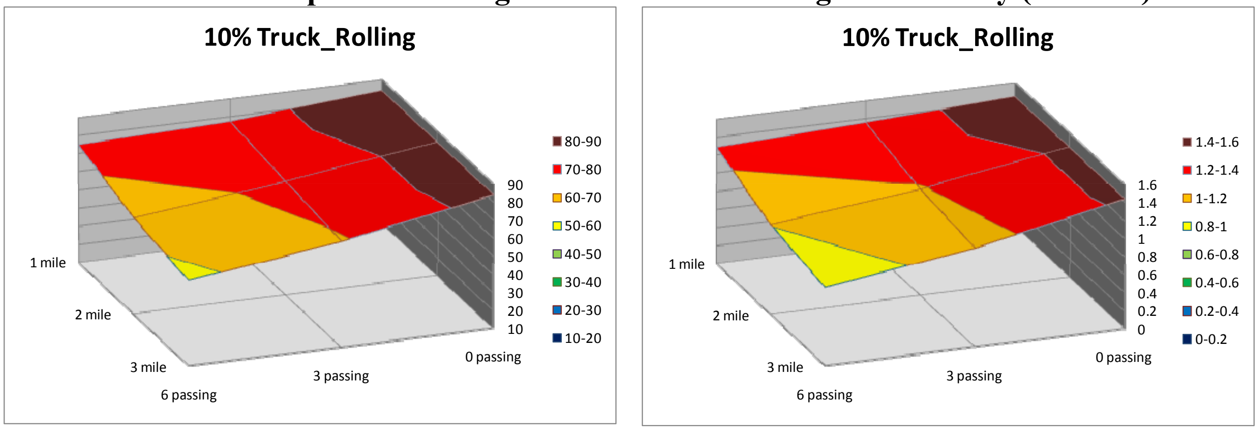


Figure 54. Performance Measures on Different Passing Lane Configurations – 14,667 ADT, 10% Truck, Rolling Terrain.

The completed MOEs comparisons for all variables are listed in Appendix A. Analysis of the contour maps for all scenarios indicates that:

- Adding new passing lanes and extending their length will help to improve operational performance.
- The improvements achieved by increasing the number of passing lanes usually are greater than the improvements achieved by increasing the length. The transition along the x-axis (number of passing lanes) in the contour map has a steeper slope and more changes in the color band than along the y-axis (length of passing lanes). This supports previous research that platoons tend to break up within a certain distance along a passing lane; beyond that distance, there is little added benefit within the same lane. However, adding another lane will help to disperse platoons that develop elsewhere.
- Similarly, the difference in MOEs between the 1-mile and 2-mile length scenarios is greater than the difference between the 2-mile and 3-mile length scenarios, suggesting a trend of diminishing returns for added passing lane length.
- The incremental improvement achieved by adding passing lanes along the 10-mile long corridor also tends to be diminished. The improvement in adding three lanes to a no-lane scenario is greater than the improvement in adding three more lanes to produce a six-lane scenario. There is still an improvement, but it is not as great. From a practical standpoint, the more passing lanes are added, the more like a continuous four-lane corridor the road will become, thereby minimizing the cost savings of installing a Super 2

treatment instead of full four-lane widening. It is therefore reasonable that incremental operational benefits diminish accordingly.

Comparison of Performance for Different Traffic and Geometric Combinations

Similar to the previously mentioned analyses to determine changes in operational performance for specific traffic volume and geometric layout scenarios, it is useful to evaluate how varying ADT and other traffic variables will affect performance measures under a fixed passing lane configuration.

Appendix B shows full results, but two examples are given in Figure 55 and Figure 56. The former shows results from examining various performance measures, terrain, and truck percentage in a scenario with no passing lanes; the latter displays results for the same performance measures in a scenario with six passing lanes, each 1 mile in length.

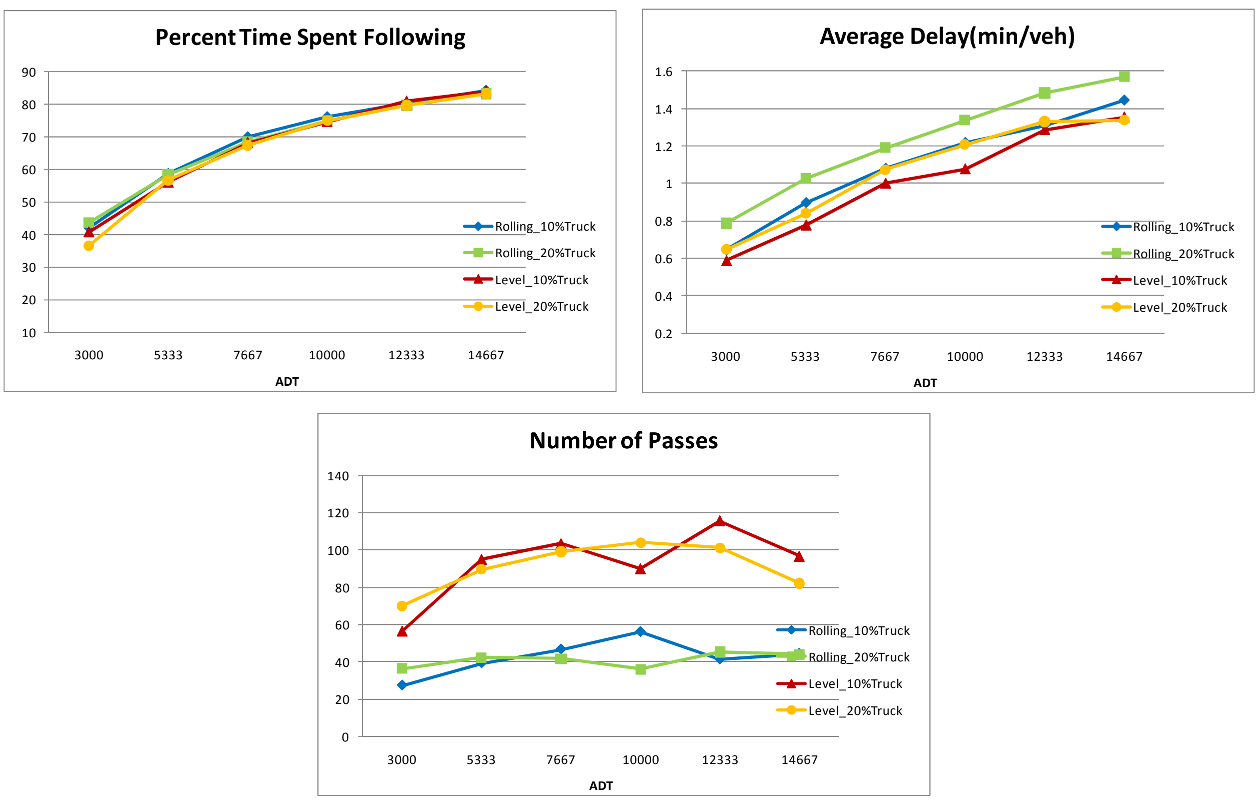


Figure 55. Performance Measures on Varied Geometric and Traffic Conditions – No Passing Lanes.

The results shown for number of passes in Figure 55 is noteworthy in that the number of passes does not have a definable mathematical relationship (i.e., linear, exponential) relative to increases in ADT. Instead, the results indicate that the number of passes reached a maximum threshold between 7667 and 12,333 ADT due to the limited passing opportunity present in the scenario with no passing lanes.

Figure 56. Performance Measures on Varied Geometric and Traffic Conditions – 6 Passing Lanes, 1 Mile Length.

The trendlines in the Number of Passes chart in Figure 56 diverge as the ADT increases. A conclusion can be made that as the volume level increases, the number of passes increases because there are more opportunities to pass slower vehicles, particularly trucks, when a passing lane is provided. Not surprisingly, though, for a given roadway corridor, the percent time spent following and the average delay increase as ADT increases. A comparison of results for level terrain and 20 percent trucks in Figures 8 and 9, however, indicates a delay savings of about 0.18 min (about 11 seconds) for each vehicle with the addition of six passing lanes and

14,667 ADT, a total delay savings of almost 4.3 hours during the test period. Similar savings can be calculated for the other three scenarios and the other ADT values, as shown in Table 39.

Table 39. Estimated Delay Savings (hr) From Installation of Six 1-Mile Passing Lanes in the 10-Mile Simulation Corridor.

	Rolling		Level	
ADT	10% Trucks	20% Trucks	10% Trucks	20% Trucks
3000	0.6	1.5	1.3	1.1
5333	2.0	2.7	2.4	2.4
7667	2.5	3.1	3.7	5.2
10,000	2.5	4.7	3.1	5.2
12,333	3.4	3.6	6.2	6.2
14,667	3.9	2.9	6.5	4.3

The completed MOE comparisons for all variables are listed in the appendices. Analysis of the figures for all scenarios led researchers to conclude that:

- ADT levels, truck percentage, and terrain types all have impacts on operational performance, but to different degrees.
- ADT level had the biggest impact: the percent time spent following for the lowest ADT level was often about 40 percent of that for the highest ADT, and the average delay roughly tripled as ADT increased from 3000 to 14,667.
- Relative to ADT, truck percentage and terrain types had very limited impact on the performance measures, particularly the influence by truck percentage on percent time spent following. In most scenarios the difference produced by increased truck volume is almost negligible.
- While the truck percentage and terrain types show negligible impact on the measure of percent time spent following, their impacts on the measure of average delay and number of passes are more pronounced. For example, as one would expect, the number of passes in the no passing lanes scenario went up substantially in level terrain, as compared to rolling terrain. Accordingly, the average delay on level terrain tended to be lower than on rolling terrain.
- As mentioned previously, the percent time spent following increases as ADT increases, but at a decreasing rate, approaching a peak value at or above 14,667 ADT. Conversely, the number of passes appears to increase exponentially with increasing ADT, which is

intuitive because the number of possible passes increases as the number of vehicles increases.

- In most scenarios, the number of passes for 20 percent truck volume is higher than that for 10 percent trucks. It is believed that the higher percentage of slower trucks provided more opportunity for overtaking by faster passenger cars (using both the opposing lane and the passing lanes to complete their passing maneuvers).
- The largest difference in number of passes by terrain is in the scenario with no passing lanes. For scenarios with three or six passing lanes, regardless of passing lane length or truck percentage, the number of passes is very similar for rolling and level terrain.

CHAPTER 10
CONCLUSIONS AND RECOMMENDATIONS

This chapter of the report summarizes the work completed throughout the project, as documented in the previous chapters of this report, and provides a listing of the researchers' key conclusions from the work. This chapter also includes the researchers' recommendations for future action based on those conclusions.

FINDINGS FROM LITERATURE

General Considerations

During the course of this research project, researchers have reviewed relevant literature and research findings, as well as current policies in other states. Observations from those efforts led to several conclusions on general considerations in the location and design of passing lane sections for Super 2 roadways, which are summarized below:

- Super 2 roadways are most suitable for level and rolling terrain, particularly the latter, where sight distance is often restricted.
- Intersections and driveways, especially those with high volumes, should be avoided within a passing lane section if possible. Where a low-volume side road intersection is inevitable within a passing lane, the passing lane should be located so that the intersection is as close as possible to the middle of the passing lane. Side road intersections within lane drops and lane additions should be avoided.
- The location and configuration of a passing lane may be influenced by the need to alleviate an operational problem, adjacent development, terrain, or other factors. Some guidelines for location of passing lanes include:
    - To address operational problems, identify areas with high levels of platooning and/or large delays that occur regularly.
    - The location should appear logical to the driver, (e.g., on grades or where passing sight distance is restricted).
    - Location should provide adequate sight distance on the approach and departure tapers.
    - Avoid locating passing lanes on highway sections with low-speed horizontal curves.

- The geometrics of the passing lane section (particularly lane widths and shoulder widths) should be similar, if not identical, to the adjacent two-lane section of the highway.
- In relation to urban areas and major intersections, it is preferable that passing lanes be located where traffic departs an urban area, rather than on the approach.

Traffic Volumes

Previous studies described in the literature indicate that there is not a defined upper limit to the traffic volumes at which a Super 2 corridor should be considered. There is, however, a practical limit of the capacity of a two-lane highway, which is estimated by the *Highway Capacity Manual* to be 1700 passenger cars per hour (pc/hr) in one direction or 3200 pc/hr in both directions. This theoretical capacity is reduced by various real-world effects, but regardless of the exact number, volumes above that will be best served by a four-lane alignment.

Passing Lane Length

Previous studies consistently show that most passing occurs within the first mile of a passing lane, though this length increases somewhat with volume. As volumes increase, there can be some added benefit to longer passing lanes. Recommended minimum lengths in other states are typically 1000 ft or 0.25 mile.

The practical application of passing lane length, however, is that the length of the passing lane should be influenced by traffic characteristics (e.g., volumes, length of platoons), location of major intersections, geometrics, and distances between successive passing opportunities.

Passing Lane Spacing

Similar to passing lane length, actual spacing of passing lanes can vary at each site and should depend on the traffic volumes, right-of-way availability, and existing passing opportunities. However, some states have regular interval spacing, while others allow more flexibility. Common recommended values for passing lane spacing are 3 to 10 miles, which contains the typical range of distances where the downstream operational benefits of a passing lane end. Where possible, it may be desirable to provide passing lanes at longer spacing with plans for intermediate passing lanes as the traffic volume increases. However, the spacing must be flexible to permit selection of suitable and inexpensive sites.

Buffer Area and Configuration of Passing Lanes

On highways where right-of-way and alignment are accommodating, passing lanes can be located in either side-by-side (Figure 15k) or adjoining head-to-head or tail-to-tail (Figure 15f and Figure 15e, respectively) configuration. However, the side-by-side passing lane configuration is less desirable in close proximity to urban areas and major intersections.

The design of tail-to-tail passing lanes is the simpler of the adjoining configurations because there is not a need for the large transition or buffer area between the lanes that is required for head-to-head lanes. When adjoining passing lanes meet near the merge area and oncoming traffic is approaching from opposite directions, it is necessary to provide a sufficient buffer between the two lanes to accommodate for late merges and situations when two cars in the same direction approach the merge area side-by-side. Specific values for the length of that buffer are not commonly recommended, but a prudent rule of thumb is that the stopping sight distance (SSD) be provided where possible.

The needed tapers to accomplish the addition or removal of the added lane are equivalent to those of other tapers at highway speeds, with the latter being more generous than the former, generally by a factor of two. A minimum sight distance at the lane removal (or lane drop) taper of 1000 ft or 1500 ft is common; a convenient way to calculate a more site-specific taper length is provided by the equation (L = WS), where:

- L= Length of taper.
- W=Lane width.
- S= Posted speed.

USAGE IN TEXAS

Based on the 21 responses to the questionnaire distributed to TxDOT Area Engineers as part of this project, researchers drew the following conclusions about the current usage of Super 2 corridors in Texas:

- Use, or proposed use, of Super 2 corridors has increased in recent years.
- These corridors are distributed across the state, though the northern and western parts of the state seem to be more active.

- The most common reason for installing passing lanes was the presence of limited passing opportunities on the existing roadway. Restricted sight distance, high traffic volumes, and passing safety were also commonly cited as reasons for installation.
- Current guidelines in the *Roadway Design Manual* are most often used to determine length and spacing of passing lanes, with some influence by terrain, sight distance, and traffic conditions.
- Anecdotal evidence suggests that Super 2 corridors are well received by the driving public when they are installed.

CRASH DATA ANALYSIS

Researchers reviewed and analyzed crash data for identified Super 2 corridors for the years 1997–2001 and 2003–2009. In that analysis, researchers found the following:

- Most of the passing lanes identified in the questionnaire were installed recently (e.g., after 2004), which limited the number of sites for which post-installation crash data were available.
- Empirical Bayes analysis of 53 centerline-miles on five Super 2 corridors showed that there is a statistically significant crash reduction of 35 percent for segment-only crashes (KABC) on the study corridors, as compared to the expected number of crashes without passing lanes. This finding is consistent with findings of previous safety-related studies of Super 2 corridors, which show improvements in safety with installation of passing lanes, even at traffic volumes higher than those considered under previous guidance in Texas.

FIELD STUDIES

Researchers collected field data on Super 2 corridors on SH 121 in the Paris District and on US-183 in the Yoakum District. That data provided a look at drivers' behavioral trends and patterns associated with driving in passing lanes. Some of the key findings from analysis of the field data are as follows:

- Super 2 corridors do improve operations on rural two-lane highways, in agreement with previous research.

- Observation of lane selection at the entrance to the passing lane section indicates that large numbers of vehicles began passing maneuvers at the beginning of the section; however, not all vehicles in the left lane actually used the left lane for passing, contrary to Texas law. As many as 92 percent (and as few as 21 percent) of left-lane vehicles began a passing maneuver near the beginning of the section.
- Large trucks tended to utilize the right lane at the entrance to passing lane sections, allowing faster vehicles to pass. Truck compliance with the right lane was 74 percent or better at each site.
- While not a specific focus of this study, differences in traffic patterns at the two study sites suggest that pavement markings, signing, and enforcement may have measurable effects on lane choice at the entrance to the passing lane section, which supports findings from previous research that signing and marking are important elements in Super 2 design. A dedicated study with detailed analysis based on additional study sites could provide useful information on these effects.
- Between 40 and 66 percent of vehicles in the passing lane at the point of departure were engaged in a passing maneuver. Though there was a high level of non-compliance with the "left lane for passing only" law, it was more consistent at the departure than at the entrance, perhaps indicating that many vehicles complete their passing maneuvers early in the passing lane section and then do not change lanes prior to leaving the section.
- Truck utilization of the right lane at departure was lower than at entrance, though it was still high overall, ranging from 67 to 91 percent.
- Analysis of the headway data collected for this project to determine the impacts of passing lanes beyond their physical limits indicates that the downstream effects of passing lanes on congestion may be limited at higher volumes.

RESULTS FROM SIMULATION

Members of the research team used field data to create and calibrate a simulation model to analyze operational characteristics of Super 2 corridors under a variety of traffic conditions and passing lane design parameters. Researchers created a Super 2 corridor 10 miles in length to serve as the test bed for the simulation model. The total number of simulations was 648: 6 ADT values × 2 terrain values × 2 heavy vehicle proportions × 3 passing lane lengths × 3 passing lane spacing values resulted in 216 scenarios, which were each run three times, producing 648 total simulation runs. As a result of the simulation process, researchers concluded the following:

- Calibration of the simulation model indicated that the IHSDM's Traffic Analysis Module is an appropriate modeling tool for future analyses of passing lanes on other Super 2 corridors.
- Separate analyses of individual variables showed varying degrees of improvement in those variables (i.e., reduction in delay, reduction in percent time spent following) as the number of passing lanes increased.
  - Results indicated that the percent time spent following when no passing lanes are provided varied from about 40 percent to about 85 percent. The corresponding range in delay per vehicle was about 0.6 minutes to 1.6 minutes, respectively.
  - The percent time spent following when six passing lanes were provided was about 13 percent to about 75 percent. Corresponding delay per vehicle was about 0.25 minute and 1.5 minutes, respectively.
  - The incremental improvement achieved by adding passing lanes along the 10-mile long corridor tends to diminish. The benefit in having six passing lanes compared to three is not as great as the benefit in having three passing lanes compared to zero. There is still an additional benefit when the next three lanes are added, but the second three lanes are not as effective as the first three.
  - In comparing the scenarios with the highest following percentage or delay for each variable, there is an apparent trend that performance measures tend to converge as performance degrades. It suggests that under the most restrictive scenarios (e.g., high volumes with short passing lane length), the incremental benefit from additional passing lanes after a certain threshold value is minimal.
- Analysis of the completed MOEs comparisons for all variables indicates that:
  - Adding new passing lanes and extending their length will help to improve operational performance.
  - Improvements achieved by increasing the number of passing lanes usually are greater than the improvements achieved by increasing the length. This supports previous research that platoons tend to break up within a certain distance along a passing lane;

beyond that distance, there is little added benefit within the same lane. However, adding another lane will help to disperse platoons that develop elsewhere.

- Similarly, the difference in MOEs between the 1-mile and 2-mile length scenarios is greater than the difference between the 2-mile and 3-mile length scenarios, suggesting a trend of diminishing returns for added passing lane length.
- The incremental improvement achieved by adding passing lanes along the 10-mile long corridor also tends to be diminished. From a practical standpoint, the more passing lanes are added, the more like a continuous four-lane corridor the road will become, thereby minimizing the cost savings of installing a Super 2 treatment instead of full four-lane widening. It is therefore reasonable that incremental operational benefits diminish accordingly.

- ADT levels, truck percentage, and terrain types all have impacts on operational performance, but to different degrees.
- ADT level had the biggest impact on operations; over the range of scenarios simulated by the model, the percent time spent following for the lowest ADT level was often about 40 percent of that for the highest ADT, and the average delay roughly tripled as ADT increased from 3000 to 14,667. MOEs particularly degrade at or above ADT of 10,000.
- Percent time spent following increases as ADT increases, but it does so at a decreasing rate, approaching a peak value at or above 14,667 ADT. Conversely, the number of passes appears to increase exponentially with increasing ADT, because the number of possible passes increases as the number of vehicles increases.
- Relative to ADT, truck percentage and terrain types had very limited impact on the performance measures, particularly the influence by truck percentage on percent time spent following. In most scenarios the difference produced by increased truck volume was almost negligible.
- While the truck percentage and terrain types show negligible impact on the measure of percent time spent following, their impacts on the measure of average delay and number of passes are more pronounced. The number of passes in the no passing lanes scenario was much higher in level terrain than in rolling terrain. Accordingly, the average delay on level terrain tended to be lower.

- In most scenarios, the number of passes for 20 percent truck volume was higher than that for 10 percent trucks. It is believed that the higher percentage of slower trucks provided more opportunity for overtaking by faster passenger cars (using both the opposing lane and the passing lanes to complete their passing maneuvers).
- The largest difference in number of passes by terrain was in the scenario with no passing lanes. For scenarios with three or six passing lanes, regardless of passing lane length or truck percentage, the number of passes was very similar for rolling and level terrain.

RECOMMENDATIONS

Based on the findings from the literature, review of current practices, analysis of field data and crash data, and simulation conducted in this project, researchers recommend the following guidelines for use of Super 2 corridors in Texas:

- The use of ADT as an upper limit on the installation of passing lanes should be eliminated. As budget, terrain, and other factors allow, passing lanes may be added or lengthened to provide additional passing opportunities regardless of volume. There is, of course, the proviso that as passing lanes are added and lengthened, the highway more closely resembles a four-lane undivided alignment and the incremental cost and operational benefits of each added lane diminish.
- While ADT need not be a limiting factor in installation, it can be used to prioritize candidate sites for passing lanes, particularly when considering truck volumes. A traffic analysis of candidate sites will help the designer to determine which locations may receive greater benefit from lengthening existing passing lanes or installing new passing lane sections.
- Where terrain, available budget, and other considerations allow, the addition of another passing lane is preferred over adding length to an existing one. Passing lane lengths over 2 miles show less incremental benefit than higher frequency of lanes, particularly for ADT less than 10,000 vpd. Regardless of volume, passing lanes longer than 3 miles should be used sparingly, and lengths of more than 4 miles should be avoided.
- In lieu of guidelines related to specific ADT values, other general principles should be used to assist designers in the decision to install Super 2 corridors. Key principles are as follows:

- The designer should consider existing width of right-of-way (ROW), terrain, and structures to evaluate the feasibility of a Super 2 corridor and determine the best locations to install passing lanes with a minimum of ROW acquisition, earthwork, and structure widening.
- The location of major traffic generators, such as intersections with other state highways or driveways to large developments, should be identified as the proposed alignment is planned. It is preferable to avoid locating high-traffic intersections and driveways within the boundaries of a passing lane. When such generators are unavoidable, it is preferable that they be located near the midpoint of the passing lane to provide as much separation from the opening and closing tapers.
- Avoid locating passing lanes at locations with restrictive geometry (e.g., sharp horizontal curves) or other impediments to traffic flow (e.g., approaches to urbanized areas). However, providing passing lanes downstream of these features is beneficial for dispersing platoons.
- Where passing lanes are terminated, sufficient sight distance must be provided to avoid conflicts with oncoming traffic or constraints such as guard rail, guard fences, or narrow bridges. Stopping sight distance is recommended.

RECOMMENDED REVISIONS TO TXDOT HIGHWAY SAFETY IMPROVEMENT MANUAL

Based on the results of the review and analysis of crash data on existing Super 2 corridors as part of this project, researchers recommend revisions to the portion of the June 2008 TxDOT *Highway Safety Improvement Program Manual* (*49*) that provides information on crash reduction effects of passing lanes on two-lane highways. Revisions are shown as additions or ~~deletions~~ to correspond to findings from this project. The research team recommends the following revisions to Section 9 – HSIP Work Codes Table:

540	Install Passing Lanes on 2-Lane Road	
	Definition	Install passing lanes on a 2-lane road.
	Reduction Factor (%):	~~25~~ 35
	Service Life (Years):	10
	Maintenance Cost:	N/A
	Preventable Crash:	(Roadway Related = 1 or 2 or 3) AND (Intersection Related = 3 or 4) AND (Crash Severity = 1-4) ~~OR (Vehicle Movements/Manner of Collision = 20-24 or 30)~~

Researchers also recommend adding the following table to Section 10 – Preventable Crash Decoding, to provide the severity information referenced in the revised table in Section 9:

Crash Severity

0	Unknown
1	Incapacitating Injury
2	Non-Incapacitating
3	Possible Injury
4	Killed
5	Not Injured

RECOMMENDED REVISIONS TO TXDOT ROADWAY DESIGN MANUAL

Previous *Roadway Design Manual*

The Texas *Roadway Design Manual* provides the current description of and guidance for the use of passing lanes on two-lane highways; specifically, Chapter 4, Section 6 contains the guidelines on Super 2 highways. At the time this research project began, guidance on passing lane length and spacing was based primarily on the ADT of the roadway; Table 4-6 of the October 2006 *Roadway Design Manual*, reproduced here as Table 40, specified the details of those guidelines (*2*).

Table 40. Super 2 Passing Lane Length and Spacing by ADT (*2*).

Two-Way ADT (vpd)	Recommended Passing Lane Length (mi)	Recommended Distance between Passing Lanes (mi)
< 2000	1.0	5–9
2001–5000	1.5–2.0	4–9
> 5000	Conversion to four-lane highway should be considered	

Design criteria for passing lane sections were the same as the 3R design guidelines for other rural two-lane highways. Those guidelines were also based on ADT, as shown in Table 4-6 of the October 2006 *Roadway Design Manual*, reproduced here as Table 41.

Table 41. 3R Design Guidelines for Rural Two-Lane Highways, US Customary Units (*1*).

Design Element [a]	Current Average Daily Traffic		
	0–400	400–1500	1500 or more
Design Speed [b]	30 mph	30 mph	40 mph
Shoulder Width (ft)	0	1	3
Lane Width (ft)	10	11	11
Surfaced Roadway (ft)	20	24	28
Turn Lane Width (ft) [c]	10	10	10
Horizontal Clearance (ft)	7	7	16
Bridges [d]: Width to be retained (ft)	20	24	24 [e]

NOTES:
[a] These values are intended for use on rehabilitation projects. However, the designer may select higher values to provide consistency with adjoining roadway sections, to provide consistency with prevailing conditions on similar roadways in the area or to provide operational improvements at specific locations.
[b] Considerations in selecting design speeds for the project should include the roadway alignment characteristics as discussed in this chapter.
[c] For two-way left turn lanes, 11 ft–14 ft usual.
[d] Where structures are to be modified, bridges should meet approach roadway width as a minimum. (Approach roadway width is the total width of the lanes and shoulders.) Greater bridge widths may be appropriate if the rehabilitation project increases roadway life significantly or if higher design values are selected for the remainder of the project. Existing structure widths less than those shown may be retained if the total lane width is not reduced across or in the vicinity of the structure.
[e] For current ADT exceeding 2000, minimum width of bridge to be retained is 28 ft [8.4 m].

The October 2006 *RDM* provided additional general guidance that passing lanes should be located to best fit existing terrain and field conditions: "Uphill grades are preferred sites over downhill grades. Passing lanes on significant uphill grades should extend beyond the crest of the hill. Passing lane sections and transitions should be placed to avoid major intersections. If present, minor intersections that do not require deceleration lanes should be located near the midpoint of passing lane sections and also avoid transition areas to the extent practical."

The previous *RDM* added that providing a passing lane section downstream of a traffic signal for platoons exiting an urbanized area is particularly beneficial in dispersing the platoons and improving operations in rural areas. The *RDM* also stated that "significant lengths or segments of passing lanes are not encouraged. If traffic volumes are such that significant lengths or segments of passing lanes are necessary, then construction of another category of roadway should be considered." However, the *RDM* added that "a passing lane is appropriate for areas where passing sight distances are limited. The location of the proposed lane addition should offer adequate sight distances and lane taper. The location selection should also consider the presence of intersections and high volume driveways in order to minimize the volume of turning movements on a roadway section where passing is being encouraged" (*2*).

Current *Roadway Design Manual*

Since the beginning of this project, TxDOT engineers have discussed a number of changes to the previous edition of the *RDM*; some changes were introduced in conjunction with the research activities on this project, while others were based on experiences of TxDOT designers in implementing existing Super 2 corridors. In May 2010, a revised version of the *RDM* was released (*50*), with one of the major changes being a complete rewrite of Chapter 4, Section 6. In that version, the guidelines tying ADT to passing lane length and spacing were removed, giving designers more flexibility to use a Super 2 alignment that meets the needs of a particular roadway. The removal of ADT requirements and a number of other changes made in the May 2010 version are consistent with findings from this project, but other findings were not yet completed at that time. The following pages show the May 2010 version of Chapter 4, Section 6 of the *RDM*, with further suggested revisions shown as additions or ~~deletions~~ to correspond to additional findings from this project made after the release of the latest version of the Manual.

Section 6
Super 2 Highways

Passing Lanes (Super 2 Highways)

A Super 2 highway is ~~where a~~ defined as a two-lane rural highway in which periodic passing lanes have been ~~is~~ added ~~to a two-lane rural highway~~ to allow passing of slower vehicles and the dispersal of traffic platoons. The passing lane will alternate from one direction of travel to the other within a section of roadway allowing passing opportunities in both directions. A Super 2 project can be introduced on an existing two-lane roadway where there is a significant amount of slow-moving traffic, there is limited sight distance for passing, and/or ~~the~~ existing traffic volume has increased ~~exceeded the two-lane highway capacity~~, creating the need for vehicles to pass on a more frequent basis.

Recent research has showed that providing periodic passing lanes on two-lane rural highways provides a benefit in reduced delay and time spent following, which improves operations and reduces the need for drivers to pass on two-lane sections. A single passing lane has a carryover benefit into the downstream two-lane section, because previous platoons are partially or completely dispersed and traffic flow is improved. This carryover benefit of a single passing lane exists for high-volume locations, but it is even greater for low-volume sites where a single slower vehicle can delay a higher proportion of trailing vehicles. The improvement in operations also contributes to an improvement in safety, as drivers are less likely to execute a passing maneuver in a two-lane section of the corridor.

Widening of the existing pavement can be symmetric about the centerline or on one side of the roadway depending on right of way (ROW) availability and ease of construction.

Some issues to consider when designing a Super 2 project:

- Research indicates that adding passing lanes improves performance for a wide variety of traffic conditions, but the designer should not omit the consideration of other treatments, such as adding capacity to the roadway section or adding auxiliary turning lanes. As with any improvement project, designers should select the treatment most appropriate for conditions.
- Existing ROW width considerations must be analyzed to determine feasibility of upgrading to a Super 2.

- 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 miles, even if additional length is provided. More frequent passing lanes result in reduced delay as compared to longer passing lanes.
- There is not a "maximum" spacing for passing lanes; isolated passing lanes and passing lanes that are a large distance apart still provide some benefit to operations where other opportunities for passing are limited.
- Where practical, avoid substantial traffic generators such as state highways or high-volume county roads or driveways within passing lanes, or consider providing ~~a left turn~~ an auxiliary lane (for left turns or right turns, as applicable) if ~~a significant~~ the traffic generator falls within the limits of a ~~Super 2~~ passing lane.
- Consider providing full shoulders (8'–10') in areas with high driveway density.
- The location of large drainage structures and bridges should be evaluated when considering the placement of passing lanes.
- Consider providing the passing lane in the direction leaving an incorporated area for potential platoons generated in the urban area.

Additional considerations for terminating a passing lane include:

- Avoid terminating passing lanes on significant uphill grades. Evaluate traffic operations, including truck volumes, if consideration is given to terminating passing lanes on significant uphill grades. Coordinate passing lanes with climbing lane needs to improve operating characteristics.
- Avoid closing a passing lane over a hill or around a horizontal curve where the pavement surface at the end of the taper ~~isn't~~ is not visible from the beginning of the taper.
- 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.

- Allow adequate distance (recommend stopping sight distance) between the end of a lane closure taper and a constraint such as metal beam guard fence, a narrow structure, or major traffic generator.
- ~~Consider providing the passing lane in the direction leaving an incorporated area for potential platoons generated in the urban area.~~

Design Criteria

Recommended design values are shown in Table 4-6.

Table 4-6. Design Criteria.

	Minimum	Desirable
Design Speed	See Table 4-2	
Horizontal Clearance	See Table 4-2	
Lane Width	11 ft	12 ft
Shoulder Width	3 ft [a]	8–10 ft
Passing Lane Length	1 mi	1.5–2 mi [b]

[a] Where ROW is limited
[b] Longer passing lanes are acceptable, particularly for ADT ≥ 10,000, but ~~not recommended~~ lengths more than 4 miles are not recommended. Consider switching the direction if more than 4 miles.

The length for opening a passing lane (Figure 4-1), should be based on the following:

L= WS/2

Where:

L= Length of taper,

W=Lane width, and

S= Posted speed.

The taper length for closing a passing lane (Figure 4-1) should be based on:

L = WS,

Where

L= Length of taper,

W=Lane width, and

S= Posted speed.

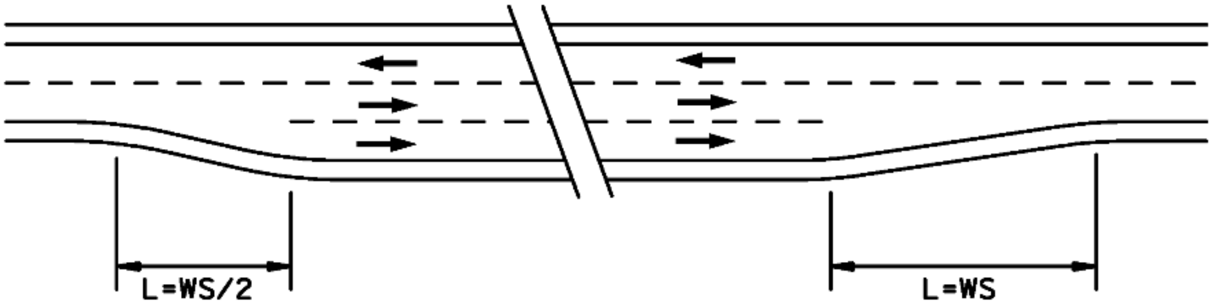


Figure 4-1. Opening and Closing a Passing Lane.

When switching the passing lane from one direction to another (closing the passing lane in each direction), provide a taper length from each direction based on **L = WS**, with a minimum 50 ft buffer (stopping sight distance (SSD) desirable) between them (Figure 4-2).

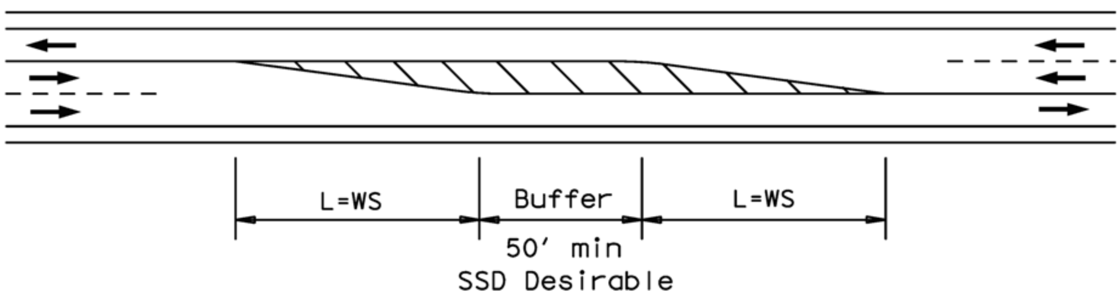


Figure 4-2. Closing the Passing Lane from One Direction to Another.

When opening a passing lane in each direction (Figure 4-3), provide a taper length based on L= WS/2.

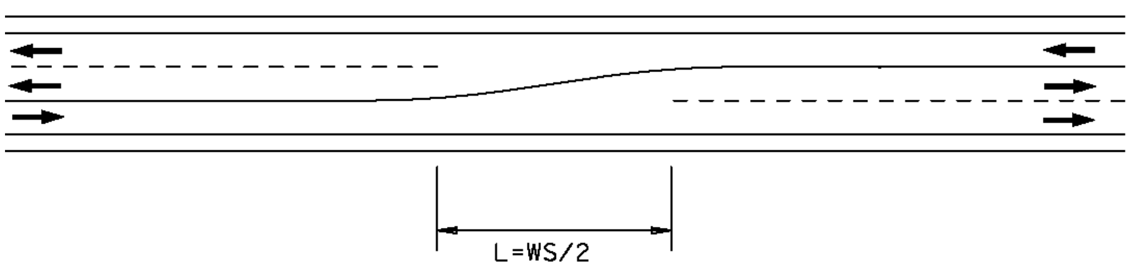


Figure 4-3. Opening the Passing Lane from One Direction to Another.

When widening to the outside of the roadway to provide a passing lane opportunity (Figure 4-4), provide an opening taper length based on L = WS/2 and a closing taper length based on L=WS.

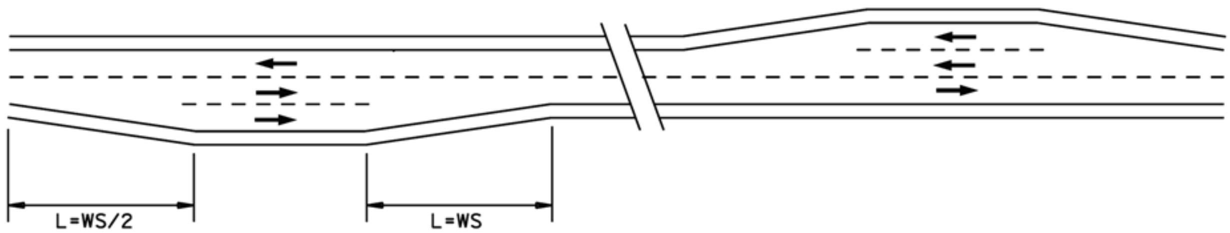


Figure 4-4. Separated Passing Lanes with Widening to the Outside of Roadway.

Passing lanes in each direction may ~~overlap~~ be installed side-by-side if ROW is sufficient (Figure 4-5). Provide an opening taper length based on L = WS/2 and a closing taper length based on L=WS.

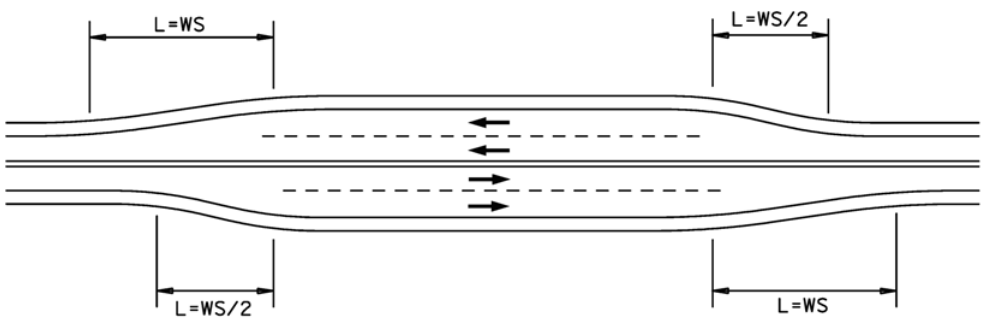


Figure 4-5. Side-by-Side Passing Lanes.

APPENDIX A
COMPARISON OF PERFORMANCE FOR DIFFERENT PASSING LANE CONFIGURATIONS

This appendix displays results of the comparison of performance measures under varying passing lane configurations. In the 10-mile simulation corridor, the number of passing lanes was set at zero, three, or six, and the length of passing lanes was set at 1 mile, 2 miles, or 3 miles. Researchers also had two values for truck percentage—10 and 20 percent—and two types of terrain—level and rolling. Figure A-1 shows performance measures resulting from simulation with an ADT of 3000. Figures A-2 through A-6 shows the same measures with increasing ADT, at an interval of 2333, up to an ADT of 14,667 in Figure A-6.

Percent Time Spent Following | Average Total Delay (min/veh)

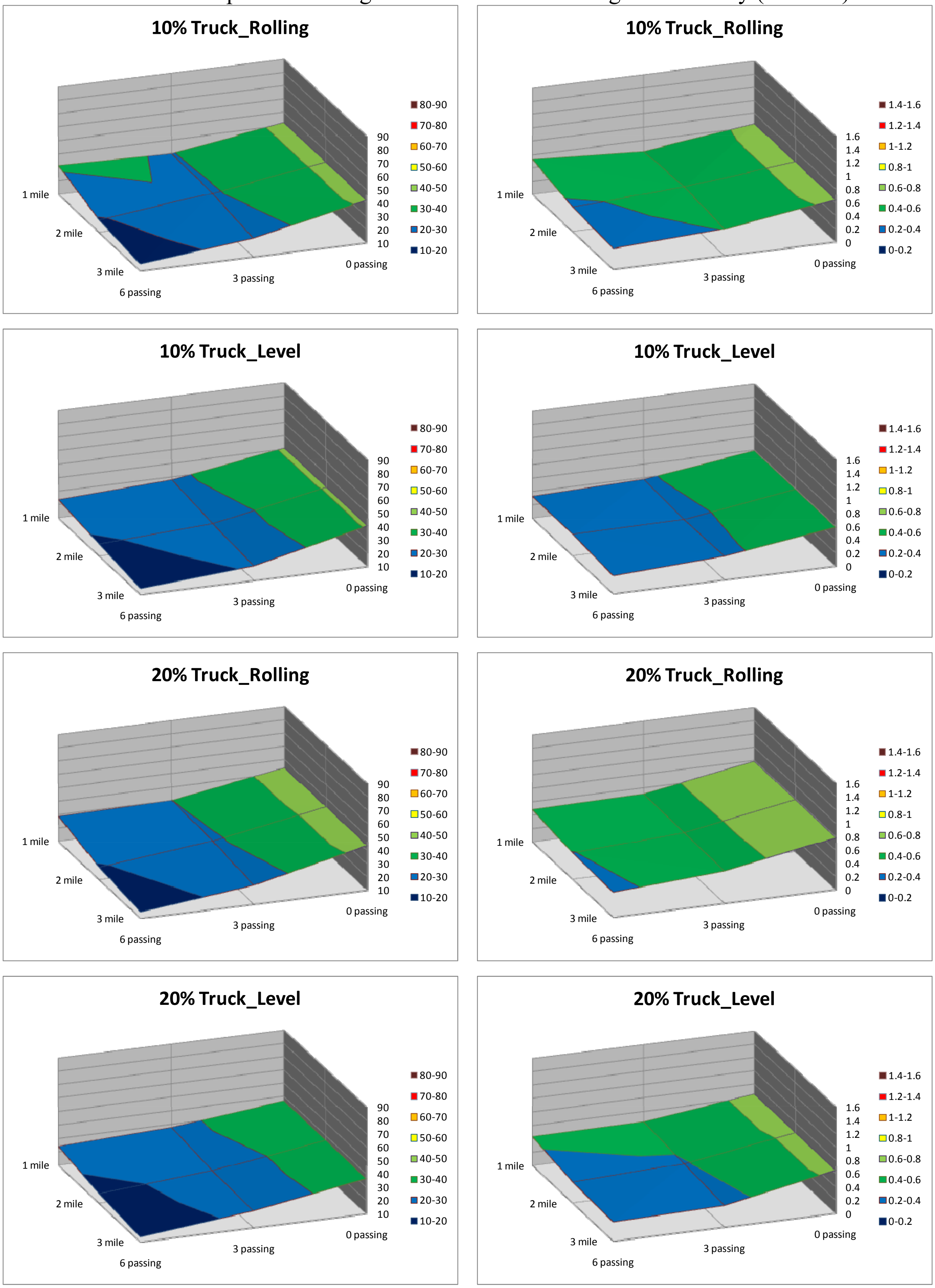


Figure A-1. Performance Measures for Different Passing Lane Configurations – 3000 ADT Scenarios.

Percent Time Spent Following | Average Total Delay (min/veh)

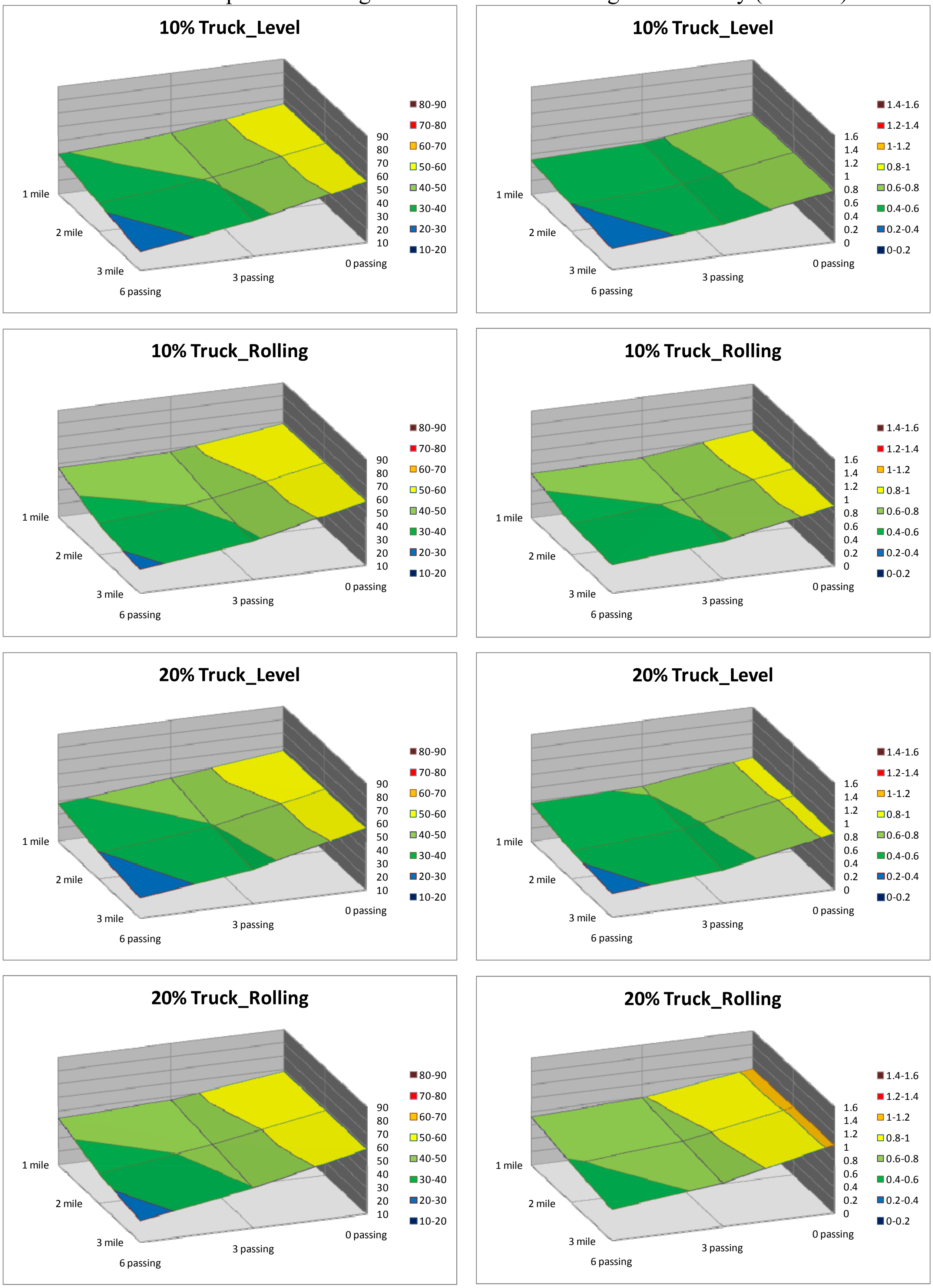


Figure A-2. Performance Measures for Different Passing Lane Configurations – 5333 ADT Scenarios.

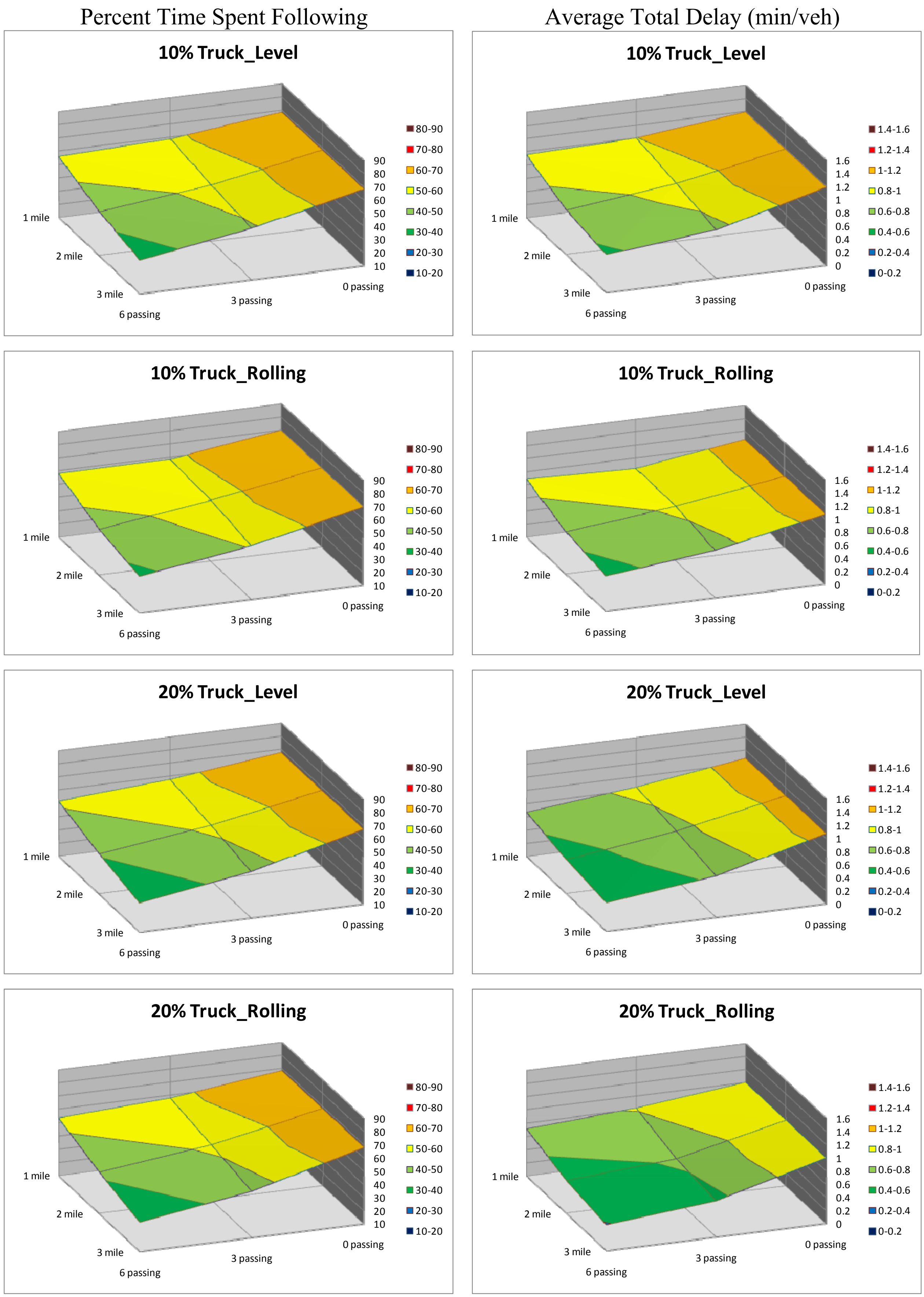


Figure A-3. Performance Measures for Different Passing Lane Configurations – 7667 ADT Scenarios.

Percent Time Spent Following | Average Total Delay (min/veh)

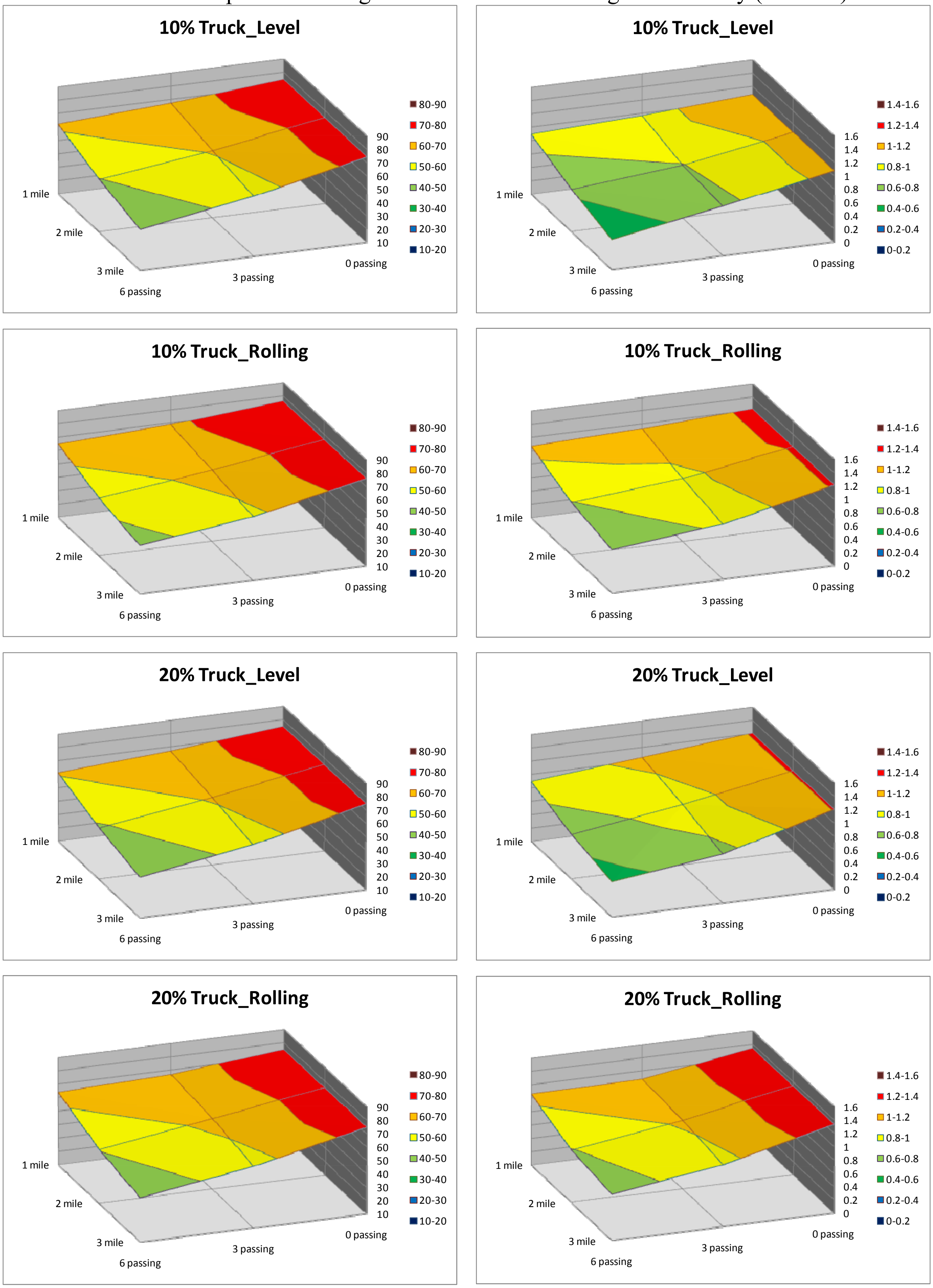


Figure A-4. Performance Measures for Different Passing Lane Configurations – 10,000 ADT Scenarios.

Percent Time Spent Following | Average Total Delay (min/veh)

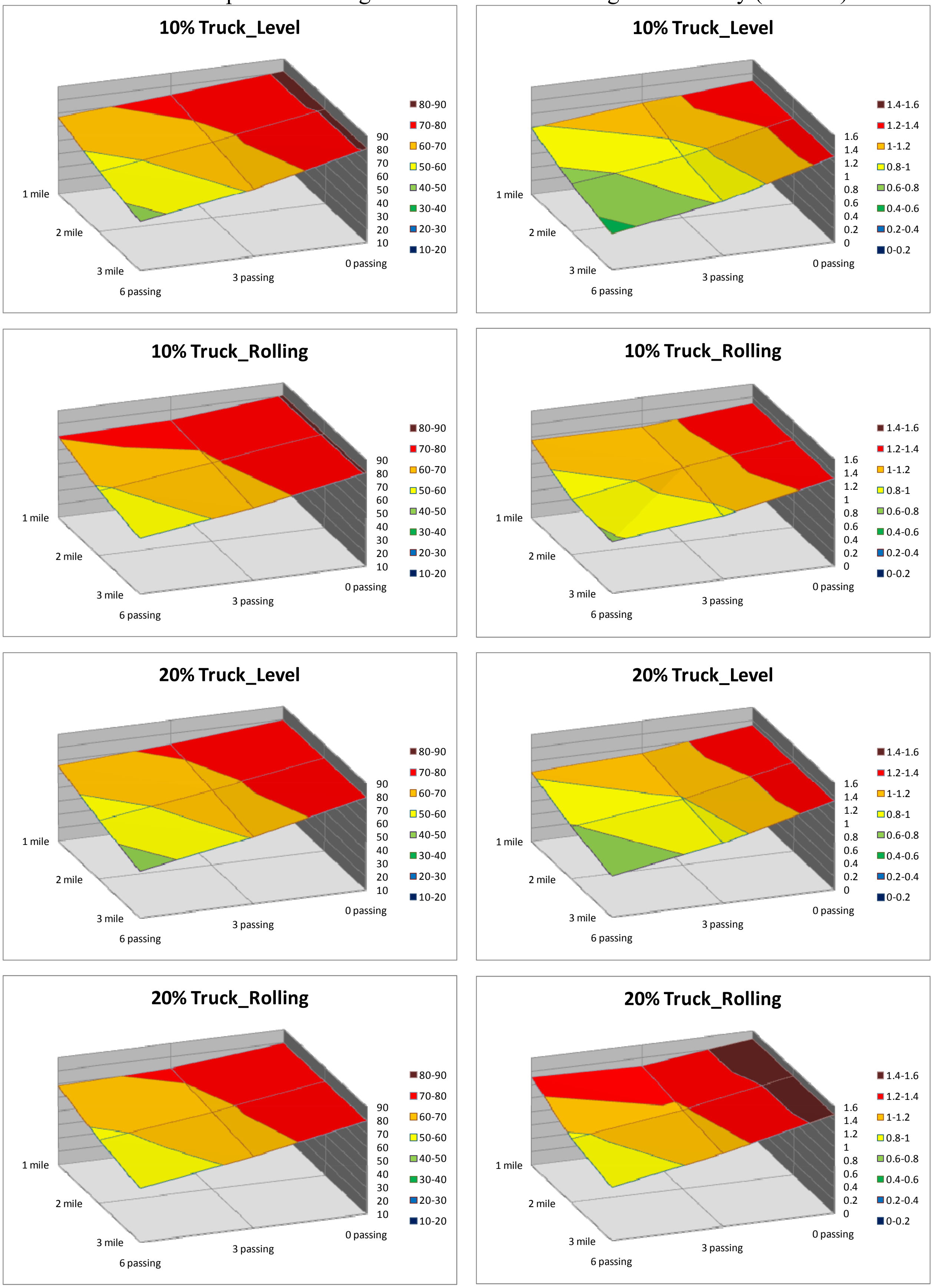


Figure A-5. Performance Measures for Different Passing Lane Configurations – 12,333 ADT Scenarios.

Percent Time Spent Following

Average Total Delay (min/veh)

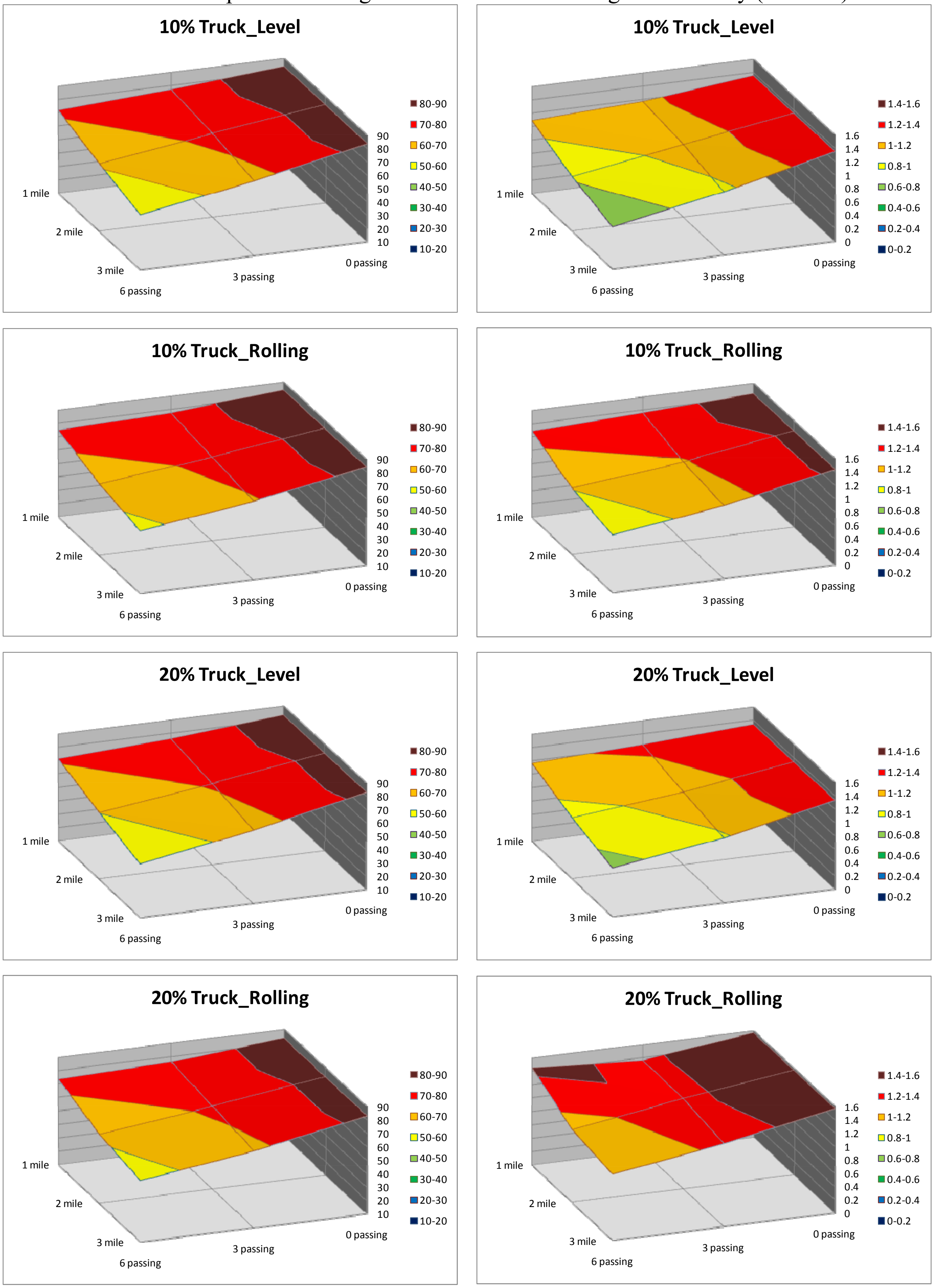


Figure A-6. Performance Measures for Different Passing Lane Configurations – 14,667 ADT Scenarios.

APPENDIX B
COMPARISON OF PERFORMANCE FOR DIFFERENT TRAFFIC AND GEOMETRIC COMBINATIONS

This appendix displays results of the comparison of performance measures under varying traffic volumes and geometric configurations. In the 10-mile simulation corridor, the number of passing lanes was set at zero, three, or six, and the length of passing lanes was set at 1 mile, 2 miles, or 3 miles. The ADT varied from 3000 to 14,667, with an increment of 2333. Researchers also had two values for truck percentage—10 and 20 percent—and two types of terrain—level and rolling. Figure B-1 shows performance measures resulting from simulation with zero passing lanes in the 10-mile corridor. Figures B-2 and B-3 shows the same measures for a corridor with three passing lanes and six passing lanes, respectively.

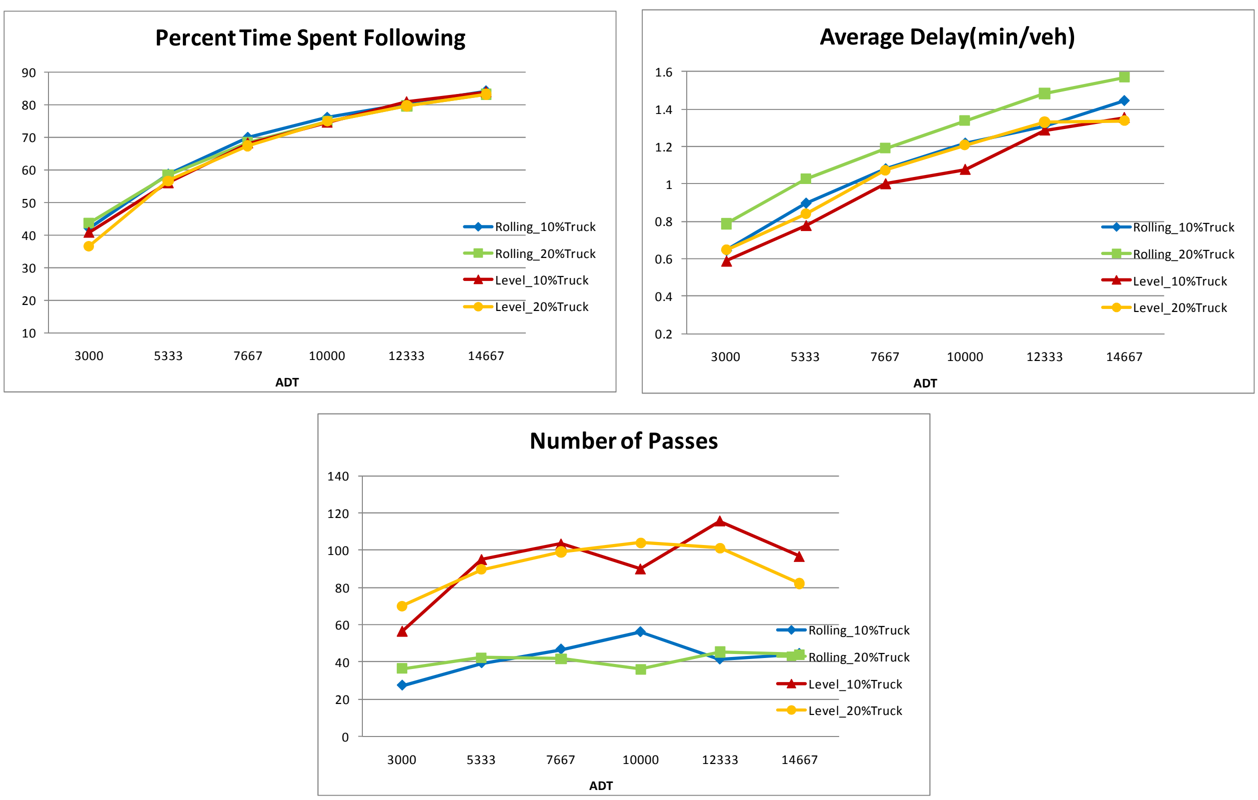


Figure B-1. Performance Measures for Various Geometric and Traffic Conditions – No Passing Lanes.

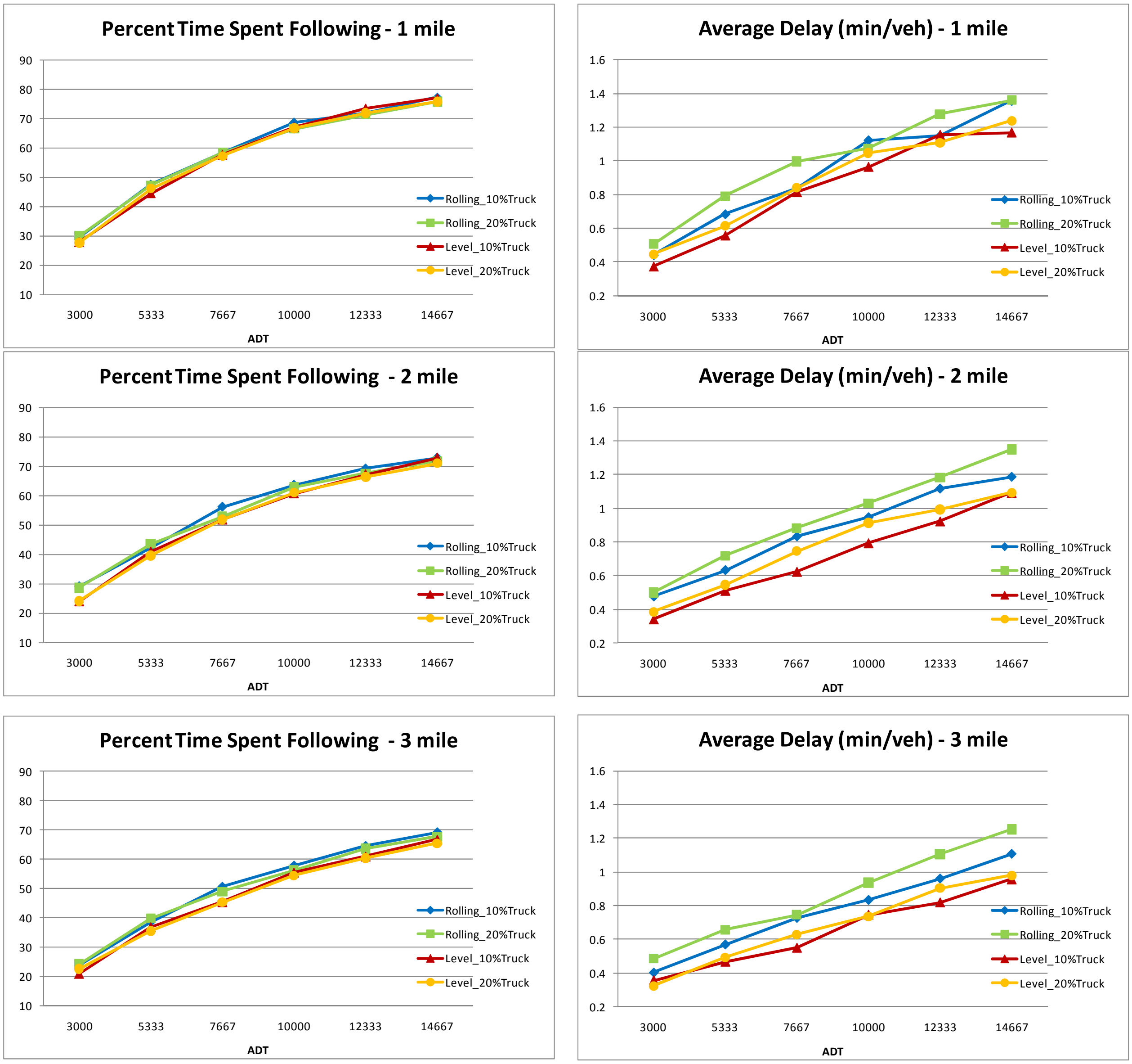


Figure B-2. Performance Measures for Various Geometric and Traffic Conditions – Three Passing Lanes.

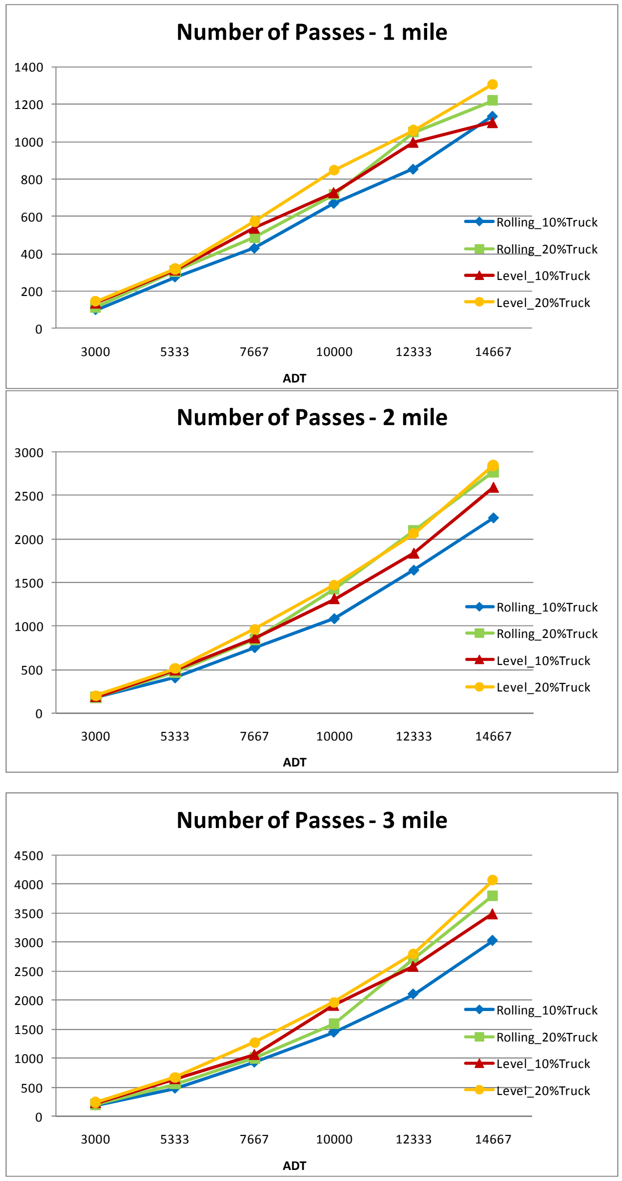


Figure B-2. Performance Measures for Various Geometric and Traffic Conditions – Three Passing Lanes (continued).

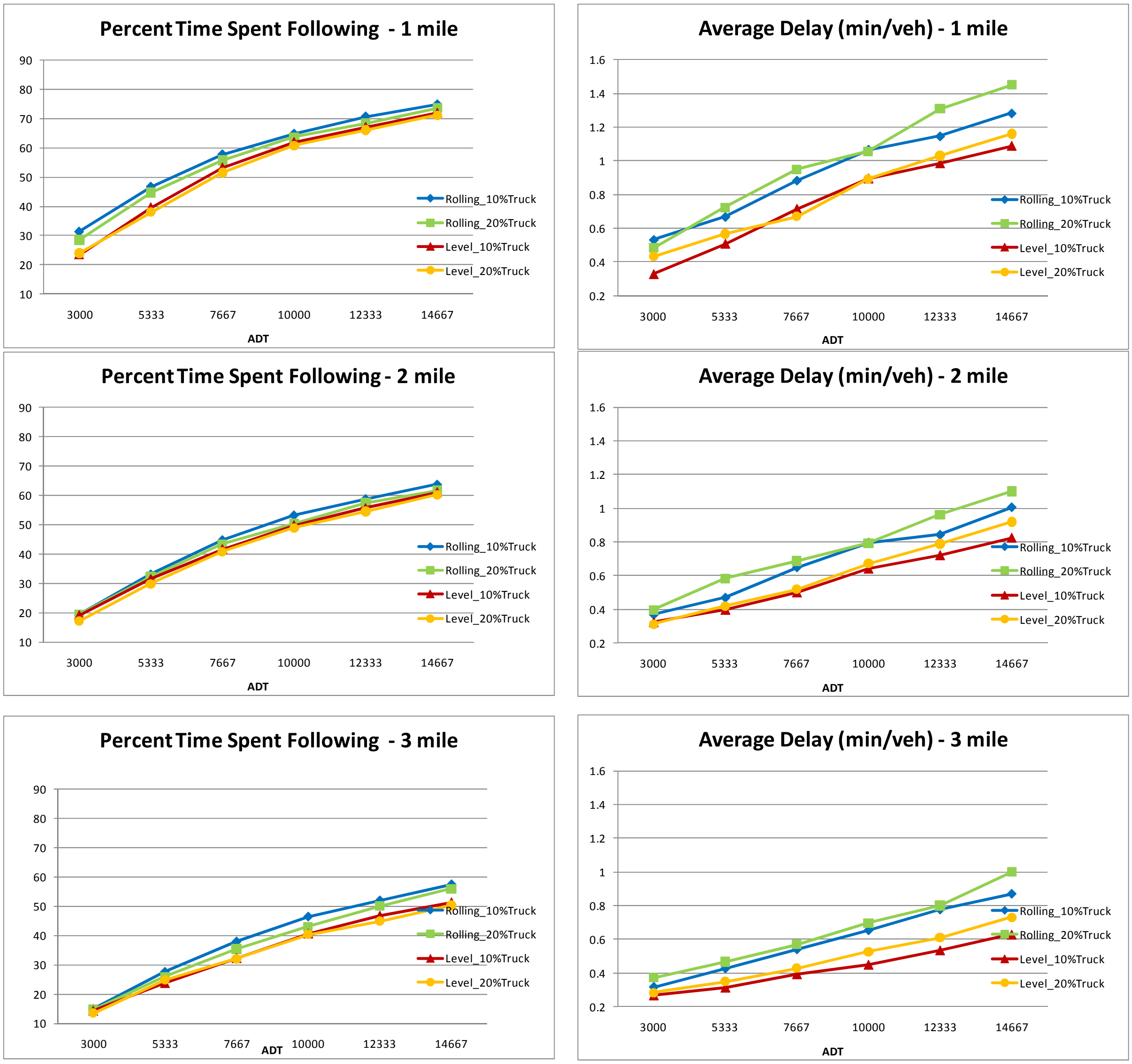
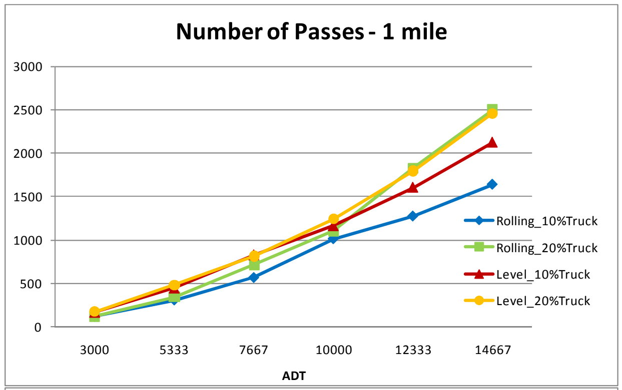
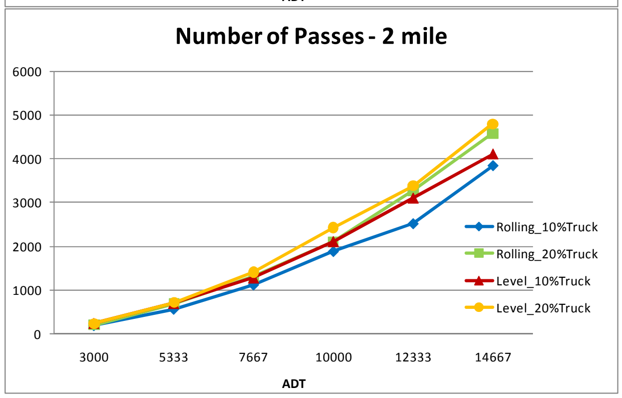


Figure B-3. Performance Measures for Various Geometric and Traffic Conditions – Six Passing Lanes.





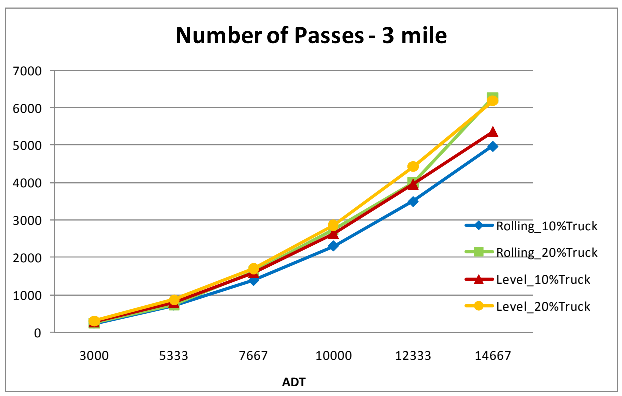


Figure B-3. Performance Measures for Various Geometric and Traffic Conditions – Six Passing Lanes (continued).

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