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Final Report

REVIEW OF THE EFFECTIVENESS, LOCATION, DESIGN, AND SAFETY OF PASSING LANES IN KANSAS

Madaniyo Mutabazi
Eugene R. Russell
Robert W. Stokes

Kansas State University
Manhattan, Kansas

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16 Abstract

Existing passing lanes in place on Kansas highways were studied from an operational and safety perspective. It was found that they generally operated well, improved operational efficiency and were well liked by the public.

Determination of highway segments that would need passing lane(s) to improve their operational performance should be accomplished in a two-level process; i.e., Network and Project Level. At the Network Level, two-lane rural highway segments that operate at a level-of-service below a predefined acceptable level are identified. At the Project Level, highway segments identified at the network level are ranked for the purpose of prioritization. The number of highway segment passing lane projects to be implemented will depend on the funding level.

At the project level, a detailed economic analysis of different passing lane lengths, spacing, and configurations can be undertaken to set parameters with an objective of minimizing percent time delay. Computer simulation using TWOPAS is a valuable tool to use at this level.

The location of passing lanes should be planned along with their spacing. The location guidelines are based in part on the results of field studies, engineering judgement, and “common sense.” The location guidelines can be grouped into four main considerations: safety, improved traffic performance, a design consistent with driver’s expectation, and minimized construction costs.

Spacing between any two successive passing lanes is intended to make the passing lanes function as a coordinated system. The effective length of a passing lane depends on traffic volume and composition, passing lane length, and downstream passing opportunities.

The report also addresses the geometric elements relating to passing lanes, including lane and taper length and cross-sections, and pavement marking and signing.

Passing lanes in Kansas are too new to make any statistically significant conclusions regarding safety. However, available statistics show a downward trend in crashes and subjectively, the authors believe that improved operational efficiency should provide safer traffic flow.

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By

Madaniyo Mutabazi
Research Associate

Eugene R. Russell
Professor

and

Robert W. Stokes
Associate Professor

Kansas State University
Department of Civil Engineering
Seaton Hall
Manhattan, KS 66506-2905

Prepared for

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Steve King
KDOT Monitor

FINAL REPORT

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PREFACE

This research project was funded by the Kansas Department of Transportation K-TRAN research program and the Mid-America Transportation Center (MATC). The Kansas Transportation Research and New-Developments (K-TRAN) Research Program is an ongoing, cooperative and comprehensive research program addressing transportation needs of the State of Kansas utilizing academic and research resources from the Kansas Department of Transportation, Kansas State University and the University of Kansas. The projects included in the research program are jointly developed by transportation professionals in KDOT and the universities.

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ACKNOWLEDGMENTS

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Last but not least, special thanks to Douglas Harwood of Midwest Research Institute in Kansas City, Missouri for his assistance on providing guidance on how to run the TWOPAS model.

EXECUTIVE SUMMARY

DESIGN GUIDELINES FOR PASSING LANES ON TWO-LANE TWO-WAY RURAL ROADS IN KANSAS

LOCATION AND DESIGN GUIDELINES

WARRANTS

Determination of highway segments that would need passing lane(s) to improve their operational performance should be accomplished in a two-level process; i.e., Network, and Project level.

At the Network Level, two-lane rural highway segments that operate at a level-of-service below a predefined acceptable level are identified. At the Project level, highway segments identified at the network level are ranked for the purpose of prioritization. The number of highway segment passing lane projects to be implemented will depend on the funding level. The Highway Capacity Manual (HCM) level-of-service procedures for rural two-lane highways were used to develop Average Annual Daily Traffic (AADT) levels at which passing lanes should be considered at the Network level. Warrants for passing lanes shown in Tables S1 and S2 were constructed using HCM procedures.

At the project level, a detailed economic analysis of different passing lane length, spacing, and configuration can be undertaken to set these parameters with an objective of minimizing percent time delay. Computer simulation using TWOPAS is a valuable tool to use at this level, supplemented with spacing, location, and configuration guidelines presented in the following sub-sections.

Table S1: Suggested Minimum AADT for Rural Two-Lane Highways for Level of Service (LOS) B and C in Level Terrain that would Warrant Passing Lane(s)

		Projected Design Year AADT									
% Trucks		**10**		**15**		**20**		**30**		**40**	
LOS		**B**	**C**	**B**	**C**	**B**	**C**	**B**	**C**	**B**	**C**
% “no-passing” zones	**0%**	3900	6200	3700	5890	3520	5600	3210	5110	2950	4690
	20%	3460	5630	3290	5340	3130	5080	2850	4630	2620	4260
	40%	3030	5190	2880	4930	2740	4690	2500	4280	2290	3930
	60%	2740	4900	2600	4660	2480	4430	2260	4040	2080	3710
	80%	2450	4760	2330	4520	2220	4300	2020	3920	1860	3600
	100%	2310	4620	2190	4380	2090	4180	1900	3800	1750	3490

Assumptions: K=0.15, directional split = 60/40, PHF=0.92, Lane width 12 ft, shoulder width 6ft

Table S2: Suggested Minimum AADT for Rural Two-Lane Highways for Level of Service (LOS) B and C in Rolling Terrain that would Warrant Passing Lane(s)

		Projected Design Year AADT									
% Trucks		**10**		**15**		**20**		**30**		**40**	
LOS		**B**	**C**	**B**	**C**	**B**	**C**	**B**	**C**	**B**	**C**
% “no-passing” zones	**0%**	3000	4850	2630	4240	2340	3770	1910	3090	1620	2610
	20%	2660	4500	2320	3940	2070	3500	1690	2870	1430	2430
	40%	2190	4040	1920	3540	1710	3140	1400	2570	1180	2180
	60%	1960	3690	1720	3230	1530	2870	1250	2350	1060	1990
	80%	1730	3460	1520	3030	1350	2690	1100	2210	940	1670
	100%	1500	3230	1320	2830	1170	2520	960	2060	810	1740

Assumptions: K=0.15, directional split = 60/40, PHF=0.92, Lane width 12 ft, shoulder width 6ft

SPACING

Spacing between any two successive passing lanes is intended to make the passing lanes function as a coordinated system. Two approaches for spacing of passing lanes are suggested:

ž The first approach is subjective and relies on intuition and “common sense” from a traffic operations point of view. Initial spacing is at a large spacing of, say 10-15 km (6.25-9.375 miles), then at a later stage when volumes have increased or when additional improvement is needed, intermediate passing lanes are constructed at 3-5 km (1.875-3.125 miles) spacing.

ž The second approach is more objective and uses the concept of the "effective length" of a passing lane. The effective length is the length of the passing lane plus the distance downstream to the point where traffic conditions return to those that existed before the vehicles entered the passing lane. (Refer to Figure S1). It is suggested that the distance to the next passing lane should be equal to the effective length of the preceding passing lane. The effective length of a passing lane ranges from 4.8-12.8 km (3-8 miles) depending on traffic volume and composition, passing lane length, and downstream passing opportunities. For most cases, effective length can be estimated from Figure S2, with adjustments for factors, which might hasten or slow the downstream overtaking or catch-up process.

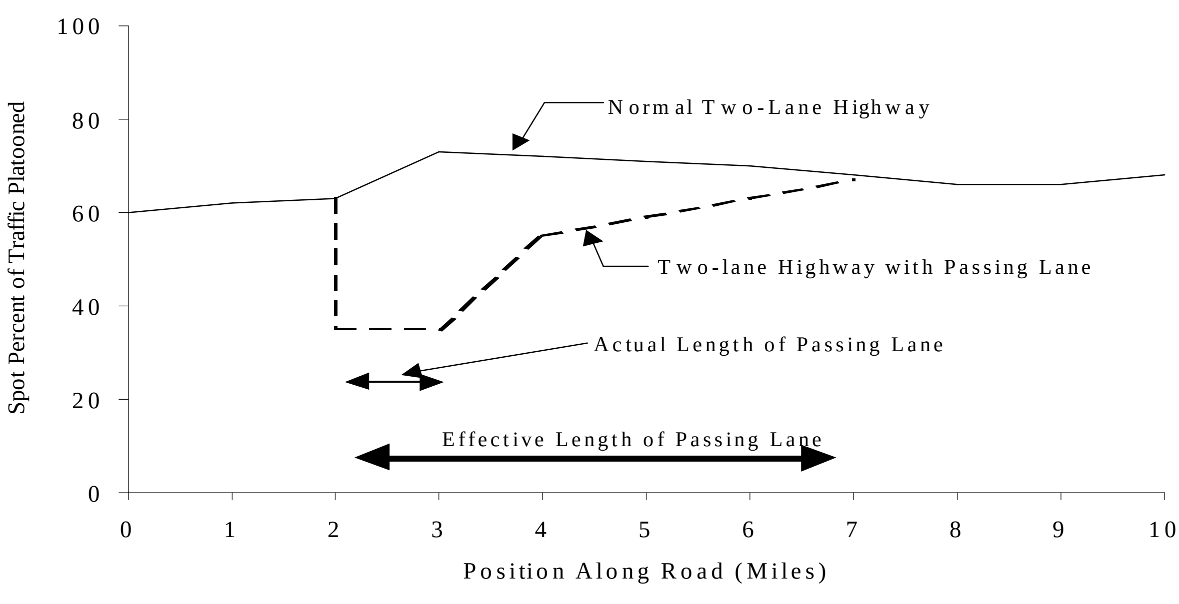
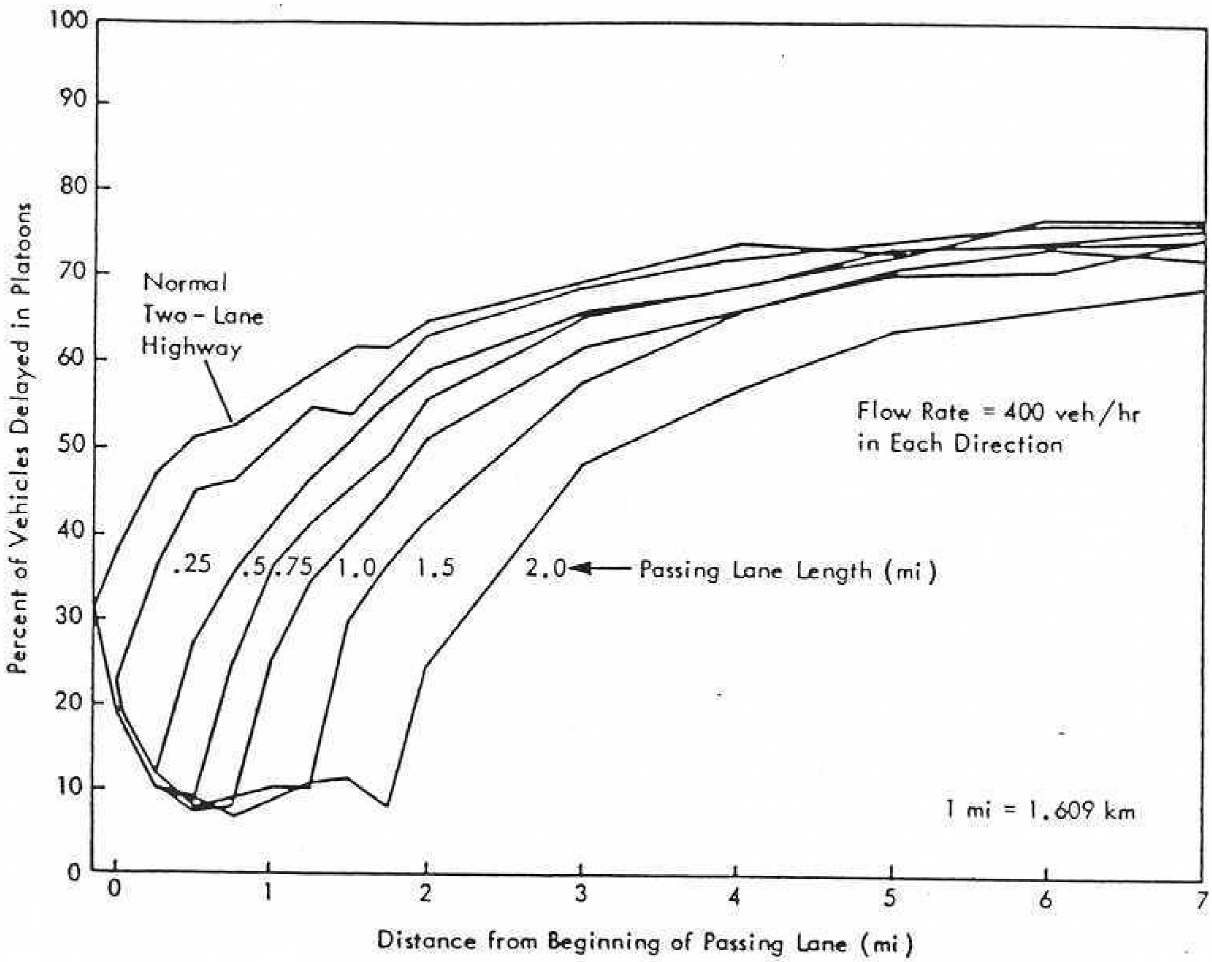
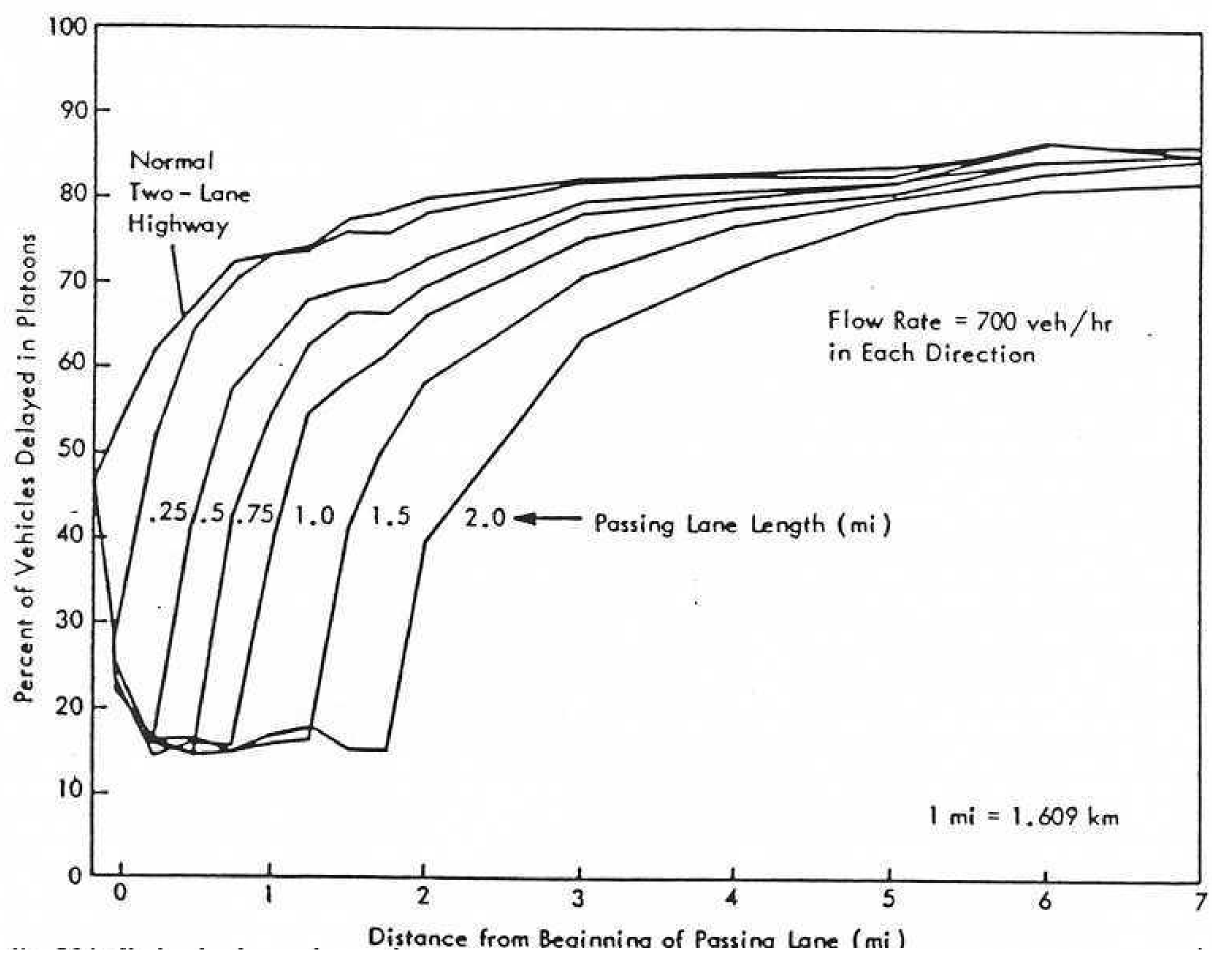


Figure S1: Effective Length of a Typical Passing Lane


Percent of Vehicles Delayed in Platoons
Distance from Beginning of Passing Lane (mi)
Normal Two - Lane Highway
Flow Rate = 400 veh/hr in Each Direction
.25 .5 .75 1.0 1.5 2.0 Passing Lane Length (mi)
1 mi = 1.609 km
0 10 20 30 40 50 60 70 80 90 100
0 1 2 3 4 5 6 7


Percent of Vehicles Delayed in Platoons
Distance from Beginning of Passing Lane (mi)
Normal Two - Lane Highway
Flow Rate = 700 veh/hr in Each Direction
.25 .5 .75 1.0 1.5 2.0 Passing Lane Length (mi)
1 mi = 1.609 km
0 10 20 30 40 50 60 70 80 90 100
0 1 2 3 4 5 6 7

LOCATION

The location of passing lanes should be planned along with their spacing. The location guidelines presented in these guidelines are based in part on the results of field studies, engineering judgement, and “common sense.” These location guidelines can be grouped into four main considerations: 1) those that address safety, 2) those that should improve traffic performance, 3) those that result in a design consistent with drivers expectation, and 4) those that minimize construction costs.

Safety:

ž Side road or driveway intersections should be avoided if feasible within and immediately after the passing lane section. High-volume side road intersections should be avoided within and immediately after the passing lane section. (A high-volume side road intersection is defined as an intersection where left-turn volume from a main highway without a passing lane would warrant a separate left-turn lane.)

ž Where a low-volume side road or driveway intersection cannot be reasonably avoided, it should be located close to the middle of the passing lane rather than in lane-drop, lane-addition, or immediately after the passing lane areas.

ž A minimum sight distance of 303 m (1,000 ft) should be available at the lane-drop and lane-addition tapers.

ž All passing lane locations should meet or exceed prevailing geometeric design criteria after the passing lane is constructed.

Traffic Performance:

- Select locations where there have been limited passing opportunities for approximately two miles or where a field study shows substantial platooning;
- Passing lanes can be located on a downgrade or an upgrade, where there is significant speed differential, and climbing lanes are not warranted; i.e., they can be effective even where speed difference is significant;
- Consider passing lanes on level terrain where a field study shows a platooning up;
- Locate passing lanes leading away from rather than into areas of traffic congestion. If the passing lane preceeds the area of traffic congestion, the congestion area should be beyond the effective length of the passing lane determined from Figure S2. Traffic congestion areas includes sections with significant no-passing zones, in communities where the speed limit of the highway is reduced, etc. Locating passing lanes upstream of congestion will diminish the benefits (which normally extends some distance downstream of the physical location of a passing lane) gained at the passing lane; (Refer to Figure S1)
- Locate passing lanes leading away from rather than into points where a significant number of vehicles may end their trips or leave the highway system, such points includes urban areas, major intersections, recreation areas, etc.
- Avoid locations in proximity to four-lane sections. Locations near upstream of the four-lane section will reduce the benefits gained at the passing lane. Locations near downstream of the four-lane location will make the passing lane less effective because of lack of demand for passing in the traffic stream as it reaches the passing lane;
- If passing lanes must be started on a horizontal curve, it is generally desirable to start passing lanes on a horizontal curve to the left rather than to the right because it directs traffic to the

outside lane and hence improves operational effectiveness of a passing lane, unless the curve is very flat.

ž If available, use traffic, computer simulation, such as the TWOPAS program to choose locations to minimize percent time delay. The TWOPAS computer program is in public domain and can be obtained from Federal Highway Administration (FHWA) by contacting them at the following contact information:

Highway Research Engineer
FHWA T303
Tuner-Fairbank Hwy. Research Ctr.
6300 Georgetown Pike
McLean, VA 221-1-2296
Telephone Number: (202) 493-3318.

Drivers' Expectation:

ž Select locations that appear logical to the driver; i.e. on or immediately after restricted passing opportunities causes undesirable platoon buildup, such as on a segment with 100 percent no passing. The length of the segment will be a function of traffic volume, passing opportunities and should be determined by field observation. However, about two miles is suggested in the literature.

Construction Costs:

ž Consideration should be given to costly physical constraints such as bridges, culverts, deep cuts and high fills;

ž to reduce construction unit cost, consider passing lanes as part of any other planned highway reconstruction.

CONFIGURATION

Any of the nine possible different configurations shown in Figure S3 could be considered for site specific conditions.

- The isolated case *a* is usually used to reduce delays at a specific one direction bottleneck;
- Configuration *e* (adjoining head-to-head) is unfavorable from a safety stand point due to the merging areas being opposite each other;
- Type *f* and *g* are sometimes appropriate where sufficient width for passing lanes is available. For highways where double lines are used within a passing lane, drivers may feel unduly constrained when passing is prohibited on the other 50 percent of the road length if sight distance is good and traffic is low;
- Overlapping type *h* may be used in sags where there are upgrades in both directions;
- Overlapping types *i* are often used at crests where passing or climbing lane is provided on each upgrade;
- This study found that side-by-side type *j* used in Kansas work well, and nothing found in this study that would suggest any of the others are better than side-by-side for the conditions studied.

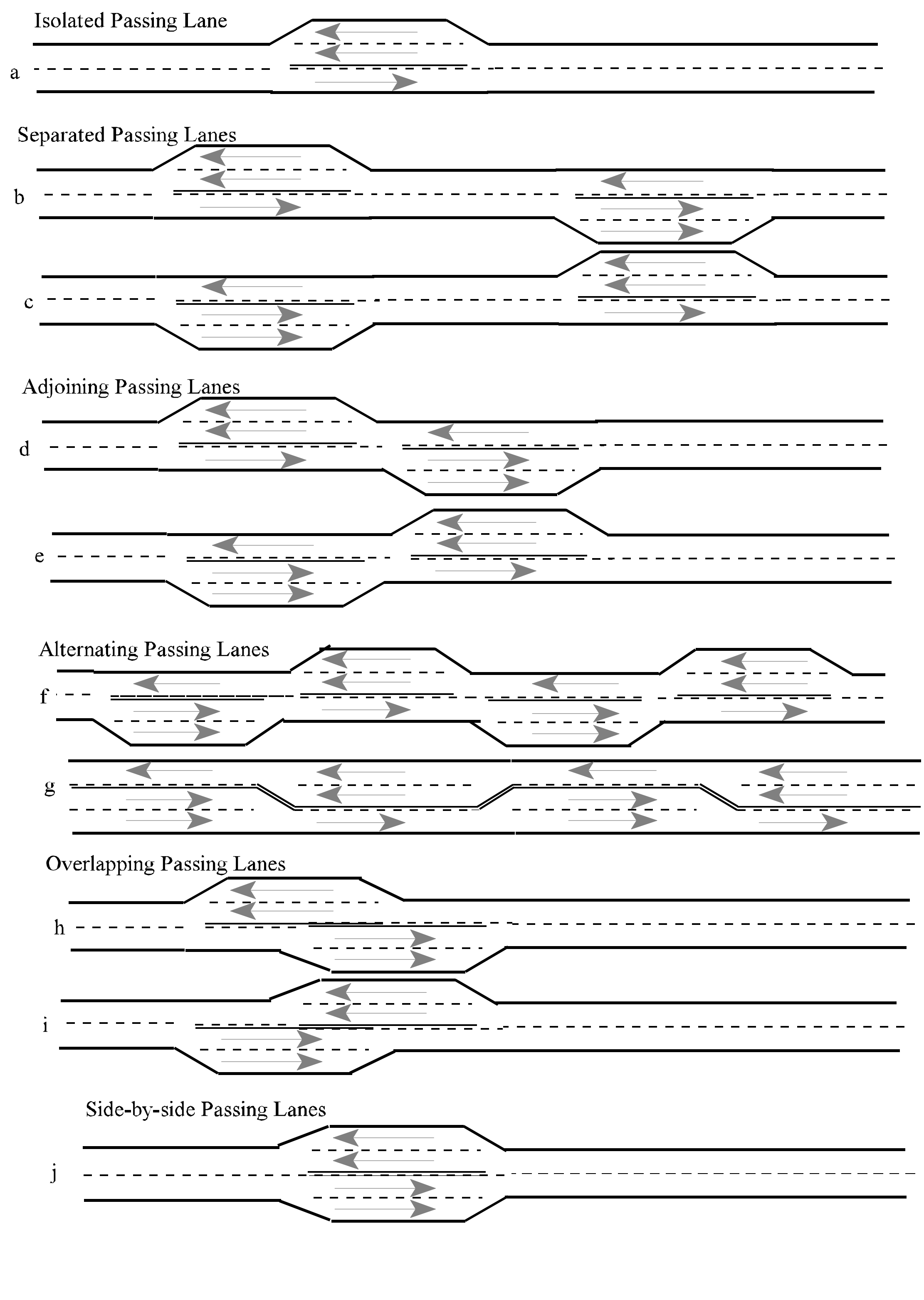


Figure S3: Passing Lane Configurations**.**

GEOMETRIC ELEMENTS

The geometric elements provided in this guideline includes lane and taper lengths and cross-section.

Length

The optimum length, (without tapers) should be 0.8 to 1.6 km (0.5 to 1.0 miles), with a minimum length of 0.8 km (0.5 miles). Table S3 presents optimum passing lane length as a function of traffic volume.

Table S3: Optimum Design Lengths for Passing Lanes

One-Way Flow Rate (Veh/hr)	Optimal Passing Lane Length	
	Miles	Kilometers
100	0.50	0.8
200	0.50 - 0.75	0.8 - 1.2
400	0.75 - 1.00	1.2 - 1.6
700	1.00 - 2.00	1.6 - 3.2

Lane-Drop

The minimum length (ft) of the lane-drop taper should be the product of the lane width (ft) and the speed limit or 85th percentile speed (mph).

Lane-Addition

The minimum length of the lane-addition taper should be two-thirds of length of the lane-drop taper.

Cross-Section

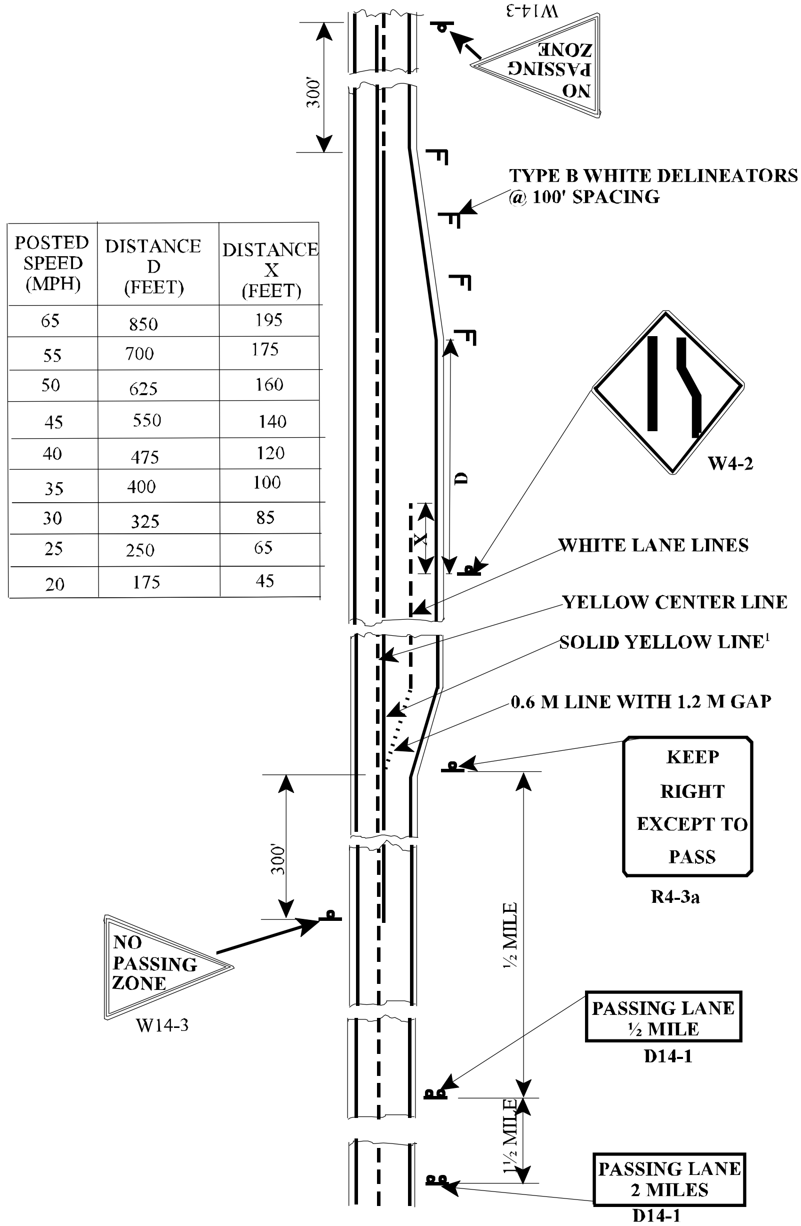
Passing lane widths should not be less than the width of the lanes in the adjoining sections, but may have reduced shoulder width with a minimum of 1.2 meters (see Table 2.4) at the passing lane section. The cross slope should be the same as the adjacent lane.

PAVEMENT MARKING AND SIGNING

Guidelines address signing and marking in the following areas: 1) advance zone; 2) lane addition zone; 3) lane drop zone; and 4) opposing lane.

Typical signing and marking for a passing lane is shown in Figure S4.

- Use pavement markings to direct traffic to the right.
- Use the sign, "Keep Right Except to Pass" at the beginning of the passing lane.
- As a minimum, advance signs should be installed at 3.2 km (2 miles) and 0.8 km (½ mile).
- Use double yellow lines to prohibit passing in the opposing lane when one-way hourly volume is greater than 700, when there are sight distance restrictions, or when side-by-side configuration is used.
- Use a symbolic merge sign at the end taper.



POSTED SPEED (MPH)	DISTANCE D (FEET)	DISTANCE X (FEET)
65	850	195
55	700	175
50	625	160
45	550	140
40	475	120
35	400	100
30	325	85
25	250	65
20	175	45

[1]*Center line is marked by double yellow line if one-way hourly volume is greater than 700 vehicles*

Figure S4: Typical Signing and Marking of a Passing Lane

Chapter 1

INTRODUCTION

1.1 BACKGROUND

A passing lane usually is a lane added in one or both directions of travel on a two-lane, two-way highway to improve passing opportunities. In essence, a passing lane works in the same way as the more common climbing lane. The major difference is that a climbing lane is provided for the purpose of passing heavy vehicles on sustained grades (grades where heavy vehicles cannot maintain normal highway speed due to their low performance characteristics), while the passing lane is provided at places other than sustained grades to facilitate the passing of all types of slower vehicles. For this report, the term “passing lane” is limited to an auxiliary lane provided on a two-lane highway in rolling or level terrain with the primary objective of increasing passing rates and/or improving traffic flow. This definition excludes climbing lanes, short 4-lane sections, turnouts, and two-way left turn lanes from being considered as passing lanes.

While some states in the United States (US) and countries like Canada, Australia, etc. have used passing lanes since the early 1980s, the Kansas Department of Transportation (KDOT) started constructing passing lanes in 1994. Currently the state road network contains only nine sections with passing lanes and another two were under construction during the period of this study. These sections are found along US 50 and US 54 highways.

1.2 PROBLEM STATEMENT

Previous studies have shown that passing lanes can improve the Level-of-Service (LOS) and safety of two-lane highways (Emoto and May 1988; Harwood and St. John 1985; Morrall and Blight 1984; Staba et al. 1991; Taylor and Jain 1991). The public and some Kansas legislators have requested more extensive use of passing lanes, and the 1990-1997 Kansas Comprehensive Highway Program (KDOT 1996) identified selected routes as possibly needing passing lanes. Since the performance and effectiveness of passing lanes depends on their location, design, construction, signing, and familiarity to drivers, there is a need to evaluate the effectiveness of the state's existing passing lanes in order to provide guidelines for future planning, design and construction decisions.

Crossroads within passing lane sections may create potential safety problems since crossing traffic is required to cross four lanes of traffic at current side-by-side passing lane sections, instead of crossing two lanes at intersections located at standard two-lane highway sections. KDOT is particularly interested in the effect of crossroads and the extent to which they need to be taken into account in the guidelines.

Lane-addition and lane-drop sections of the passing lanes are usually thought to have a negative impact on safety. The length of lane-addition and lane-drop sections depends on anticipated highway speed. Passing lane geometric elements of the existing passing lanes were designed to operate at the 55 mph (90 km/hr, using KDOT conversion) highway speed limit prevailing on those highways before 1996. With the rise of the speed limit across the state in 1996, to 60 or 65 mph (100 or 110 km/hr), some of these elements might not be compatible with the current speed limits, hence creating safety concerns. There is a need to evaluate the safety of these elements.

1.3 STUDY OBJECTIVES

The objectives of this research are to evaluate the effectiveness of existing passing lanes in Kansas from an operational, safety, and public perception standpoint, and to provide recommendations for improvements where appropriate. This study provides information for locating, planning, designing, constructing and signing passing lanes in a way that maximizes their safety and efficiency.

1.4 STUDY METHODOLOGY

The study methodology included: 1) conducting a literature review of current practices to identify previous relevant research and the experience with passing lanes in other states and countries; 2) conducting field studies to assess the operational effectiveness and safety of existing passing lanes in Kansas; 3) evaluating accident data to determine the effectiveness of passing lanes on highway safety; 4) conducting postcard surveys of drivers to assess public opinion on the operation and safety of passing lanes in Kansas; 5) comparing different passing lane configurations using a traffic, computer simulation model; and 6) assessing current KDOT design practices and recommendations concerning any changes that may be needed in current KDOT warrants and design standards.

1.5 ORGANIZATION OF REPORT

This report consists of an executive summary, six chapters, and an appendix. Executive summary presents the summary of proposed design guidelines for passing lanes on two-lane two-way rural roads in Kansas. Chapter 1 gives the definition of the term "passing lane" as used in the context of this report. The problem statement, objectives of the study and the methodology used to arrive to the study objectives are given in Chapter 1. Chapter 2 presents findings from documented previous research related to passing lanes. While Chapter 3 presents the experimentation and data collection of the seven sub-studies done in this research, Chapter 4 discusses the results from those studies. The seven sub-studies are: 1) monitoring traffic behavior using a video camera, 2) taking traffic count and time headway measurements, 3) making traffic conflict observations at the lane-drop sections of several passing lanes, 4) making traffic conflict observations at several crossroad intersections, 5) analyzing accident data for the highway sections with passing lanes, 6) surveying drivers on their perception of the passing lane program in the state, and 7) comparing passing lane configurations using a computer traffic simulation model. Chapter 5 describes the development of warrants for passing lanes in Kansas, and presents the guidelines on location of passing lanes in relation to crossroad intersections. The conclusions and recommendations are presented in Chapter 6. The Appendix presents a sample of the survey card used in the drivers' survey.

Chapter 2

LITERATURE REVIEW

A literature review was conducted to assess documented benefits of passing lanes and to determine the state-of-the-art of their design, location, and signing.

2.1 OPERATIONAL PERFORMANCE MEASURES

Several studies that have tried to evaluate the effectiveness of passing lanes have used percent time delay, speed, and passing rates as major measure of effectiveness (Morrall and Blight 1984; Harwood and St. John 1985; Emoto and May 1988; and Staba et al. 1991). Percent time delay, speed and capacity utilization are used by the Highway Capacity Manual (HCM) (TRB 1994) to define Level-of-Service (LOS) for a two-lane highway. Table 2.1 summarizes the performance measures used in the above mentioned studies.

Effectiveness of passing lanes can be evaluated in two ways: 1) an evaluation to compare the effectiveness of the passing lane to a standard two-lane highway, and 2) an evaluation to measure the effect of different passing lane elements (geometry, signing, and marking). All measures of effectiveness shown in Table 2.1 can be used for the second type of evaluation; however, passing rate and lane utilization cannot be used for the first type of evaluation. Passing rate is unsuitable for comparison to standard two-lane highways for two reasons: 1) it is difficult to select comparable sections because the effect of a passing lane in the direction of travel extends downstream beyond its physical length, and this extension cannot be easily determined in the field; 2) a passing lane may affect the passes in the opposing direction in the downstream section, and a valid comparison would need to take this into account.

Lane utilization cannot be used for comparison with a standard, two-lane, highway section because it is not applicable to a standard two-lane highway.

Table 2.1: Operational Performance Measures for Evaluating Passing Lanes

Study	Measures of Effectiveness (MoE)	Suggested Major MoE
Passing Lane Research Study for the Trans-Canada Highway in Banff National Park. (Morrall and Blight 1984).	• Percent vehicles in platoon[a] • Passing rate • User opinion	None
Passing Lanes and Other Operational Improvements on Two-lane Highways. (Harwood and St. John 1985).	• Speed • Percent vehicles in platoon • Passing rate	Passing rate
Operational Evaluation of Passing Lanes. (Emoto and May 1988).	• Speed • Percent time delay[b] • Passing rate[c] • Lane utilization	Percent time delay
Development of Comprehensive Passing Lane Guidelines. (Staba et al. 1991)[d].	• Speed • Time headway • Percent vehicles in platoon • Passing rate • Lane utilization • Platoon structure	Passing rate

[a] Surrogate measure of percent time delay (TRB 1994).
[b] In the field is measured by the percent of vehicles following at headway less than five seconds.
[c] Results were inconclusive because of limited data.
[d] Four of the five study sites were climbing lanes.

2.1.1 Passing Rates

The primary objective of a passing lane is to increase the opportunity of a vehicle to pass a slower, moving vehicle. The HCM (TRB 1994) uses percent time delay as a primary criterion when evaluating the Level-of-Service on two-lane highways. The percent time delay depends on the availability of passing opportunities in both directions. Passing demands in one direction of travel depends on traffic characteristics in that direction. Wardrop (1952) showed that passing demand depends on volume and speed distribution, as shown in Equation 2.1,

$$N \quad 0.56\frac{Q^{2}\ _{s}}{v_{s}^{2}} \qquad \text{. . . . .} \qquad \mathbf{2.1}$$

Where: N = The number of desired passes per unit length per unit time,

Q = Flow in one direction (vehicles per unit time),

$_s$ = Standard error of desired space-distribution speeds, and

v_s^{2} = Space mean speed.

Equation 2.1 assumes a normal distribution of desired speeds, and one type of vehicle in the traffic stream. The normality assumption can be met only at low flow. Troutbeck (1982) expanded Wardrop's model to determine the number of passes when there is more than one type of vehicle, with each group having its own speed distribution. For a case where the traffic stream contains car and truck sub-streams, one may assess the effect of the percentage of trucks on the total number of passes. This is possible because the model can estimate four different types of passes: 1) cars passing trucks, 2) cars passing cars, 3) trucks passing cars, and 4) trucks passing trucks.

Troutbeck (1982) also noted the following limitations of both models: the inability to consider the trailing time before commencing a passing maneuver; the non-homogeneity of the roadway section; and the fact that all drivers are not traveling at their desired speed. It is also noted that these models yield only the overtaking (catch-up) rates since they don't consider the time required to complete the passing maneuver, which in turn depends on the composition of traffic. Passing time depends on both the length of the passing and passed vehicle and their performance characteristics (FHWA 1990). Table 2.2 shows the minimum length of required passing zones for various passing scenarios, considering vehicle lengths and their acceleration performance. Trucks have difficulty passing compared to passenger cars, due to their low operating characteristics such as low speed. Even where they can maintain speeds comparable to cars, their relatively larger size can create problems for the passing vehicle. The increased difficulty of passing trucks was demonstrated by McLean (1989) in Figure 2.1. The data for Figure 2.1 were derived from Australian field study results on passing behavior on two-lane roads.

Troutbeck (1982) further noted that at low traffic volumes, the time spent following behind slower vehicles is very small compared to journey time. This tends to make the models more accurate at low traffic volumes than at high traffic volume. Troutbeck (1982), for example, suggests that his model produces reasonable estimates for traffic volume up to 150 vehicles per hour. Because of these limitations, the predicted number of passes would always be higher than the actual number of passes.

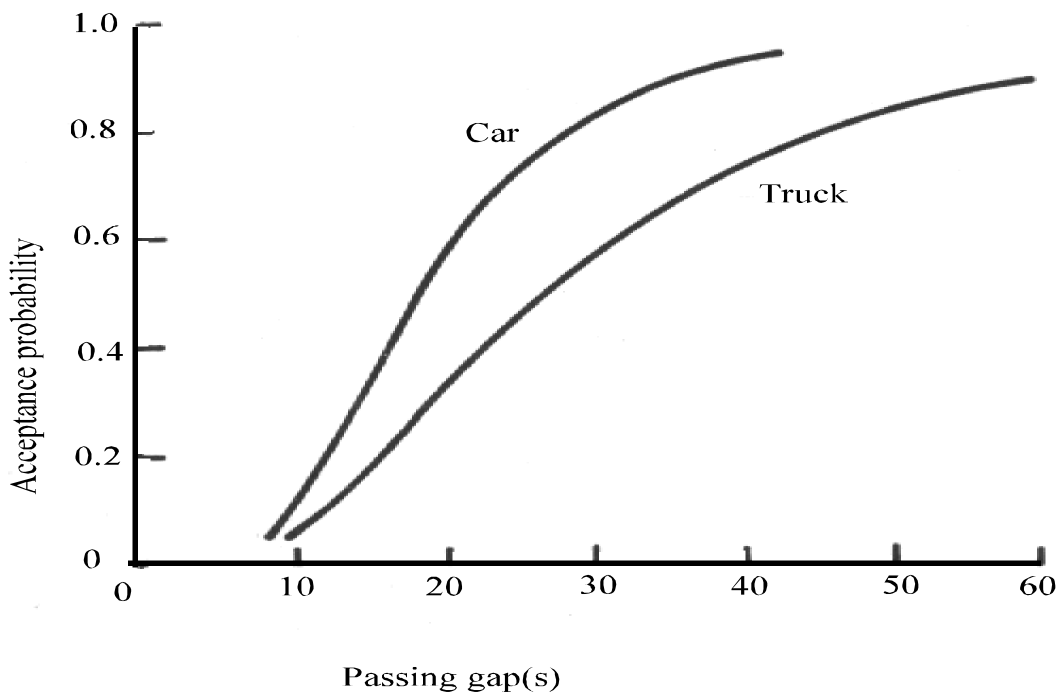
These models are likely to estimate passing rates much better at passing lane sections than at an ordinary two-lane section, because for a passing lane section, passing is not limited by the opposing

Table 2.2: Passing Zone Length Required to Complete a Pass for Various Passing Scenario

Design speed (mi/h)	Passing vehicle speed, (mi/h)	Speed difference, (mi/hr) used by passing vehicle		Minimum length of passing zone, (ft)			
		Car[a]	Truck	Car passing Car	Car passing truck	Truck passing Car	Truck passing truck
20	20	13	6.5	150	225	275	350
30	30	12	6	350	475	600	725
40	40	11	5.5	600	825	975	1,175
50	50	10	5	975	1,250	1,450	1,750
60	60	9	4.5	1,475	1,850	2,025	2,450
70	70	8	4	2,175	2,650	2,900	3,400

[a] Passenger car

Source: Federal Highway Administration (1990).



Source: McLean (1989)

Figure 2.1: Acceptance Probability for Passing Gaps for Cars Passing Cars and Cars Passing Trucks.

traffic as it is on an ordinary section. Yet models may not provide good results on passing lanes because existing passing lanes are likely to be on high-volume roads where these models are less accurate. During this study, no documented research to quantify the overestimation of Wardrop and Troutbeck models was found. Passing rates in previous studies (Morrall and Blight 1984; Harwood and St. John 1985; Emoto and May 1988; and Staba et al. 1991) were determined manually from videotapes.

For a given flow rate, passing opportunities (hence passing rates) can be increased by improving roadway geometry. Roadway geometry improvement to increase passing opportunity can take the form of either increasing the number of sections with adequate passing sight distance or providing formal/informal auxiliary lanes for passing. Sections with adequate passing sight distance may be increased by flattening both horizontal and vertical curves or by increasing lateral clearances at horizontal curves especially in cut sections.

Morrall and Hoban (1986a) gave detailed definitions of auxiliary lanes, passing lanes, climbing lanes and the evolution of passing lanes. Formal auxiliary lanes for passing purposes are called either “climbing lanes” or “passing lanes” (Morrall and Hoban 1986a). As stated previously, in this study the term “passing lane” is restricted to an auxiliary lane provided for increasing passing opportunities at a place other than sustained grades.

Many studies (Morrall and Blight 1984; Staba et al. 1991; Harwood and St. John 1985) have recognized the ability of passing lanes to improve traffic operations by providing passing opportunities.

Harwood and St. John (1985) developed a model to predict passing rates at the passing lane. The relationship is shown in Equation 2.2 ($R^2 = 0.83$):

PR = 0.127 FLOW - 9.64 LEN + 1.35 UPL;   for 50 vph   FLOW   400 vph **2.2**

Where: PR = Passing rate (passes per mile per hour),

FLOW = Flow in one direction,

LEN = Length of passing lane (mi),

UPL = Percentage of vehicles platooned upstream, and

Vph = vehicles per hour.

Staba et al.(1991) performed regression analysis to predict the number of passes in the passing lane section as a function of a five-minute vehicle count. For the three climbing lanes and one passing lane studied, the intercept of the regression equation ranged from -8 to -2.7, the slope of the regression equation ranged from 0.5 to 0.78, and the regression coefficient (R^2) ranged from 0.31 to 0.76. The best fit was obtained for climbing lanes.

Emoto and May (1988) used a floating car technique to count the number of passes that a test vehicle made in the passing lane sections. The highway test section was 9.4 miles (15 kilometers) long and contained two passing lanes in each direction. They observed that a larger number of passes occurred in the first of the two passing lanes. However, this was not true when number of passes per unit length of passing lane were analyzed. They stated that the results were inconclusive but still felt that the number of passes depends on:

- length of the passing lane,

- length of the platoon preceding the passing lane section,
- speed of the vehicles,
- magnitude of the traffic level, and
- position in the queue of the test vehicle as it enters the passing lane.

Gattis et al. (1997) analyzed the passing activity at short (less than 1,400 ft (427 m)) and long (over 2,500 ft (762 m)) passing lanes. The proportions of vehicles that attempted to pass and the proportion of attempts that were successful were recorded. The conclusion from this study was that a slightly smaller proportion of vehicles attempted to pass on the short passing lanes than did on the long lanes.

2.1.1.1 Shoulder Use

Roadway sections with wide, paved shoulders sometimes are used as informal passing lanes when slower drivers pull to the shoulders leaving the basic lane for faster vehicles to pass them. Morrall and Blight (1984) in their study in Canada, observed some slower drivers pulling to a shoulder (3-meter, paved) to let a faster vehicles pass them. However, they cautioned that this good gesture is usually limited to low volume conditions, because at higher volumes drivers are reluctant to pull to the shoulders due to the difficulty of reentering the main stream.

In regard to shoulder use, Harwood and St. John (1985) observed that in sections where shoulders were designated for use by slow-moving vehicles, up to eight percent of the total traffic and 40 percent of the platoon leaders used the shoulders. However, they cautioned that at flow rates below 100 vph, shoulder use provides only minimal operational benefits. Even at flow rates above 100 vph, accrued benefits from shoulder use are only 20 percent of those of a passing lane.

Use of the shoulder for increasing passing opportunities is gaining more popularity in the US. In 1987 Harwood and Hoban (1987) stated that only the state of Washington had designated shoulder sections for passing. Seven years later, the HCM (TRB 1994) cited five states that allow shoulder use for slow-moving vehicles at all times, and 10 more states that allow such use under special conditions.

2.1.2 Speed

Speed and capacity utilization are used as secondary measures in defining LOS for a two-lane highway in the HCM procedures (TRB 1994). Percent time delay is used as a primary measure. The speed used is the average travel speed. This speed is calculated by taking the length of the highway segment under consideration and dividing by the travel time of all vehicles traversing the segment in both directions. This speed is also called space mean speed.

The HCM uses speed and other measures to define LOS for most uninterrupted flows, such as on basic freeway sections, weaving areas, ramps and ramp junction, multi-lane highways, and two-lane highways. The 1965 HCM used operating speed, but since the 1985 edition was published, average travel speed has been used in place of operating speed (Morrall and Werner 1990b).

In assessing the operational benefits of a passing lane from a speed point of view, researchers have been comparing speed in three different ways (Harwood and St. John 1985; Emoto and May 1988; and Staba et al. 1991): 1) at locations before, within, and after a passing lane section; 2) between basic lanes and passing lanes; and 3) between the direction with passing lanes and the opposing direction without passing lanes. If speed differences between lanes exist, this implies that the passing lanes have a significant impact on vehicle speeds. Yet results from these studies (Harwood and St. John 1985; Emoto and May 1988; and Staba et al. 1991) have produced mixed results, and at times were inconclusive.

On five sites in a California study (Staba et al. 1991), speed differences between the basic lane and the passing lane at the beginning of the passing lane section had the following characteristics:

- for two climbing lanes (short length in rolling terrain and medium length in rolling terrain), the speed difference between lanes was not significantly different between sites;
- for two climbing lanes (short length in rolling terrain and long length in mountainous terrain) the speed difference between lanes was significantly different for each site;
- for one passing lane (short length in level terrain), the speed difference between lanes was not significant.

May (1991) did not report speed as a performance measure of passing lanes when he was summarizing the results of this study.

Harwood and St. John (1985) observed that the mean speeds upstream, within, and downstream of a passing lane were only slightly affected by the presence of a passing lane. The difference in mean speed (downstream location speed minus upstream location speed) varied between a high of +8.3 mph (+13.3 km/hr) to as low of -6.7 mph (-10.7 km/hr). Their conclusion was that spot speed was more highly influenced by local geometry at upstream and downstream sites than by the presence of a passing lane.

In Emoto and May’s study (1988), the space mean speeds for several (11) runs of a test car in a floating mode were compared for two directions with and without a passing lane. Results showed that the space mean speed for the direction with the passing lane was higher than the space mean speed for the direction without a passing lane by two to seven mph (3.2 to 11.2 km/hr).

2.1.3 Percent Time Delay and Percent of Vehicles in Platoons

The HCM (TRB 1994) defines percent time delay as the average percent of time that all vehicles traveling in platoons are delayed due to the inability to pass. In determining the LOS of a two-lane highway by HCM procedures, percent time delay is used as a primary measure because it reflects both functions of a highway; i.e., mobility and accessibility, and it is a measure meaningful to users of the road.

The 1965 HCM used only operating speed and volume to capacity (v/c) ratio in defining LOS of a two-lane highway. The major breakthrough was in the 1985 HCM edition that replaced operating speed with average traveling speed, introduced the effect of directional split, and introduced percent time delay as a primary measure (Morrall and Thomson 1990a). However, HCM procedures cannot be used to evaluate LOS of a two-lane highway with a passing lane or any other special treatment.

To illustrate the weakness of HCM procedures in analyzing highways with passing lanes, consider a road section from upstream to downstream of a passing lane having no side roads or other interruption. This implies that any part of this section can be considered as an uninterrupted flow facility on which HCM procedures can be applied. The HCM procedure requires inputs of geometric and traffic data. Geometric data includes lane and shoulder widths, design speed, percent no passing, and terrain type. Traffic data includes two-way hourly volume, directional distribution, traffic composition, and peaking characteristics. The traffic characteristics, as required by the procedures, are expected to remain constant in both upstream and downstream sections. The difference in LOS, determined by the HCM procedures between the upstream and downstream sections will be due solely to the difference in their geometric characteristics and not in the passing lane between them. Commenting on the weakness of HCM procedures on analyzing passing lanes, Morrall et al. (1986b) cited a study in Canada in which passing lanes were rejected in favor of a four-lane highway for 19 out of 20 improvement projects. Because of the inability of HCM procedures to analyze passing lanes, most studies analyzing the benefit of passing lanes have opted to use percent vehicles in platoons to approximate percent time delay.

2.1.3.1 Field Measurement of Percent Time Delay

Theoretically the field measurement of percent time delay for a highway section could be conducted by tracking the time spent by each individual vehicle traveling through the section. The time each vehicle was delayed due to inability to pass would be recorded and its corresponding percentage computed. The average percent delay would then be the average delay for all vehicles tracked. This type of measurement would be a difficult and very costly exercise. An approximate method would be to measure percent distance traveled while the vehicle is traveling in a platoon (delayed) using the spot platooning method. Percent distance delayed will always underestimate the percent time delay. (This will be elaborated on in sections below). The HCM recommends using the spot platooning measurement in the field as a surrogate measure of percent time delay. However, it does not give guidelines on how to conduct the measurement besides defining a platoon vehicle as one following at a headway of less than five seconds.

For measuring percent time delay in the field, Harwood and Hoban (1987) suggested taking spot platooning at several points on the road and then averaging them to obtain percent distance delay as an approximate measure of the percent time delay. While Harwood's methodology seems reasonable, using percent distance delay seems questionable. This will become apparent after a discussion of the relationship between spot platooning, percent distance delay and percent time delay in the following subsection.

Spot Platooning as the Measure of Percent Time Delay.

A platoon is a group of vehicles traveling together in the same direction. A vehicle is considered a member of a platoon if its time headway is less than the critical time headway. There is no clear definition for the critical time headway. Values ranging as low as three seconds to as high as six seconds have been suggested in the literature. For the purpose of field measurements, the HCM (TRB 1994) defines the critical headway as five seconds. The five-second headway was suggested after correlation of field data on percent vehicle following and actual time delay (Messer 1983).

Some literature includes the platoon leader in its definition of platoon size, while others restrict the definition to those vehicles behind the platoon leader. Whichever definition is used depends on the purpose of classifying a platoon. To understand platooning characteristics over a section of a road, one needs to track the movement of the platoon as it progresses downstream. However, as discussed earlier, such an exercise will be very expensive and difficult. Instead, platooning at one spot on the section is observed, then it is assumed that platooning will be similar throughout the section. This assumption may be valid if the section is short. However, for longer sections this might not be true, because trailing vehicles will most likely get an opportunity to pass a platoon leader and disperse or reduce the platoon size.

A roadside observer can observe the proportion of vehicles trailing behind the platoon leader in a platoon, known as spot percent platooned. If the road section under analysis has only one spot observation, the vehicle(s) delayed behind the platoon leader at only that one spot is (are) assumed to have traveled the whole distance while being delayed, thus yielding 100 percent distance delay. Similarly, if the vehicle(s) spent their whole time of travel through the whole section delayed behind the platoon leader, then there is also a 100 percent time delay. But if the vehicle(s) passed the platoon leader, within the road section under analysis, then both percent distance and time delayed will be less than 100 percent. This implies that one spot observation is likely to overestimate actual delay. For a section having more than one spot observation, percent distance delayed is assumed to be proportional to spot percent platooned. Percent distance delayed will be equal to percent time delay only if the spot percent platooned for each spot is the same; i.e., platooning didn't change throughout the road section. Otherwise the percent time delay would be expected to be higher than percent distance delay if platooning increased because the speed of the additional delayed vehicle(s) would have been higher in an un-delayed section than in the delayed section (delayed and un-delayed section assumed to be of equal length).

From an example cited by Harwood and Hoban (1987), a delayed vehicle traveling five kilometers behind the platoon leader at a speed of 50 km/hr (time is six minutes) and traveling the next five kilometers at a free speed of 60 km/hr (time is five minutes) after passing the platoon leader, will have 50 percent distance delay and 54.5 percent time delay. This suggests that the percent distance delay is likely to underestimate percent time delay.

The logic of using multi-spot platooning observations is apparent, but the practice of averaging the spot percent platooned, over all spots, as suggested by Harwood (1987), doesn't seem to produce good results in estimating either percent time delay or percent distance delay. Consider the previous example in which a vehicle was trailing behind the platoon leader in the first five-kilometer section (spot percent platooned = 50), and was traveling at its free speed in the second five-kilometer section where each vehicle is traveling at its free speed (spot percent platooned = 0). In this case, Harwood's analysis would suggest a percent distance delay of 25 percent (average of 50 percent and zero percent). But assuming the total section is 10 km, it has already been shown that percent distance delay is 50 percent.

Single-spot platooning has been used to evaluate operational benefits of passing lane treatments. Assuming the traffic volume is the same both upstream and downstream of a passing lane, the section with lower spot percent platooned suggests that fewer vehicles are being delayed. The larger the number of following vehicles, the more likely there will be larger percent time delay. Nevertheless, the magnitude of percent time delay is not known. Some researchers have questioned the use of a five-second headway as a cut off value when estimating the percent delay suggested by HCM (TRB 1994).

Mathematical formulation for analyzing multi-spot platooning data in the context of percent distance delay or time delay is complex, difficult and beyond the scope of this study.

2.1.3.2 Observed Values in Previous Studies

Harwood and St. John (1985) developed a regression equation for predicting the difference in percentage of vehicles platooned upstream and downstream of a passing lane as a function of length, and entering platoon. The relationship is shown in Equation 2.3 ($R^2 = 0.33$):

$$PL = 3.81 + 0.1\ UPL + 3.99\ LEN \qquad \textbf{2.3}$$

Where: PL = Difference in percentage of vehicles platooned upstream and downstream of passing lane,

UPL = Percentage of vehicles platooned upstream,

LEN = Length of passing lane (mi).

The coefficient of passing lane length (LEN) in Equation 2.3 is far bigger than the coefficient for percent of vehicles platooned upstream (UPL). This implies that the length of the passing lane is more critical in reducing the number of vehicles in platoons than the size of the platoon. (Neither the range of volumes, nor the passing lane lengths on which Equation 2.3 was derived from, were given.)

For one of the passing lanes observed in the Staba et al. (1991) study, the percent of vehicles in platoons at the exit location of the passing lane was higher by one percent than at the entrance location. For the other four sites, which are climbing lanes, the percent of vehicles in the platoon at the entrance location was lower by two to 19 percent.

Gattis et al.(1997) reported a relationship between the number of vehicles in platoons and one-way volume as follows:

Number of vehicles in platoon/hr = -151 + 1.22 Volume **2.4**

The range of volume for which this equation was developed was from 325 to 525 vph. A very high R^2 of 0.97 was reported. However, the definition of a platooned vehicle was modified. The vehicle was considered to be in a platoon if its time headway as it entered the passing lane was equal or less than five seconds, or if the vehicle passed other vehicles within the passing lane section even if the headway was more than five seconds.

Emoto and May (1988) reported good correlation between percent time delay and percent of platooned vehicles. They found that percent time delay for the direction without a passing lane ranged between 65 to 90 percent, while for the passing lane section it ranged from 30 to 45 percent. The definition of a delayed vehicle within a passing lane included only vehicles in the shoulder (outer) lane that were traveling at headways less than five seconds.

2.1.4 Lane Utilization

In the section where a passing lane is provided, the outer lane (shoulder lane) is supposed to be used by slow-moving vehicles thereby leaving the inner lane (passing lane) for passing vehicles. The proportion of vehicles in the inner lane may reflect passing activities within the section. In essence, lane utilization may be considered an indirect measure of passing rates. However, this assumes that motorists understand and follow the postulated concept of lane assignment.

Emoto and May (1988) developed a linear equation for predicting percent flow in a passing lane given the total flow in the passing lane direction for a range of 50 to 1,300 vehicles of total flow in the passing lane direction.

$$\%FLOW_{PL} = 10 + 0.025(TF_{PLD}) \; ; \quad \text{for } 50 < TF_{PLD} < 1{,}300 \qquad \mathbf{2.5}$$

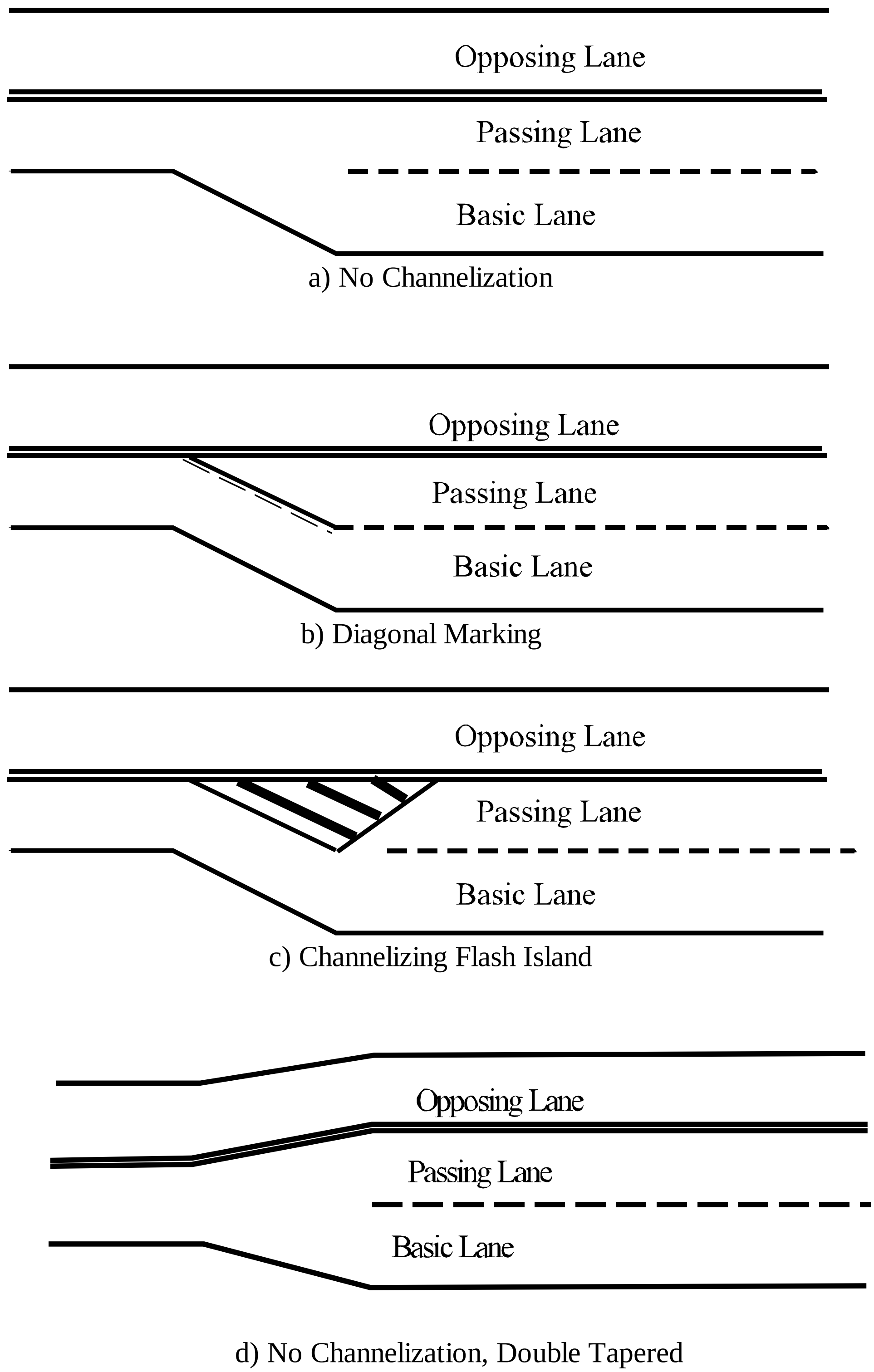
Where: $\%FLOW_{PL}$ = Percentage of vehicles in the passing lane,

TF_{PLD} = Total flow in the passing lane direction.

A plot of absolute flows suggests a concave parabola. No passing rates were observed to check the correlation with lane utilization.

Staba et al. (1991) observed that lane utilization depends on the passing lane addition, pavement marking (pavement marking for lane additions are shown in Figure 2.2). They noted that when the passing lane flows directly from a single channel entrance (see Figure 2.2a), 80 percent of all directional traffic chose the passing lane as opposed to 20 percent for a design in which the entering traffic is channeled to the basic lane (see Figure 2.2b). The latter had 12 percent of vehicles performing passing maneuvers at the passing lane entrance compared with only six percent for the design where the passing lane flows from a single channel entrance.

Morrall and Blight (1984) found lane utilization immediately after the lane-addition point to be 30 percent, decreasing to 15 percent toward the end of the passing lane. The lane-addition pavement marking of the passing lanes in Morrall's study was such that entering traffic flows partially to the outer lane and partially to the inner lane; i.e., no channelization. (Figure 2.2d).



Sources: *May (1991); Batz (1989); Morrall and Blight (1984);*

Figure. 2.2: Lane-Addition Pavement Marking

In another extensive study done in California, Fong and Rooney (1990) studied 20 passing lanes. The study found that the proportion of vehicles that went to the inner lane changed from 36 percent at sites where passing lanes flow directly from a single channel entrance to 22 percent at sites in which the entering traffic is channeled to the basic (outer) lane.

2.1.5 Users' Opinion

The performance and effectiveness of passing lanes, like any other highway infrastructure, depends on their design, construction, signing, and familiarity with and acceptance by drivers. An effective design has to take into account three elements of the highway system: prevailing roadway conditions, vehicle performance, and driver performance (human factors). Traffic operations and safety on the highway system are a result of the interactions of these three elements. Of these three elements, roadway conditions and vehicle performance are more easily predicted than driver performance.

Human factors can be classified into two main groups. The first, physiological, which deals with vision, strength, reaction capabilities, etc. The second, psychological, deals with motivation, attention, temperament, etc. Psychological characteristics are relatively more variable than are physiological ones. Most highway and traffic engineering studies involving human factors have concentrated on physiological aspects, such as determining drivers' reaction time, tolerable acceleration and deceleration rates, drivers' understanding of signing and pavement markings, etc.

The emphasis on physiological aspects by highway engineers most likely stems from the role these aspects play in designing a road's geometric elements and traffic control devices. The psychological aspects of human factors have gained a wider application in social sciences and business. These public sentiments carry more weight in determining the fate of public programs than does evaluation of technical findings (Harwood et al. 1988). However, KDOT engineers felt that drivers' perception of existing passing lanes would be valuable for deciding future use of such lanes.

Only one study (Morrall and Blight 1984) was found in the literature which sought users' opinion as part of monitoring a new passing lane program in the Trans-Canada highway in Banff National Park. Due to budget limitations, the Canadian survey was limited to users in a few categories; namely, professional drivers from the trucking firms and bus lines, Parks Canada employees, and Royal Canadian Mounted Police (RCMP) officers. The user opinion survey asked general questions related to the quality (such as rating of the opportunities to pass other vehicles) of a two-lane section (between Banff and Lake Louise) containing passing lanes; rating relative importance of factors felt to influence the drivers' perception of two-lane highways; questions specific to passing lanes; and comments on passing lane signing, geometry, and operation. The majority of respondents described the delay as slight to moderate, with two-thirds of the respondents either slightly or not frustrated by the prevailing delay. Comments on location, length, and signing of passing lanes were positive. Negative responses commented on driving attitudes, such as drivers disregarding "Keep Right Except to Pass" signs, etc. Fifty percent of respondents had safety concerns contrary to the improved accident record of the highway section evaluated. It should be noted that only one year of accident records was available, and that respondents' concerns are likely to reflect only their perception.

2.1.6 Platoon Structure

The patterns of platoon structures at the entrance and exit of a passing lane could be an indirect measure of passing activities within a passing lane section. An ideal passing lane should be able to convert a platoon at the upstream location of the passing lane to free flowing single-vehicles at the downstream location of the passing lane. Staba et al. (1991) used a platoon structure as a measure of effectiveness to evaluate passing lane effectiveness. They observed that the number of free-flowing single-vehicles was higher at the exit of the passing lane than at the entrance of the passing lane.

2.1.7 Time Headway Distribution

Time headway is another measure of the effectiveness of passing lanes in breaking up platoons. The percent of vehicles with a specified headway is compared at the upstream and downstream locations of a passing lane to determine the effect of the passing lane. It is expected that the percentage of vehicles with short headways should decrease from the upstream to the downstream location. Results reported in a study by Staba et al.(1991) were mixed and inconclusive. For the passing lane, the percent of vehicles with 2-second headways increased from the passing lane to the exit, while the percent with greater than 10-second headways remained unchanged. For the other four climbing lanes, the percentage with 2-second headways decreased but for the greater than 10-second headways, results were mixed.

2.2 PLANNING FOR PASSING LANES

Planning for the provision of passing lanes starts with selection of highway sections which are in need of operational improvements. The selection is made using predefined criteria (warrants). Once the decision has been made to provide passing lanes on a two-lane highway, planning for passing lanes is important so as to maximize their effectiveness. The planning process involves determining their location, spacing and configuration to enhance efficient flow and safety.

2.2.1 Warrants for Provision of Passing Lanes

This section discusses warrants for provision of a passing lane as cited in the literature. These warrants may be divided into four main groups: 1) those which employ a supply-demand model, 2) those which employ a benefit-cost ratio model, 3) those which employ a maximum-queue model, and 4) those which employ a level-of-service model. Reid Crowther and Partners (1990) have summarized several passing lane warrants used by different jurisdictions in North America and Australia. These warrants plus the one used by KDOT are shown in Table 2.3.

2.2.1.1 Supply-Demand Models

These models are based on the supply and demand of commodities theory. The commodity is the passing opportunity, the supply is the availability of those opportunities, and the demand is the need for a passing maneuver. The objective is to add supply where and when it is needed; i.e., where the demand exceeds the supply. Passing opportunities on a two-lane highway can be increased by providing a passing lane, or other means such as geometric improvement. In supply-demand modeling, provision of a passing lane is considered the only way for increasing passing opportunities. Two models in this category have been suggested: 1) the passing ratio model, and 2) the passing opportunity model.

Table 2.3: Passing Lane Warrants in a Sample of Jurisdictions

Jurisdiction	Passing lane warrant
Canada[1] (Transportation Association of Canada)	When the available assured passing opportunity is approaching 30%.
British Columbia[a]	Level of service
Alberta[a]	AADT, no passing zones, truck and RV[b] traffic
Ontario[a]	30% assured passing opportunity
Canadian Parks Services[a]	Maximum 60% platooned determined by traffic simulation.
Australia NAASRA (1990)[a]	Traffic volume, percent slow vehicles, overtaking opportunities over preceding 5 km
US (FHWA procedures)[a]	1985 Highway Capacity Manual
KDOT	Benefit-Cost Analysis

a Adapted from Reid Crowther and Partners Ltd. (1990).
b RV = Recreation Vehicle.

Passing Opportunity Model

The passing lane is warranted when the supply (passing opportunities) provided for one direction of travel falls below a prescribed value. Passing opportunities available for one direction of travel depend on road geometry in that direction (direction 1), and on gap distributions in the opposing direction (direction 2). The required gap distribution depends on the traffic characteristics and roadway geometry of direction 2.

The passing opportunity model determines the available supply (Net Passing Opportunities (NPO)) analytically as the product of the proportion of the road with passing zones in direction 1, and the proportion of gaps greater than a critical time suitable for passing in direction 2. For example, a traffic stream with no opposing stream in an entire road section that permits passing at any location would have 100 percent passing opportunities on that section. At the other extreme, a traffic stream on a section marked no-passing for the entire length, or encountering very heavy opposing traffic such that there are no gaps adequate for passing within the opposing stream, would have zero passing opportunities. For a given highway section, NPO can be interpreted as the percent of gaps within a given period of time that can be used for passing purposes by vehicles in the direction of travel that is being analyzed.

Passing Ratio Model

The passing ratio model was first introduced by Morrall and Werner (1990b). This model computes the ratio between the supply and demand for the highway section under consideration. Both supply and demand are measured by passing rates (passes per mile per hour). The supply is the passing rate provided by the given highway and given traffic level. Demand is the passing rate created by the same traffic level on the same highway provided with passing lanes in both directions. This ratio is known as the passing ratio and is computed by the following formula:

$$\textit{Passing Ratio} \quad \frac{\textit{Achieved Passing}}{\textit{Desired Passing}} \qquad \textbf{2.5}$$

Achieved passing is the actual passing rate achieved for a given prevailing highway section. Desired passing is the passing rate that could be achieved on the same or a similar highway under the same traffic conditions if the highway were provided with passing lanes in both directions throughout its entire length (basically a short four-lane section). Both achieved passing and desired passing are obtained using a computer traffic simulation model. The passing ratio would assume a value of one for a road section with passing lanes in both directions throughout its entire length, and would assume the value of zero for the road which is marked with double yellow lines throughout, or when traffic is so heavy in both directions that no gaps adequate for passing are available for traffic in either direction. A passing lane is warranted when the passing ratio falls below a prescribed value. To the authors' knowledge, no agency has put this model into practice.

The developers of this model used the TRAffic Rural Road (TRARR) traffic simulation model to simulate traffic on two-lane highways with passing lanes and found that the passing ratio changes faster with changing traffic volumes than with changing percent time delay. Percent time delay is the primary measure which the HCM uses to determine the level-of-service for a two-lane highway. Furthermore, they observed that the passing ratio is more appropriate than the HCM procedures for evaluating the LOS of sections with passing lanes. The authors recommended using passing ratio as a measure of level-of-service to supplement existing measures.

.2.2.1.2 Benefit - Cost Ratio Models

The benefit-cost ratio model uses economic analysis procedures to compare benefits and costs resulting from the provision of a passing lane. All benefits and costs have to be quantified in terms of their monetary values. The objective of the benefit-cost ratio model is to maximize the benefits per dollar spent on the project. Warrants developed using this model result in a threshold volume beyond which the benefits accrued from the provision of a passing lane exceed the costs of providing it. At this threshold volume, a passing lane is warranted. Existing warrants in use by KDOT were developed using this model in which the benefits are accident reduction and time savings, and cost include construction and maintenance costs over the life span. The use of this model has also been suggested in the literature by other researchers (Kaub and Berg 1988; Taylor and Jain 1991).

The benefits used by Taylor and Jain (1991) include time savings and accident reduction. Kaub and

Berg (1988) used travel time savings and reduction of potential passing conflicts as benefits. Both studies used computer simulation to estimate travel time savings.

2.2.1.3 Maximum Queue Model

The maximum queue model is well documented in Safety Design and Operation Practice (SDOP) for Streets and Highways (FHWA 1980). In this model a passing lane is warranted on a relatively long no-passing zone where overtaking of vehicles behind the slower, leading vehicle will result in a queue whose size is set at a maximum number of vehicles. Assuming uniform flow and no queue at the beginning of the no-passing zone, the distance required to build up the maximum queue is determined analytically. If this distance is less than the length of a no-passing zone, then the passing lane is warranted.

2.2.1.4 Level-of-Service Models

In this warrant the passing lane is justified when the level of service falls below a desired level. While in the US the level-of-service as defined by the HCM is used, the Canadian parks service uses percent time delay determined by computer simulation. It should be remembered that HCM level-of-service uses percent time delay as its primary measure of level of service for two-lane rural highways.

2.2.2 Location

Following is a summary of suggestions/guidelines on passing lane location by various researchers in the literature reviewed for this study.

Morrall et al. (1984, 1985, 1986a, 1986b, 1990a) presented the following guidelines:

- Avoid locations at/near campgrounds, day-use areas, intersections and driveways (especially where diverging and merging is taking place and where left turns are needed from passing lanes) to avoid conflicts between turning movements and passing maneuvers.
- Avoid locations upstream or downstream from a four-lane section because at those places, passing lanes are less effective.
- A choice between a grade location and a level location should consider relative costs, delays on the grade and the nature of the traffic demand on the road; however, it is more likely that grades where speed differences are often greater should receive priority because of greater effectiveness regarding the number of passes.
- In level and rolling terrain, avoid locations where passing opportunities are presently provided for in both directions.
- Passing lanes can be located on a downgrade where speed difference is significant.
- A location should appear logical to the driver; i.e., immediately after restricted passing opportunities, such as on a road segment with solid barrier lines.
- Consider passing lanes on level terrain where demand for passing opportunities exceeds supply.
- Consider passing lanes where adequate sight distance is available at the diverge and merge tapers.
- Avoid including costly physical constraints that restrict continuity of shoulder width, such as bridges, culverts, cuts and fills.
- Consider passing lanes at sections which need realignment because of a safety problem.
- Locate passing lanes to minimize construction costs where possible.

- Locate passing lanes leading away from rather than into areas of traffic congestion and
- Avoid highway sections with reduced design standards, since they are not suitable for passing.

Harwood et al. (1987, 1988) presented the following guidelines:

- Minimize construction costs subject to other constraints.
- Passing lanes should be logical to the driver (where passing is restricted by geometry).
- Passing lanes should have adequate sight distance at the lane-addition and lane-drop tapers.
- Avoid major intersections and high-volume driveways.
- Consider passing lanes when realignment is done for safety problems to reduce construction unit cost.
- Avoid sections with low-speed curves because passing may be unsafe.
- Passing lanes are preferred on grades if delay problems are severe, or on level terrain if platooning delay exists for some distance along the road.
- Consider passing lanes on level terrain where demand of passing opportunities exceeds supply and
- Avoid bridges.

Underwood (1996) presented the following guidelines:

- Minimize construction costs subject to other constraints.
- Passing lanes should be logical to the driver (where passing is restricted by geometry).
- Passing lanes on flat terrain are more effective.

- Provide adequate sight distance at the lane-addition and lane-drop tapers.
- Avoid major intersections and
- Avoid physical constraints which restrict width; e.g., bridges.

Reid Crowther and Partners Ltd. (1990) presented the following guidelines:

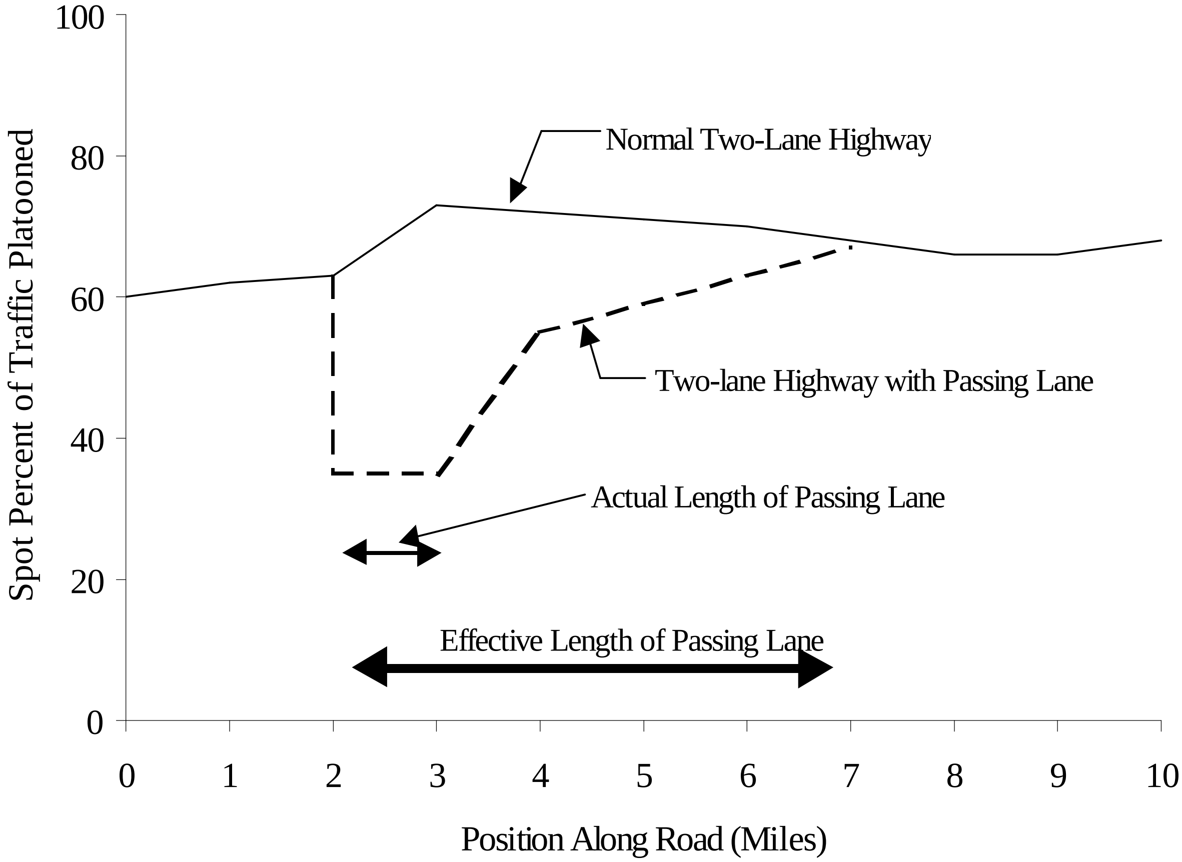
- Engineering costs: avoid cuts, fills, culverts and bridges.
- Intersections: avoid intersections especially within close proximity to the start and end points.
- Passing opportunities: sections with good passing opportunities should not be considered for passing lane.
- Geometric design standard: avoid sections with reduced geometric standards.
- Logical to the driver: location should appear logical to the driver.
- Four-lane sections: avoid proximity to four-lane sections.
- Through traffic simulation: base on effective length of the passing lane (Figure 2.3).
- Beginning: start on a horizontal curve to the left rather than to the right because it directs traffic to the outside lane, hence improving operational effectiveness of a passing lane and
- Terminus: terminate at a section with good sight distance.

2.2.3 Spacing

In comparing the passing lane spacing experiences of Australia and Canada, Morrall and Hoban (1986a) described the spacing strategy in Australia as a stage by stage process. For standard two-lane roads, it is cost-effective to space passing lanes at a distance of about 10-15 km. Then at a later stage, when volumes have increased or when greater operational improvement is needed, they should be spaced closer; e.g., three to five kilometers. In Canada the spacing practice is variable, depending on the length and number of lanes determined necessary to improve passing opportunities. In rolling terrain, where the objective is to combine climbing and passing lanes, spacing may be closer than in level terrain.

Morrall et al. (1984, 1985) described the spacing as a function of traffic volume, traffic composition, available road sight distance, amount of desired improvement and configuration. It was suggested that it is better to have several short passing lanes closely spaced than a few long ones at longer spacing. For the Trans-Canada Highway in Yoho National Park, 2-km passing lanes at frequent intervals arranged in a tail-to-tail configuration (refer to Figure 2.4) were suggested.

Harwood and Hoban (1987) suggested that the spacing should be equal to the effective length of the preceding passing lane. The effective length is the length of the passing lane plus the distance downstream to the point where traffic conditions return to the platooning level before entering the passing lane. Figure 2.3 shows the concept of the effective length. The effective length of a typical, one mile passing lane ranges from three to eight miles (4.8-12.8 km) depending on traffic volume and composition, passing lane length, and downstream passing opportunities. Harwood and Hoban suggested variable spacing to permit avoidance of expensive locations. For a road without passing



Source: Harwood and Hoban (1987)

Figure 2.3: Effective Length of a Passing Lane

lanes, they recommended the Australian approach of long initial spacing of 10-15 miles (16-24 km) scaled down to three to five miles (4.8-8 km) by adding additional, intermediate passing lanes when major improvements are due or when there is an increase in traffic volume.

2.2.4 Configuration

There are nine different configurations for passing lanes. Figure 2.4 shows these configurations. The isolated case *(a)* is usually used to reduce delays at a specific bottleneck. Other alternatives allow some interaction between two consecutive passing lanes in different directions and should be used where traffic improvements are needed in both directions over an extended length of a route. As explained by Morrall and Hoban (1986a), the interaction can take the following forms:

> 1. "If double yellow lines are used, an auxiliary lane in one direction becomes a no-passing zone for opposing traffic, thus reducing the quality of service in that direction.
>
> 2. The break-up of platoons in one direction results in fewer long gaps (between platoons) for passing by opposing vehicles on a two-lane road section.[1] This results in more platooning in the opposing stream, which produces more long gaps for passing by vehicles traveling in the first direction, and can create a self-reinforcing distinction between the two directions."

The following discussion of tail-to-tail vs head-to-head adjoining configurations in Figure 2.4 is from the literature (Harwood and Hoban 1987; Morrall and Blight 1984; Mclean 1989). Consider two passing lanes in opposite directions located at the same place (adjoining passing lanes, case *d* and *e* of Figure 2.4) on a high-volume highway. Because of high traffic volume, traffic in the opposing direction of a passing lane is usually restricted by double yellow lines. In such a case, configuration *d* (Figure 2.4), known as tail-to-tail, is believed by some researchers (Harwood and Hoban 1987; Morrall and Blight 1984; McLean 1989) to be more effective than the head-to-head configuration, *e* (Figure 2.4). (In one report Harwood et al.(1987) configurations *c* and *e* are referred to as head-to-head.) Those who favor the tail-to-tail configuration, claim that it is more effective because it creates a process of platoon formation opposite the opposing passing lane, followed by platoon break-up so that vehicles are not in a platoon as they leave the passing lane. For head-to-head configuration *e*,

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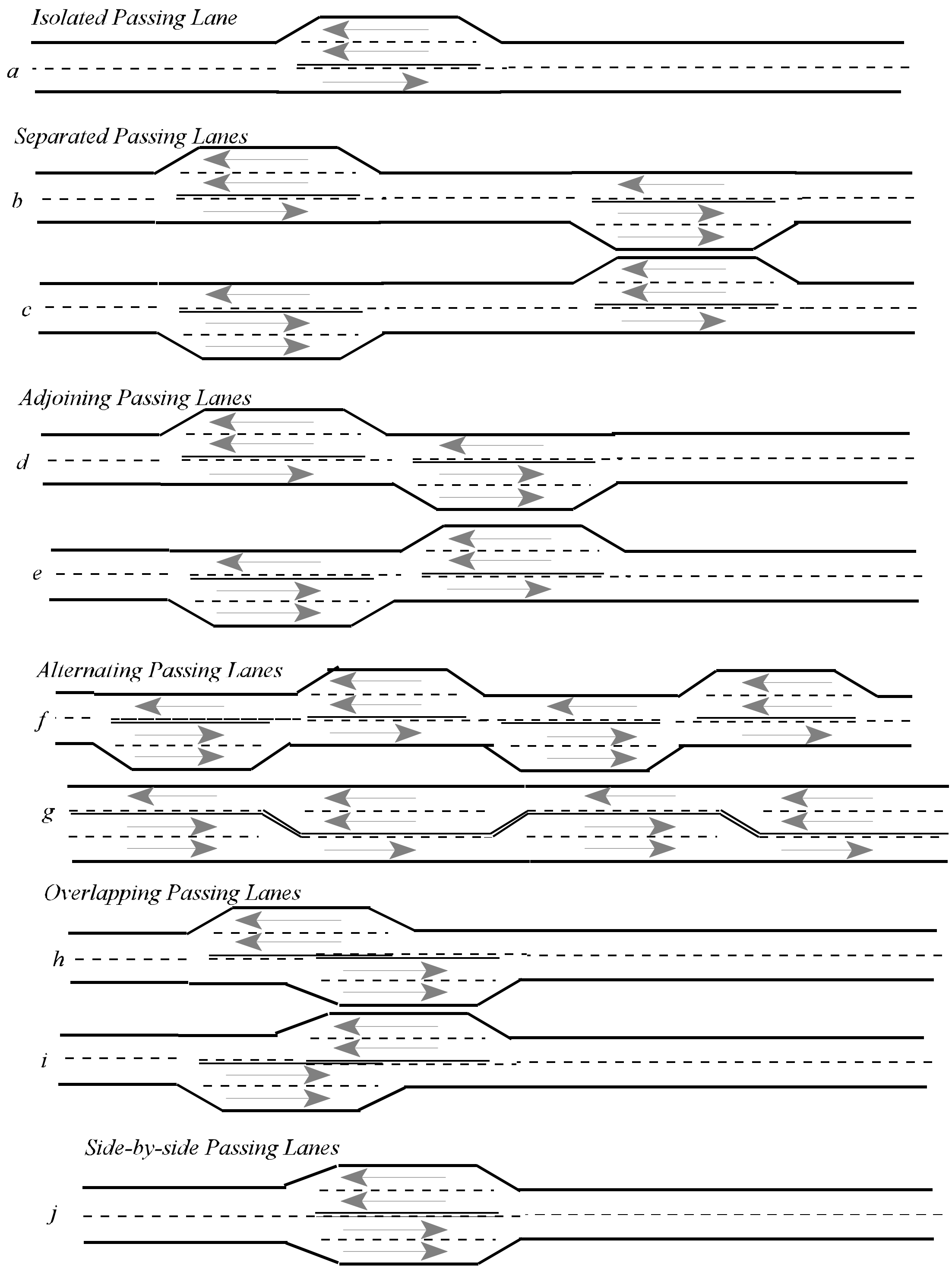
[1] This assumes that the traffic is flowing in a series of relatively short platoons with headway between platoons adequate for passing by vehicles in the opposing direction.



Source: Harwood and Hoban (1987)

Figure 2.4: Passing Lane Configurations

(Figure 2.4) the break up of platoon occurs at the passing lane, but then vehicles may be re-platooned at the opposing passing lane, implying that vehicles may leave passing lane sections in platoons.

Others, including the authors, don't think there is sufficient evidence to conclude that one is more efficient than the others. However, configuration *e* of Figure 2.4 (adjoining head-to-head), is unfavorable from a safety stand point due to the merging areas being opposite each other.

The above arguments of tail-to-tail vs head-to-head are based on short sections of passing lanes. A better comparison would extend beyond the physical length of the passing lanes to cover the whole section influenced by the passing lane; i.e., the effective length. More research would be necessary to clearly determine that one or the other configurations of Figure 2.4 is superior.

If the distance between opposing passing lanes in cases *b* and *c* (Figure 2.4), is sufficient, such that the passing lane has no influence to the opposing passing lane, they would perform similar to isolated passing lanes (case *a* of Figure 2.4). Otherwise, they would perform like adjoining passing lanes (case *d* and *e* of Figure 2.4).

Type *f* and *g* (Figure 2.4) are sometimes appropriate where sufficient width for the passing lanes is available. For highways where double lines are used within a passing lane, drivers may feel unduly constrained when passing is prohibited on the other 50 percent of the road length if sight distance is good and traffic is low. Overlapping types (*¡*) are often used at crests where a passing or climbing lane is provided on each upgrade, and overlapping type (*h*) may be used in sags where there are upgrades in both directions.

Side-by-side type *j* (Figure 2.4) may be more appropriate where:

- location of a passing lane is constrained by non-flexible factors such as, obtaining right-of-way, avoiding intersections in the passing lane, or assuring that there is sufficient sight distance at passing lanes termini. When these factors are favorable at a certain location, it is convenient and generally cost-effective to construct passing lanes in both directions;
- heavy traffic volume is the main cause of platooning rather than no-passing zones, and hence no-passing zones don't significantly influence the location of a passing lane;
- the need for passing lanes exists in both directions.

However, side-by-side is not appropriate near major urban areas or major intersections.

2.3 GEOMETRIC FEATURES OF A PASSING LANE

Passing lane geometric features include horizontal alignment, vertical alignment, lane and taper lengths, cross-section and shoulder width. Alignments are not discussed here as they relate more to passing lane location discussed above in section 2.2.2. Reid Crowther and Partner Ltd. (1990) summarized passing lane design guidelines for Canada, US, and Australia. This summary is presented in Table 2.4.

2.3.1 Length

There is no clear definition of the length of a passing lane. Some consider the whole length including both tapers, while others exclude tapers. It has been suggested that length, without tapers, should be based on the passing rate (number of passes per unit length per unit time). The study of Trans-Canada

Jurisdiction	Spacing (km)	Length (km)	Taper Length		Lane Width (m)	Shoulder Width (m)	Signing	Passing in Opposing Lane	Configuration	Other Factors
			Diverge	Merge						
British Columbia	-	Minimum 0.8; Desirable 1.0	20:1	25:1	3.6	1.8		Yes if AADT < 4000	Mostly 4-lane sections	
Alberta	-	2.0	25:1	50:1	3.7	1.5	Figure 2.6	Yes if AADT < 4000	Mostly alternating sections	
Canadian Parks Service	Determined from warrants	Trans-Canada 2.0 Other highways minimum 0.5	100 m	200 m	Elsewhere 3.7	1.2 (min)		Yes if AADT < 4000	Mostly alternating sections	- Start of PL on left hand curve - intersections avoided (if possible) -Wide shoulder at merge for recovery area
Ontario	10 - 25	1.5 - 2.0	$\frac{V \times W^{a,b}}{1.6}$	$\frac{V \times W}{1.6}$	3.4 desirable: 3.25 minimum	equal to approach shoulder; 1 min.	Figure 2.7	Yes, except for certain cases	Mostly alternating	
FHWA[d] (USA)	5 - 13	0.3 minimum 0.8-1.6 optimal	$\frac{2 \times V \times W}{3}$	$V \times W$	3.7	1.2	Figure 2.9	Yes, if signed and left turn avoided	Depends on situation	
Australia	3-5 to 10-15	Depends on design speed; Normal maximum length 1.2	$\frac{V \times W}{3}$	$\frac{V \times W}{2}$	3.5 not less than lane width provided elsewhere	Minimum: 1	Figure 2.8	Yes, P.S.D.[c] permitting	Alternating and overlapping	Two short auxiliary lanes are most cost effective rather than one long one in excess of the normal maximum length

[a] v = 85th Percentile Speed [b] W = Lane Width [c] P.S.D. = Passing Sight Distance [d] FHWA = Federal Highway Administratio

Source: Reid Crowther and Partner Ltd. (1990)

Table 2.4: Comparison of Design Guidelines Between Canada, US and Australia

Highways by Morrall and Thomson (1990a), concluded that the length depends on traffic volume, with taper lengths being directly proportional to the product of the 85th percentile speed and the widening or narrowing of the pavement. They suggested an optimum length of two km including tapers of 150 m and 200 m for lane-addition and lane-drop, respectively. Reporting on the experience of the Province of Alberta, Morrall et al.(1986b) could not find any guidelines for passing lane length; however, they found the length in the order of two kilometers depending on site specifics, with 25:1(80 m) and 50:1(175 m) for tapers.

The study by Harwood and Hoban (1987) suggested an optimum length (excluding tapers) of 0.5 to 1.0 miles (0.8 to 1.6 km). They stated that lengths of more than 1.0 mile (1.6 km) usually are not cost-effective, while lengths shorter than 0.5 miles (0.8 km) are not effective in creating additional passing opportunities. The effect of shorter lanes on passing was also observed by Gattis et al. (1997) in Arkansas. In the Arkansas study, a smaller proportion of vehicles attempted to pass upon reaching the shorter passing lane (0.27 miles, i.e., 0.43 km) than on longer lanes (0.47 miles, i.e., 0.75 km). It was suggested that this could be the result of the drivers' perception of inadequate distance to complete the pass on shorter lanes.

2.3.1.1 Lane-Drop

Although some aspects of lane-drop were mentioned in the preceding section, its role in the safety and operation of passing lanes is important enough to warrant additional explanation. Probably the most critical element of a passing lane is the lane-drop section where the two lanes in one direction converge to one lane. The lane-drop is conceived as a crucial element from both the operational and safety stand points. In regard to traffic operations it acts like a bottleneck to the traffic stream which was on a two lane section but now is forced to merge into one lane. Merging could be to the left, where slower vehicles in the outer lane being terminated merge to the inside lane, or to the right where the passing vehicles in the inner lane merge to the slower stream in the outer lane. From a safety stand point, in the former mode of terminating the right lane, a slower merging vehicle has to estimate and choose a suitable gap in the adjacent stream to merge safely. This process will likely produce a “race track” phenomena where the merging slower vehicle increases its speed to merge with faster passing vehicles, while the passing vehicle increases its speed to avoid ending up behind the slower vehicle after the passing lane section. Guidelines for the location of passing lanes, summarized in the previous section, have emphasized the need of avoiding placing the passing lane where there is insufficient sight distance at the terminus. Termini at vertical and horizontal curves are not desirable.

2.3.1.2 Lane-Addition

If a lane-addition is not properly designed, marked, and signed, the result may degrade operation and safety of the passing lane. Most design guidelines recommend that taper lengths for lane- additions be shorter than those for lane-drop. Pavement markings are shown in Figure 2.2.

2.3.2 Cross-Section

Most agencies suggest that the passing lane width should not be less than the width of the adjoining sections, but may have reduced shoulder width at the passing lane section. The cross slope should be the same as the adjacent lane.

2.4 SIGNING AND MARKING

There are four places within a passing lane system that need signs to supplement the information provided by the roadway geometry: 1) advance zone; 2) lane addition zone; 3) lane drop zone; and 4) opposing lane, as it approaches the end of a passing lane. The practice mentioned in this report includes both passing and climbing lanes. Figure 2.5 shows different zones for the passing lane. Sign naming used in this section conforms to the Manual on Uniform Traffic Control Devices (MUTCD) (FHWA 1988), unless otherwise stated.

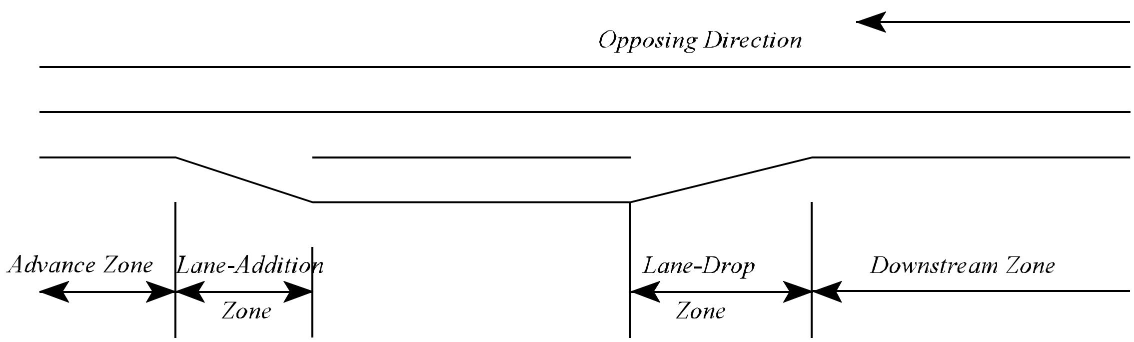


Figure 2.5: Passing Lane Zones

2.4.1 Advance Signing

A passing lane is more effective in dispersing platoons if it is located at the downstream of a low-passing-opportunity section. However, drivers being delayed in platoons for a considerable time due to their inability to pass, may become frustrated and perform risky passing maneuvers in front of opposing traffic. It is advisable to inform such drivers of the presence of a passing lane ahead which may reduce such incidents. Signs informing motorists of the distance to the beginning of a passing lane serves this purpose. There is no specific location for these signs, but the best policy would be to locate such signs where they will constantly remind motorists of a passing lane ahead, and possibly reduce risk-taking, passing behavior.

2.4.2 Lane-Addition Signing and Marking

At the beginning of the passing lane, drivers are normally reminded of the lane assignments; i.e., slower drivers should use the outside lane. Some agencies remind motorists with the **SLOWER TRAFFIC KEEP RIGHT** (R4-3) sign, while other agencies use a **KEEP RIGHT EXCEPT TO PASS** sign (this sign is not specifically defined in the MUTCD). There is no clear-cut agreement among highway and traffic engineers as to which sign is best. While some believe there is a difference between the two, others contend they work in a similar manner. Morrall and Hoban (1985) reported that in Australia and many parts of Canada (except British Columbia) the **SLOW TRAFFIC KEEP RIGHT** sign has been phased out in favor of the **KEEP RIGHT EXCEPT TO PASS** on all passing lanes, including climbing lanes. Those who favor the latter sign argue that operational experience and driver surveys have shown that **KEEP RIGHT EXCEPT TO PASS** is more effective since it encourages greater use of an outer lane thus increasing the number of passes, and that drivers favor it because it is less ambiguous. Even those who view the **SLOW TRAFFIC KEEP RIGHT** as an acceptable alternative agree that their choice provides less definitive instructions to drivers.

Channeling traffic to the outer lane is highly recommended because slower vehicles tend to flow naturally to the basic (outer) lane. Fong and Rooney (1990) conducted an extensive study of lane-addition pavement marking. Ten passing lanes with lane-addition channelization and 10 passing lanes without channelization were filmed for two hours each. At lane-additions without channelization, 36 percent of the total vehicles, 6.4 percent of the platoon leaders, and 57.7 percent of the followers were in the inner lane at the beginning of the passing lane. The numbers for sites with channelization were 22 percent, 4.9 percent, and 47 percent respectively; i.e., more vehicles were "channeled" to the outer lane creating greater passing opportunity.

Staba et al. (1991) conducted a before-and-after experiment in California to assess the effect of marking the lane-addition. In the before study without channelization (case (a) of Figure 2.2, page 22), it was found that 80 percent of entering traffic flowed directly into the passing lane. In the after study with channelization (case (b) of Figure 2.2) 80 percent of entering traffic flowed directly into the outer lane.

In another study conducted by Batz (1989) in New Jersey, 41 percent of the platoon leaders (platoon was defined as a three seconds critical headway) flowed to the passing (inner) lane of the unchannelized case a (Figure 2.2), whereas only one percent flowed to the passing (inner) lane of channelized case c (Figure 2.2). In yet another study in Canada (Morrall and Blight 1984), 30 percent of all vehicles flowed into the passing (inner) lane of unchannelized case d of Figure 2.2 (page 23). Although only one case was studied in California, only platoon leaders were considered in New Jersey, and only two passing lanes were studied in the Canada study, the magnitude of the change in lane utilization is so great that it is unlikely to be due to chance.

With significant differences of lane utilization between "channelization" and "no channelization" at the lane-addition of a passing lane, one might expect that the number of passes would be higher in the channelization case. However, results from some studies have not supported this hypothesis. In Staba's study (Staba et al. 1991), 458 passes were made from a volume of 1,059 vehicles (0.43 passes per vehicle) for the "no-channelization" condition, while for the "channelization" condition the passes were 413 out of 1015 vehicles (0.41 passes per vehicle). When the data was analyzed in a five-minute period, the "channelization" condition showed a higher pass rate (May 1991). Another study in California comparing 10 passing lanes with channelization and 10 passing lanes without channelization, it was concluded that there was no significant difference in the amount of passing between the two pavement marking practices (Fong and Rooney 1990). However, Batz (1989) concluded that results were inconclusive when analyzing numbers of passes of platoon leaders for situations with and without channelization.

The authors believe, that all passing lane sections should be provided with clear channelization to the outer lane with appropriate pavement markings.

2.4.3 Lane-Drop Signing and Marking

Where the lane is dropped at the end of a passing lane section, some agencies use only one sign to alert motorists to this, while other agencies use more than one sign (Morrall et al. 1984, 1985 1986a; Harwood and Hoban 1987). Those who use one sign, use a symbolic lane reduction, transition sign (W4-2) as defined in MUTCD (FHWA 1988) near or at the beginning of the lane-drop taper. For those who prefer two signs, the first, with wording such as **RIGHT LANE ENDS** (W9-1) serves as an advance notification sign upstream from the merging area, while the second, symbolic sign (W4-2) serves to inform motorists of the location where the lane-drop taper begins. As far as marking is concerned, the most common practice suggested in the literature (Morrall et al. 1984, 1985 1986a, Harwood and Hoban 1987) is to terminate the pavement marking line that delineates basic and passing lanes just before the beginning of the lane-drop taper to simplify efficient merging. The same method is suggested in the MUTCD. (Refer to Figure 3-10 page 3B-13 of MUTCD (FHWA 1988) for complete guidance on lane reduction signing and marking.)

2.4.4 Opposing Lane Signing and Marking

Within the length of the passing lane section, passing in the opposing direction can be allowed or restricted. Signing and marking for traffic approaching from the opposite direction has to reflect the restriction or permission. It is recommended that where passing is allowed, signing must clearly show the priority of the opposing, passing lane for traffic in the passing lane direction.

2.4.5 Signing and Marking Practice in Alberta, Canada

Signing practice in Alberta, Canada is shown in Figure 2.6a and 2.6b. The advance sign, RB-37 of the Roads and Transportation Association of Canada (RTAC 1987) is posted two kilometers upstream of a passing lane (Morrall et al. 1986b). Reid Crowther and Partners Ltd. (1990) have recommended a lane-addition sign (RB-37T of RTAC) with wording **PASSING LANE 2 km** to be used together with RB-37. The lane-addition sign **KEEP RIGHT EXCEPT TO PASS** is placed at the beginning of the taper and at an interval of 500 m along the lane. Pavement marking for the lane-addition consists of a broken line separating the basic lane and the passing lane from the start of the lane-addition taper to the beginning of the lane-drop taper. A lane-drop sign WA-33R (RTAC 1987) sign similar to the W4-2 is used. Reid Crowther and Partner Ltd. (1990) have recommended that it be at 300 m and supplemented with a **300 m** sign plate.

For opposing lane signing and marking, the Alberta province in Canada uses Average Annual Daily Traffic (AADT) as a criteria for permitting or restricting passing in the opposing direction of the passing lane (Morrall et al. 1986b). If AADT is less than 4,000 vehicles a day, passing is permitted in the opposing direction provided there is sufficient sight distance. When passing is permitted, a broken line pavement marking is used in the opposing directions (except on lane-drop and lane-addition taper sections marked with solid lines), supplemented by three types of signs: 1) **DO NOT PASS** sign, both symbolic and word signs (similar to the MUTCD R4-1 sign) together at 310 m before the end of the lane-drop taper and also at 10 m before the end of the lane-addition taper; 2) **DO NOT PASS WHEN TRAFFIC ONCOMING** at 500 m intervals between the two tapers. Reid Crowther and Partner Ltd. (1990) recommended replacing it with **YIELD CENTER LINE TO OPPOSING TRAFFIC** (RTAC's RB-36); and 3) **PASSING PERMITTED**, both symbolic and word sign at 275 m after the end of the lane-drop taper, provided there is sufficient sight distance. When passing is restricted, sign (2) above is replaced by sign (1), and the solid line is marked throughout. Other requirements remain the same as in the passing permitted mode.

2.4.6 Signing and Marking Practice in Ontario, Canada

Signing practice in Ontario, Canada is shown in Figures 2.7a and 2.7b. For the advance warning, two signs are used to notify motorists of the upcoming passing lane. The signs used are the same as those discussed earlier for Alberta, Canada. The lane-addition sign **KEEP RIGHT EXCEPT TO PASS** similar to that used in Alberta, is placed at the beginning of lane-addition taper. Also pavement markings for the basic lane and passing lane delineation is similar to that used in Alberta. For lane-drop notification, three signs at two locations are used: 1) the symbolic lane reduction sign (MUTCD W4-2) and **"RIGHT LANE ENDS 300 m**" sign on one post are placed 300 m before the start of lane-drop taper; and 2) W4-2 sign and "**RIGHT LANE ENDS**" sign are placed at the beginning of the lane-drop taper. Passing in the opposing direction is permitted except in the lane-drop section, where sight distance does not allow, or for the side-by-side or overlapping configurations. Where passing is permitted, a sign "**PASS ONLY WHEN CENTER LANE IS CLEAR**", is located at the beginning of the opposing lane-drop taper and 800 m from the beginning of opposing lane-drop taper.

2.4.7 Signing and Marking Practice in British Columbia, Canada

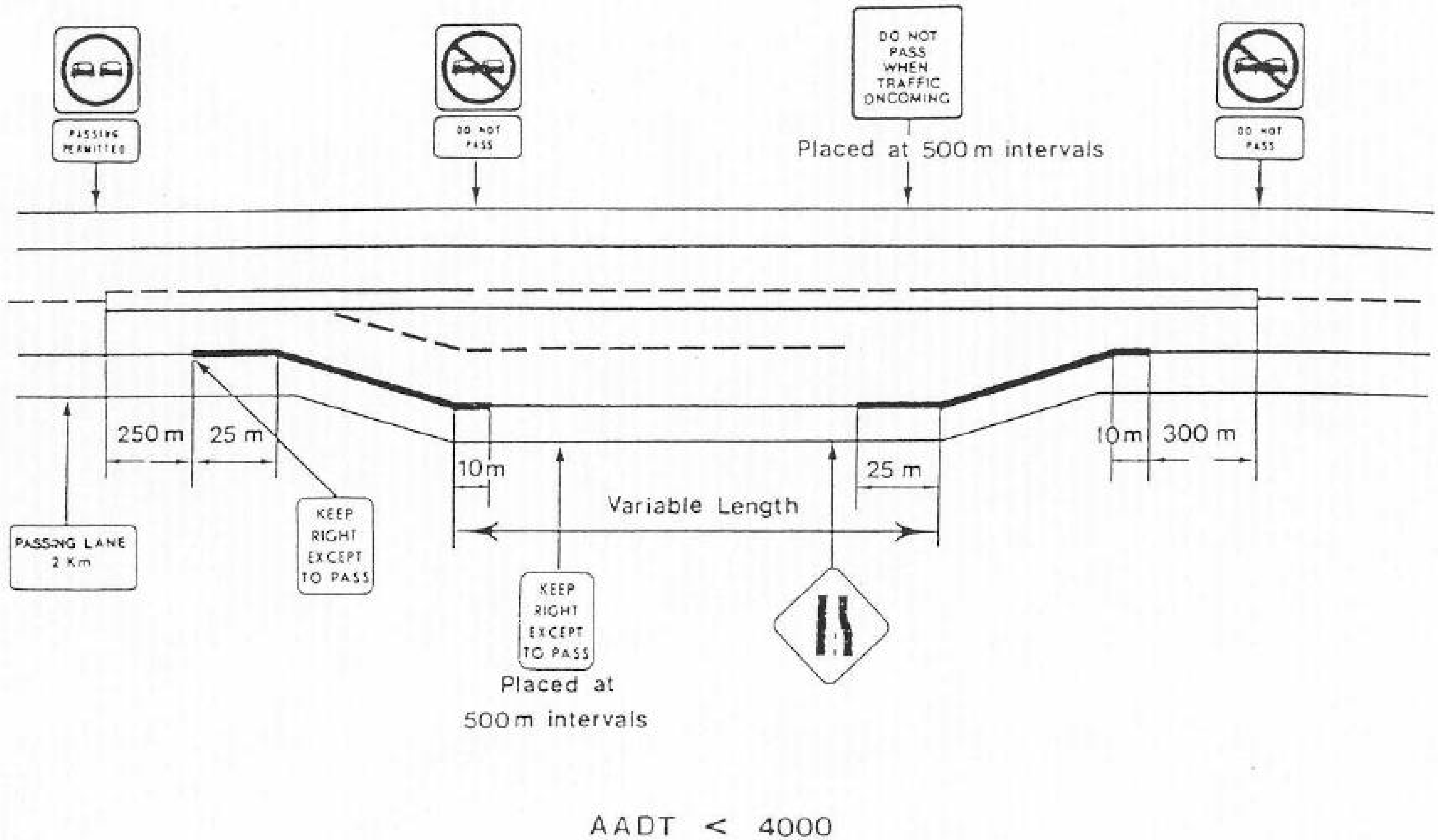
British Columbia uses one advance sign (**PASSING LANE 2 km AHEAD**) at two kilometers upstream of the passing lane. At the lane-addition, it uses the sign **SLOWER TRAFFIC KEEP RIGHT**. At the lane-drop point, two signs (**RIGHT LANE ENDS** and **FORM SINGLE LINE**) are used concurrently. For the opposing lane, either **YIELD CENTER LANE TO OPPOSING TRAFFIC** or **DO NOT PASS** is used depending upon whether passing in the opposing direction is permitted or restricted.

2.4.8 Signing and Marking Practice in Australia

Figure 2.8 shows typical signing practice in Australia. The most common place for the advance sign is 300 m before the passing lane. Although the Australian Manual for Uniform Traffic Control Devices has a provision for advance signs for up to three kilometers ahead, they are not extensively used because the state road authorities are not required to follow this manual (Morrall et al. 1986b). A lane-addition sign **KEEP LEFT UNLESS OVERTAKING**, equivalent to **KEEP RIGHT UNLESS PASSING** in the US and Canada is placed at the end of the lane-addition taper; i.e., at the point where full widths for two lanes are available for traffic in the passing lane direction. Traffic is channeled to the outside lane by a solid line beginning at the lane-addition taper and continuing up to 15 m before the end of the taper, where it opens up to introduce the inner lane. Australia uses two or three signs at the lane-drop, depending on the length of the passing lane: 1) **LEFT LANE ENDS** which is equivalent to right lane ends in the US and Canada, and **500 m** signs on one post are used when the outer lane is more than 1 km long; 2) **LEFT LANE ENDS** and **MERGE LEFT** (with an arrow) signs on one post are used at a distance of 60 to 260 m from the beginning of lane-drop taper, depending on the 85th percentile speed; and 3) a **FORM 1 LANE** sign is used at the beginning of the lane-drop taper. For the entire lane-drop taper, a wide broken line maintains the width of the inside line to the end of passing lane section where it becomes the basic lane. Australia permits passing in the opposing lane except on the end tapers. A symbol sign with two 'up' arrows and one 'down' is placed at the end of passing lane-drop taper on the opposite side of passing lane, similar to the two-way traffic sign W6-3 (MUTCD 1988).

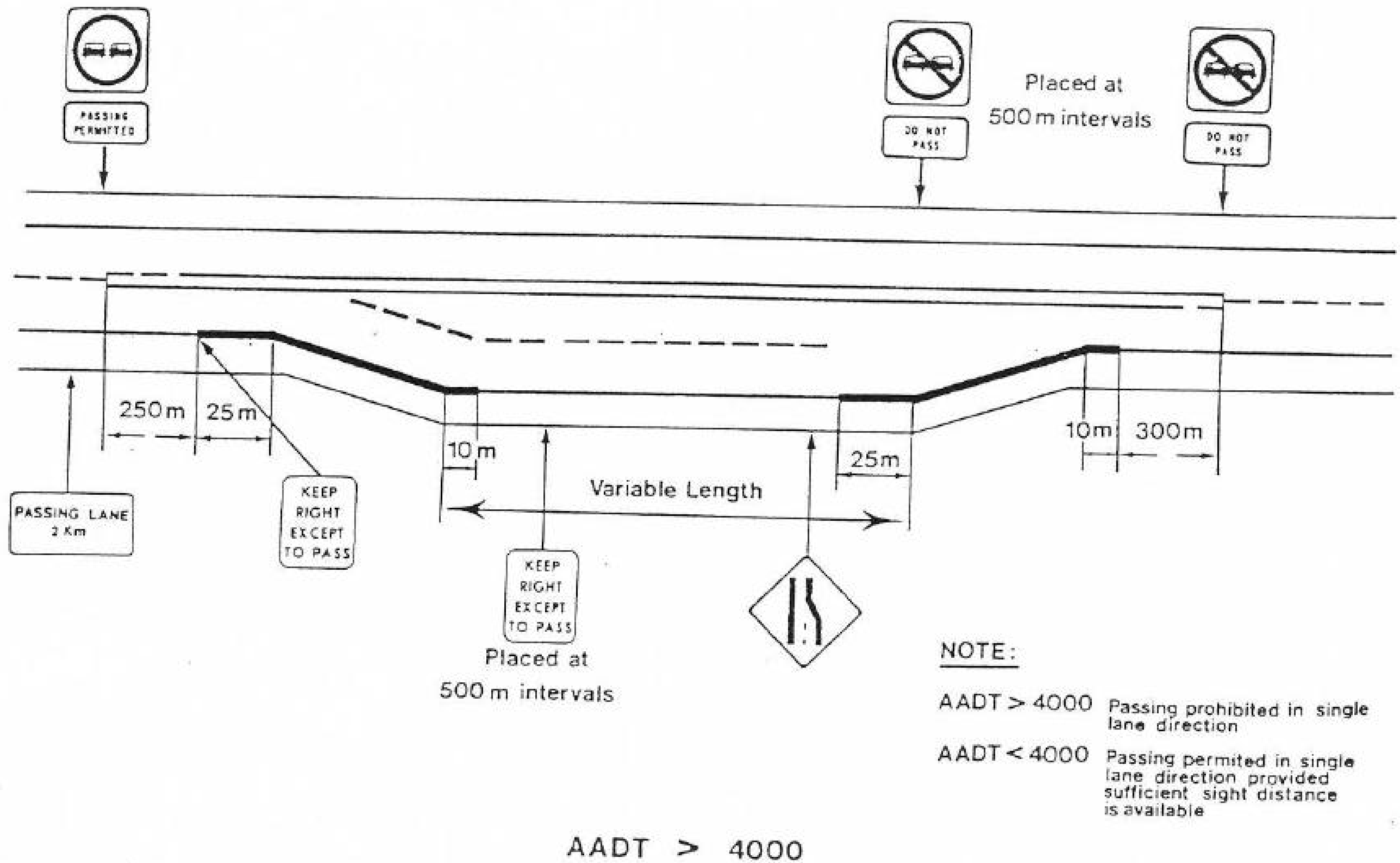
2.4.9 Signing and Marking Practice in the United States

Figure 2.9 shows the typical signing practice in the US. FHWA (Harwood and Hoban 1987) recommends placing advance signs at two to five miles (3.2 to 8 km), and ½ mile (0.8 km) upstream. The two to five miles' (3.2 to 8 km) sign is intended to reduce risky passing by impatient drivers. The ½ mile (0.8 km) sign serves to prepare both slow and fast vehicles for effective use of the passing lane. The sign recommended by Harwood and Hoban (1987) at the beginning of the lane-addition taper is **KEEP RIGHT EXCEPT TO PASS** supplemented by a broken line pavement marking to delineate the basic lane and the passing lane. This pavement marking extends to just before the beginning of the lane-drop. For dropping the lane, Harwood and Hoban (1987) recommended two signs: 1) **RIGHT LANE ENDS** (MUTCD sign W9-1) and a lane reduction sign (MUTCD sign W4-2), at 1,000 ft and 500 ft (305 m and 152 m) respectively, before the beginning of the taper. For the signing for the opposing traffic, signs used for passing and no-passing zones on conventional two-lane highways are recommended.



Source: Morrall et al. (1986b)

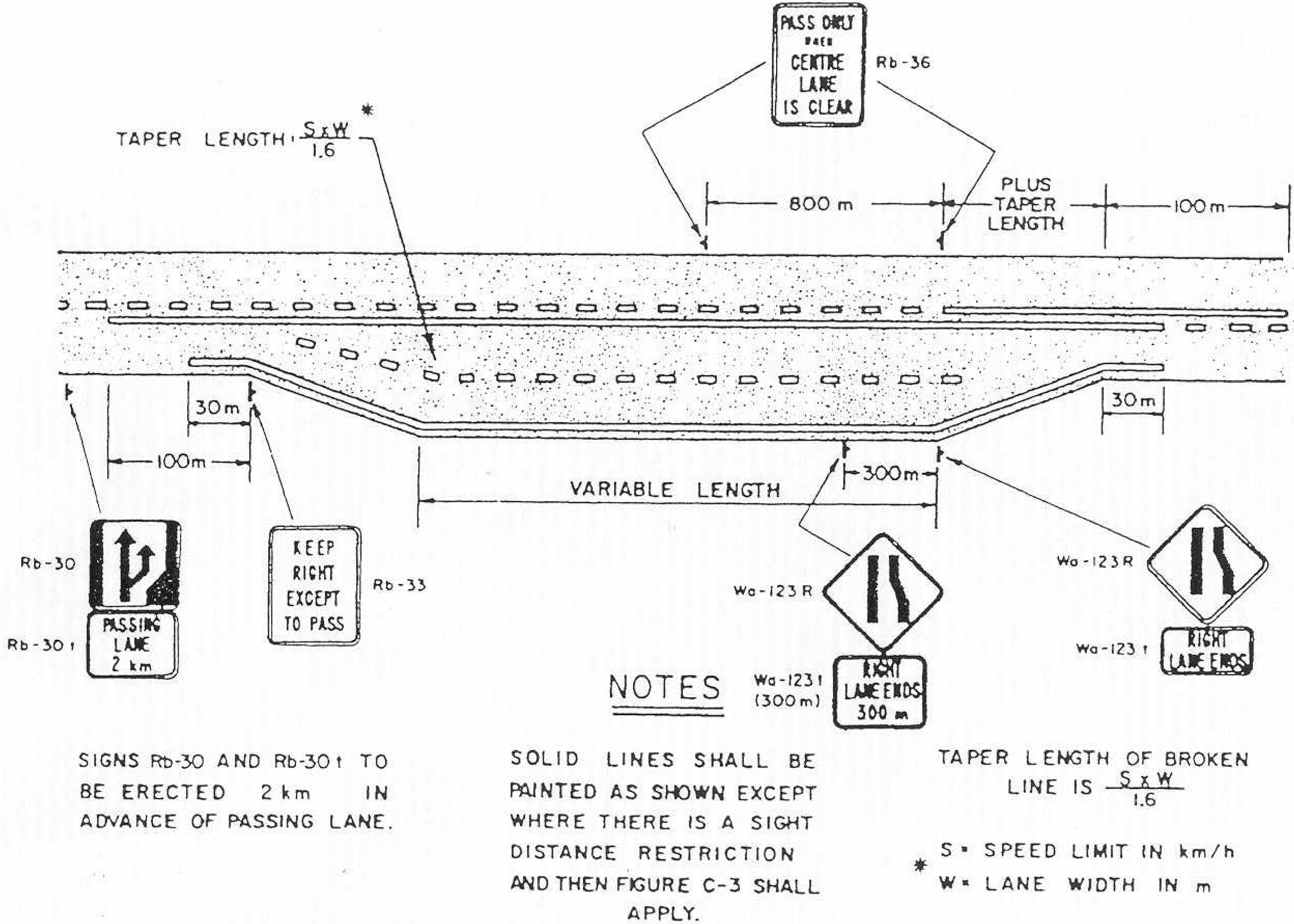
Fig. 2.6a: Alberta Transportation Passing Lane Signing and Marking.



Source: Morrall et al. (1986b)

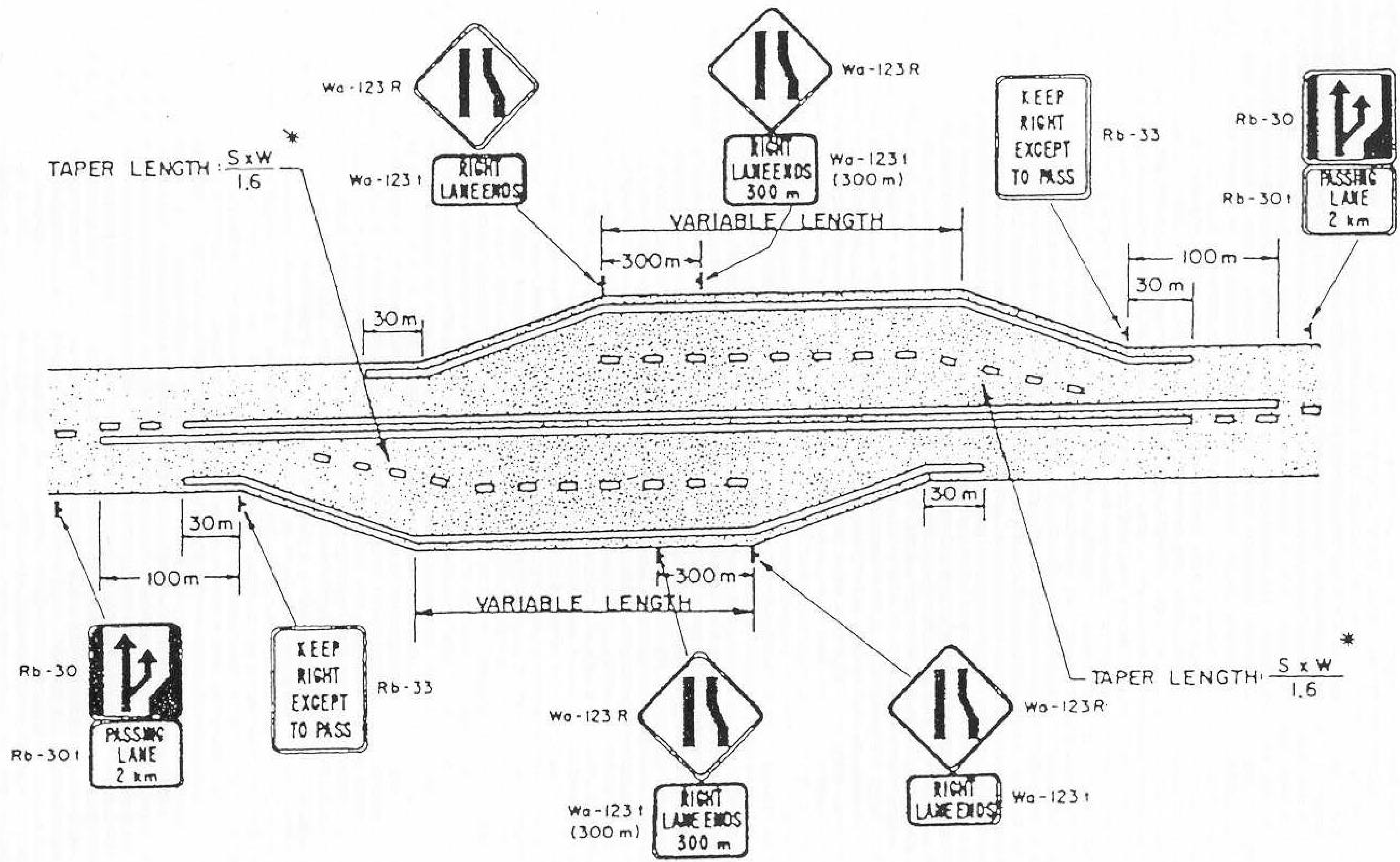
Figure 2.6b: Alberta Transportation Passing Lane Signing and Marking.

Fig. 2.6a: Alberta Transportation Passing Lane Signing and Marking.



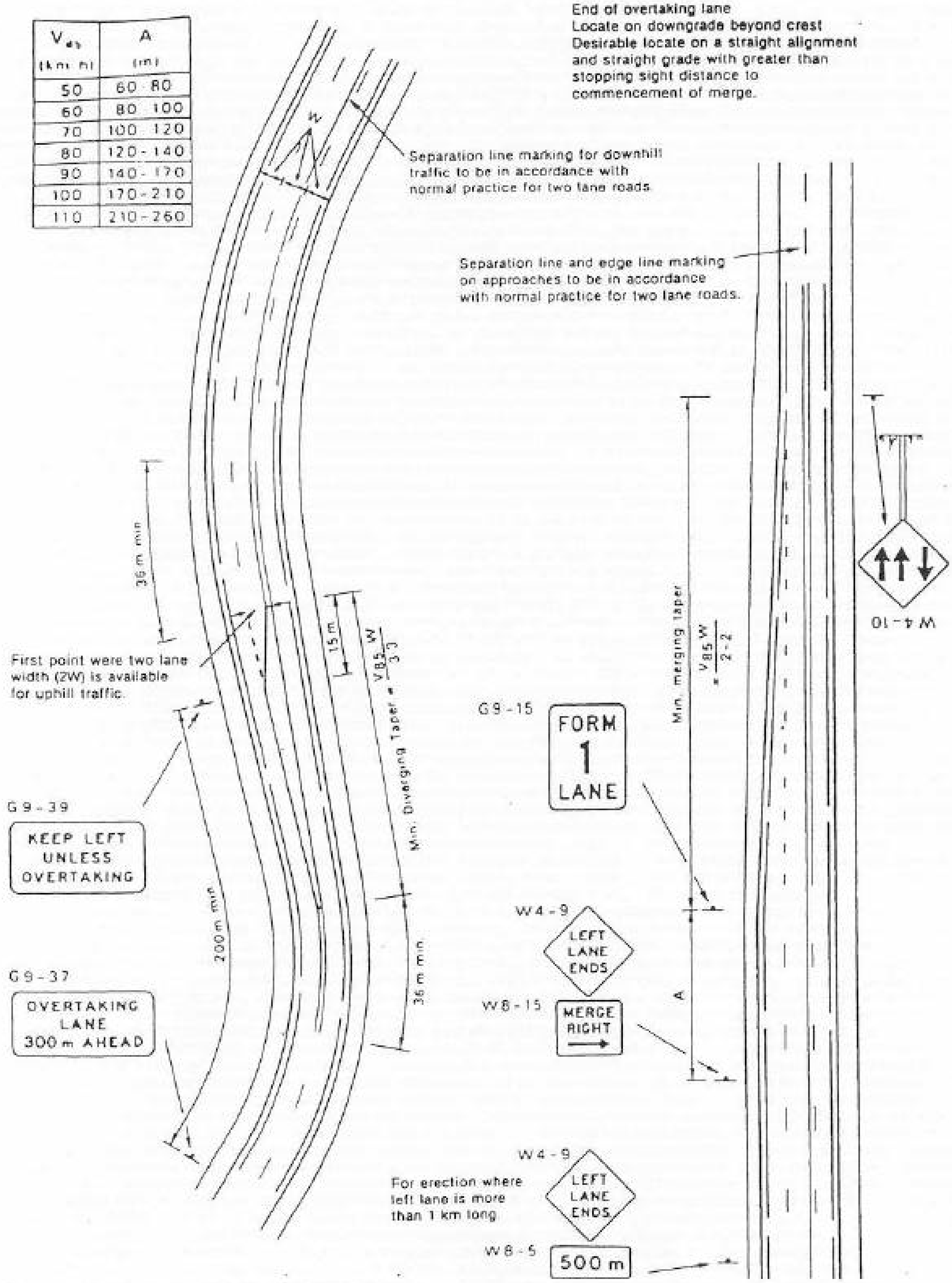
Source: Morrall and Hoban (1986a)

Figure 2.7a: Ontario Passing Lane Signing and Marking.



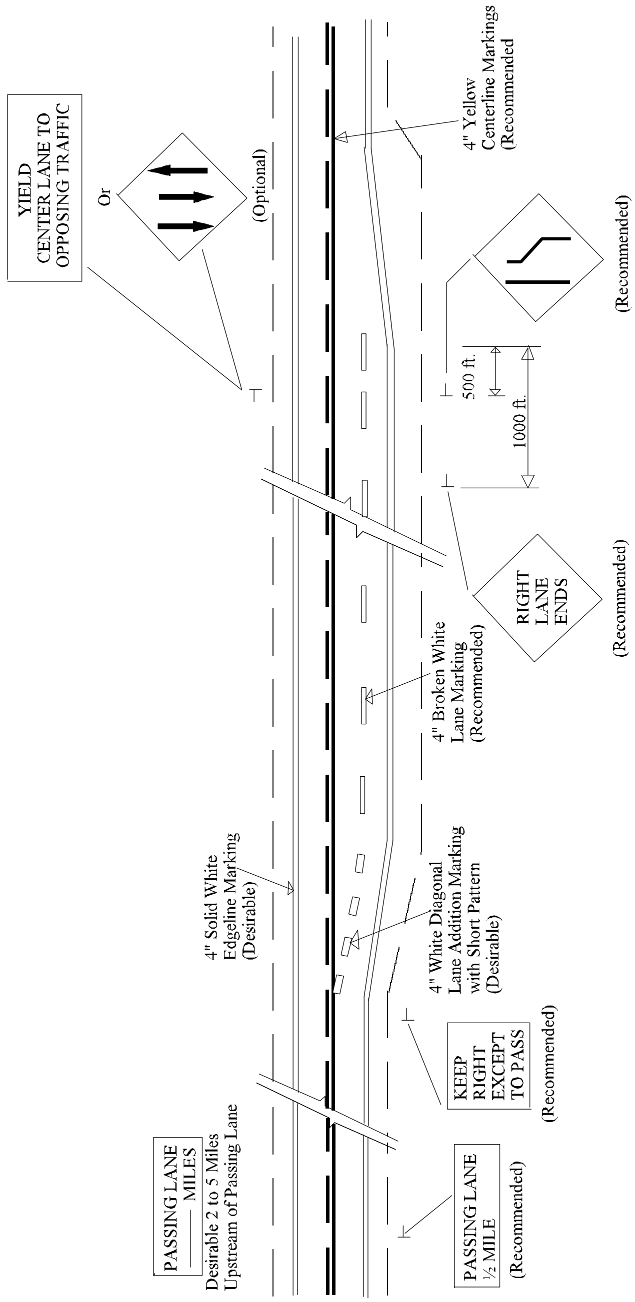
Source: Morrall and Hoban (1986a)

Figure 2.7b: Ontario Passing Lane Signing and Marking.



Source: Morrall and Hoban (1986a)

Figure 2.8: Australia Passing Lane Signing and Marking.



Source: Harwood and Hoban (1987)

Figure 2.9: US Recommended Passing Lane Signing and Marking

2.5 SAFETY

Theoretically, the effect of passing lanes on safety can be measured by before-and-after comparison of accident history, or comparison with similar sections without passing lanes. However, accident records are unreliable, and it can take a long time before a site experiences enough accidents for meaningful statistical analysis. Therefore, some researchers (Glauz and Migletz 1980; Glennon et al. 1977; Zegeer and Dean 1978) have advocated the use of the traffic conflict techniques as a supplementary measure of highway safety.

2.5.1 Accident History

Morrall and Hoban (1985) summarized previous studies on the effect of auxiliary lanes (passing and climbing lanes) on two-lane roads. Evaluating the relative safety of climbing lanes Jorgensen (1966) reviewed a small number of climbing lanes in the US and found no change in accident experience. Martin and Voorhees Associates (1978) found a 13 percent reduction in accidents on crawler (climbing) lanes in the United Kingdom (UK) due to the lanes.

The lane-drop section of the passing lane is thought to be a major safety problem on passing lanes because of merging maneuvers on those locations (Homburger 1987; Harwood and St. John 1985). Homburger (1987) investigated merging-related accidents for a five-year period on 21 locations of climbing lanes in California. He found only seven percent of the accidents were related to merging maneuvers and he concluded that merging does not seem to cause a serious safety concern. Harwood and St. John (1985) compared accident rates at lane-addition, middle, and lane-drop sections of passing lanes and came up with these results: lane-drop and lane-addition sections had the same accident rate (accidents/mile), and both had higher accidents per mile than the middle section of passing lanes.

Probably the most extensive study on passing lane safety was done by Harwood and St. John (1985) who examined 76 passing lanes in 12 US states. Comparison with untreated sites (two-lane with no passing lanes) revealed a reduction of accident rates in the order of 30 to 50 percent for nearly all cases. The study had four major conclusions: 1) passing lane sites did not increase accident rates, and probably reduced the accident rate; 2) the provision for allowing vehicles in the opposing direction to pass does not appear to lead to any safety problems at the types of sites and the flow-rate levels (up to 400 vehicles per hour) where it has been permitted by the participating states; 3) there was no indication of any marked safety problem in the lane addition and lane drop transition areas of passing lanes; and 4) there was no evidence of safety problems associated with left-turns within passing lane sections.

Taylor and Jain (1991) compared accidents in Michigan between sections with and without passing lanes for a period of five years to determine the effectiveness of passing lanes on safety. The findings in that study suggested passing lanes are effective in reducing accident rates on two-lane highways. Using a five year period accident data, the comparison of accident rates or highway sections with and without passing lanes, indicated that the passing lanes can reduce accident rates by nine to 15 percent depending on the traffic volume.

2.5.2 Traffic Conflicts

Provision of a passing lane creates two sections, lane-drop and lane-addition, which may degrade the safety of the passing lane section. Only one study was found in the literature (Harwood and St. John 1985) which reported a study on traffic conflicts at lane-drop and lane-addition sections of passing lanes. The study concluded that these areas operated smoothly because the observed value of 1.3 conflicts per 100 vehicles were much smaller than traffic conflicts found in lane-drop transition areas at other highway locations. The study cited traffic conflict rate of five to 15 conflicts per 100 vehicles reported in transition tapers of work zones.

Traffic conflict studies have been used widely in evaluating intersection safety and some investigators have used the method in studying lane-drops on four-lane highways (Graham and Sharp 1977; Cima et al. 1977). Graham and Sharp (1977) used traffic conflicts and speed to compare lane-drop taper lengths computed from two different formulas at construction zones. The two formulas, both by MUTCD are $L = WS$ and $L = WS^2/60$. Where L is the minimum taper length (ft), W is the width of the lane to be dropped (ft), and S is the speed limit or 85th percentile speed (mph). The later formula was being proposed as an alternative to the existing formula (the former). Based on traffic conflict results, Graham and Sharp (1977) concluded that there was no evidence that the new proposed taper lengths were more hazardous than the conventional lengths.

2.6 COMPUTER SIMULATION

Traffic conditions on a two-lane highway create a complex system. It depends on many factors interacting together. In analyzing such a complex system, simulation is better suited than analytical approaches (Dai et al. 1996). Harwood et al. (1988) offered the following comments regarding evaluating quality of traffic on a two-lane highway by field and computer simulation methods:

> "Field evaluation can not compare the quality of traffic operations on a highway section with and without passing lanes, but comparisons of this type can be made with a computer simulation model".

Several computer simulation programs have been developed, but the most widely known and used are the two microscopic models TWOPAS and TRARR. TWOPAS is a modified version of the TWOWAF (TWO Way Flow) model developed by the Midwest Research Institute of Kansas City (MRI) for the FHWA (Dai et al. 1996). Basic data presented in chapter eight of the 1985 HCM was generated by the TWOWAF model. TRARR is an acronym for TRAffic on Rural Roads, and was developed at the Australia Road Research Board (ARRB) between 1978 and 1990.

Comparing these two models for USA conditions, Botha et al. (1992) concluded the following:

- Both models are generally comparable in their capability to simulate traffic operations on a two-lane, two-way highway.
- TWOPAS' results for a 50 mph design-speed road in level and rolling terrain compared better with field data than did the TRARR results.
- Both models require further work before they can be applied without reservations to many situations that might arise on two-lane roads.

Botha's comparison used travel time and percent time delay. He also observed that TRARR needs more improvements to customize it to US conditions. This is probably because it was developed outside the US.

Chapter 3

STUDY METHOD AND DATA COLLECTION

The study method included a review of KDOT design practices, the study of existing passing lanes, and seven sub-studies. The seven sub-studies were conducted to assess traffic operation, safety, and users' opinions of the existing passing lanes in the state. "Video data" and "traffic count" studies assessed the traffic operation, while lane-drop conflict studies, intersection conflict studies, and accident analysis studies assessed the safety of passing lanes. The users' opinions sub-study was intended to evaluate the operation, safety, and acceptability of passing lanes from the users' perspective. Traffic conflict studies were intended to supplement the limited accident history. Comparison of passing lane configurations using a traffic simulation model was intended to compare passing lane configurations from a traffic operation performance stand point.

KDOT was responsible for the collection of traffic count data at all sites where filming took place, and provided Kansas State University (KSU) with available accident data. The KSU team was responsible for traffic conflict data collection and evaluating video data.

The intent was to conduct all studies using a statistical experiment framework. However, the "video data" and "traffic count" studies are classified as observational studies, because randomization within the limited number of sites was not considered practical. Priority for site selection was based on available power and suitable camera location.

3.1 KDOT DESIGN PRACTICES

Since there is no formal design handbook(s) for the design of passing lanes, procedures used for designing existing passing lanes were compiled from interviews with design personnel at KDOT.

3.1.1 Planning

The topics considered under passing lane planning include warrants, location, spacing, and configuration. The decision to provide passing lanes on a section of highway in Kansas is based on the traffic volume. A threshold value of 3,000 vehicles a day with 25 percent heavy commercial vehicles was recommended. The two Kansas highways (US 50 and US 54) provided with passing lanes had the highest AADT among the two-lane highways in the state road network. The 1992 AADT for US 50 was 3,900, while for US 54 was 5,000 vehicles per day. Heavy commercial vehicle statistics for 1992 were not available.

KDOT spaced passing lanes using guidelines from a FHWA report (Harwood and Hoban 1987). The guidelines suggest initial spacing of 10 to 15 miles (16 to 24 kilometers) on highways that need only moderate improvement in passing opportunities. The spacing can then be reduced to three to five miles (4.8 to 8 kilometers) when the passing demand to the highway increases later. The spacing on US 50 is three to four miles (4.8 to 6.4 kilometers), and that on US 54 is 5 to 10 miles (8 to 16 kilometers).

Passing lanes in Kansas were located more or less for convenience on highway sections that were being reconstructed or improved, while side road intersections were avoided as much as possible. The strategy of avoiding crossroads was difficult because of the high density of crossroads in Kansas. On the highways studied, crossroads averaged one every 1.12 miles (1.79 kilometers). At the passing lane sites studied, seven crossroads and seven driveways were within the passing lane sections. The KDOT approach can be summarized as follows:

- consider passing lanes where adequate sight distance is available at the lane-addition and lane-drop tapers.
- avoid major intersections.
- consider passing lanes on proposed highway improvement projects to reduce construction unit cost by combining overhead costs.
- determine termini using sight distance guidelines from the FHWA information guide report (Harwood and Hoban 1987). (The guide recommends locating passing lane termini where there is an adequate sight distance of a minimum of 1,000 feet.)

Existing passing lanes in Kansas have a side-by-side configuration. On the sections studied the beginning of each passing lane is aligned with the end of the opposing passing lane, except at one site (4 miles west of Pratt), where the beginning of the east-bound lane and the end of west-bound lane are offset by about 800 feet. However, within this offset, there is an intersecting side road with relatively high AADT of 310 (1994 counts). It is possible that the designer was trying to avoid an intersection within the full width (4-lanes) of a passing lane section.

3.1.2 Geometric Elements

Passing lane geometric elements discussed under this section include lane length, lane-drop and lane-addition taper lengths, lane and shoulder widths, and cross slope. KDOT uses the FHWA report (Harwood and Hoban 1987) to determine the length (minus tapers) of the passing lanes. Lengths for existing passing lanes range from 0.6 to 1.1 miles (0.96 to 1.76 kilometers). Table 3.1 shows the FHWA recommended passing lane lengths as a function of one-way volume.

Table 3.1: Optimum Design Lengths for Passing Lanes

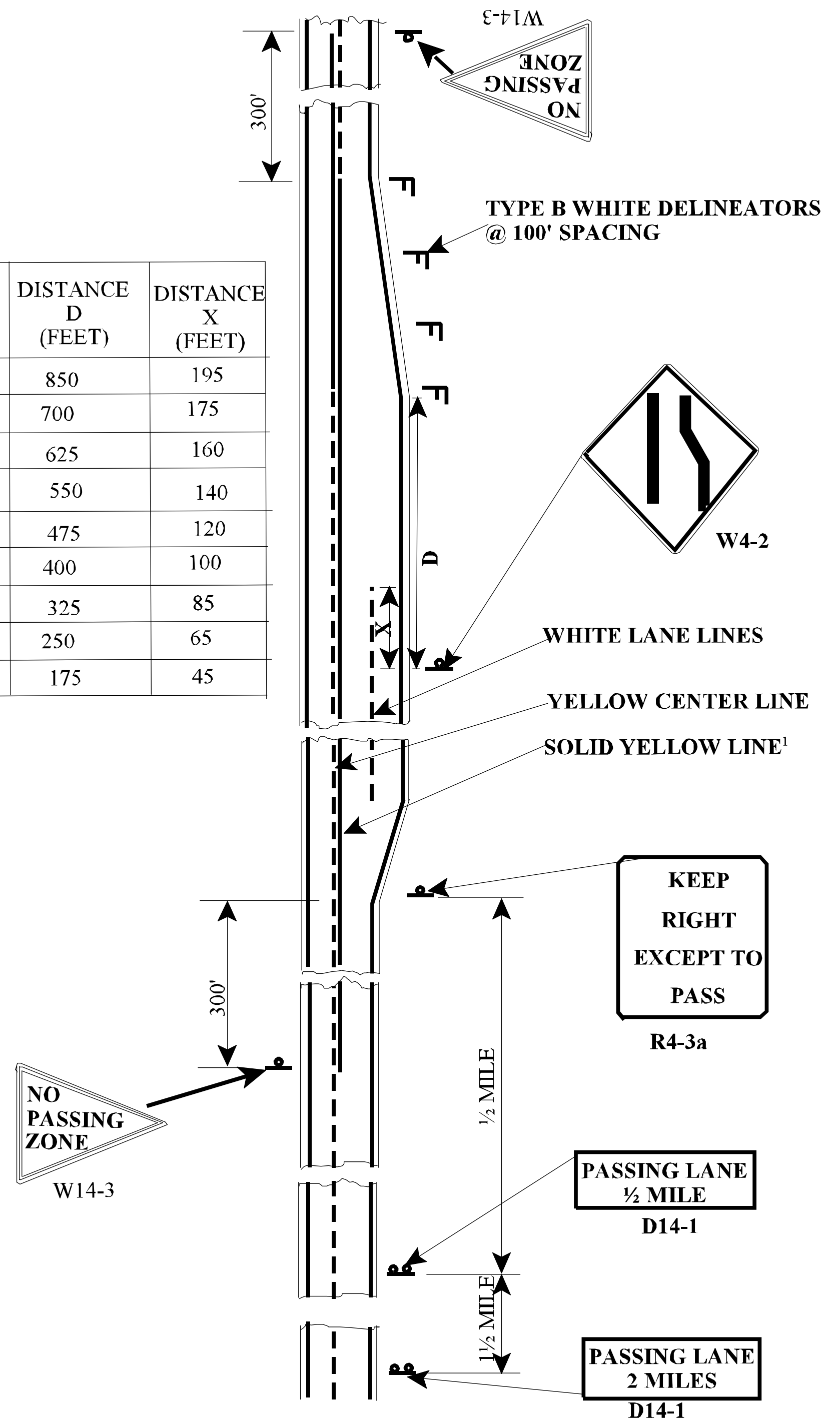
One-Way Flow Rate (Veh/hr)	Optimal Passing Lane Length (mi)
100	0.50
200	0.50 - 0.75
400	0.75 - 1.00
700	1.00 - 2.00

Source: Harwood and Hoban (1987)

Lane-drop and lane-addition taper lengths were determined using MUTCD and FHWA guidelines, respectively. MUTCD guidelines, which are also cited by FHWA (Harwood and Hoban 1987), recommend a minimum length (in feet) for the lane-drop taper equal to the product of lane width (in feet) and off-peak 85th percentile speed or the speed limit (in mph). The lane-addition taper length recommended by FHWA is ½ to ⅔ of the lane-drop taper length. With a 12-foot lane width and 60 mph assumed as the off-peak 85th percentile speed, the minimum taper lengths for the lane-drop and lane-addition as suggested by the guides would be 720 and 360 feet (220 and 110 meters), respectively. The minimum lengths observed in the field are 518 and 243 feet (158 and 74 meters), respectively. The guidelines used by KDOT for determining cross-section elements are the same as those used to determine cross-section elements on conventional two-lane highway sections.

3.1.3 Signing and Marking

Figure 3.1 show the typical signing and marking for an auxiliary passing lane in Kansas developed in 1995 by KDOT. Because this standard was developed after most of the existing passing lanes were in place, there is variation from the signing and marking shown on Figure 3.1 and those found in the field. These are summarized as follows: 1) the second advance sign in the field at all passing lanes is placed only ¼ mile upstream instead of the ½ mile as indicated in Figure 3.1; 2) the standard suggests two advance signs at 2 and ½ mile upstream of the passing lane (passing lanes on US 54 have two advance signs: the first sign at 2 miles and the second sign at ¼ mile upstream of the passing lane, while passing lanes on US 50 have only one advance sign, at ¼ mile upstream the passing lane); 3) at the beginning of the lane-addition section of passing lanes, those on US 50 are signed **KEEP RIGHT EXCEPT TO PASS**, while those on US 54 are signed **SLOWER TRAFFIC KEEP RIGHT**, the standard suggests **KEEP RIGHT EXCEPT TO PASS**.



POSTED SPEED (MPH)	DISTANCE D (FEET)	DISTANCE X (FEET)
65	850	195
55	700	175
50	625	160
45	550	140
40	475	120
35	400	100
30	325	85
25	250	65
20	175	45

[1]Due to side-by-side configuration, center line is marked by double yellow lines
Source: KDOT
Figure 3.1: Typical Signing and Marking for Auxiliary Passing Lane.

The five major differences between the signing and marking recommended by Harwood, shown on Figure 2.9, with that recommended by KDOT, shown on Figure 3.1, are summarized in Table 3.2: 1) FHWA recommends two advance signs for the lane-drop, while KDOT recommends only one sign; 2) FHWA recommends a diagonal lane-addition marking, while KDOT does not mark the lane-addition section; 3) KDOT uses post-mounted delineators along the length of the lane-drop taper length; 4) the FHWA guideline indicate that opposing traffic may be informed of a passing lane for the opposite direction by a sign showing one arrow 'up' and two arrows 'down', (somewhat similar to MUTCD sign W6-3). This is considered as an optional sign, and according to KDOT standards there is no specific information to inform the driver of the coming passing lane in the opposite direction; and 5) FHWA permits passing in the opposing direction throughout the entire passing lane, while KDOT restricts passing in the opposing direction only at the lane-drop section and in 300 ft (91 m) downstream of the passing lane.

Table 3.2: Major Differences Between KDOT and the Harwood Study Regarding Passing Lane Signing and Marking.

Signing/Marking Issue	**KDOT**	**Harwood Study**
Advance signing	One sign	Two signs
Lane-addition marking	No marking	White diagonal marking
Lane-drop treatment	Post-mounted delineators	No treatment
Opposite direction signing	No signing	A sign showing one arrow up and two arrows down
Passing in the opposing direction	Restricted at the lane-drop	Permitted throughout the entire passing lane

3.2 EXISTING PASSING LANES

3.2.1 Locations, Geometric and Traffic Conditions at Passing Lanes in Kansas

A field visit was made to all passing lanes prior to the selection of the study sites. Table 3.3 and Figure 3.2 shows the location and features for the nine passing lane sites in the state at the time of this study, and Table 3.4 shows the geometric and traffic data for these passing lanes sites. On eight sites, the passing lanes have a side-by-side configuration. One site, west of the city of Pratt, has an overlapping configuration as discussed earlier. Signing and marking were explained above in section 3.1.3. The parameters of Tables 3.3 and 3.4 were obtained from construction drawings and field observations during a site visit.

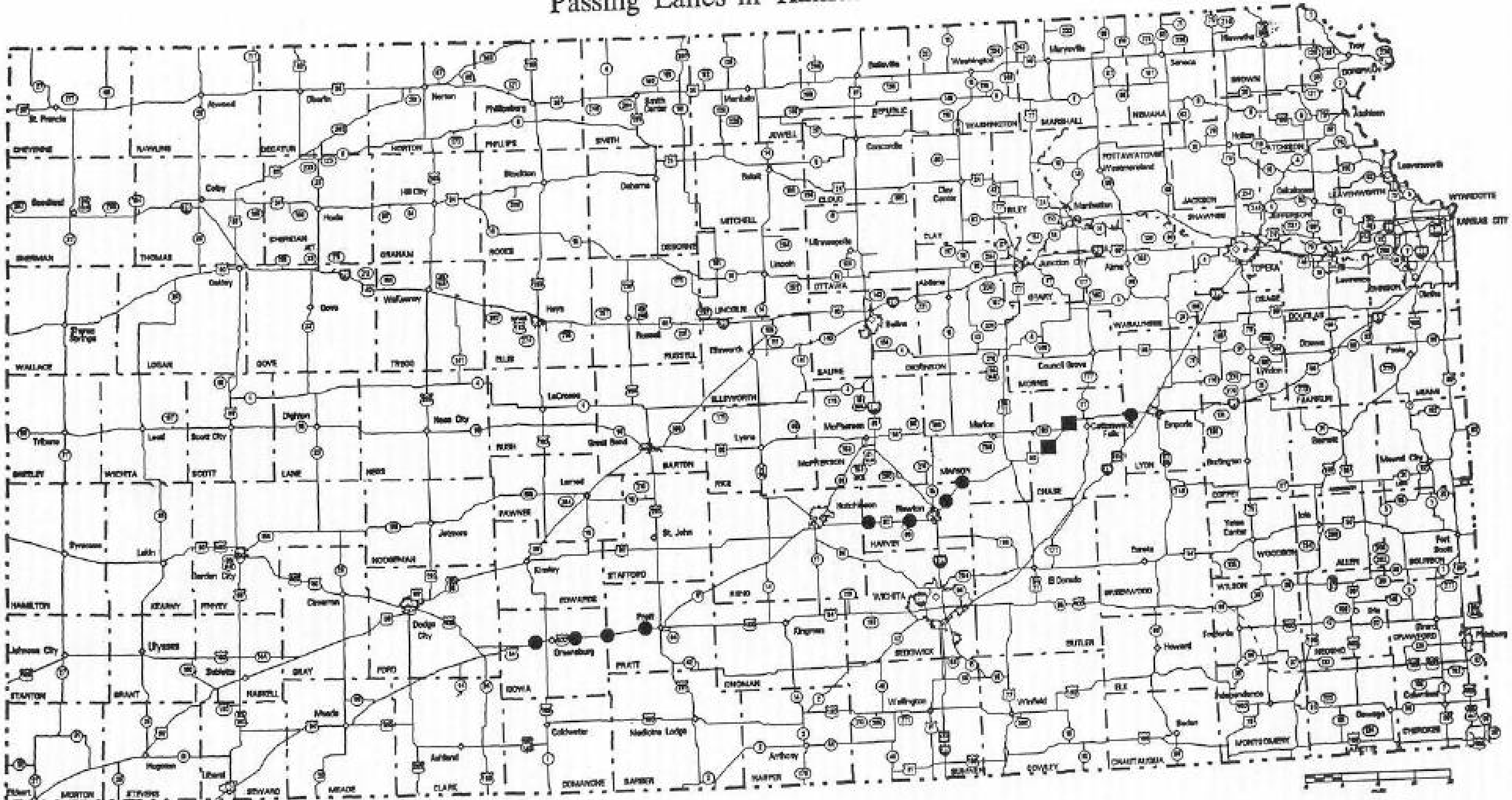
Table 3.3: Locations of Passing Lane Sites[a] in Kansas.

S/N[b]	KDOT PROJECT No.	HIGHWAY	LOCATION		COUNTY
			Route Milepost	Location	
1	54-49 K-3180-01	US 54	101 to 103	5 Miles West of Greensburg	Kiowa
2	54-49 K-3196-01 Part I	US 54	113 to 115	3 Miles West of Haviland	Kiowa
3	54-76 K-4045-01	US 54	125 to 127	3 Miles West of Cullison	Pratt
4	54-76 K-4045-01	US 54	132 to 134.5	4 Miles West of Pratt City	Pratt
5	50-40 K-3386-01	US 50	259 to 261	3 Miles West of K-89	Harvey
6	50-40 K-3386-02	US 50	265 to 267	3 Miles East of K-89	Harvey
7	50-40 K-4058-01	US 50	284 to 286	3 Miles North-East of Walton	Harvey
8	50-57 K-3219-01	US 50	289 to 291	2 Miles West of Peabody	Marion
9	50-56 K-2853-01	US 50	338 to 340	6 Miles West of Emporia	Lyon

[a] Each site have two passing lanes, one in each direction.

[b] S/N = Site Number.

Passing Lanes in Kansas



KANSAS DEPARTMENT OF TRANSPORTATION
BUREAU OF TRANSPORTATION PLANNING

● Existing ■ Under Construction

Source: Kansas Department of Transportation

Figure 3.2: Location of Passing Lane Sites

Table 3.4: Geometric, Traffic Data and Completion Date for Passing Lanes in Kansas.

S/N[1]	DIRECTION	DIVERGING SECTION		MERGING SECTION		PASSING LANE		SIGNING[3]	ADT (1997)		COMPLETION DATE
		LENGTH [Ft]	HA[2]	LENGTH [Ft]	HA[2]	LENGTH [Ft]	HA[2]		ALL	TRUCKS	
1	Eastbound	300	Curve	840	Curve	5800	Curve	Type 2	4255	1460	11/20/92
	Westbound	300	Tangent	840	Curve	5800	Tangent	Type 2			
2	Westbound	300	Tangent	840	Tangent	4000	Tangent	Type 2	4775	1100	6/24/94
	Eastbound	300	Tangent	840	Tangent	4000	Tangent	Type 2			
3	Westbound	250	Tangent	720	Tangent	3880	Tangent	Type 2	4710	1095	8/19/94
	Eastbound	250	Tangent	720	Tangent	3880	Tangent	Type 2			
4	Westbound	250	Tangent	518	Curve	4020	Tangent	Type 2	6325	1120	8/19/94
	Eastbound	243	Tangent	720	Tangent	4720	Tangent	Type 2			
5	Westbound	250	Tangent	840	Tangent	4000	Tangent	Type 1	5030	1050	10/94
	Eastbound	250	Tangent	840	Tangent	4000	Tangent	Type 1			
6	Westbound	250	Tangent	840	Tangent	4000	Tangent	Type 1	6005	1095	10/94
	Eastbound	250	Tangent	840	Tangent	4000	Tangent	Type 1			
7	Westbound	560	Tangent	840	Tangent	3300	Tangent	Type 1	5410	1210	8/26/94
	Eastbound	560	Tangent	840	Tangent	3200	Tangent	Type 1			
8	Eastbound	560	Tangent	840	Tangent	4360	Tangent	Type 1	3925	1195	7/19/94
	Westbound	560	Tangent	840	Tangent	4360	Tangent	Type 1			
9	Eastbound	243	Tangent	840	Tangent	4594	Tangent	Type 1	5160	1535	12/27/94
	Westbound	250	Tangent	735	Tangent	4522	Tangent	Type 1			

[1] S/N = Site Number.
[2] HA = Horizontal Alignment.
[3] Type 1 = "**KEEP RIGHT EXCEPT PASSING**", plus one advance sign at ¼ mile upstream;
Type 2 = "**SLOWER TRAFFIC KEEP RIGHT**", plus two advance signs at 2 and ¼ miles upstream.

3.3 VIDEO DATA

The first meeting between the KSU research team and the KDOT advisory committee was held in Topeka on June 24, 1996. During this meeting it was agreed that a video camera was to be used to monitor traffic behavior in the passing lanes. The objective was to observe traffic behavior at passing lanes in terms of passing maneuvers and lane utilization. Passing rates, keep right compliance, platooning characteristics, and passing to the right behavior were analyzed at the passing lane sites.

3.3.1 Site Selection

Because of constraints on both time and budget, it was not possible to film all passing lanes. The process of selecting sites is known as sampling. The main objectives of sampling are: 1) to produce a sample which will yield unbiased estimator(s) of the population, and 2) to minimize the variance of the estimator. To achieve the first objective, stratification sampling was used. Each of the two highways (US 50 and US 54) were used as a strata to select a number of sites from each strata. The second objective is usually achieved by selecting sufficiently large, random sample. In this sub-study, a trade-off had to be made between sample size and cost, and randomization of the sites was not possible.

Four sites were selected for video filming and traffic count data collection: two sites on US 50, one at three miles west of K-89 junction (Burton site) in Harvey County, and the other two miles west of Peabody in Marion County, and two sites on US 54, one at four miles west of Pratt in Pratt County and the second at three miles west of Cullison. The primary criterion used in selecting the sites was the presence of a power line near one of the two lane-addition sections. The Pratt site was selected so that the effect of a major side road intersecting within the passing lane could be observed. Because sites were not selected at random, inferences are limited to only those sites studied. For this reason, this study is classified as an observational study.

3.3.1.1 Site Characteristics

Figure 3.3 shows the vertical alignment for the Peabody and Pratt passing lane sections. The Peabody passing lane section (westbound) is 0.826 miles (1.321 kilometers) long, begins on a + 1.17 percent grade on a crest vertical curve, changes to +0.92 percent grade followed by +2.05 percent grade, and ends on a -0.56 percent grade on a crest vertical curve. The Burton passing lane section (westbound) is 0.758 miles (1.212 kilometers) long and on level grade. The Pratt passing lane section (eastbound) is 0.894 miles (1.43 kilometers) long, begins on a -0.55 percent grade, changes to a -0.8 percent grade, then to +0.5 percent grade, and ends on a -0.09 percent grade. The Cullison passing lane section (eastbound) is 0.735 miles (1.176 kilometers) long on a level grade.

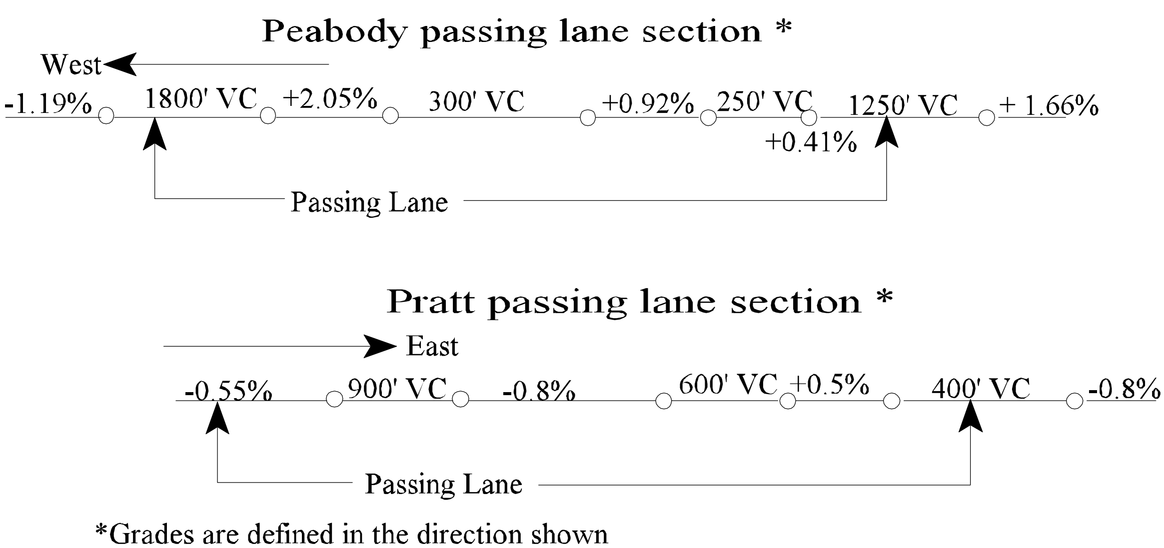


Figure 3.3: Vertical Alignments for Peabody and Pratt Passing Lane Sections

Table 3.5 shows the list of filmed passing lanes and some key attributes. For each site except Pratt, the camera was placed upstream of the passing lane. The camera at the Pratt site was placed after the start point of the east-bound lane. Filming was done continuously day and night for periods of time ranging from three to seven days.

Table 3.5: List of Filmed Passing Lanes

Highway	Site Name and Location	Length	Direction
US 50	Peabody (2 miles west of Peabody)	0.826 mi	Westbound
	Burton (3 miles west of K-89)	0.758 mi	Westbound
US 54	Pratt (4 miles west of Pratt)	0.894 mi.	Eastbound
	Cullison (3 miles west of Cullison)	0.735 mi	Eastbound

3.3.1.2 Experimental Set Up and Data Collection

Four passing lane sections were filmed using time-lapse, video cameras to observe traffic patterns and behavior at the beginning of each passing lane. Three sites (Peabody, Burton, and Pratt) were filmed using a black and white, high resolution video camera from Mission Electronics in Kansas City. The fourth site (Cullison) was filmed using a low resolution color, video camera from ATD Northwest in Washington state.

The system from Mission Electronics relied on power from utility companies. The ATD system was powered by a marine battery. The original plan was to record traffic movements towards the end section of one lane and the beginning section of the opposing passing lane (because the configuration is side-by-side) continuously for seven days and nights.

The camera was mounted on a pole approximately 150 feet (45 meters) from the passing lane. The height above the ground was variable depending on the terrain, but not less than about 18 feet (5.5 meters). The pole was placed at a distance of 30 feet (9 meters) from the edge of the outside lane line. The poles for the original three sites were erected by the power companies operating in those areas. The pole at the Cullison site was erected by the KDOT staff from District Five, Area One, in Pratt.

The Peabody site was the first site videotaped. Filming was done at the east-end of the passing lane continuously for one week, and the tapes were changed daily. A preliminary review of the tapes from the Peabody site revealed some problems associated with the height of the camera and poor contrast between lane lines and the concrete pavement surface. The height problem was due to the terrain (embankment).

The Burton site was the second that was videotaped. The camera was placed at the east-end of the passing lane section. The camera was mounted at about 25 feet (7.5 meters) above the ground and the terrain was relatively flat. Filming was accomplished similar to the manner at the Peabody site, except that about 12 hours of data were lost when heavy rains switched off the circuit breaker on the power outlet.

Filming at the Pratt site was also completed in one week using two cameras at the west-end of the passing lane. One camera recorded the passing lanes, while the second camera recorded the intersection with the side road. Because of the overlapping configuration at this site, the pole was placed at about 800 feet (244 meters) from the end of the westbound passing lane and about 250 feet (76 meters) from the beginning of the eastbound passing lane.

The Cullison site was filmed for four days using the system from ATD Northwest. The camera, equipped with a wide angle lens was mounted at the west-end of the passing lane section. After a review of the first recorded tape, it was found that the system was unsuitable for this job because it was difficult to view the whole lane-addition section. Discussion with the ATD staff revealed that the wide angle lens was only suitable for intersection filming. ATD Northwest then sent a zoom lens more suitable for corridor-type recording. With the zoom lens, it was possible to continue normal filming. However, the system had two main problems: 1) a loose connection between the battery power cable and the camera resulted in the loss of data for a whole day; and 2) the focusing and zooming controls could not be operated while the camera was mounted. The KDOT staff in Pratt assisted in changing tapes and charging batteries.

3.4 TRAFFIC COUNTS

The objective of this study was to assess the impact of passing lanes by comparing traffic platooning characteristics and speed at three locations on a passing lane site: 1) immediately upstream of a passing lane; 2) within a passing lane; and 3) immediately downstream of a passing lane.

3.4.1 Sites

Since the initial plan was to conduct traffic counts during the same period the video camera was recording, the same sites selected for video filming were also used for traffic counting. The Halstead site (3 miles west of K-89 junction), added later by KDOT, is the only site which was counted and not filmed. Due to the same limitation as those noted in the video data study, this study is classified as an observational study.

3.4.2 Experimental Set Up and Data Collection

Although it was planned to count traffic during the same period the video camera was recording, it was difficult to maintain such an arrangement due to logistical problems and the tight schedule of the KDOT traffic count team. Thus the two activities were done at different times.

Traffic count data were collected by KDOT's planning division using automatic traffic counters known as TrafiCOMP III. These counters have the capability of counting volume, speed, vehicle classification and time headway. Vehicle classification was done according to the 13 FHWA classes referred to as "Scheme F" which is used as the guideline for all of the KDOT's normal vehicle classification data. Figure 3.4 shows the FHWA vehicle classification scheme. Speed data was grouped in five mph intervals. Headway data was also grouped in intervals ranging from 1 second to 10 seconds. Counts were taken at the middle of the passing lane section (4 lanes) and at both ends of the passing lane (2 lanes), approximately 1,000 feet (305 meters) after the passing lane termini.

Data was collected during the fall of 1996 and 1997. The Peabody site was counted during the week of September 3-6, 1996. Unfortunately, the sensors at the "within the passing lane" location operated only for a short period (six hours west bound and 13 hours east bound) before they were malfunctioning. No attempt was made to count traffic at the "within passing lane" location for the subsequent sites. The Pratt site was counted on September 23 -25, 1996. The Cullison site was counted on October 13 -15, 1997. The Halstead and Burton sites were counted on October 13-14, 1997. Table 3.6 show the data collection features of the counted sites.

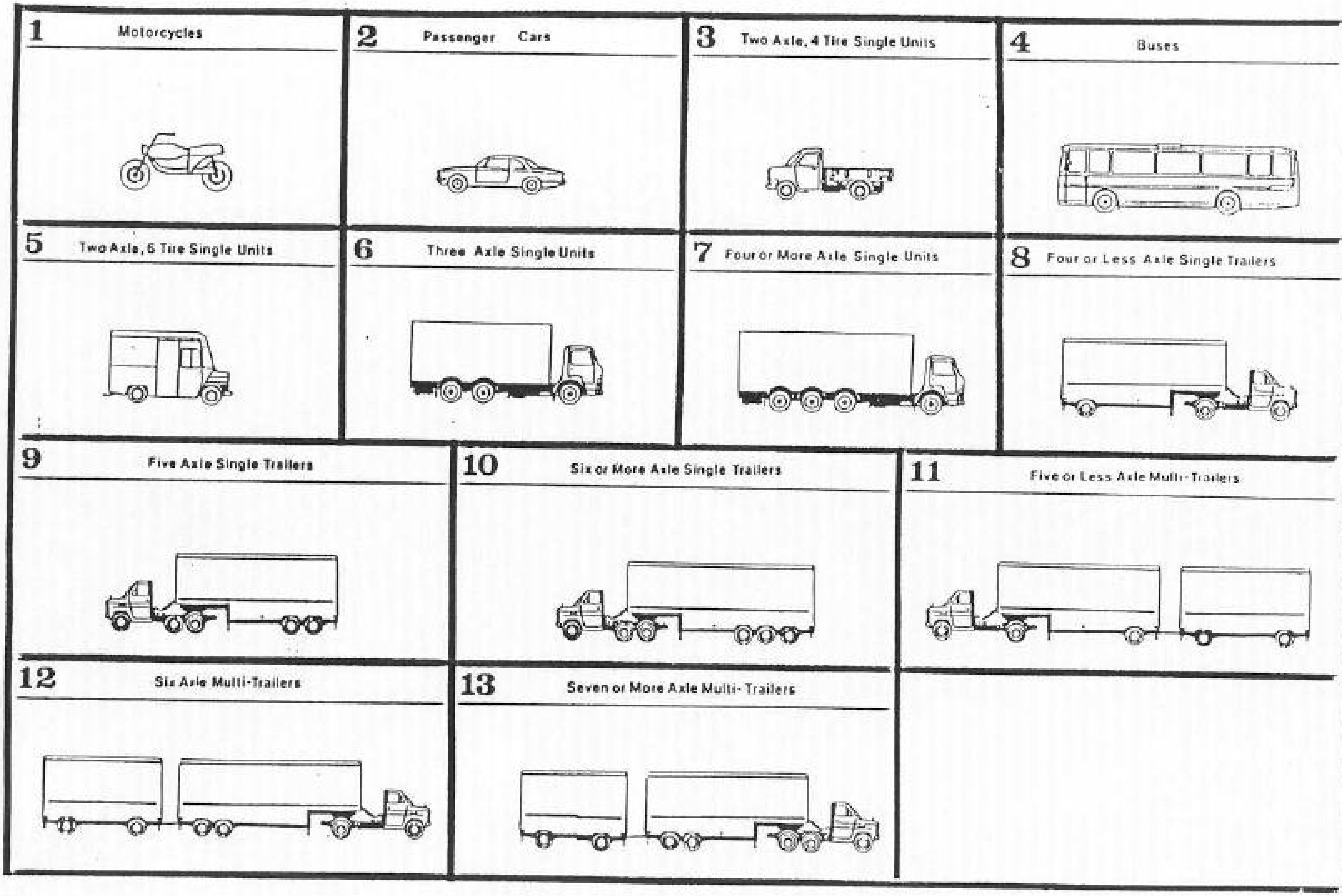


Figure 3.4: FHWA Vehicle Classification

Table 3.6: Summary of Passing Lane Site Traffic Data Collection

Site	Counting Dates	Direction	Hours Counted	Headway Classification	Speed Classification	Vehicle Classification
Peabody	Sept. 3-6, 1996	East	50	Type A[1], Type B[2]	Type 1[4]	FHWA Scheme F
		West	66			
Pratt	Sept. 23-25, 1996	East	17	Type B[2]	Type 2[5]	
		West	24			
Cullison	Oct. 13-15, 1997	East	40	Type C[3]		
		West	10			
Halstead	Oct. 13-14, 1997	East	24			
		West	24			
Burton	Oct. 13-14, 1997	East	15			
		West	24			

[1]Type A = (10), (10-19), (20 - 29), and (30) seconds.
[2]Type B = (2), (2-4), (4-6), and (>6) seconds.
[3]Type C = (1), (1-2), (2-3), (3-4),(4-5), (5-6), and (>6) seconds.
[4]Type 1 = (40), (40- 45), (45 - 50), (50 - 55), (55 - 60), (60 - 65), (65 - 70), (70 - 75), (75 - 80), and (> 80) mph.
[5]Type 2 = (50), (50 - 55), (55 - 60), (60 - 65), (65 - 70), (70 - 75), and (> 75) mph.

3.5 LANE-DROP TRAFFIC CONFLICTS

The rise of highway speed limits in 1996 has resulted in some lane-drop taper lengths being shorter than the recommended minimum length for the new speed limits. One of the objectives of this study was to find out whether shorter taper lengths cause more traffic conflicts than longer taper lengths. The study was limited to a merge-to-the-left movement because of the type of lane-drop used by KDOT, allows merging to the left lane.

3.5.1 Experimental Design

The experimental design considered the lane-drop section of a passing lane as the experimental unit with an intended inference space of all existing lane-drop sections on passing lanes in Kansas. The number of traffic conflicts and erratic maneuvers were used as response variables. Erratic maneuvers and conflicts are considered as late merges. For low volumes, most vehicles will merge upstream before reaching the merging taper, while for heavy flow there may be late mergers within the merging taper which couldn't find an opportunity to merge early. From a safety point of view, early merges create less accident risk than late merges in which drivers have to "force" into the continuing stream.

Taper length, signing type, horizontal alignment, site, traffic volume, traffic composition, time of the day, and geographical location of the highway may cause variations within the response variables. However, only taper length, highway location, traffic volume, and traffic composition were considered the most important and, therefore, accounted for in the experiment designed for this study. The number of factors which could be included in this study was constrained by the available number of sites. Only nine sites; i.e., 18 passing lane sections, were available for the study.

Taper Length: It was hypothesized that longer taper lengths are safer than shorter ones because with longer taper lengths, a merging driver will have more time to adjust vehicle speed and obtain a suitable gap to merge into the adjacent traffic stream. The length required to make all merges risk free would likely be uneconomical. MUTCD (FHWA,1988) recommends a desirable minimum taper length (L) of 780 feet (238 meters) for a speed limit of 65 mph (110 km/hr). Both highways in this study are posted with a 65 mph speed limit. The factor, "length," had two levels: 1) low (less than 780 feet (238 meters)); and 2) high (greater or equal to 780 feet (238 meters)).

Highway Location: Both US 50 and US 54 highways run in an east-west direction. US 50 is in the middle of the state and US 54 is in the southern part of the state. It is hypothesized that US 54, being close to the border of the state, will have a significantly higher proportion of out-of-state motorists, and hence drivers less familiar with the highway as compared to US 50. The highway location is extraneous information, hence used as a blocking factor in the experiment design.

Traffic Volume: Since traffic volume would influence the levels of traffic conflicts resulting from merging maneuvers, the response variable was transformed from "number of conflicts" to "number of conflicts per 100 vehicles." Glauz and Migletz (1980) suggested that conflict rates (conflicts/volume) rather than counts are more appropriate when analyzing traffic conflicts. Also, Graham and Sharp (1977) analyzed lane-drop traffic conflicts using rates instead of the number of conflicts.

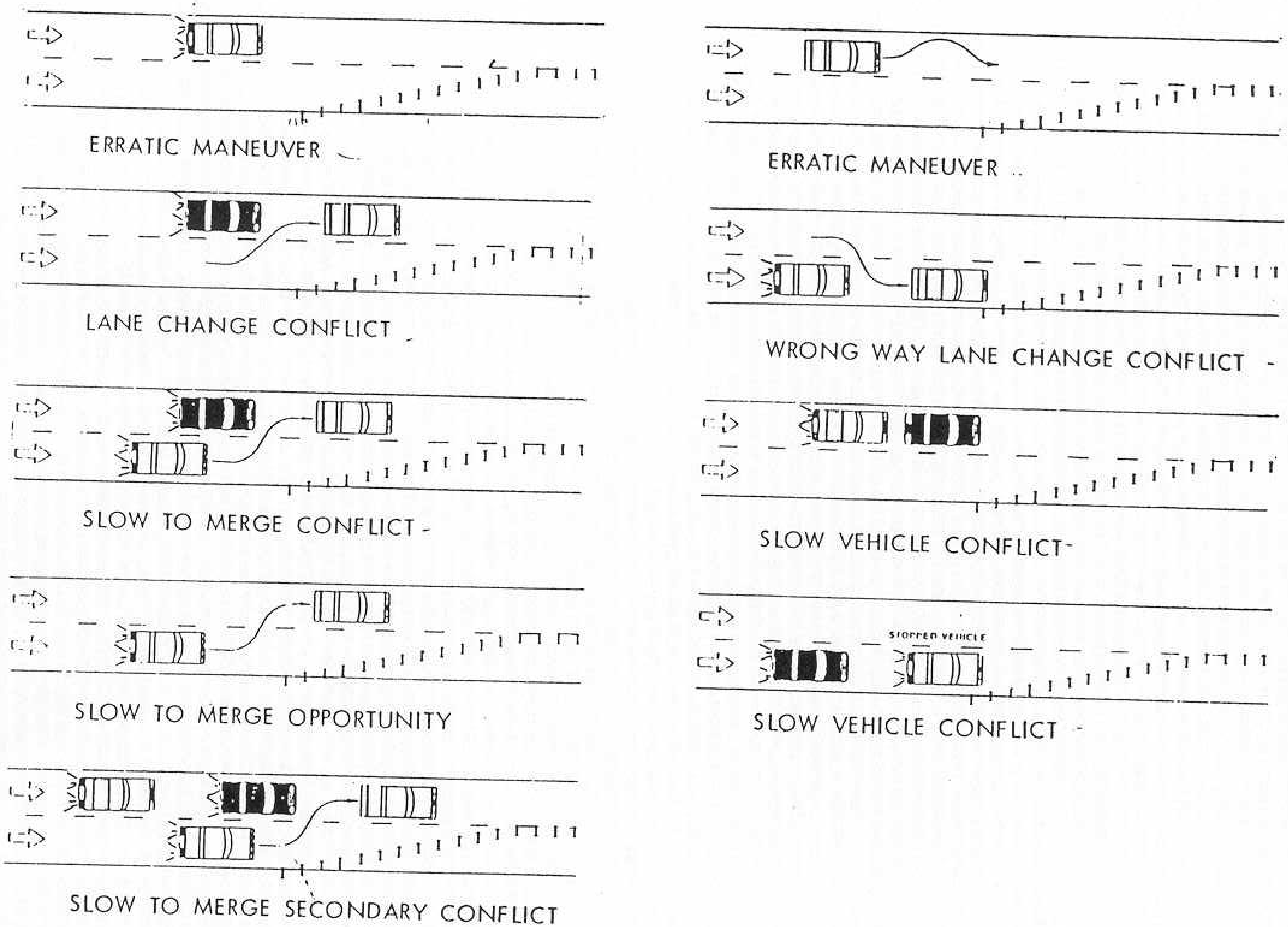
Traffic Composition: The proportion of larger vehicles (mostly trucks) would be expected to influence the levels of traffic conflicts resulting from merging maneuvers. If two streams have the same flow but different proportions of larger vehicles, the average gap in the two streams will be larger for the stream with shorter vehicles, and the size of the gap the driver chooses to accept for merging determines the probability of the conflict. Since the levels of this factor couldn't be set by the experimenter, it was considered as a covariable (covariate) factor.

3.5.2 Experimental Set Up and Data Collection

Merging conflicts were observed from the point upstream from the lane-drop taper. For a comparison study such as this one, an arbitrary but consistent location would suffice. The location of the lane-drop sign was chosen to mark the beginning of the highway section in which conflicts were observed. Conflicts occurring from the location of the lane-drop sign to the end of the lane-drop taper were counted.

Traffic conflicts were observed on 17 of the total 18 lane drop-sections during the summer and fall of 1997. Of the 17 lane-drop sections observed, four were observed for eight hours and the rest for four hours. Erratic maneuvers include swerving of a single vehicle within a lane, while rear brake lights indicated a conflict. Figure 3.5 shows the types of conflicts and erratic maneuver considered at a lane-drop section.

Two observers were positioned at a place where they could clearly monitor the traffic in the lane-drop areas, but far enough from the roadway to minimize motorists' reactions to the presence of observers, and at least 30 ft (9 meters) from the edge of roadway for observers' safety reason. One of the observers counted volume while the other counted erratic maneuvers and conflicts.



Source: Grahm et al. (1978)

Figure 3.5: Lane-drop Erratic Maneuvers and Conflicts.

3.6 INTERSECTION TRAFFIC CONFLICTS

The objective of this sub-study was to compare the relative safety of different locations of crossroad intersections. KDOT was concerned with safety at these intersections where side roads intersect a main highway within a passing lane section. At these intersections, the side-by-side configuration of passing lanes forces a through vehicle from the side road to cross four lanes. Also, the left-turning vehicles are required to cross two lanes to complete their maneuver. These maneuvers require double crossing distances compared to similar intersections on standard two-lane highway sections.

3.6.1 Experimental Design

The number of conflicts to the through-vehicles on the main highway as a result of the presence of vehicles turning into or out of the side road, was used as the response variable. The study considered three intersection locations: 1) within the full width of the passing lane; 2) immediately downstream from the passing lane; and 3) at isolated locations where there were no passing lane influences. The locations within the lane-drop and lane-addition sections were not considered because there are only two passing lanes in Kansas with an intersection in these areas.

Intersections Within the Full Width of a Passing Lane, A left-turning vehicle at this location uses the inside lane as a *defacto* left-turn lane. Left-turn, same direction conflicts will depend on the proportion of vehicles using the inside lane. Because the potentially conflicted vehicle has the option of using the second lane (if not occupied), the number of left turn, same direction conflicts were expected to be lower, than at intersections on the standard two-lane highway sections, other parameters being equal.

Intersections at a Conventional Section: This includes intersections on conventional two-lane highway sections immediately downstream from the lane-drop of the passing lane, and intersections on conventional sections isolated from the influence of the passing lane. These locations may or may not be provided with right-turn or left-turn lanes. Where the right-turn lane is provided, it is intended normally to accommodate the right-turning vehicle. However, in practice it plays a role similar to that provided by a turning bay by accommodating the through vehicles when they are likely to be delayed by the left-turning vehicles. As the left-turning vehicle occupies the through lane, waiting for a gap in the opposing traffic, the trailing vehicle can maneuver at reduced speed around the stopped/decelerating left-turn vehicle . This situation is considered better than having the through vehicle stopping and waiting until the left-turn vehicle has turned through the intersection, where a turning bay is not provided.

If a left turning vehicle just exiting a lane-drop section will need only normal deceleration to a stop at an intersection, the intersection is considered to be isolated; otherwise, it is considered to be within the influence of the passing lane. It is hypothesized that the speed of the vehicle at the lane-drop is relatively higher due to the merging process. A left-turning vehicle that will decelerate at a rate higher than acceptable is likely to impact the following through vehicles which might not be able to cope with the high deceleration rate of the lead vehicle. The critical distance downstream from the end of a lane-drop which sets the boundary between isolated intersections and intersections under the influence of the passing lane, is made up of two components: 1) the distance traveled during perception-reaction time, and 2) distance traveled during deceleration to a stop at an acceptable deceleration rate.

Lewis and Michael (1963) reported a comfortable deceleration rate as 8 to 9 ft/sec^2, and up to 16 ft/sec^2 without causing severe discomfort to the driver. Drew (1968) in his examination of the "rule of thumb" method used to compute braking distance to a stop at an intersection, concluded that a deceleration rate of 16 ft/sec^2 is appropriate. The deceleration rate of 8 ft/sec^2 was considered as a normal deceleration beyond which the driver will feel uncomfortable.

Reaction time is defined as the time taken by the driver between the instance the driver recognizes the object and the instant the driver actually applies the break. AASHTO (1990) uses a value of 2.5 seconds for braking reaction time for an average driver. Assuming an average speed of 65 miles per hour (104 kilometer per hour), the critical distance is computed from the following expression:

$$D_{critical} \quad V\times t_{P/R} \quad \frac{V^2}{2a} \quad 65\times1.47\times2.5 \quad \frac{(65\times1.47)^2}{2\times8} \quad 1{,}048\,\boldsymbol{ft}\ (319\ \boldsymbol{m})$$

The critical distance of 1,048 ft is greater than the minimum stopping sight distance of 550-725 ft (168 - 220 m) recommended for a design speed of 65 mph (AASHTO 1990, p 120). This implies that there is a high probability that all conflicts due to the influence of the intersection were included in the conflict counts. Sites were selected randomly except that the intersections within a highway section with reduced speed limit, (those within the influence of urban areas) were avoided.

3.6.2 Experimental Set Up and Data Collection

Figure 3.6 shows the field experimental set up for a typical intersection at the middle of the passing lane. Intersections at other locations had a similar set up. The observer at a distance of about 1,000 ft (305 m) from the intersection was based on the critical distance determined earlier in Section 3.6.1. The observer distance of 100 to 300 ft (30 to 91 m) was suggested for urban intersections by Glauz and Migletz (1980).

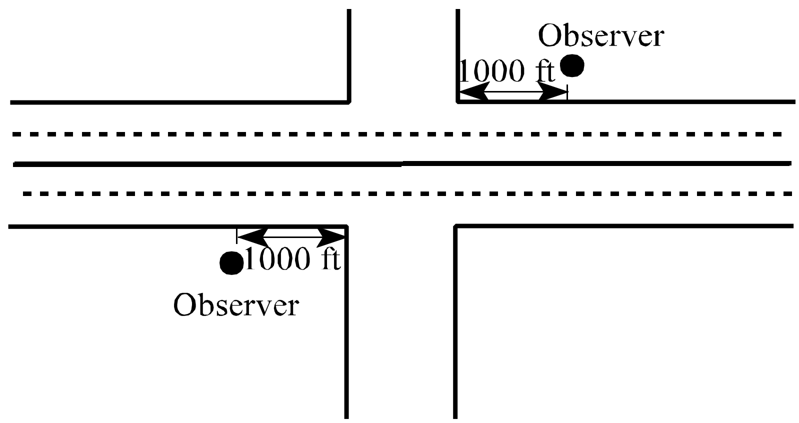


Figure 3.6: Experimental Set Up for Intersection Conflict Study.

Similar stratification sampling used for obtaining video data in Section 3.3.1 was also used in this sub-study. Randomization during site selection was possible, but the sample size had to be balanced with available resources. Three intersections from each highway were selected at random while avoiding those intersections within the lane-drop and lane-addition sections of the passing lane, and those within highway sections with reduced speed limits.

Traffic conflicts on three intersections from each highway were observed for a total of eight hours per intersection (four hours during the morning peak, and four hours during the afternoon peak period). Each of the three intersections was chosen at random at the following locations: 1) within the full width of passing lane section (four lanes), 2) immediately after the lane-drop of the passing lane (2-lane section), and 3) an isolated section free from the influence of the passing lane (2-lane section). Intersection turning, and through volumes were counted at the same time when the conflicts to the through vehicles on the main highway were being counted.

3.7 ACCIDENT ANALYSIS

Accident data for the segments on the two highways with passing lanes (US 50 and US 54) were provided by the Division of Planning of KDOT. The data consisted of individual accident information for accidents that happened between the years 1990 and 1996 inclusive, the average annual accident rate for control sections with similar characteristics in the state to those being studied, and the maps showing the names and boundaries of the control sections. A control section which is used by KDOT for spatial classification of accidents, is defined as a road section with similar physical characteristics, such as pavement width, surface, etc.

3.8 DRIVERS SURVEY

This sub-study was done with the objective of interpreting drivers' attitudes, understanding and acceptance of passing lanes in Kansas.

3.8.1 Questionnaire Design

The objective of the questionnaire design for this study was to optimize information collected while maximizing the response rate and minimizing cost. A self-addressed, postage-paid, postcard questionnaire was used. It had five multiple-choice questions and one open-ended question, as shown in the appendix. The questions were designed to be easy to analyze and answer. The multiple-choice ("Yes" or "No" and "Check-list") questions are easy to analyze and easy to complete on the part of the respondent. It was felt that this type of survey would maximize the return of the questionnaire, although more open-ended questions may have provided more information. The questions related to travel behavior, vehicle mix, attributes of passing lanes, need for passing lanes, drivers' state of residence, and additional comments. As an incentive, drivers who wanted a free state road map were requested to fill in their addresses at the bottom of the card.

3.8.2 Experimental Set Up

A field survey of drivers on the two highways (US 50 and US 54) was decided upon, and a site was selected on each highway downstream/upstream (because of side-by-side configuration) of a passing lane. The number of postcards to be distributed was set at 500 cards per location. Because of the difference in signing and geographical location between US 50 and US 54 highways, it was decided to conduct separate surveys for each highway (cluster sampling). Cards were color coded to identify point of distribution.

3.8.3 Data Collection

Survey card distribution places were selected at the east end of the passing lane sections on US 54, and at the west end of the two passing lane sections between I-135, and K77 on US 50. Five hundred survey cards were distributed to drivers at each location. Sampling of drivers was such that the first 500 drivers received survey cards at each location.

3.9 COMPARISON OF PASSING LANE CONFIGURATION

Comparison of passing lane configurations was achieved by computer traffic simulation program known as TWOPAS. TWOPAS have the ability to simulate traffic operations on two-lane highways.

3.9.1 Experiment Set-Up

A 2x2x3x5 factorial experiment (percent truck, directional distribution, volume, and passing lane configuration) was conducted using a 10-mile hypothetical highway section to determine the best passing lane configuration. A hypothetical highway section was used because it was difficult to obtain an existing section long enough (at least 8 miles) to be simulated by the TWOPAS model. TWOPAS cannot simulate highway sections with more than 30 grade regions. The longest existing Kansas highway section (US-50 and US-54) with 30 grade regions was 5.75 mile. Similar experiment set-up used for the hypothetical section was used on the existing highway section to discover how the passing lane configurations may rank in a real world situation.

3.9.2 Data Collection

Traffic data needed for calibration and validation of TWOPAS for the Kansas study was collected along two highway sections, each from highway US 50 and US 54. A section of US 54 was selected for calibration while a section of US 50 was selected for validation. Because TWOPAS assumes an uninterrupted facility (a facility without traffic control and interruptions from side roads), the two sections were chosen to minimize these interruptions. The section on US 54 is located between the city of Greensburg and the city of Mullinville with a length of about three miles (4.8 kilometers) and one low-volume side road intersection within the section. The section on US 50 is between the city of Walton and the city of Peabody with a length of about 3.2 miles (5.1 kilometers) and two low-volume side road intersections.

Chapter 4

DATA ANALYSIS AND RESULTS

This chapter presents analysis of data and results for the seven sub-studies described in chapter three. These studies are: 1) video data, 2) traffic counts, 3) lane-drop conflicts, 4) intersection conflicts, 5) accident analysis, 6) users' opinions, and 7) comparison of passing lane configurations using a traffic simulation model.

4.1 VIDEO DATA

Although the camera could view the traffic in both directions of travel, the data on opposing traffic, and that filmed during night time and during rainy weather were difficult to see, unreliable and therefore omitted from the analysis. The usable daytime data were extracted from the videotapes in 5-minute intervals, and included the following: 1) the number of vehicles entering the passing lane section; 2) the number of vehicles entering the basic lane at the beginning of the passing lane section by headway categories of a) less than five seconds and b) more than five seconds; 3) the number of passes for each type of pass; i.e., car passing car, car passing medium vehicle, car passing heavy vehicle, medium vehicle passing car, medium vehicle passing medium vehicle, medium vehicle passing heavy vehicle, heavy vehicle passing car, heavy vehicle passing medium vehicle, and heavy vehicle passing heavy vehicle; 4) the number of passes performed in the basic lane and opposing lane; 5) number of vehicles keeping right to the shoulder lane; 6) number of vehicles keeping right after the beginning of lane lines; 7) the size and number of platoons. Because of a good camera location at the

Burton and Cullison sites, an extra count was added to differentiate between the number of passes initiated at the beginning of the passing lane and those initiated in the middle of the passing lane.

Table 4.1 shows the checklist of the items analyzed at each site. Lane usage at Peabody was not analyzed because lane lines were not clearly visible due to poor camera positioning (too low) and poor contrast between fading lane lines and the concrete pavement surface. Vehicles keeping right and platooning characteristics for the Pratt site were not analyzed because the camera view did not include the beginning of the passing lane. Tables 4.2 and 4.3 summarize the data obtained from videotapes at all four sites.

4.1.1 Volume

Vehicles were classified into three classes: 1) passenger cars, 2) medium size vehicles, and 3) heavy vehicles. Passenger cars include all 4-tired vehicles; medium vehicles including recreation vehicles and light trucks (6-tired pickups), while heavy vehicles include all vehicles with more than two axles (trailers, semi-trailers) and farm equipment. Of the total vehicles counted for all sites, small vehicles were 76 percent, medium vehicles were 4 percent, and heavy vehicles were 16 percent.

Table 4.1: Data Extracted From Videotapes

Passing lane	Vehicle Mix	Passes	Keeping Right	Mid-Keeping Right	Platoons
Burton	X	X	X	X	X
Peabody	X	X	-	-	X
Pratt	X	X	X	-	-
Cullison	X	X	X	X	X

Table 4.2: Traffic Pattern (Platooning and Lane Utilization) at Passing Lanes

Site	Number of Hours	Total Volume	Hourly Volume Range	Vehicle Class	% of Volume	% in Right Lane				
						By Class	Free and Platoon Leaders	Following in Platoons	End of Lane-Addition	All
Burton	33	6586	96 - 420	Small Medium Heavy	79.8 3.9 16.2	45.1 66.5 68.9	55.4	41.3	16.3	49.8
Peabody	69	9671	80 - 201	Small Medium Heavy	74.5 3.4 22.2	NA	NA	NA	NA	NA
Pratt	48	6745	75 - 188	Small Medium Heavy	75.2 4.3 20.5	61.8 85.4 71.7	NA	NA	NA	64.8
Cullison	22	3333	111 - 211	Small Medium Heavy	72.2 5 22.8	52.1 82.5 66.7	66.2	40.9	10.6	57

Table 4.3: Passing Behavior and Platooning at Passing Lanes

Site	Vehicle Class	% Passes			% Following in Platoons
		By Class	In Outer Lane	In Opposing Lane	
Burton	Small Medium Heavy	84.5 1.6 13.9	16.5	0.2	39.6
Peabody	Passenger Medium Heavy	78.8 1.2 19.9	6.9	0.1	32.4
Pratt	Small Medium Heavy	75.3 1.8 22.9	4.5	0	NA
Cullison	Small Medium Heavy	74.2 3.3 22.5	2.8	0.5	36.5

4.1.2 Lane Utilization

At a passing lane section it is desirable that all vehicles drive in the basic lane (right lane) leaving the inner lane for passing vehicles. Although the lane utilization shown in Table 4.2 exhibits a large variation within each vehicle class between sites, it consistently shows that a higher proportion of non-passenger cars, compared to the proportion of passenger cars, prefer to use the right lane. Due to the geometry of the passing lane sections studied, a driver has to make a deliberate effort to move to the right lane (at the beginning of the passing lane) and then has to merge back to the left at the end of the passing lane. This might be a great concern to the passenger car drivers.

Platoon leaders and single vehicles use the right lane more often than the vehicles following behind platoon leaders. A vehicle was classified as “following” if its time headway was less than or equal to five seconds (TRB, 1994). It is noted from Table 4.2 that only 10 to 16 percent go into the right lane at the beginning of the lane lines (i.e., at a point where both the outer and inner lanes attain full widths). This value probably reflects the effect of the lane-addition pavement marking (no channelization to the outer lane). These values compare well with the 20 percent obtained in California (Staba et al. 1991), although in the California study it was not known whether the 20 percent was taken at the beginning of the lanes or reflects lane utilization for the entire lane length.

4.1.3 Keep Right Compliance

Probably the best variable to describe the keep right compliance (among those in Table 4.2 and Table 4.3) is the proportion of platoon leaders and single vehicles that go into the right lane. For following vehicles, what makes the vehicle go to the right lane is not known--it could be an attempt to pass on the right, etc. At US 50 sites, the proportion of single and platoon leader vehicles that went to the right lane was 55 percent, and at US 54 sites was 66 percent. As explained earlier, supplemental signs to guide non-passing motorists into the right lane differ for the passing lanes on US 50 and US 54. The US 50 passing lanes are signed by **KEEP RIGHT EXCEPT TO PASS** while the US 54 passing lanes are signed by **SLOWER TRAFFIC KEEP RIGHT.** At the beginning of this study, the researchers considered **KEEP RIGHT EXCEPT TO PASS** better than **SLOWER TRAFFIC KEEP RIGHT**. The results of users' opinions (explained later in Section 4.6), indicated a preference for the former sign by the drivers. However, the results from video data showed that the motorists on US 54 complied better than those on US 50. However, it should be noted that other factors may attribute to these results. Such factors could include the difference in proportion of unfamiliar drivers between US 50 and US 54 highways (to be explained later in Section 4.6) and/or the two advance signs on US 54 versus one advance sign on US 50. Advance signs should serve the purpose of preparing drivers early for the effective use of passing lanes.

4.1.4 Passes

The state of Kansas discourages passing on the right. At the sites studied, passing to the right is discouraged through the **KEEP RIGHT EXCEPT TO PASS** or **SLOWER TRAFFIC KEEP RIGHT** signs at the beginning of the lane-drop section. Also at the sites studied, passing using the opposing lane(s) is prohibited by double yellow line pavement markings, and by "No passing Zone" sign placed about 300 ft downstream of the passing lane. Passing to the right, or using the opposing lane, may reflect the frustration of drivers who have been traveling behind slower vehicles for a long time and then are further frustrated by leading, slower vehicles that do not move to the right lane as they enter the passing lane section. At a passing lane, this situation will mainly depend on the number of slower vehicles failing to keep right. As shown on Table 4.3 the percentage of passes performed using the opposing lane was very small compared with those performed using the right lane. The total percentage of passes made in the right lane plus the opposing lane was 16.7 and 3.3 percent, for the Burton and Cullison sites, respectively, while the percentage of “free” and “platoon leaders” keeping right was 55.4 and 66.2 percent. In this sub-study the percentage of passes made in the opposing and right lane was inversely proportional to the “free” plus “platoon leaders” that kept to the right lane. This was expected.

Figures 4.1, 4.2, and 4.3 show the relationship between the number of passes and one-way hourly volume and between pass rates and one-way hourly volume for the four sites studied. These figures show that both passes and pass rates increase linearly with one-way traffic volume. Pass rate is normally used for comparing passing lanes of different lengths. Neither the number of passes nor the pass rates exhibit differences between the sites. Even though some of the passes at the Pratt site were not counted, the number of passes did not seem to differ significantly from the other sites.

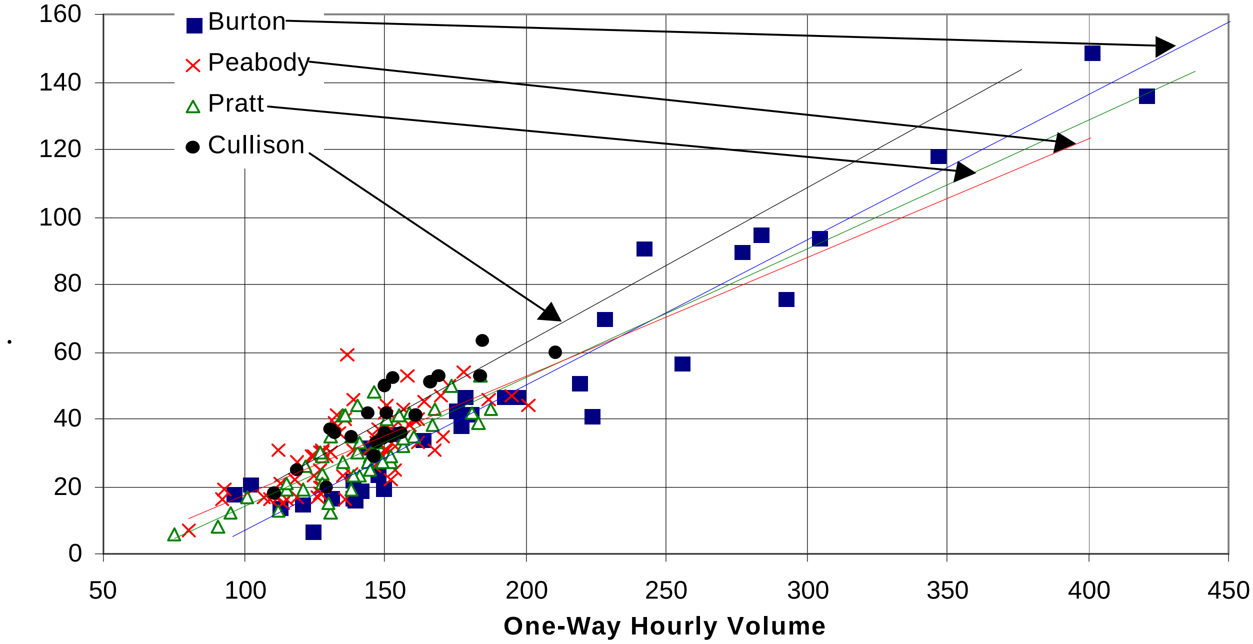


Figure 4.1: Relationship Between Number of Passes and One-Way Hourly Volume

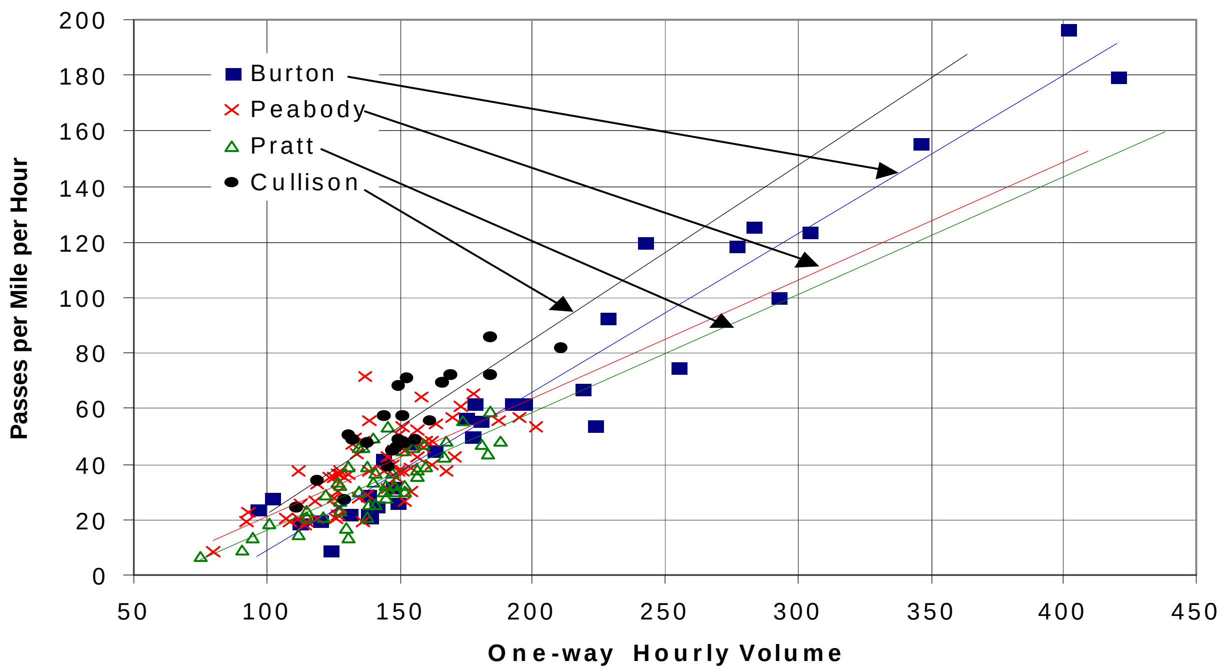


Figure 4.2: Relationship Between Pass Rates and Volume

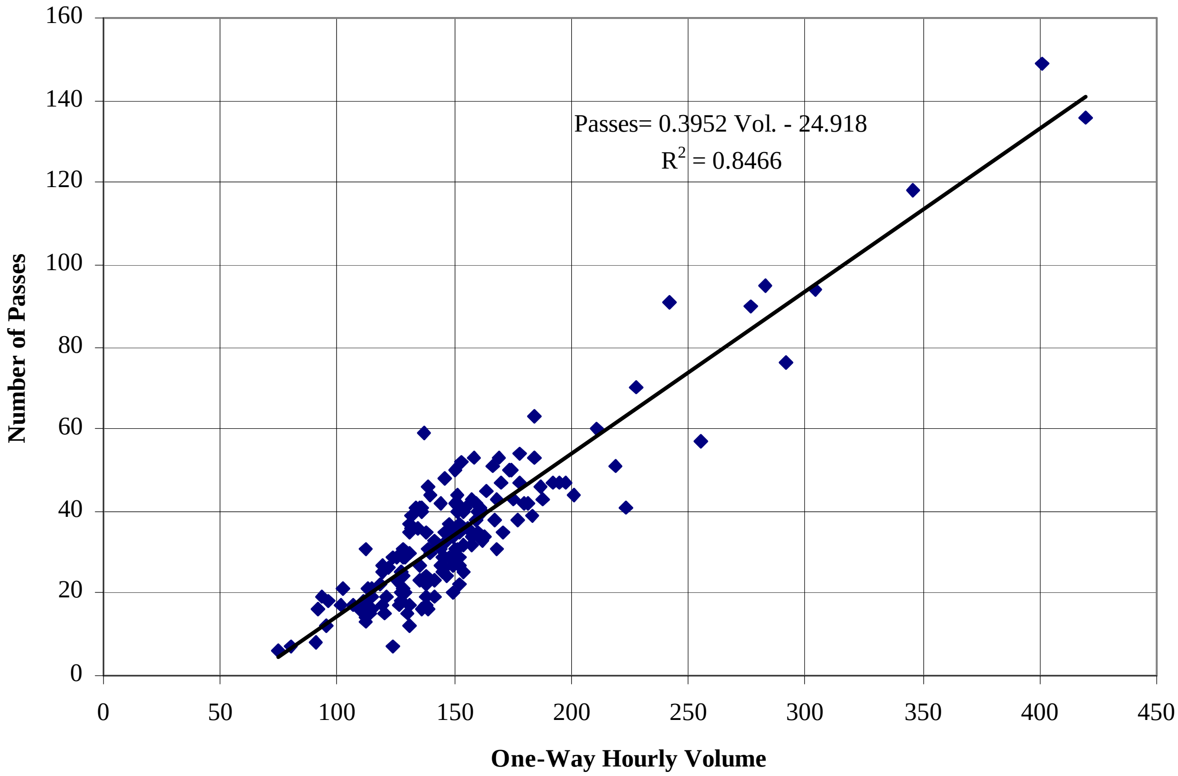


Figure 4.3: Relationship Between Number of Passes and Volume (All Sites)

4.1.4.1 Passing Frequencies Related to Vehicle Types

Table 4.4 summarizes the relationship between passing frequency and vehicle types for the combined passes at all sites. Sites were combined because it was found that there was no significant difference between sites regarding passing maneuvers. The medium and heavy vehicle classes were combined together to form a "non-automobile" class because of the small proportion of medium vehicles (4 percent) and related passes.

Table 4.4: Passing Frequencies by Vehicle Type for Combined Passes at All Sites

		Passing Vehicle		
		Auto (76)	Non-auto (24)	Total
Passed Vehicle	Auto	3,113 (50.8)	735(12)	3,848 (62.8)
	Non-auto	1,716 (28)	558(9.2)	2,274 (37.2)
	Total	4,829 (78.8)	1,293(21.2)	6,122 (100)

NOTE: Number in parenthesis are the percentages.

- non-autos are passed more than they pass;
- when an auto makes a pass, 64 percent of the time it will pass an auto and 36 percent of the time it will pass a non-auto;
- when a non-auto makes a pass, 43 percent of the time will pass a non-auto and 57 percent of the time it will pass an auto;
- when an auto is being passed, 81 percent of the time the passer is an auto and 19 percent of the time the passer is a non-auto; and
- when a non-auto is being passed, 25 percent of the time the passer is a non-auto and 75 percent of the time the passer is an auto.

These results show that a non-auto is more likely to get passed than an auto. This could partially be due to the higher percentage of non-autos that stay in the right lane, making them easier to pass. However, in general, it seems that the autos benefit more from the passing lanes than non-autos. Staba et al. (1991) also found that the autos benefited more than the non-autos when a passing lane was available.

4.1.5 Platooning

The proportion of vehicles following in platoons was 40, 32, and 36 percent for Burton, Peabody, and Cullison sites, respectively. The difference seems to be relatively small. Within the range of observed volume, the number of platoons per hour appeared to be identical at all three sites based on visual inspection of Figure 4.4. The percent of vehicles in platoons at any point along a two-lane highway depends on the traffic characteristics (especially speed distribution and volume), and geometric characteristics (especially percent no-passing zones) of the upstream section. The passing lane sites in this study were all located in rolling to level terrain indicating that the geometric characteristics are comparable for all sites. Therefore, at any particular volume level, the extent of platooning versus hourly volume is likely to be similar for all sites as shown in Figure 4.4.

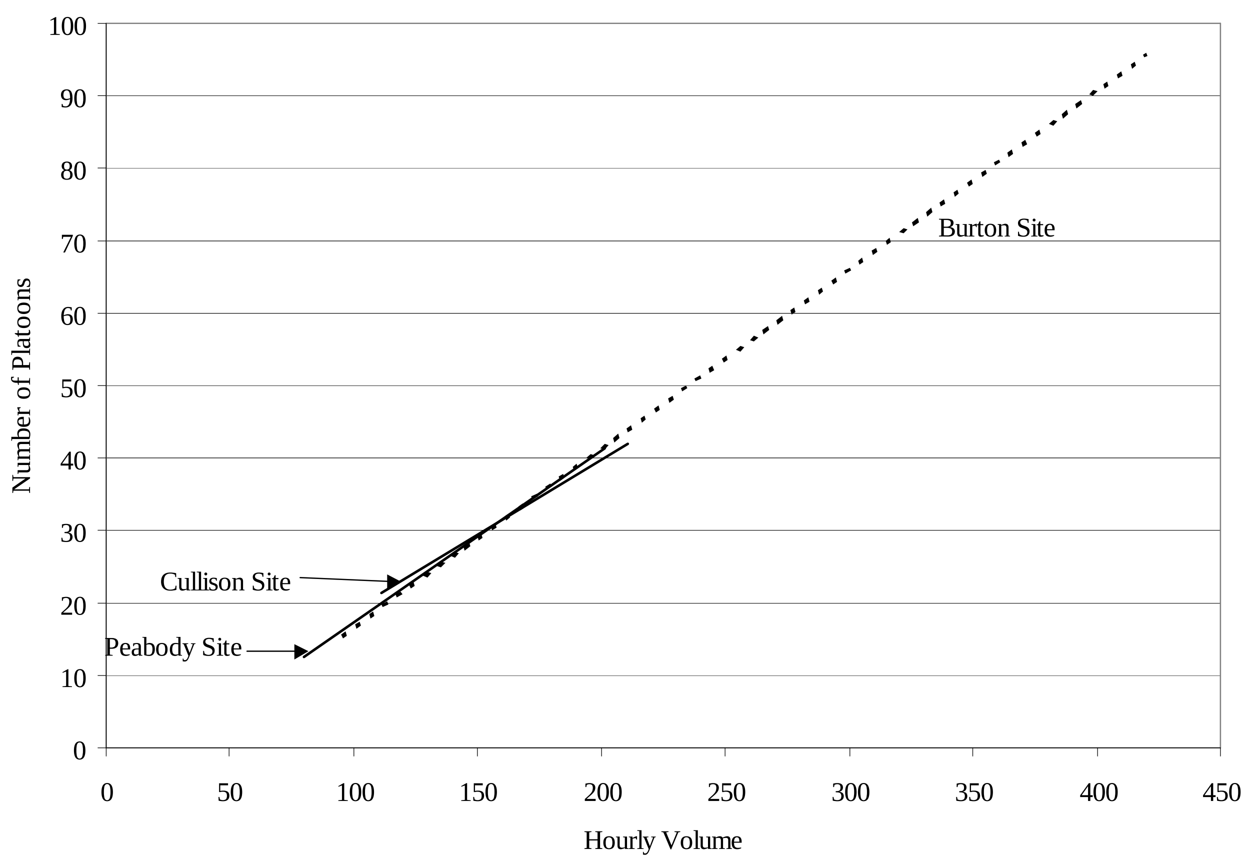


Figure 4.4: Number of Platoons Vs One-Way Hourly Volume

4.2 TRAFFIC COUNT

TrafiCOMP III counters produce data files in the ASCII digital format. The files were then formatted into spreadsheet files. For each hour the following information was summarized for each direction at each site: 1) one-way hourly volume, 2) time headway groupings depending on the range of data collected, 3) vehicle classification according to the 13 FHWA vehicle classes shown in Figure 3.2, and 4) 10 speed groups (<40, 41-45, 46-50, 51-55, 56-60, 61-65, 66-70, 71-75, 76-80, 81-100 mph). A one hour period was chosen as the base period for analysis because one-hour was the shortest period on which the data from KDOT were reported.

4.2.1 Volume Characteristics

Table 4.5 shows the two-way, peak hour period volume associated with the directional distribution, the percentage of trucks based on the total count, and the range of the directional distributions during the total period. The counts for the eastbound direction at the Burton site were discarded due to malfunctioning of the counters. The observed percentage of trucks was compared to data previously recorded by KDOT. No relationship could be found between volume and directional distribution. There was a close agreement between the observed percentage of trucks with those recorded by KDOT earlier, especially at sites along US 50. This agreement increases the confidence in the adequacy of the period used to count traffic.

Table 4.5: Traffic Characteristics

Site	Peak Hour		Total Count Period	
	Volume	Directional Distribution	Directional Distribution Range	Percent Trucks[a]
Peabody	325	41/59	21/79 - 50/50	31 (30)
Halstead	522	49/51	40/60 - 49/51	22(20)
Burton[b]				
Pratt	313	46/54	28/72 - 50/50	27(22)
Cullison	331	49/51	35/65 - 50/50	31(22)

[a] ( ) Values from KDOT's 1995 traffic count map for state highway system.
[b] Eastbound counts were discarded because they were not reliable.

At the Peabody site the data derived from the middle of the passing lane were discarded because: 1) counters were in operation for a very few hours before they got damaged; and 2) for those few hours during which the counters were in operation, the data was not considered reliable. While the counts at the upstream and downstream locations were consistent, the counts obtained within the same hour at the middle location were either too low or too high compared to the other two locations. It was concluded that the counters at the middle location generated extra or lost some counts.

There were gaps in the data from counters for the eastbound lanes at the Halstead and Burton sites during the hours when the counters were malfunctioning. For those counters, detailed investigation revealed that, for some of the hours, there were high discrepancies between counts at the upstream location and the downstream location. For example, in one hourly such period, the upstream count at Halstead was 136 vehicles, but the count at the downstream location was 211 vehicles. It is believed that the difference is too large to be attributed to one minor side road, the drift in the clocks at the counter and/or the distance lag between the two counters. In such situations, it was difficult to set a cutoff value for clock drift and distance lag which would have helped to identify and eliminate hours with unreliable data. While a difference of one vehicle between the upstream and downstream locations counts is highly likely due to clock drift and/or distance lag, the difference of 75 vehicles is very likely to be attributed to malfunctioning counters. The difference which would separate hours in which the counters were generating unreliable data was difficult to set. For this reason, caution should be exercised during interpretation of these results. It seems that the operator did not identify the malfunctioning counters before they collected unreliable data.

4.2.2 Speed

The average speed at a location downstream of a passing lane is expected to be significantly higher than at an upstream location. Figure 4.5(a) through Figure 4.5(h) show the speed distribution for both upstream and downstream speed profiles. Visual inspection reveals that for all passing lane sections except the Cullison, westbound passing lane, there is a consistent trend between the upstream and downstream speed distributions. The percentages of vehicles with speeds less than the mode[3] value are relatively higher for the upstream location than for the downstream location. For speeds greater than the mode speed, the trend is reversed with the percentage at the downstream location being higher than at the upstream location. This indicates that vehicle speeds at the downstream locations are higher than the speeds at upstream locations.

Statistical tests were performed on the difference between the upstream and downstream speed distributions. The student's t-statistic (abbreviated as t) for random samples was used to test the difference between the average speeds of upstream and downstream locations. The alternative hypothesis assumed that the downstream average speed would be higher than the upstream average

[3] Mode is the speed group with the highest frequency. For this data the mode seems to be the 66-70 mph range.

speed (one tail test) due to the platoon breakup provided by the passing lane.

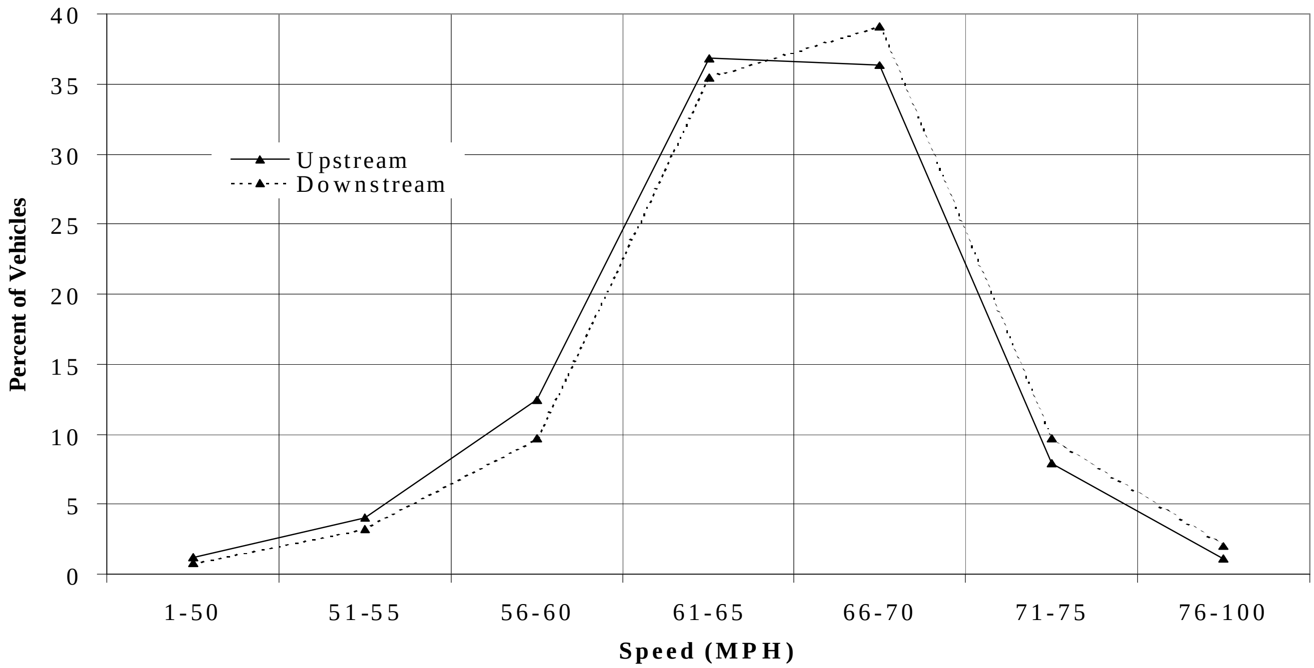


Figure 4.5(a): Speed Distribution at the Westbound Passing Lane at Peabody Site

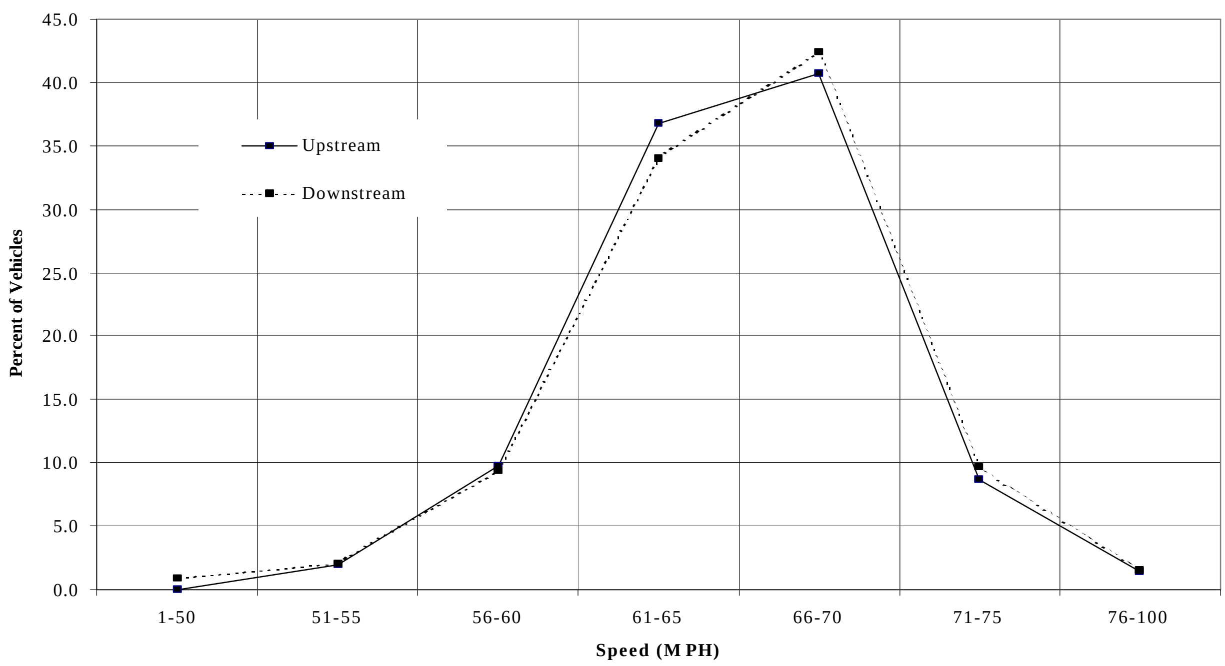


Figure 4.5(b): Speed Distribution at the Eastbound Passing Lane at Peabody Site

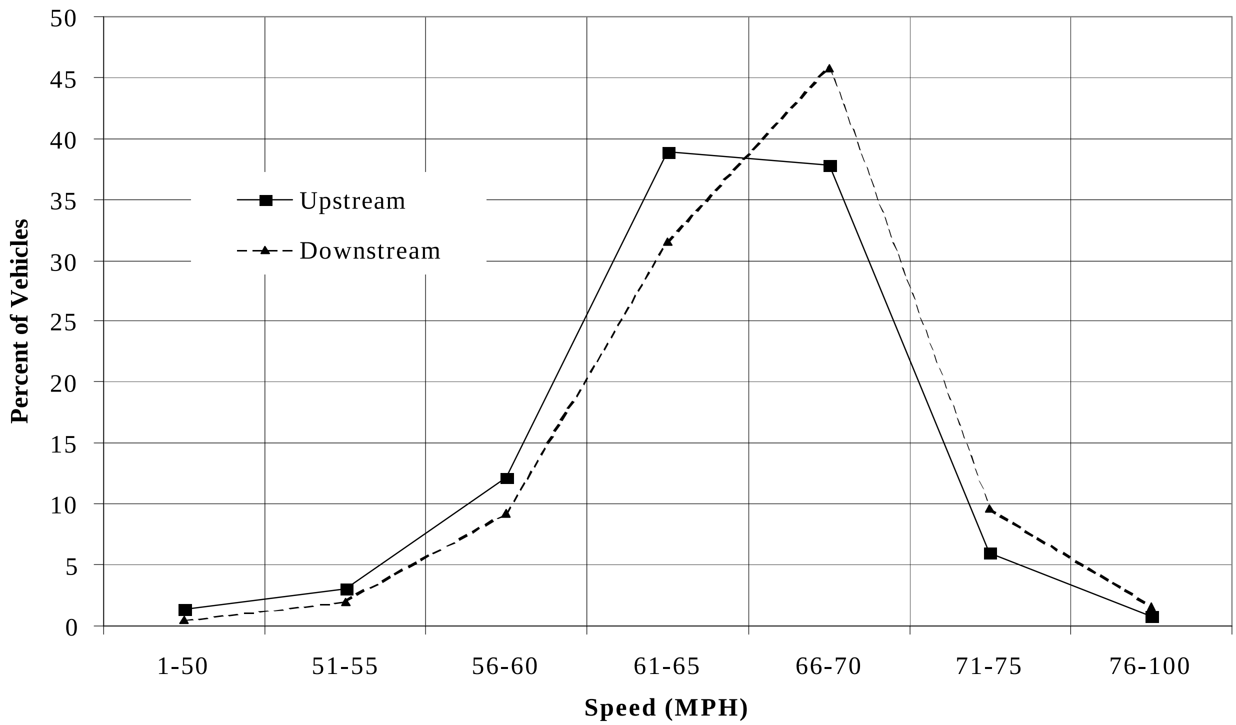


Figure 4.5(c): Speed Distribution at the Westbound Passing Lane at Halstead Site

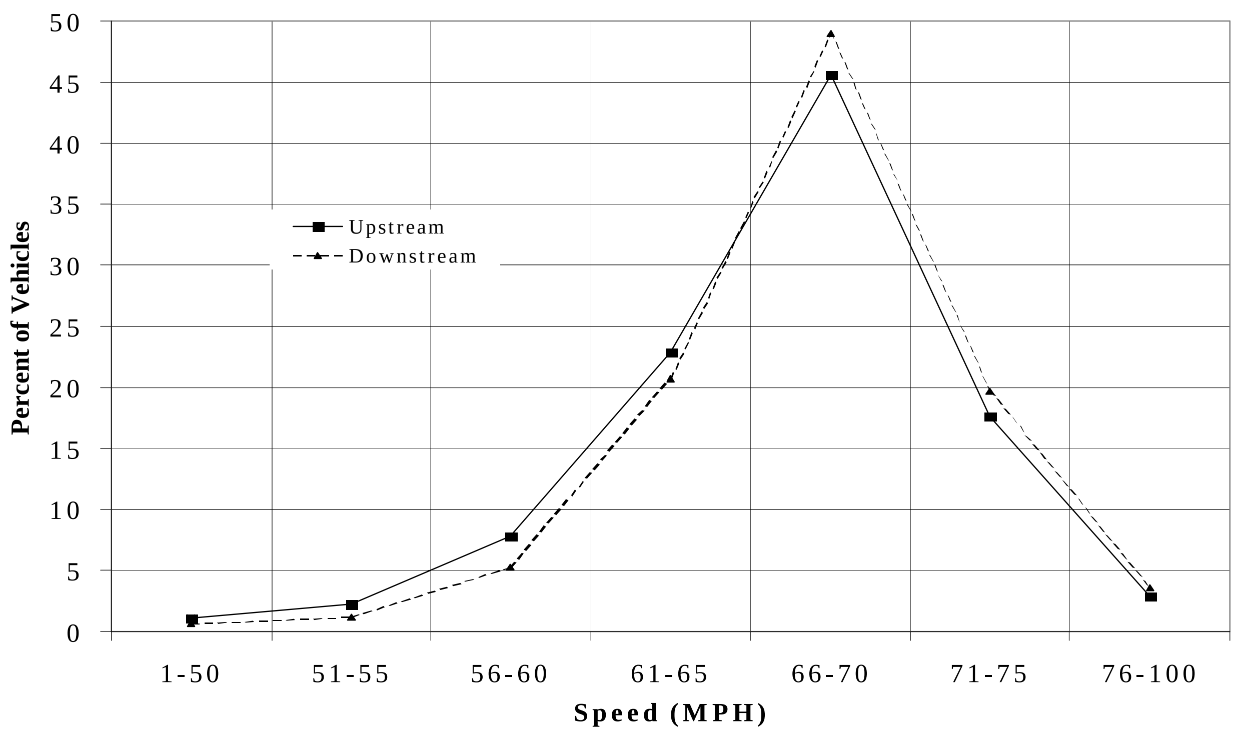


Figure 4.5(d): Speed Distribution at the Eastbound Passing Lane at Halstead Site

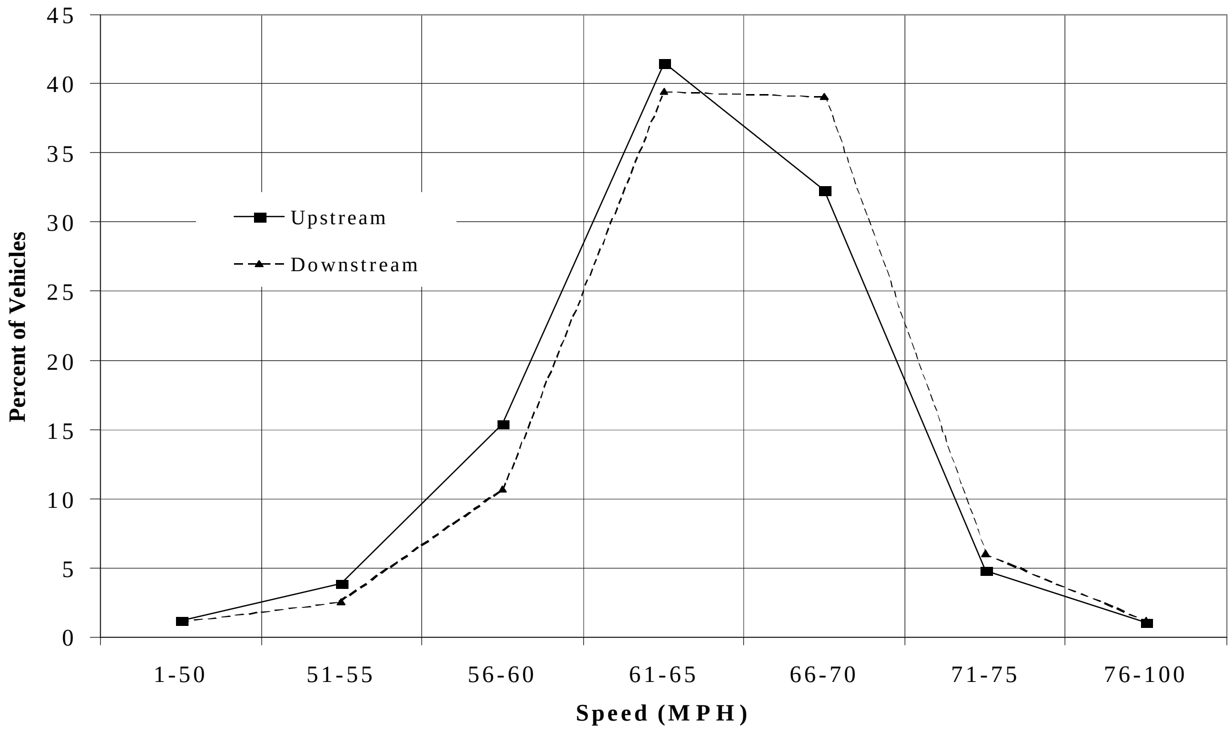


Figure 4.5(e): Speed Distribution at the Westbound Passing Lane at Pratt Site

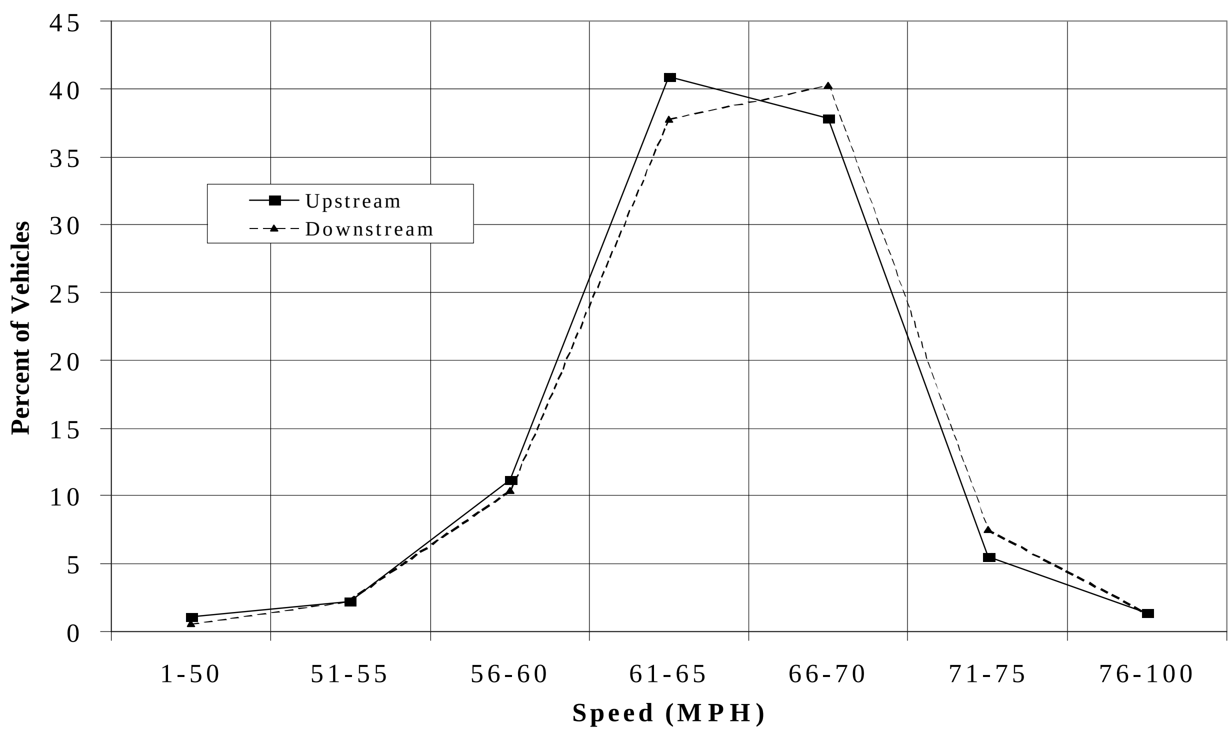


Figure 4.5(f): Speed Distribution at the Eastbound Passing Lane at Pratt Site

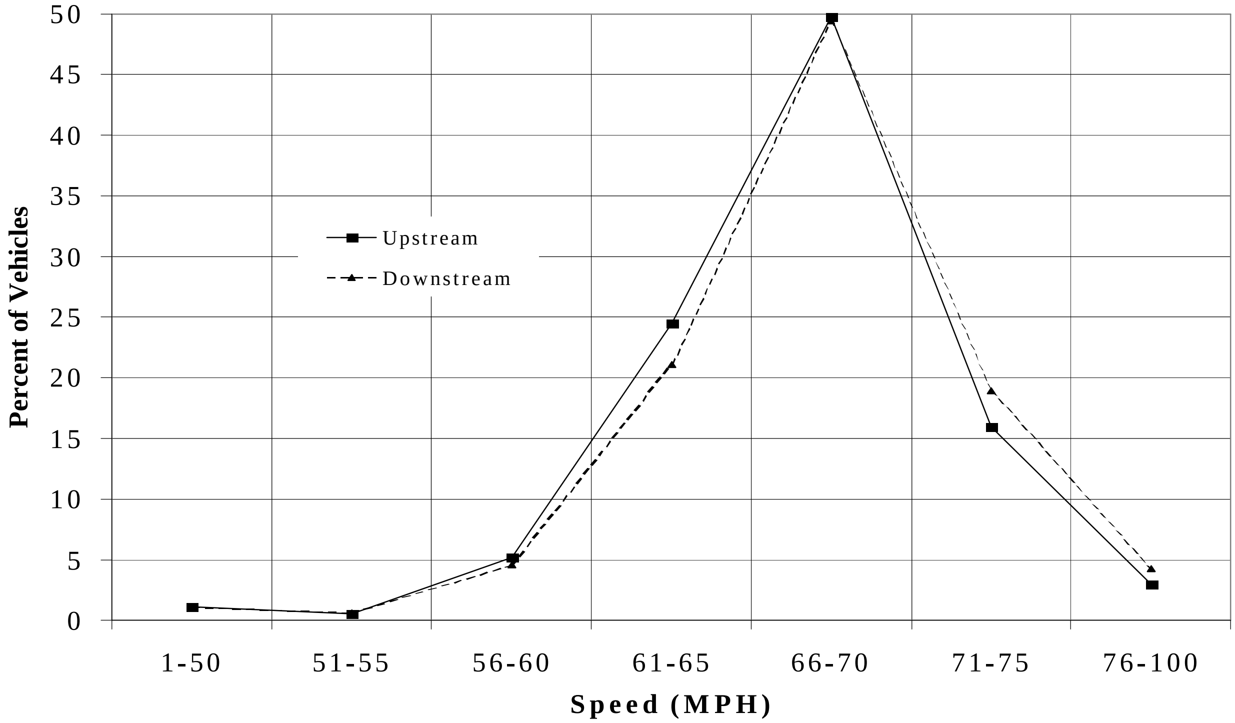


Figure 4.5(g): Speed Distribution at the Eastbound Passing Lane at Cullison Site

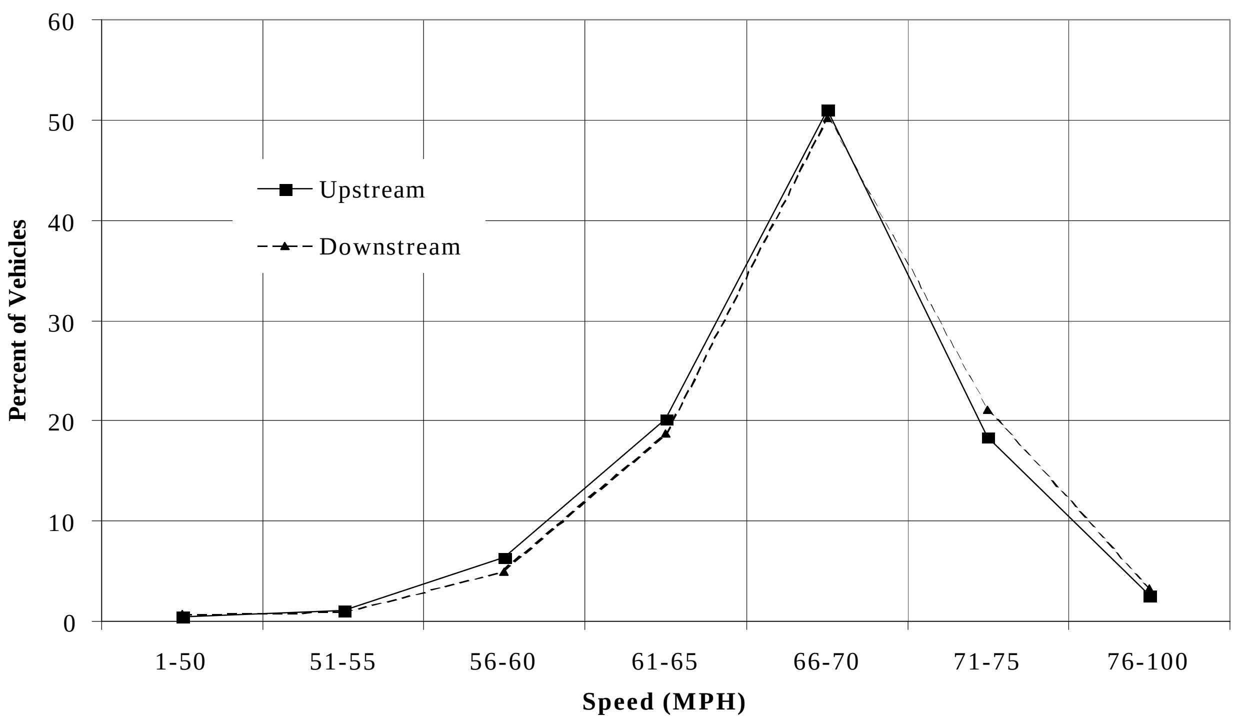


Figure 4.5(h): Speed Distribution at the Westbound Passing Lane at Burton Site

Table 4.6 shows that at all passing lanes, the average downstream average speeds become significantly larger at 95 percent confidence level, indicating that speeds at the downstream locations are significantly higher than that at the upstream locations. Although statistical tests presented in Table 4.14 show that the downstream location mean speed is significantly higher than the upstream location mean speed, the magnitude of the difference is too small to be of practical significance to a traffic engineer.

4.2.2.1 Speed Data Collection Method

The practical measurement of vehicle speeds, like any other continuous variable, involves truncation of the values. In essence, truncation groups the data into classes of "class length" equal to twice the accuracy of the instrument or method used to take measurements. For example, if the speeds are measured using a speed radar gun with an accuracy of ± 0.5 mph the recorded speed of say 50 mph includes all speeds between 49.5 to 50.5 mph, equivalent to a class length of 1 mph. One of the effects of grouping the data is to reduce the variance of the sample or population. Consider the following raw speed data initially measured to 1 mph accuracy: 46, 48, 49, 55, 54, 51, and 51 mph. The mean and variance of this data are 50.57 and 10.28 mph, respectively. Let the speeds be grouped in the interval of 5 mph of say 45-50, and 50-55, with class midpoint of 48 and 53 mph, respectively. The mean and variance of the grouped data are 50.86 and 7.14. The effect of reducing the variance on significance testing is to decrease type I error[4] which results in showing a significant difference where there is none.

[4] The probability of rejecting the null hypothesis when it is true.

Table 4.6: Speed Distributions

Site	Peabody				Halstead				Burton		Pratt				Cullison	
Direction	Eastbound		Westbound		Eastbound		Westbound		Westbound		Eastbound		Westbound		Eastbound	
Position	Up[a]	Dn[b]	Up	Dn	Up	Dn	Up	Dn	Up	Dn	Up	Dn	Up	Dn	Up	Dn
Observed vehicles	4309	5328	5799	5702	2603	3123	3200	3215	2441	2443	1037	3310	3607	2149	3142	2936
Average Speed (mph)	65.3	65.5	64.5	65.3	66.7	67.5	64.5	65.8	67.3	67.7	64.7	65.1	63.9	64.8	67.0	67.5
C.I[c].	0.2	0.2	0.2	0.2	0.4	0.4	0.4	0.4	0.4	0.4	0.6	0.4	0.4	0.4	0.4	0.4
Difference[d]	Significant		Significant		Significant		Significant		Significant		Significant		Significant		Significant	

[a] Up = Upstream location.
[b] Dn = Downstream location.
[c] C.I. = 95 percent Confidence interval length (mph).
[d] Refer to average speeds .

Sometimes grouping is done manually after data collection for the purpose of easy computation or graphical presentation. Grouping should take into consideration how the data will be used. Hald (1952, pp 51-53) suggested that when representing the distribution of observations graphically it should be aimed at choosing class intervals in such a manner that the characteristic features of the distribution are emphasized and chance variations are obscured. If the grouped distribution is to form the basis of computations, the class intervals should be small and of equal length.

4.2.2.2 Statistical vs Practical Significance

Statistical significance can be affected by the way the data is collected and summarized (as was shown in Sections 4.2.2.1), and by the sample size. This could lead to statistical significance being unimportant from a practical point of view; e.g., a difference of 0.8 mph (65.3 mph - 64.5 mph) would make little or no difference in the quality of traffic flow.

4.2.3 Time Headway

Two aspects of the time headway on a two-lane highway were studied. The first is the platooning characteristics at the upstream and downstream locations of the passing lane; and the second is the mathematical distribution of the time headway.

4.2.3.1 Percent Platooning

One of the measures that can be used to evaluate the operational effectiveness of a passing lane is the change of the proportion of vehicles in platoons from the upstream location of the passing lane to the downstream location. The vehicles following at a headway less than or equal to five seconds are considered to be members of a platoon. As shown in Table 3.6, headway classification types A and B, used to collect data at the Peabody and the Pratt sites, cannot be used directly to obtain data on platooned vehicles because the collection schemes at these sites did not explicitly count vehicles with headways less than or equal to five seconds. To determine the number of vehicles that are following at five-second or less headway at these sites, one has to establish a mathematical model relating headway to some observable, upstream roadway and traffic characteristics. Existing mathematical headway models such as a negative exponential distributions, consider only one-way, traffic volume. However, even if such a model using traffic alone as the variable could be established, it would be inappropriate for evaluation in this study because it would estimate the same number of platooned vehicles for both upstream and downstream locations since, for any analysis period, the number of vehicles passing the upstream location is the same as that passing the downstream location. For this reason, the Pratt and Peabody sites were not included in this evaluation. The data for the eastbound passing lane at the Burton site was also excluded from the analysis for the reasons stated previously—unreliable data.

Figure 4.6(a) through Figure 4.6(d) shows the headway distributions for three sites, Halstead, Cullison and Burton. Visual inspection on all plots, suggest that there is no significant difference between the upstream and downstream platooning. These plots show that the percentage of vehicles in platoons have a high correlation with the one-way volume. Table 4.7 shows the equations of the straight lines (linear fit) depicted in Figures 4.6(a) through 4.6(d).

Platooning on a two-lane highway builds up because of the inability of faster drivers to pass slower vehicles. It is apparent that if the mechanism for increasing passing opportunities is provided, then the extent of platooning should drop dramatically. Because these plots were obtained by considering all the hours on which the data was available, it is suspected that the difference between the platooning at the upstream and downstream locations will include the hour-to-hour variation. To eliminate the hour-to-hour variation, only those hours on which both upstream and downstream locations data was available were included in the analysis. The difference in platooning for these hours was computed (paired data analysis) and shown in Table 4.8.

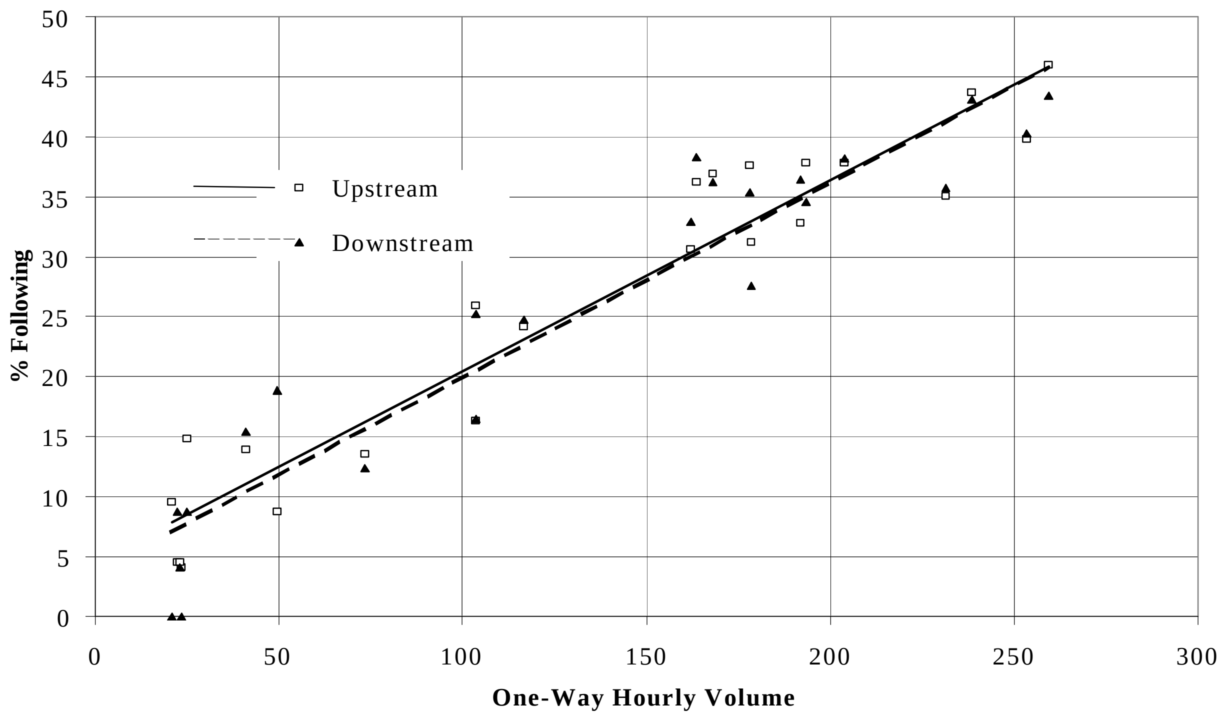


Figure 4.6(a): Platooning Characteristics for the Westbound Passing Lane at the Halstead Site

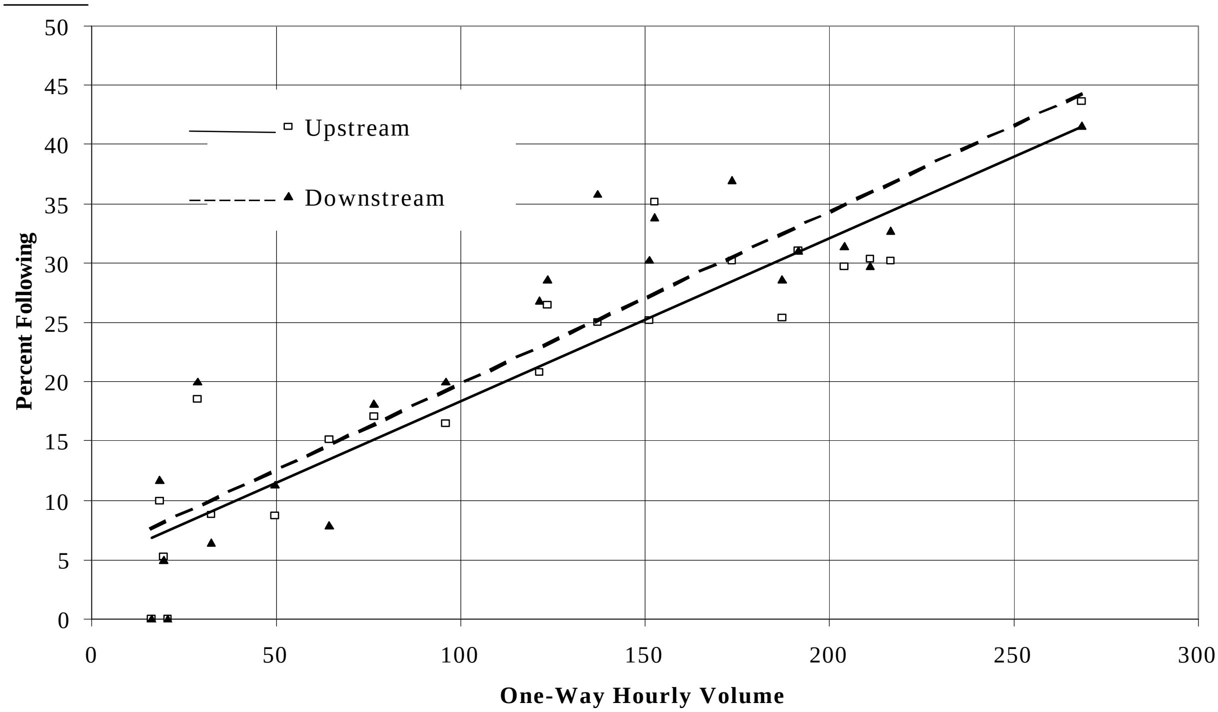


Figure 4.6(b): Platooning Characteristics for the Eastbound Passing Lane at the Halstead Site

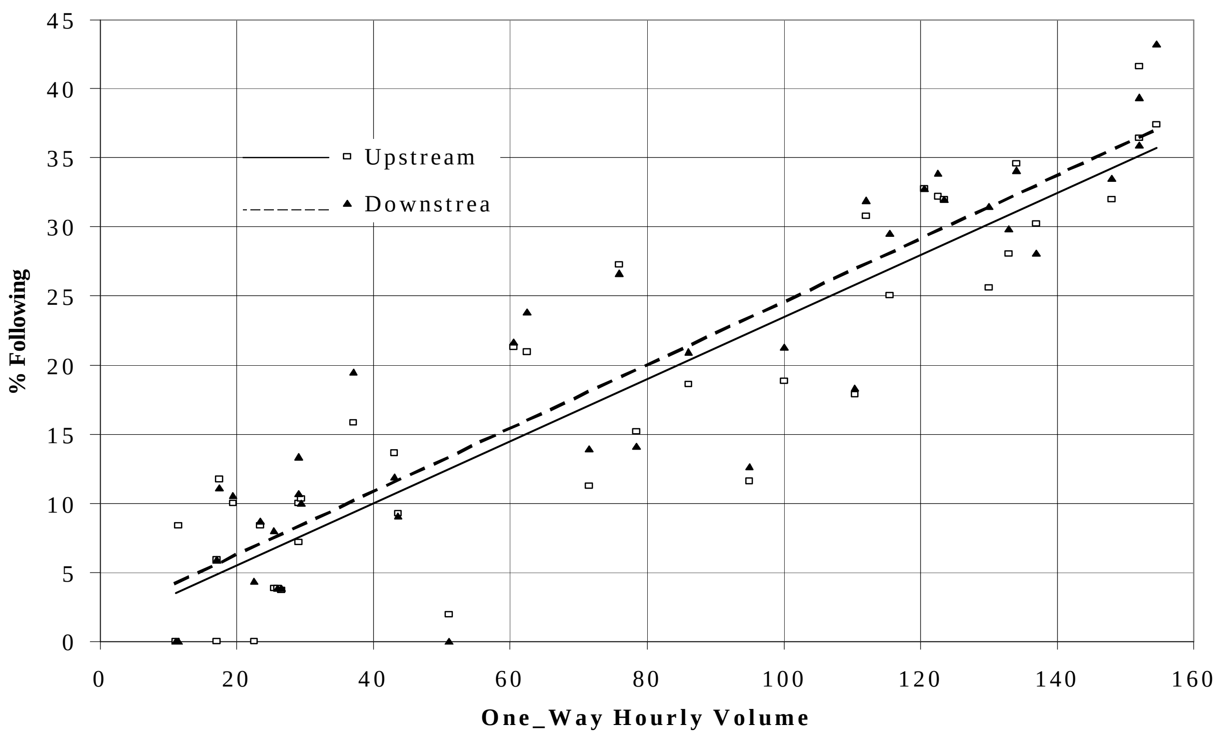


Figure 4.6(c): Platooning Characteristics for the Eastbound Passing Lane at the Cullison Site

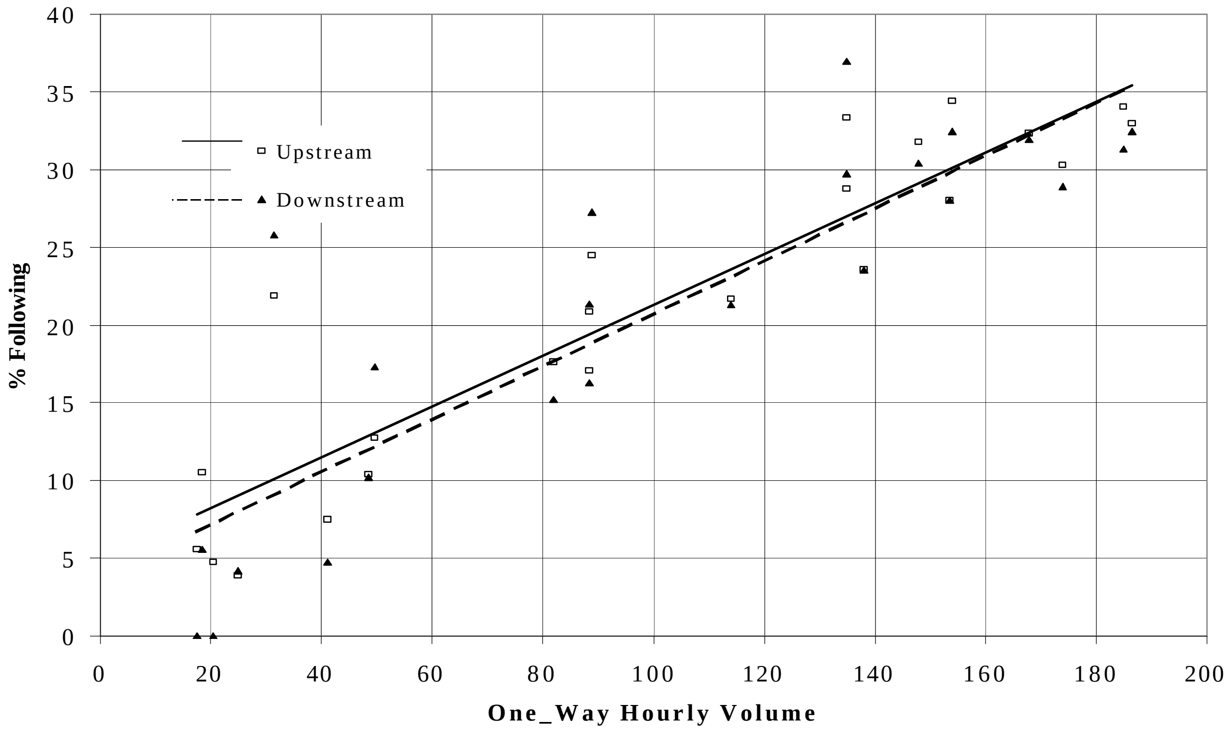


Figure 4.6(d): Platooning Characteristics for the Westbound Passing Lane at the Burton Site

Table 4.7: Platooning Characteristics at the Kansas Sites

Site	Direction	Location	Linear Fit by Location	
			Equation	R-Square
Burton	Westbound	Upstream	Y=5.177+0.162X	0.85
		Downstream	Y=3.695+0.169X	0.75
Halstead	Eastbound	Upstream	Y=4.138+0.139X	0.84
		Downstream	Y=4.765+0.148X	0.79
	Westbound	Upstream	Y=4.678+0.159X	0.92
		Downstream	Y=3.718+0.162X	0.90
Cullison	Eastbound	Upstream	Y=0.918+0.222X	0.80
		Downstream	Y=1.683+0.229X	0.83

Y = Percent in platoons
X= One-Way Hourly Volume

Table 4.8: Percent of Vehicles in Platoons at the Kansas Sites

Passing Lane	Upstream	Downstream	Change
Halstead, Eastbound[a]	27.5	29.8	+2.3
Halstead, Westbound	33.5	33.3	-0.2
Burton, Eastbound[a]	28.1	29.1	+1.0
Burton, Westbound	26.6	26.4	-0.2
Cullison, Eastbound	24.7	26.0	+1.3

Overall, the results are mixed. Some of the passing lanes recorded reduction in the percent of vehicles in platoons while others recorded an increase. It is seen that even after eliminating the hour-to-hour variation no improvement is shown. The results in this study are consistent with those obtained by Staba et al. (1991), but different from Harwood and St. John (1986) who found an average of 5.9 percent reduction in the percentage of platooned vehicles between immediately upstream and downstream locations. However, Harwood and St. John noticed a great variation between and within passing lane sites.

There are two possible explanations for failure to detect a difference in the percentage of platooned vehicles between the upstream and downstream locations of the passing lanes. The first is the difference between what is measured and what is conceived. Conceptually, passing lanes allow vehicles which follow slower vehicles to travel at their desired speed; i.e., without being constrained by slower vehicle. At the upstream location of the passing lane, where vehicles are coming from a section of relatively constrained passing there is a high likelihood that a vehicle with time headway less than or equal to five seconds, would be restrained by a slower vehicle. But immediately after the passing lane, vehicles who have just been passed by a faster vehicle might be still within five seconds headway of the now leading faster vehicle. The headway criteria would count this vehicle as a platoon member, whereas in the actual sense it is not trailing due to inability to pass. For the purpose of evaluating passing lanes, the best definition of the vehicle following in platoons should have included both time headway and speed definition as suggested by other researchers (Gattis et al. 1996). The second is the distance downstream where the measurement is made. If the time headway is the only measurement used to classify a vehicle as "following," the downstream measurement location should be far enough from the passing lane to allow faster vehicles to separate from slower vehicles.

4.2.3.2 Headway Distribution

The comparison of the time headway distribution between the upstream location and downstream location of a passing lane is depicted in Table 4.9 and in Figure 4.7(a) to (i). The percentage of vehicles with headway less than two seconds at the upstream location is higher than the corresponding percentage at the downstream location, for eight of nine passing lane sections studied. The only passing lane which has the percentage of two seconds less headway at its upstream location than the downstream location was the eastbound passing lane at the Burton site. This is one of the sites where the data is believed to be unreliable because of malfunctioning of the traffic counters. It is also likely that data, collected by mechanical means were inadequate for this type of study.

At eight out of nine passing lane sections, the percentage of vehicles with headway between two and four seconds at upstream locations was lower than the corresponding percentage at downstream

Table 4.9: Percent of Vehicles Within Each Headway Group at the Kansas Sites

Headway, h [sec]	**h < 2**		**2 h < 4**		**4 h < 6**		**6 h**	
Location	Up	Dn	Up	Dn	Up	Dn	Up	Dn
Peabody-EB	10.1	5.3	16.9	17.7	6.0	8.9	67.0	68.0
Peabody-WB	5.9	4.2	13.8	12.6	6.0	9.4	74.2	73.8
Halstead-EB	16.0	12.6	8.7	13.0	5.7	7.7	69.6	66.7
Halstead-WB	22.1	15.1	8.6	14.1	5.2	7.3	64.1	63.5
Burton-EB	12.6	12.7	8.6	11.6	14.8	8.0	64.0	67.7
Burton-WB	16.1	11.3	8.0	11.9	4.5	5.9	71.4	70.8
Pratt-EB	7.0	4.3	9.4	10.7	6.0	7.5	77.6	77.5
Pratt-WB	8.9	5.4	12.6	12.1	5.6	8.4	72.9	74.1
Cullison-EB	16.3	12.3	6.6	10.4	4.2	5.7	72.9	71.5

locations. At eight out of nine passing lane sections, the percentage of vehicles with headway between four and six seconds at upstream locations was lower than the corresponding percentage at downstream locations. (Similar to the one to two second headway group, the eastbound passing lane at Burton site was different from the rest.) At four out of nine passing lane sections, the percentage of vehicles with headway equal to or greater than six seconds at upstream locations was lower than the corresponding percentage at downstream location. These results show that passing lanes have the capability of decreasing the number of vehicles with small headways; i.e., dispersing platoons as expected. These results are similar to those obtained by Staba et al. (1991).

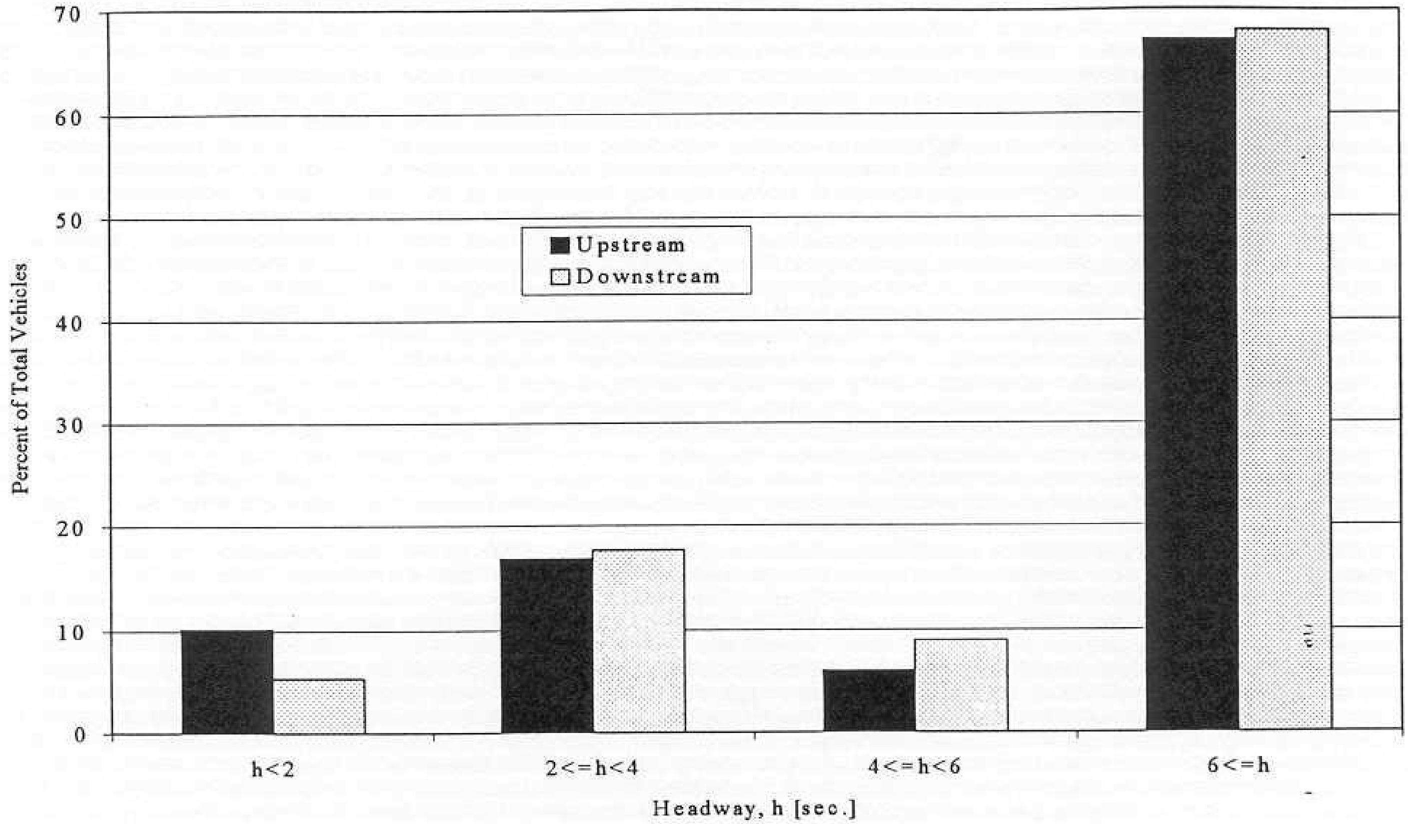


Figure 4.7(a): Headway Distribution at the Peabody Site (Eastbound)

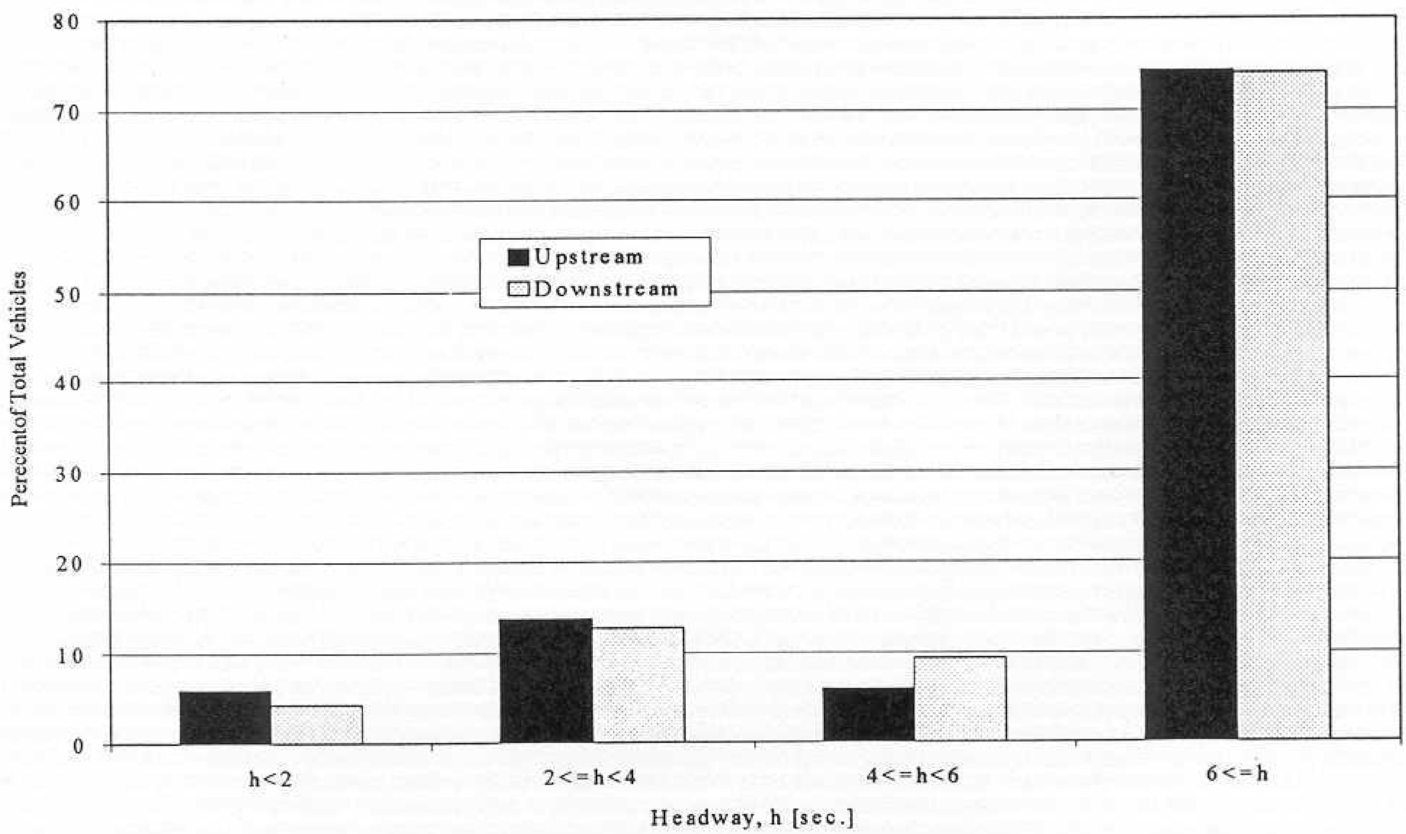


Figure 4.7(b): Headway Distribution at the Peabody Site (Westbound)

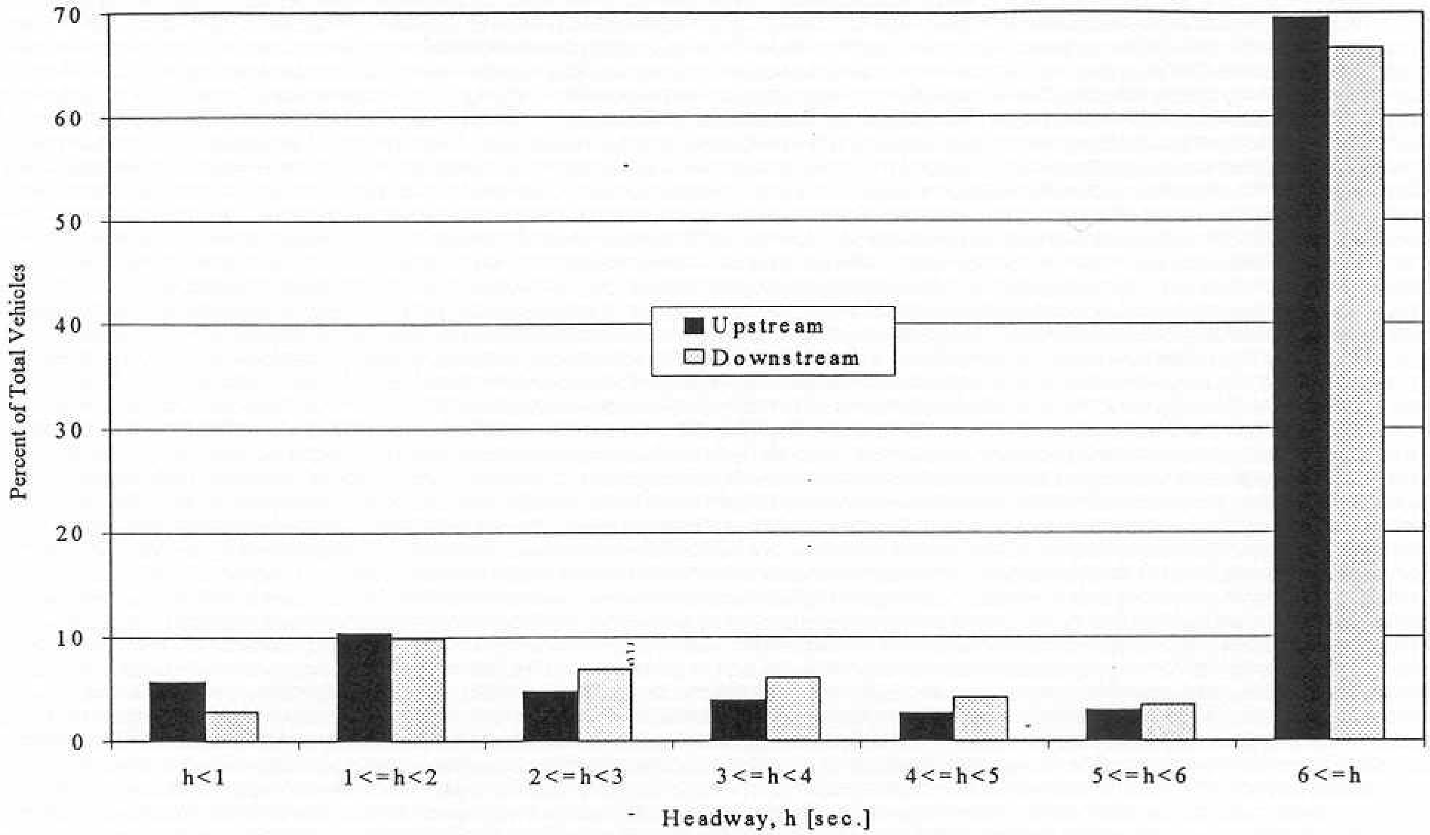


Figure 4.7(c): Headway Distribution at the Halstead Site (Eastbound)

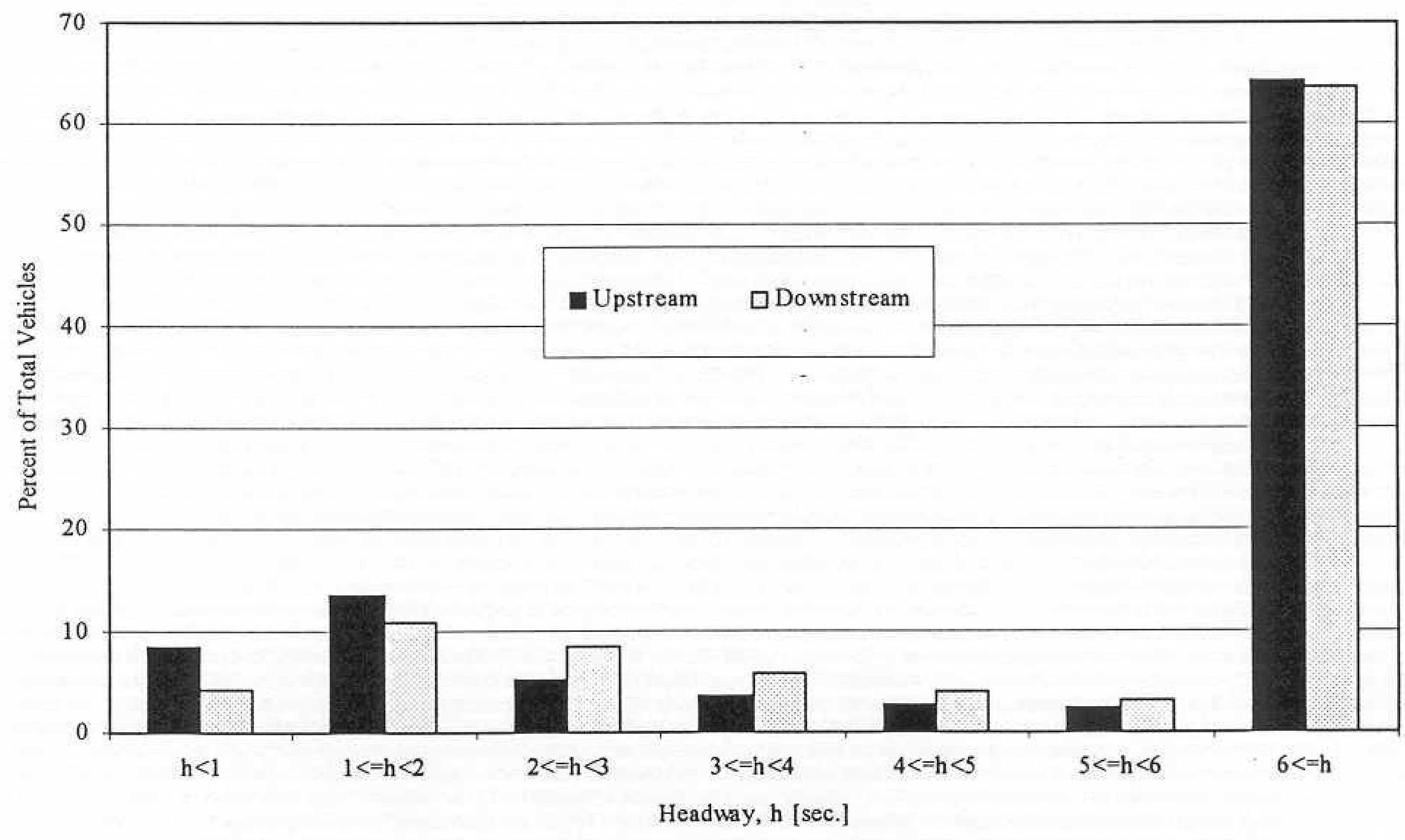


Figure 4.7(d): Headway Distribution at the Halstead Site (Westbound)

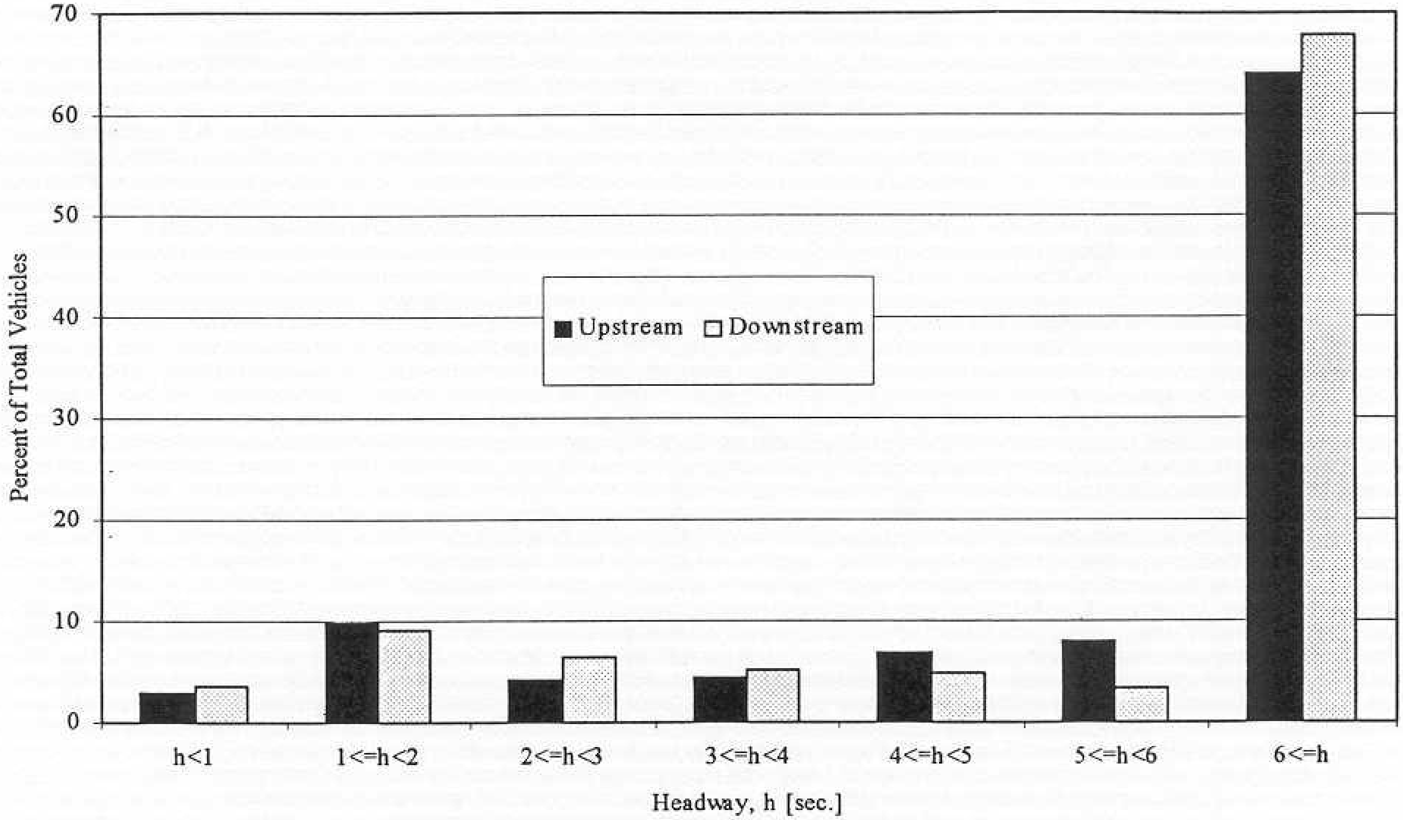


Figure 4.7(e): Headway Distribution at the Burton Site (Eastbound)

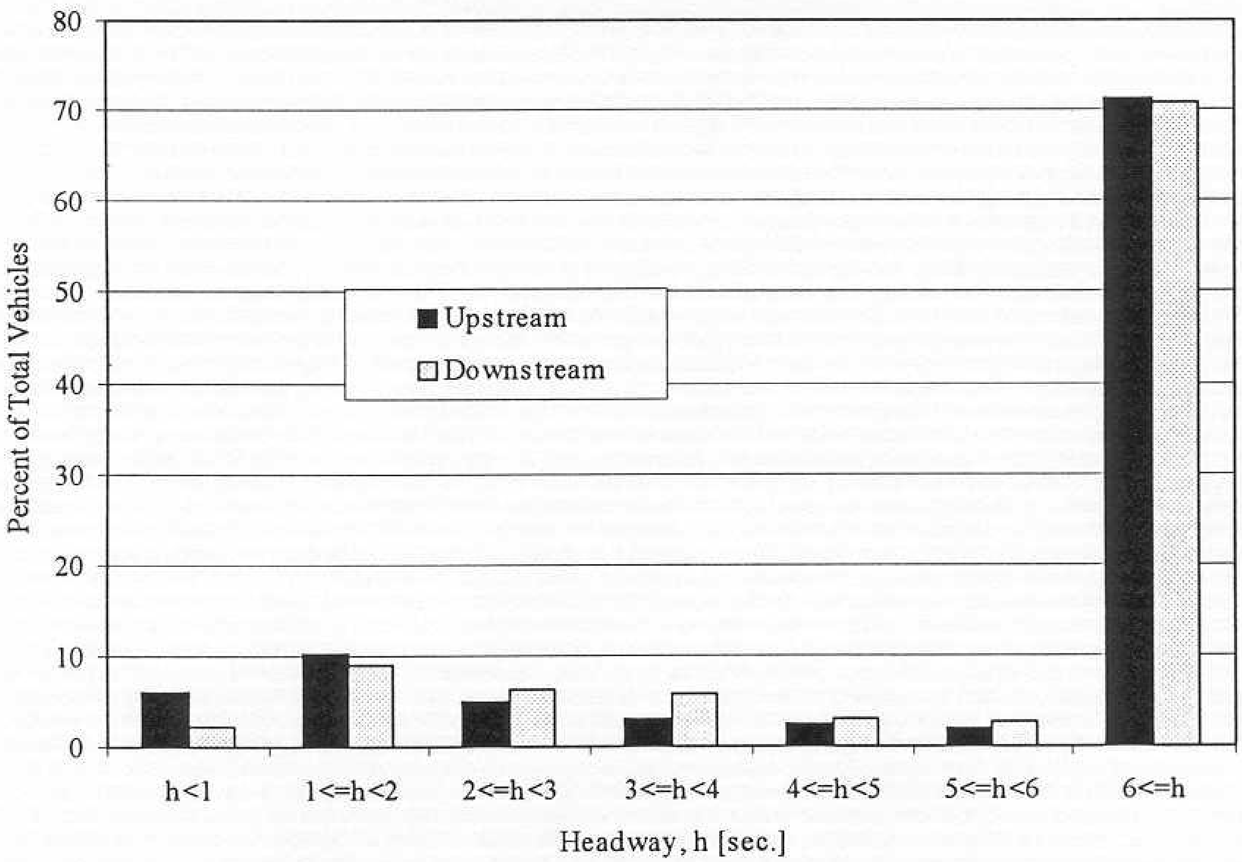


Figure 4.7(f): Headway Distribution at the Burton Site (Westbound)

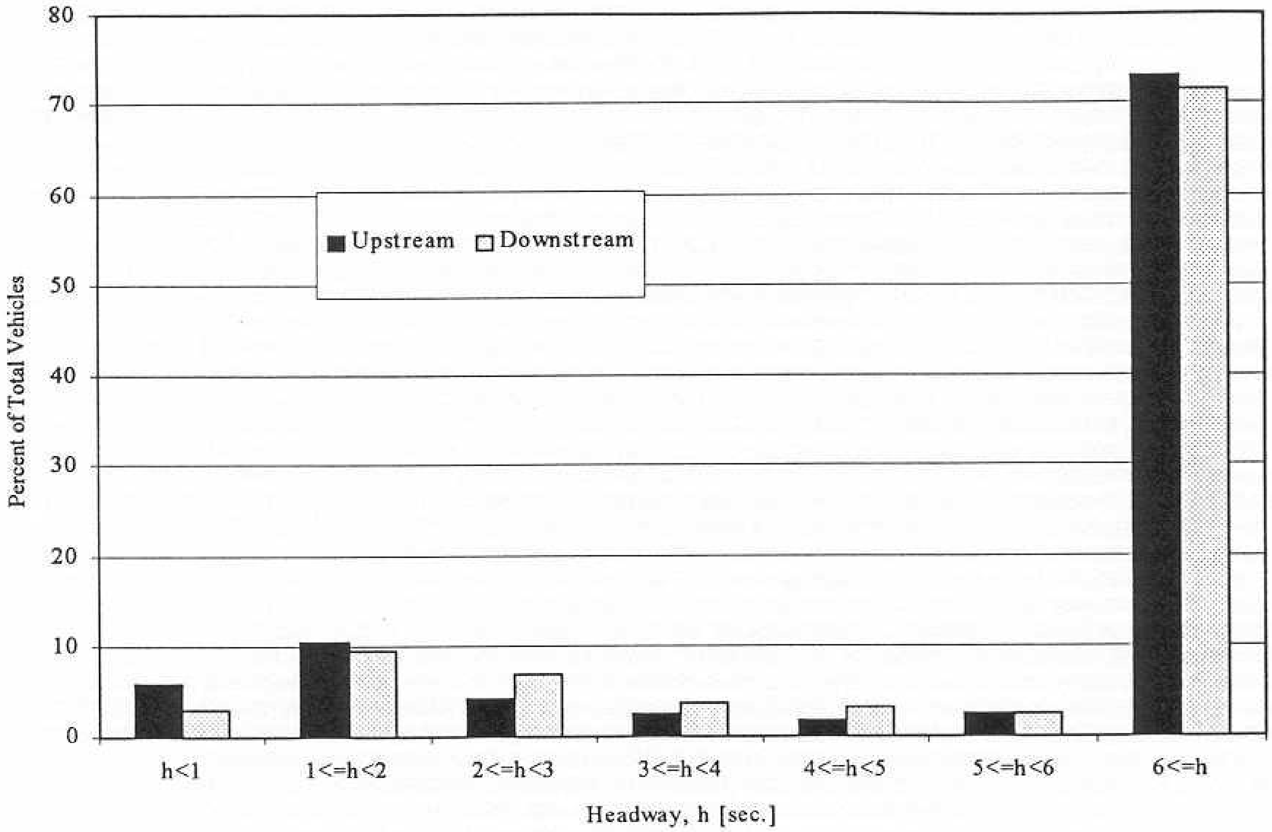


Figure 4.7(g): Headway Distribution at the Cullison Site (Eastbound)

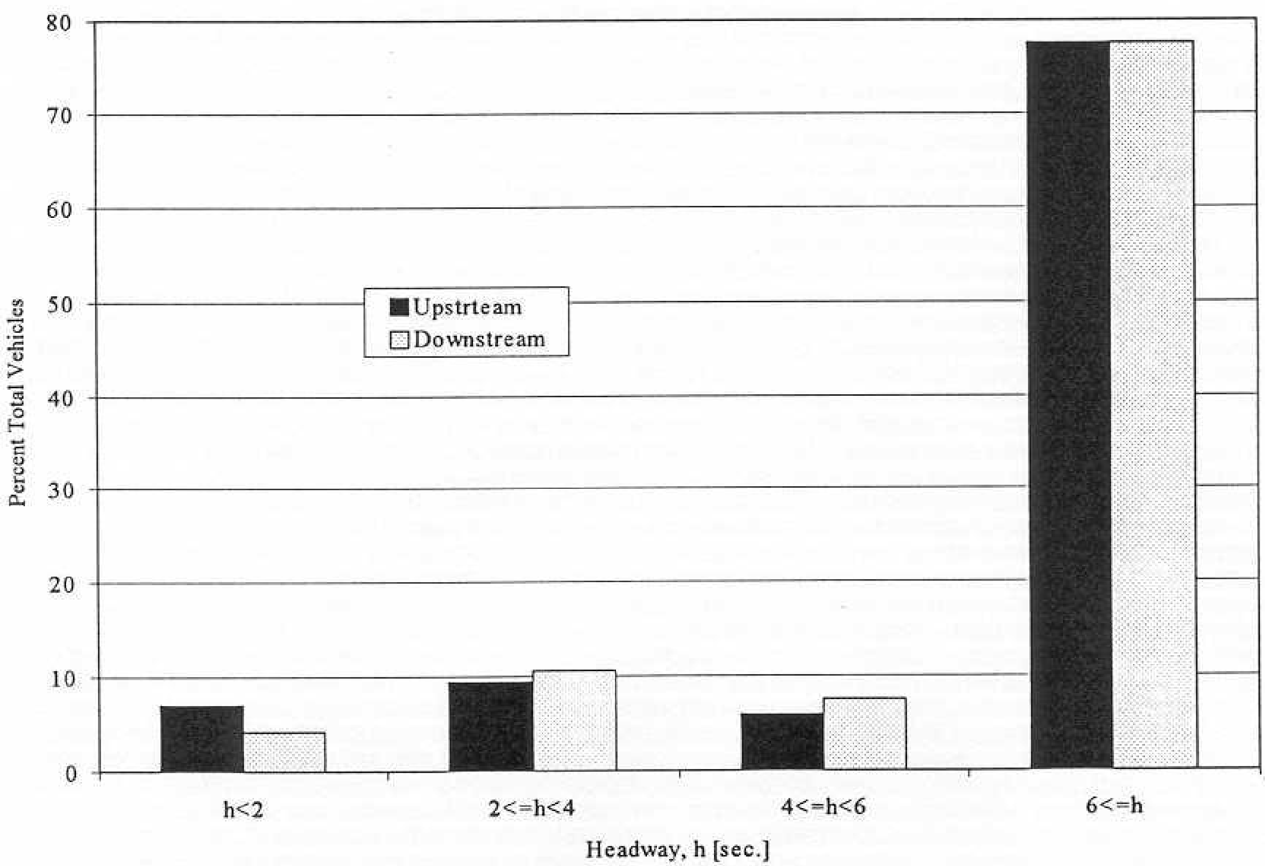


Figure 4.7(h): Headway Distribution at the Pratt Site (Eastbound)

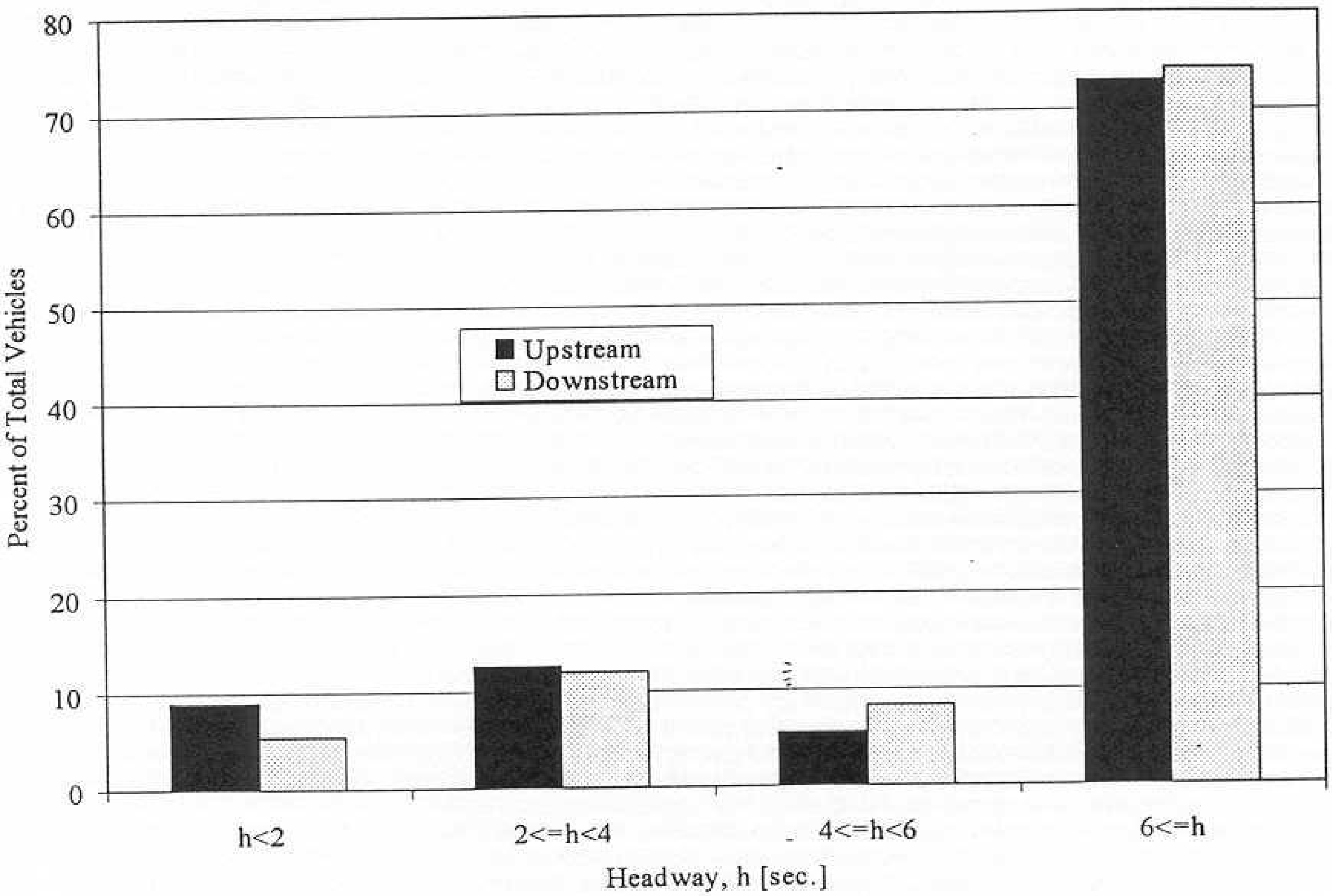


Figure 4.7(i): Headway Distribution at the Pratt Site (Westbound)

4.3 LANE-DROP TRAFFIC CONFLICTS

Table 4.10 shows the summary of the field data.

Table 4.10: Summary of Field Data for Lane-Drop Traffic Conflicts

Highway	Passing Lane Location	Period, hrs	Length	Total Volume	Non-Cars[1]	Number of Conflicts
US 50	3 Miles West of K-89	4	Long[2]	515	175	8
				500	86	12
	3 Miles East of K-89	4	Long	737	127	33
				696	82	15
	3 Miles NE of Walton	4	Long	519	89	11
				335	94	13
	2 Miles West of Peabody	4	Long	501	84	7
				540	99	11
	6 Miles West of Emporia	4	Short[3]	778	169	19
US 54	5 Miles West of Greensburg	4	Long	612	139	18
				655	220	24
	3 Miles West of Haviland	8	Long	1255	343	37
				1132	389	35
	3 Miles West of Cullison	4	Short	576	197	18
				739	142	19
	4 miles West of Pratt City	8	Short	1271	317	42
				1269	311	30

[1] Other than passenger cars.
[2] Length greater or equal to 780 ft (238 m).
[3] Length less than 780 ft (238 m).

An analysis of the variance on the conflict rates (conflicts per vehicle) was performed to find out the difference between the two taper lengths that exist on the passing lanes studied. Highway and length were the only factors analyzed, each with two levels. The percentage of non-cars was considered as a covariate, a factor which is thought to affect the response variable, but an experimenter cannot set levels on this factor. Its value is measured at the time of measuring the response variable. Before including the covariate (the proportion of non-cars), it was important to show that it satisfies the conditions assumed for incorporating covariates (analysis of covariance) in the analysis. One of the conditions is that there should be a strong linear relationship between the covariate and the response variable. The correlation coefficient (r) between the proportion of non-cars and conflict rate for short taper lengths was 0.57 and for long taper lengths it was 0.26. Kennedy and Bush (1985, p393) recommend an r-value of greater than 0.6 before considering a variable as a covariate in the analysis. Because of such low r-value, non-cars was not used as a covariate.

Three statistical tests showed no significant difference between levels of each factor at 0.05 significance level. In the first test, the analysis of variance considered the site as an experimental unit. Two options were carried out in which the interaction between the two factors were first included and then omitted. None of the analyses showed a significant difference in conflicts either between "short" and "long" or between US 50 and US 54. The second test was similar to the first one except that a single lane-drop was considered to be the experimental unit (two lane-drop per site, yields two experimental unit per site). The result showed no significant difference between sites. The third test used a different approach. It considered a vehicle as the experimental unit, and it tested whether the probability of a vehicle being conflicted was the same at all lane-drop locations using contingency tables. Two options were used by first stratifying according to "highway" and then by combining both highways together. Again no significant difference in conflicts could be detected between the two "lengths" studied.

4.4 INTERSECTION TRAFFIC CONFLICTS

An investigation of traffic interaction, created by intersecting roads (side roads), at a four-leg intersection reveals that five turning movements can cause conflicts to the through vehicles on the main highway movement. These turning movements are: 1) left-turn from the main line; 2) left-turn from the side road; 3) right-turn from the main line; 4) right-turn from the side road; and 5) through movement from the side road. Two types of analyses were conducted to determine the effect of the intersection's physical location in relation to the passing lane location. The first analysis considered the intersection to be located within the passing lane vs the intersection being located outside the passing lane. The second analysis considered the location of those intersections outside the passing lane in relation to their proximity to the passing lane. Although these two analyses seem to be related, they could not be joined because they used different response variables as it is explained below.

4.4.1 Within Vs Outside Passing Lane Location

This analysis used all conflicts to the main highway through traffic caused by all five side road turning movements mentioned in section 4.4. The argument for this aggregation of conflicts was that the geometry of four-lane and two-lane sections at within and outside the passing lane, respectively, influences the interaction between these movements. Two out of four intersections located outside the passing lane had right-turn lanes in both directions. Since the two intersections located within the passing lane did not have left or right-turn lanes, the comparison between the two locations was based on a comparison of the intersections within the passing lanes and the two intersections outside of passing lanes which have no turn lanes.

From the summarized field data shown in Table 4.11 it is apparent that the percentage of turning vehicles that cause conflicts to the through traffic on the main highway does not seem to differ between US 50 and US 54 in any of the categories. The difference between the two intersection locations without turning lanes is evident. The increasing order of the percentage of vehicles causing conflicts from within the passing lane without turn lanes, to those outside of the passing lane with turn lanes, to those outside the passing lane without turn lanes is as would be expected. Main highway through traffic at the intersection located outside the passing lane and having no turning lanes will most likely be more heavily conflicted because the through traffic has no “option lane” to go around a slowing or stopped, turning vehicle. The other two intersection location options each have an option lane for through traffic to go around these vehicles. But outside of the passing lane locations, the turn lanes have a shorter length than their counterpart at the “within” passing lane location where the relatively longer shoulder lane acts as the turn lane.

An analysis of variance was performed on the percentage of vehicles causing conflicts (conflict rate) using three main factors. The highway, the presence of turn-lanes, and the intersection’s location. Each factor was considered as a fixed factor with two levels. Due to fewer observations (only six at the six sites studied), the interaction terms between the factors were assumed negligible and hence not tested.

Table 4.12 shows the summary of the analysis of variance. Levels of the intersection location factor are significantly different at a significance level of 0.0009, with the location within passing lanes having a mean conflict rate of 3.7 percent, and the outside passing lane location with mean conflict

Table 4.11: Summarized Field Data for Intersection Traffic Conflicts

Highway	Location[1]	With Turn Lanes			No Turn Lanes		
		Turning Volume	Conflicts		Turning Volume	Conflicts	
			Total	% Causing Conflicts		Total	% Causing Conflicts
US 50	Within				43	4	9.3
	Outside	346	35	10.1	70	14	20
US 54	Within				36	3	8.3
	Outside	125	12	9.6	25	5	20
Total	Within				79	7	8.9
	Outside	471	47	10	95	19	20

[1] Relative to passing lane

Table 4.12: Summary of Analysis of Variance Results for Intersection Traffic Conflicts

Factor	**Levels**	**Mean Conflict Rate, %**	**p-value[1]**
Highway	US 50	9.6	0.2191
	US 54	9.1	
Presence of Turn Lanes	Yes	4.3	0.0011
	No	14.4	
Intersection Location	Within the Passing Lane	3.7	0.0009
	Outside of the Passing Lane	14.9	

[1] p-values 0.05 are significant at the 95% confidence level

rate of 14.9 percent. The levels of the presence of turning lanes were also significantly different at a significance level of 0.0011, with the presence of turning lanes having a mean conflict rate of 4.2, and where there are no turn lanes, the mean conflict rate was 14.4. There was not enough evidence to confirm the difference between the levels of the highway factor. The p-value for the highway was as high as 0.22, with US 50 having a mean conflict rate of 9.6 percent, and US 54 having a mean conflict rate of 9.1 percent.

4.4.2 Intersections Immediately After Passing Lane Vs Isolated From Passing Lane

The analysis of the number of conflicts for all six intersections studied showed that 33 percent of conflicts to the through vehicles on the main highway were caused by left-turning vehicles from the main highway. The remaining four side-road movements caused the remaining conflicts, with each movement causing conflicts between four and eight percent of the total conflicts. Table 4.13 shows the percentage of vehicles that caused conflicts by each side road movement.

The difference between the intersection location immediately after the passing lane and the intersection location some distance from the passing lane (isolated) is in the impact of the left turning highway traffic stream, which must slow down or stop, and wait for the gaps in the opposing traffic before turning into the side road. (A detailed account of left-turn in relation to the distance of an intersection from the end of the passing lane was given earlier in Section 3.4.1.)

A vehicle turning left from the main highway into the side road can cause two types of conflicts to the through traffic of the main highway: the first is the conflict to the through traffic in the same direction (left-turn, same-direction conflict); and the second is the conflict to the through, opposing traffic (opposing left-turn conflict). The left-turning driver has more control of the conflict with opposing traffic as opposed to the conflict with same direction traffic in which neither the conflicting nor the conflicted driver have much control. It is reasonable to expect that the same direction conflict will occur more often. As expected, of the total 46 conflicts caused by left-tun movement from the main highway, 42 (91.4 percent) were of the left-turn, same direction type. Comparable results were

Table 4.13: Intersection Conflicts Distribution by Type of Turning Movement

Highway	Data Type	LT[1] From Main Line	RT[2] From Main Line	RT From Side Road	LT From Side Road	TH[3] From Side Road
US50	Movement Volume	103	72	158	56	70
	Number of Conflicts	38	4	6	2	3
	Percent of Vehicles Causing Conflicts	37	6	4	4	4
US54	Movement Volume	36	66	27	52	11
	Number of Conflicts	8	6	1	4	1
	Percent of Vehicles Causing Conflicts	22	10	4	8	9
Total	Movement Volume	139	138	185	108	81
	Number of Conflicts	46	10	7	6	4
	Percent of Vehicles Causing Conflicts	33	7	4	6	5

[1]LT = Left Turn [2]RT = Right Turn [3]TH = Through

obtained by Glauz and Migletz (1980) in their observation of intersection traffic conflicts. They found that for intersection turning movements, same-direction conflicts had much higher rates, followed by opposing left-turn and same-direction, left-turn conflicts, with all cross-traffic conflicts having the least. For this reason, only left-turn, same-direction conflicts were used in the comparison between intersection locations immediately after the passing lane and those isolated from the passing lane section. Table 4.14 summarizes traffic conflict to the through vehicles on the main highway caused by left-turns from the main highway. Although the intent was to compare only two locations, "immediately after the passing lane," and "isolated from the passing lane," the third location, "within the passing lane," was added for comparison purposes.

Table 4.14: Summary of Intersection Traffic Conflicts Caused by Left-Turn Vehicles

Highway	Location Relative to Passing Lane	Sections With Turn Lanes			Sections Having No Turn Lanes		
		Left Turning Volume	Conflicts		Left Turning Volume	Conflicts	
			Total	% Causing Conflicts		Total	% Causing Conflicts
US 50	Within				2	1	50
					3	2	66.7
	Immediately after	67	20	29.8	3	3	100
	Isolated	16	2	12.5	12	7	58.3
US 54	Within				7	0	0
					2	0	0
	Immediately after				13	5	38.5
	Isolated	2	0	0			
		12	2	16.7			
Total	Within				14	3	21.4
	Immediately	67	20	29.8	16	8	50
	Isolated	30	4	13.3	12	7	58.3

As shown in Table 4.14, where turning lanes were present, the "immediately after" location had a higher conflict rate of 29.8 percent compared with 13.3 percent for the "isolated" location as expected. However, where turning lanes were not present, the trend was opposite; i.e., the "isolated" location has 58.3 percent conflict rate compared with 50 percent for the "immediately after" location, but the "within" location had only a 21.4 percent conflict rate, lower than both the "immediately after" and "isolated" locations. However, the analysis of variance could not detect any difference between "immediately after" and "isolated" locations due to small sample size.

4.5 ACCIDENT ANALYSIS

Analysis of accidents to determine the effect of providing passing lane(s) from a traffic safety standpoint, was done using two methods: 1) before-and-after analysis; and 2) cross-sectional analysis. In the before-and-after analysis, relatively homogenous highway sections (entities) were identified. Differences in traffic volumes, length between before-period and after-period were taken into account by using correction factors r_{tf} and r_d, respectively.

4.5.1 Entities

Entity is the highway component which receives the safety treatment. Because the safety effect of the passing lane extends beyond the physical boundaries of the passing lane section, the highway segments analyzed were defined by the boundaries of the highway improvement projects in which the passing lane sections were constructed, by an urban boundary, or by a junction with a main highway.

On US 54, a section from K-54 junction on the west and Pratt on the east was selected. This section of about 38 miles (60.8 kilometers), contains all four passing lane sites along US 54 used in this study, and is divided into three highway improvement projects.

On US 50, three highway sections were identified. The first section is bounded by the Harvey/Reno county line on the west and by the city limit of Newton on the east. It is about 17 miles (27.2 kilometers) long with two passing lane sites, and is divided into two highway improvement projects. The second section is bounded by the Walton city limit on the west and the Peabody city limit on the east. It is about 11 miles (17.6 kilometers) long with two passing lane sites within one highway improvement project. The third section is bounded by the Strong City limit on the west and the Emporia city limit on the east. It is about 15 miles (24 kilometers) long and contains one passing lane site and two highway improvement projects.

The basic unit on which KDOT keeps accident records is the "control section (CS)." The control section is a highway section with either uniform geometric characteristics or uniform pavement surface characteristics. Location of control sections and location of passing lanes are defined in different systems. While control section locations are defined by the borders of the county, passing lanes locations are defined by a route milepost system which starts at the border of the state. This discrepancy makes it difficult to correlate the passing lanes with control sections. Several adjacent control sections within the same highway improvement project were combined to form a longer section (called an entity). The objective of combining control sections was to make sure that each formed entity had experienced at least one accident in a one year period. If the section registered zero accidents in one period, it does not necessarily mean that the mean accident rate is zero (the observed value). The zero could have happened by chance. Control sections with the following (combined) attributes were chosen for analysis:

ž rural,

ž 2-lane, and

ž non-corporate.

A total of seven entities, three on US 54 and four on US 50 were formed. The resulting entities have different lengths. Resende and Benekohal (1997) investigated the effect of section length on accident modeling (using regression analysis) and found that shorter sections (less than 0.5 miles) affect the form of accident prediction models. They recommended that if such short lengths are used they should be modeled separately. However, their investigation was limited to 1.0 mile sections. The effect of section length when using before-and-after analysis is not known. Control sections within a particular highway improvement would have the same before-period and the same after-period.

To determine the estimate of the mean accident rate, two approaches are normally recommended: 1) the average of several similar sections, or/and 2) the average of the same section over a longer period. The first approach masks site to site variation (variation between sites), and the second masks time trend variation. In this sub-study, the period was fixed, and the first alternative was used. Table 4.15 shows the attributes of the seven entities selected for accident analysis.

Table 4.15: Attributes of Selected Entities

Highway	Entity ID No.	Section Boundaries	Length [miles]	Before-Period [yrs]	Construction-Period [yrs]	After-Period [yrs]
US 54	1	K 154 JCT[a] & US 83 JCT	6.970	2 (1990-91)	1 (1992)	4 (1993-96)
	2	US 83 JCT & Kiowa/Pratt County line	15.405	2 (1990-91)	3 (1992-94)	2 (1995-96)
	3	Kiowa/Pratt County line & Pratt city limit	13.213	3 (1990-92)	2 (1993-94)	2(1995-96)
US 50	4	Reno/Harvey County line & K 89 JCT.	10.319	4 (1990-93)	2 (1994-95)	1 (1996)
	5	K 89 JCT & Newton City limit	5.934	4 (1990-93)	3 (1994-96)	-----[b]
	6	Walton City limit & Peabody City limit	9.527	3 (1990-92)	2 (1993-94)	2 (1995-96)
	7	Strong City & Emporia City limit	15.201	2 (1990-91)	3 (1992-94)	2 (1995-96)

[a] JCT=Junction

[b]---- No after-period data is available as of this writing.

4.5.2 Period Length

To eliminate seasonal variations, one year was considered as the unit of time, on the assumption that all years had similar corresponding seasons. When the duration of the after period is different from the duration of the before period, the expected accident frequency in the after-period if the improvement wouldn’t have been made ( ) is equal to the observed accident frequency before the treatment is implemented ( ) adjusted by the factor r_d, where r_d is defined as the ratio of the after period to the before period; i.e.,

$$r_d \quad \frac{\textit{Duration of after period}}{\textit{Duration of before period}} \qquad \ldots\ldots\ldots\ldots \quad \mathbf{4.1}$$

4.5.3 Estimation and Prediction of Accident Frequencies

Plots of accidents shown in Figures 4.8(a) through 4.8(g) did not show any time trend in accident frequencies, probably due to the short periods. The before-period frequency ( ) and after-period frequency ( ) were estimated by the average of observed frequencies for each period. The accident frequency for the after period, assuming that the improvement was not implemented ( ), was predicted from the trend of the before period and then adjusted for change in traffic volume and differences in period lengths between before and after periods; i.e., = x r_d x t_{rf}. Where t_{rf} is the traffic factor and is defined as the ratio between the expected accidents at the traffic level corresponding to the after-period traffic volume, to the number of expected accidents corresponding to the traffic level during the before-period. Determination of accident reduction , due to the highway improvement project which includes provision of passing lanes, is shown in Table 4.16. With the assumption, that the accident frequency during the “after-period (with improvements)” follows a Poisson distribution, the minimum number of accidents reduction required for significance at the 95 percent level of confidence is 22.54. With the observed reduction of 11.1 accidents, less than the critical value of 22.54, it leads to the conclusion that the data was not sufficient to detect any safety improvement due to the highway improvement project. The value 22.54 is computed by considering the process a normal distribution adjusted by a "continuity correction factor" and a "normal approximation" correction factor as explained by Stokes and Mutabazi (1996).

Table 4.16: Determination of Accident Reduction.

Entity ID No.	**Correction Factors**		**Estimate**				
			Before-Period w/o Improvement,	**After-Period w/ Improvements,**		**After-Period w/o Improvements,**	
	r_d	r_{tf}	**Mean**	**Mean**	**Variance**[a]	**Mean**	**Variance**[b]
1	2	0.96	16	42	42	30.7	77.9
2	1	1.17	29	34	34	34	62.9
3	0.67	1.18	32	25	25	25	32.4
4	0.25	1.97	43	7	7	21	19.4
5	No After-Period						
6	0.67	1.40	20	22	22	18.6	24.3
7	1	1.11	59	54	54	65.5	158.3
All	=		199	184	184	195.1	375.2
	= 195.1-184=11.1 accidents			Std. dev. of = 23.6 accidents			

[a] is assumed to have Poisson distribution with variance = mean

[b] Var{ } = $r_d^2[r_{tf}^2 + {}^2\text{Var}\{r_{tf}\}]$

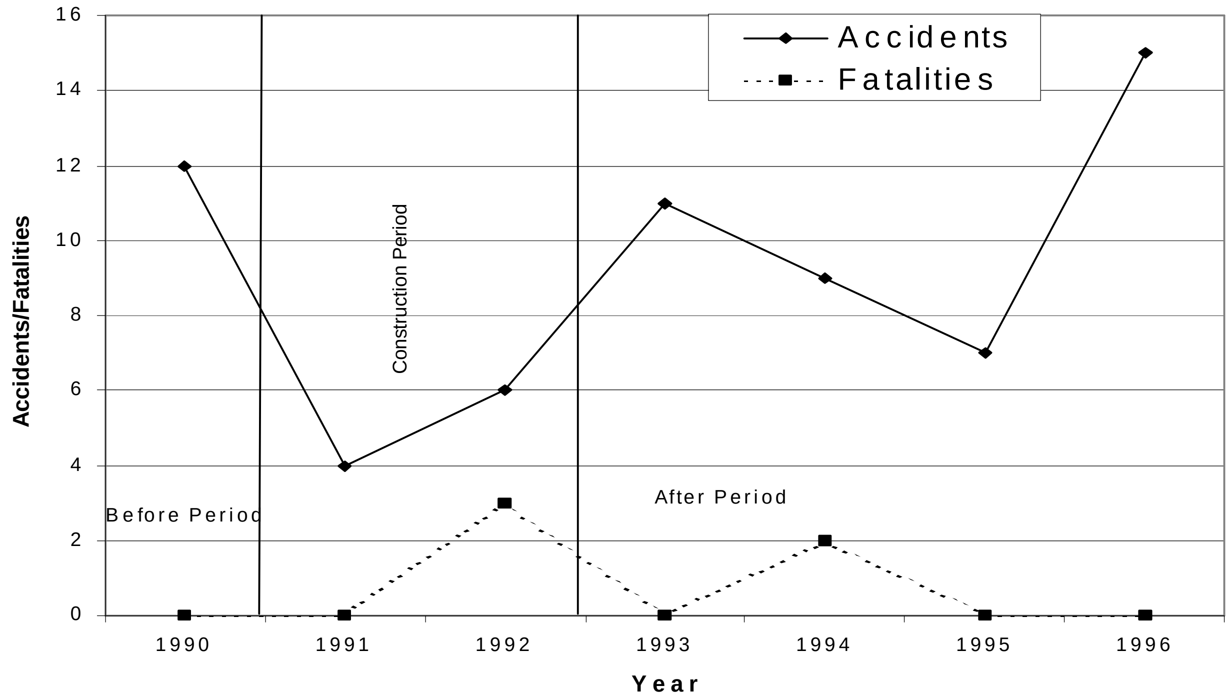


Figure 4.8(a): Number of Accidents and Fatalities by Year on US 54 Highway Between K-154 Junction and US 183 Junction

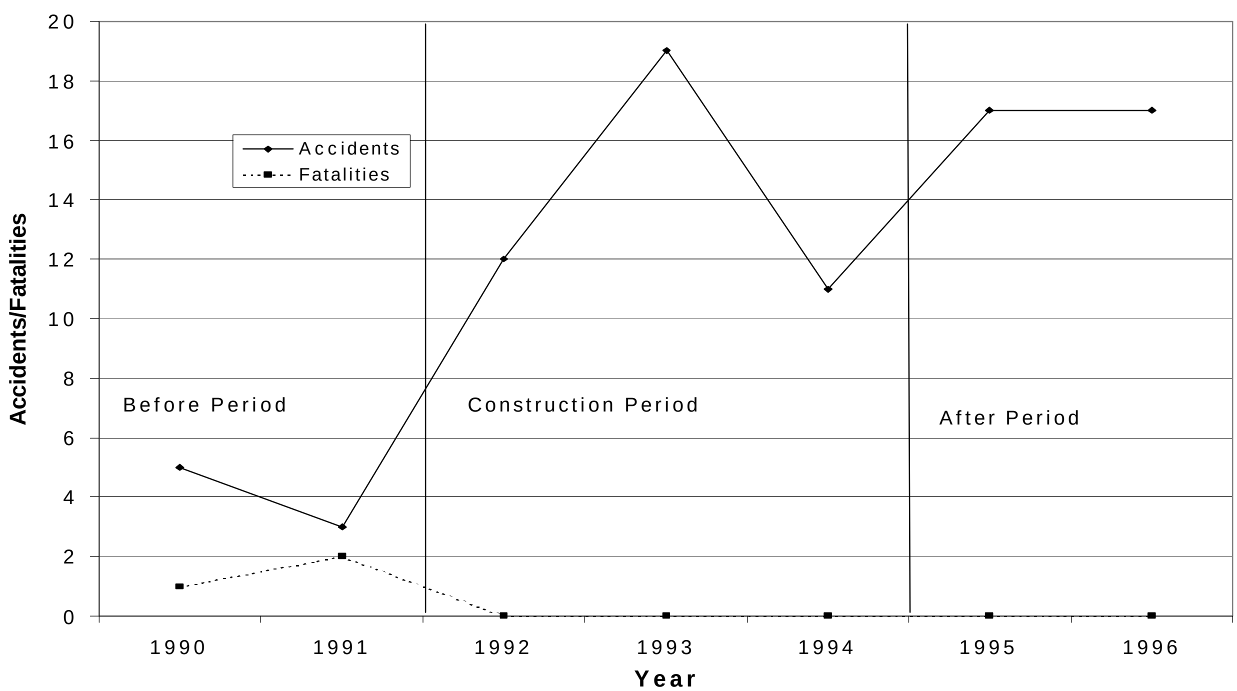


Figure 4.8(b): Number of Accidents and Fatalities by Year on US 54 Highway Between US 183 Junction and Kiowa/Pratt County Line

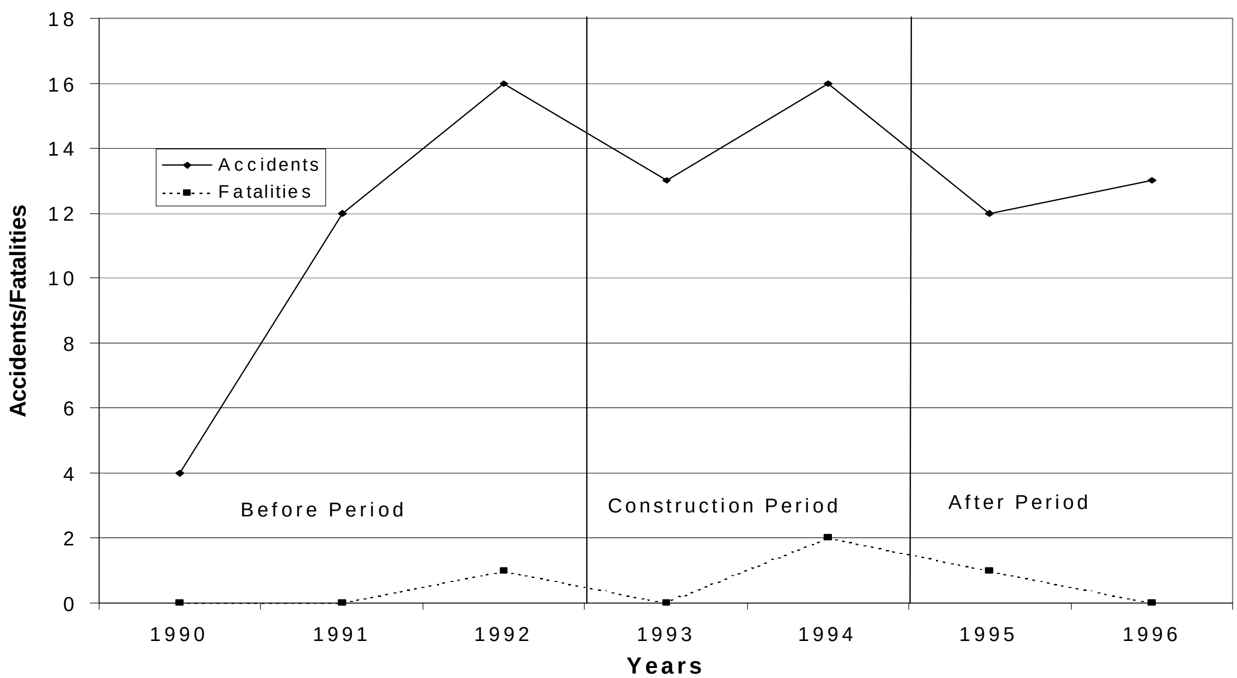


Figure 4.8(c): Number of Accidents and Fatalities by Year on US 54 Highway Between Kiowa/Pratt County Line and Pratt City Limit

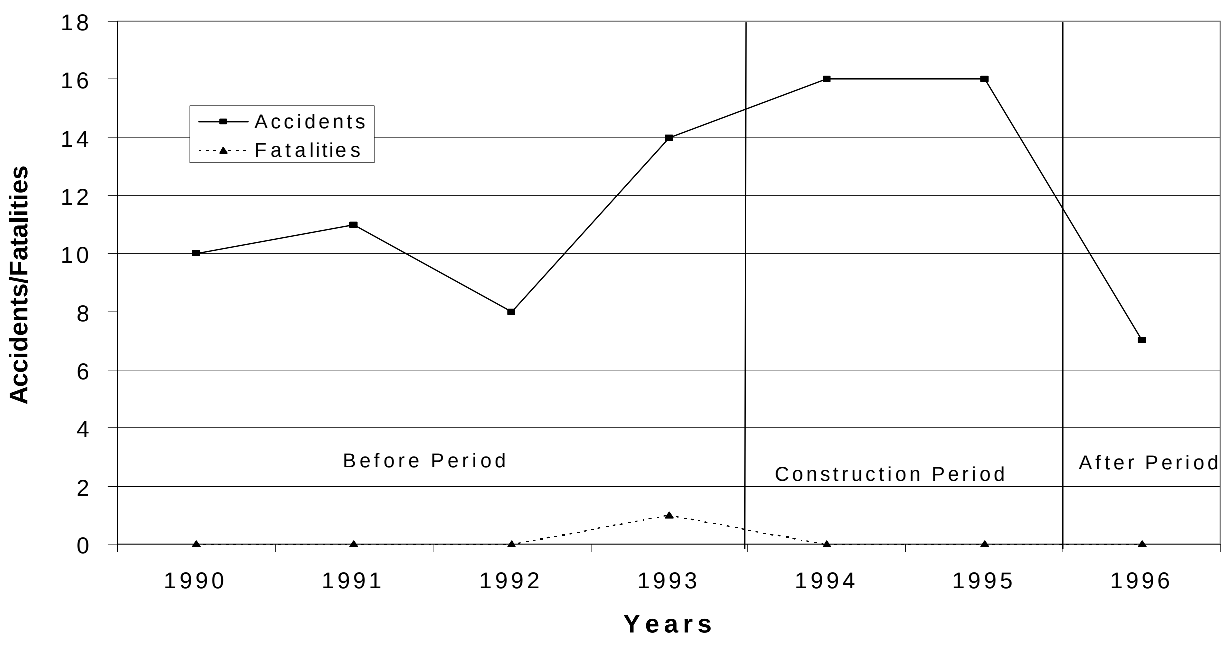


Figure 4.8(d): Number of Accidents and Fatalities by Year on US 50 Highway Between Reno/Harvey County Line and K 89 Junction

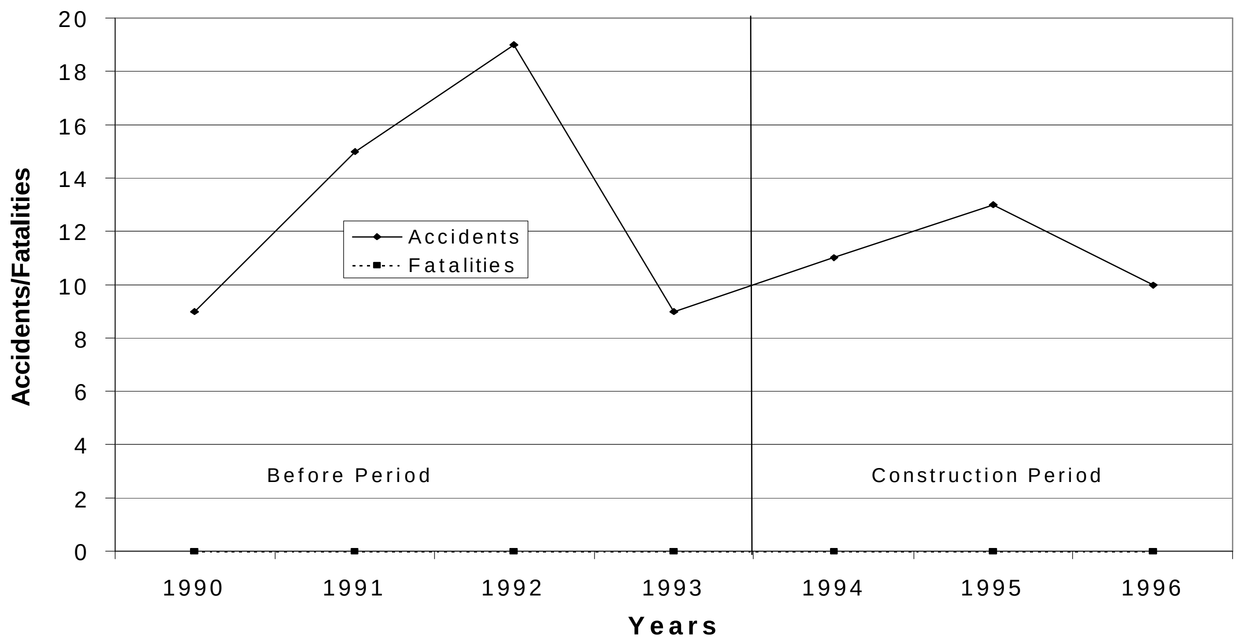


Figure 4.8(e): Number of Accidents and Fatalities by Year on US 50 Highway Between K 89 Junction and Newton City Limit

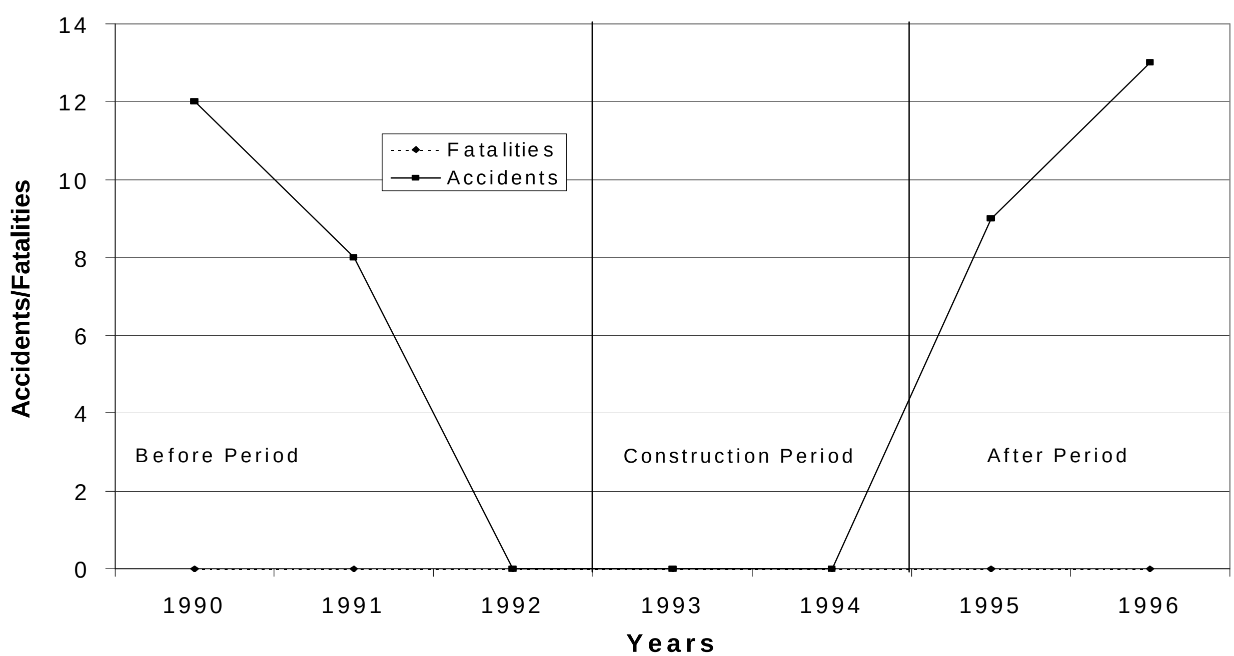


Figure 4.8(f): Number of Accidents and Fatalities by Year on US 50 Highway Between Walton City Limit and Peabody City Limit

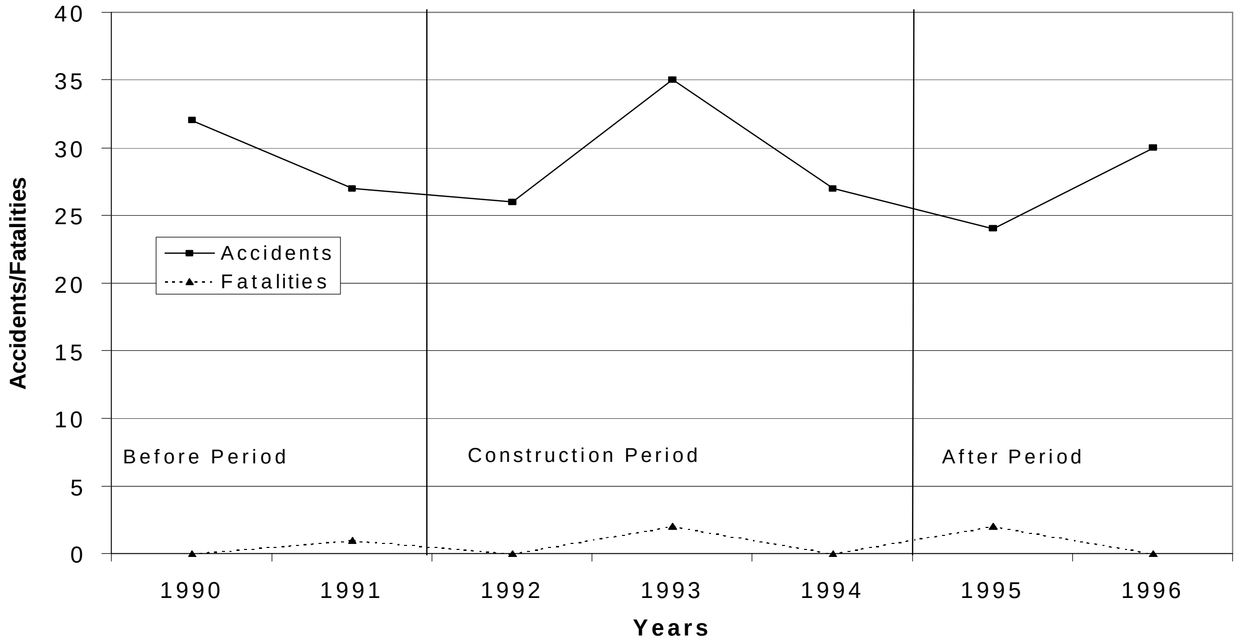


Figure 4.8(g): Number of Accidents and Fatalities by Year on US 50 Highway Between Strong City Limit and Emporia City Limit

4.5.4 Cross-Sectional Analysis

In a classical cross-sectional analysis, highways with passing lanes were to be compared with comparable highways without passing lanes to determine the effect of passing lanes on reducing accidents. Highways are considered comparable if they both could be classified as rural, non-corporate, and two-lane. This type of cross-sectional analysis is known as “high-accident location analysis.” The same entities used in the before-after analysis were used. Table 4.17 shows this comparison. In Table 4.17, all p-values for the after period are smaller than 0.005 (0.5% significance level) implying that the sections with passing lanes have significantly fewer accidents than the state average rural two-lane road.

Table 4.17: Comparison of Accident Rates on Improved Sections to the State Average

Entity ID No.	Before-Period			After-Period			
	State Average Accident Rate[a]	Entity Accident Rate	p-value[b]	State Average Accident Rate	Entity Accident Rate	p-value	Change of p-value from before-period
1	1.272	0.707	0.007	1.452	0.94	0.002	decreased
2	1.272	0.564	3.6×10^{-7}	1.488	0.622	2.5×10^{-9}	decreased
3	1.248	0.464	1.98×10^{-11}	1.488	0.508	6.3×10^{-11}	increased
4	1.292	0.715	1.23×10^{-5}	1.547	0.367	7.8×10^{-7}	decreased
5	1.292	1.138	0.583	No "After-Period" data yet			
6	1.248	0.508	2.15×10^{-6}	1.488	0.768	3.8×10^{-4}	increased
7	1.272	1.154	0.249	1.488	1.015	0.0017	decreased

[a] Number of accidents per one million vehicle miles of travel

[b] significance level at which the entity accident rate is less than the state average rate. Values less than 0.005 are significant at the 0.5 percent significance level.

4.6 DRIVERS SURVEY

This section summarizes the results of a drivers' survey that was carried out using a mail-questionnaire, self-addressed, postage-paid, postcards. One thousand cards were distributed to drivers in the field at one location for each highways US 50 and US 54.

4.6.1 Response Rate

A total of 406 out of 1000 distributed survey cards were returned. This represents an overall response rate of 40.6 percent for both locations. The response rate from US 50 was 42 percent, and from US 54 was 39.2 percent. These rates are not significantly different (p-value of 0.367).

4.6.2 Frequency of Traveling on Passing Lane Sections

Table 4.18 and Figure 4.9 shows the frequency of travel on each section. The frequency "other" was taken as less than once per month. The frequency distribution shown in Table 4.18 and Figure 4.9 between the two highways is significantly different with p-value of 0.009. The difference was expected because of geographical locations, as explained below in the following sub-section. It is important to note the higher proportion of infrequent drivers (less than once per month) on US 54 compared to US 50.

Infrequent drivers may affect the operational performance and safety of the highway; i.e., passing lane effectiveness may be reduced by infrequent drivers who are unfamiliar with passing lane locations and configurations. The higher proportion of infrequent drivers on US 54 may be attributed to: 1) US 54, being close to the state border, is likely to carry more out-of-state drivers (as later shown in Section 4.6.6) than US 50; 2) US 50, which runs parallel and close to an Interstate (I-70), is likely to have primarily local drivers making more frequent trips, because drivers making longer trips would likely be attracted to the Interstate I-70. Thus drivers would generally be more familiar with the passing lane locations and their configurations.

Table 4.18: Frequency of Travel on Passing Lane Sections

Highway	Daily N (%)	Once per Week N (%)	Once per Month N (%)	Other N (%)	Total N (%)
US 50	74(35.5)	75(36.1)	38(18.3)	21(10.1)	208(100)
US 54	63(32.3)	52(26.7)	38(19.5)	42(21.5)	195(100)
Total	137(34.0)	127(31.5)	76(18.9)	63(15.6)	403(100)
p-value	0.009				

NOTE: p-value for testing the significance difference in the response distributions between US 50 and US 54 highways.

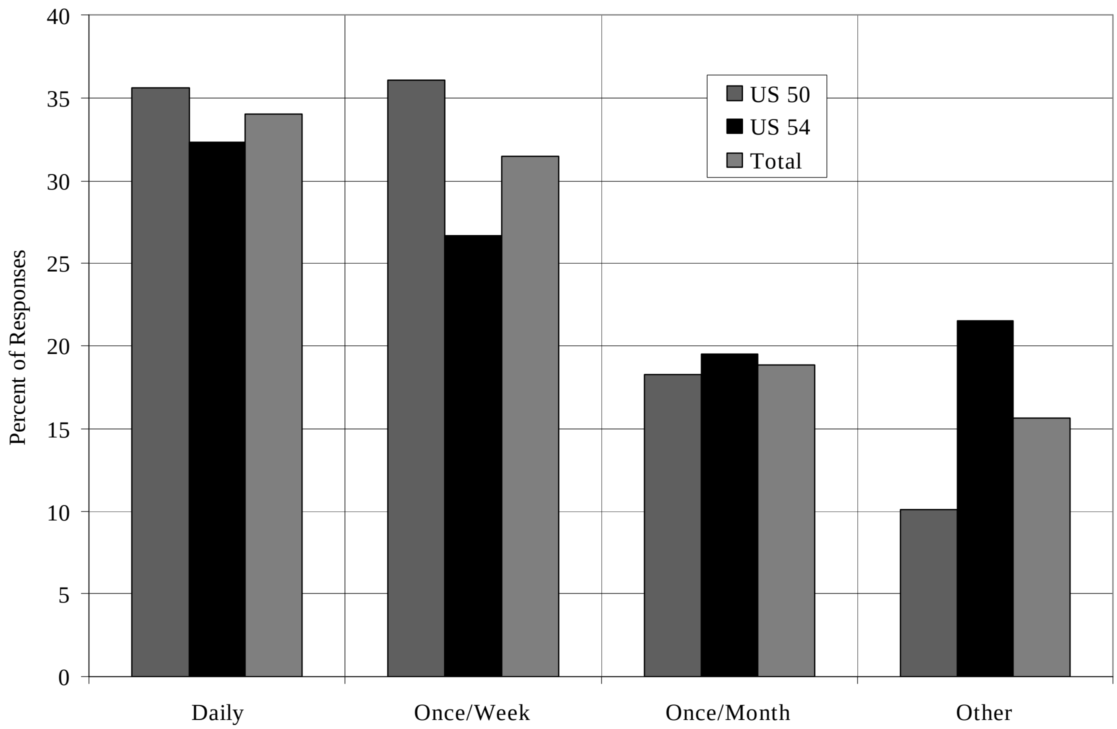


Figure 4.9: Frequency of Travel on Passing Lane Sections

4.6.3 Distribution of Vehicle Type in the Survey

Table 4.19 and Figure 4.10 shows the distribution of the vehicle mix for responding drivers. Probably this was the question that most confused drivers. It asked to determine the type of vehicle the drivers were driving at the time of survey. However, some drivers indicated more than one vehicle, and those responses were excluded. Of the 406 returned cards, only 324 were used for analysis of this question. The vehicle mix distributions for both highways are not significantly different (p-value of 0.578). Since both highways are of the same functional class they will not differ in traffic characteristics.

Table 4.19: Vehicle Mix Distribution During the Survey

Highway	Car N (%)	Pick Up N (%)	Van N (%)	Semi N (%)	Other Truck N (%)	Other Vehicle N (%)	Total N (%)
US 50	83(51.6)	26(16.2)	10(6.2)	34(21.1)	2(1.2)	7(3.7)	162(100)
US 54	82(50.3)	29(17.8)	18(11.0)	26(16.0)	3(1.8)	5(3.1)	163(100)
Total	165(50.9)	55(17.1)	28(8.6)	60(18.5)	5(1.5)	12(3.4)	325(100)
p-value	0.578						

NOTE: p-value for testing the significance difference in the response distributions between US 50 and US 54 highways

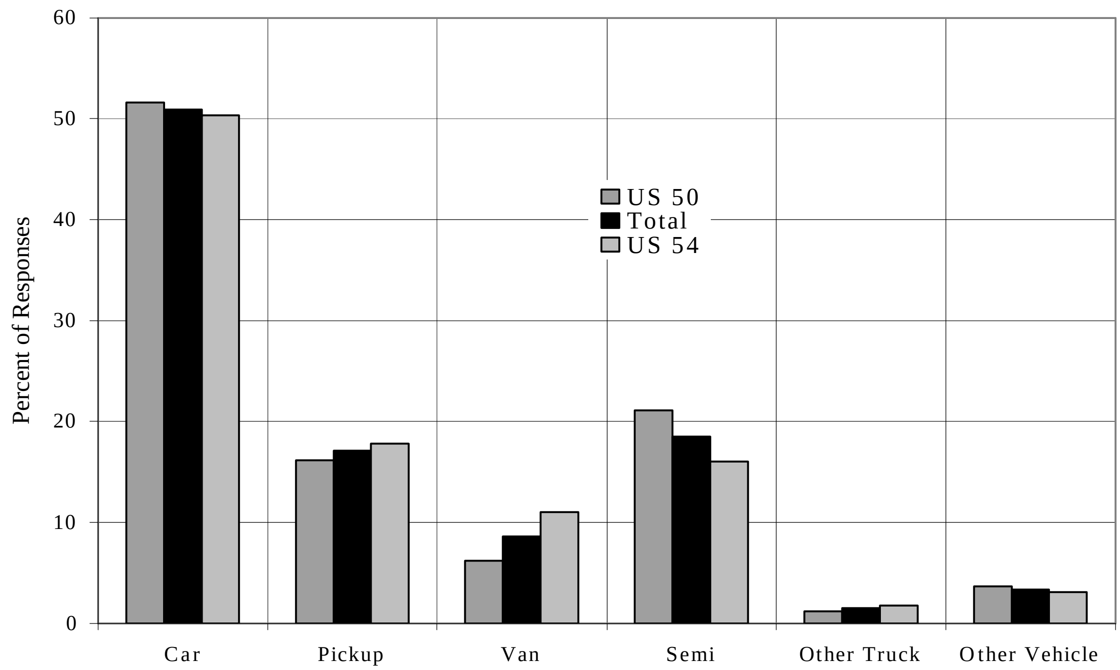


Figure 4.10: Vehicle Mix Distribution During the Survey

4.6.4 Need for More Passing Lanes

Table 4.20 and Figure 4.11 shows the responses to the question on the need for more passing lanes. Eighty-six percent of drivers agree that more passing lanes are needed in the state indicating a high degree of acceptance and satisfaction with the concept. The distribution of response between the two highways was not significantly different (p-values of 0.147). A significant proportion of drivers would like to see more passing lanes constructed.

4.6.5 Passing Lane Attributes

The question on passing lane attributes is divided in four categories: 1) length, 2) speed, 3) safety, 4) time saving. Table 4.21 and Figure 4.12 depicts the distribution of responses on this question.

Table 4.20: Need for Extra Passing Lanes in the State

Highway	Yes N (%)	No N (%)	Not Sure N (%)	Total N (%)
US 50	176(84.2)	7(3.4)	26(12.4)	209 (100)
US 54	172(87.8)	10(5.1)	14(7.1)	196 (100)
Total	348(85.9)	17(4.2)	40(9.9)	405 (100)
p-value	0.147			

NOTE: p-value for testing the significance difference in the response distributions between US 50 and US 54 highways.

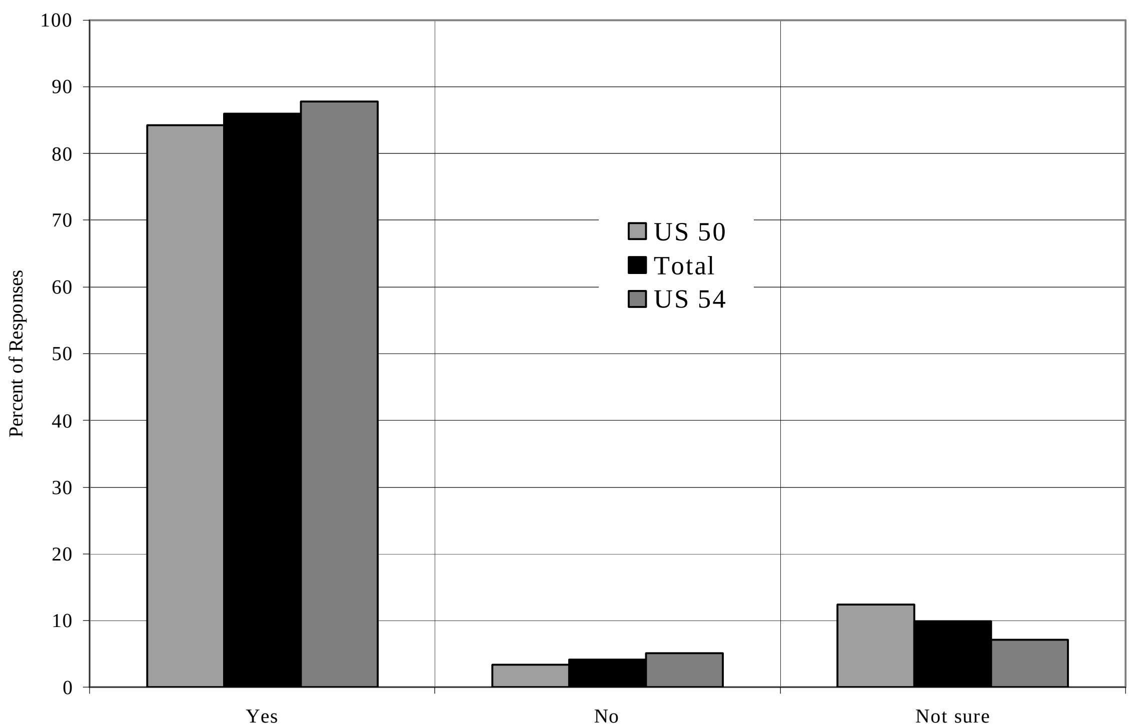


Figure 4.11: Need for More Passing Lanes in the State

Table 4.21: Passing Lane Attributes

Highway	Length			Encourage Speeding[a]	Safety		Save Time[a]	I Don't Know
	Too Short	Too Long	Just Right		Improve Safety	Cause Safety Concern		
	N (%)	N (%)	N (%)	N (%)	N (%)	N (%)	N (%)	N (%)
US 50	91(47.9)	2(1.1)	97(51)	15(7.2)	172(94)	11(6)	107(51.2)	1(0.5)
US 54	108(59.3)	0(0)	74(40.7)	16(8.2)	160(92)	14(8)	108(55.1)	0(0)
Total	199(53.5)	2(0.5)	171(46)	31(7.6)	332(93)	25(7)	215(53.1)	1(0.2)
p-value	0.048			0.711	0.564		0.378	

[a] Percent of total cards received; i.e., 406

NOTE: p-value for testing the significance difference in the response distributions between highways.

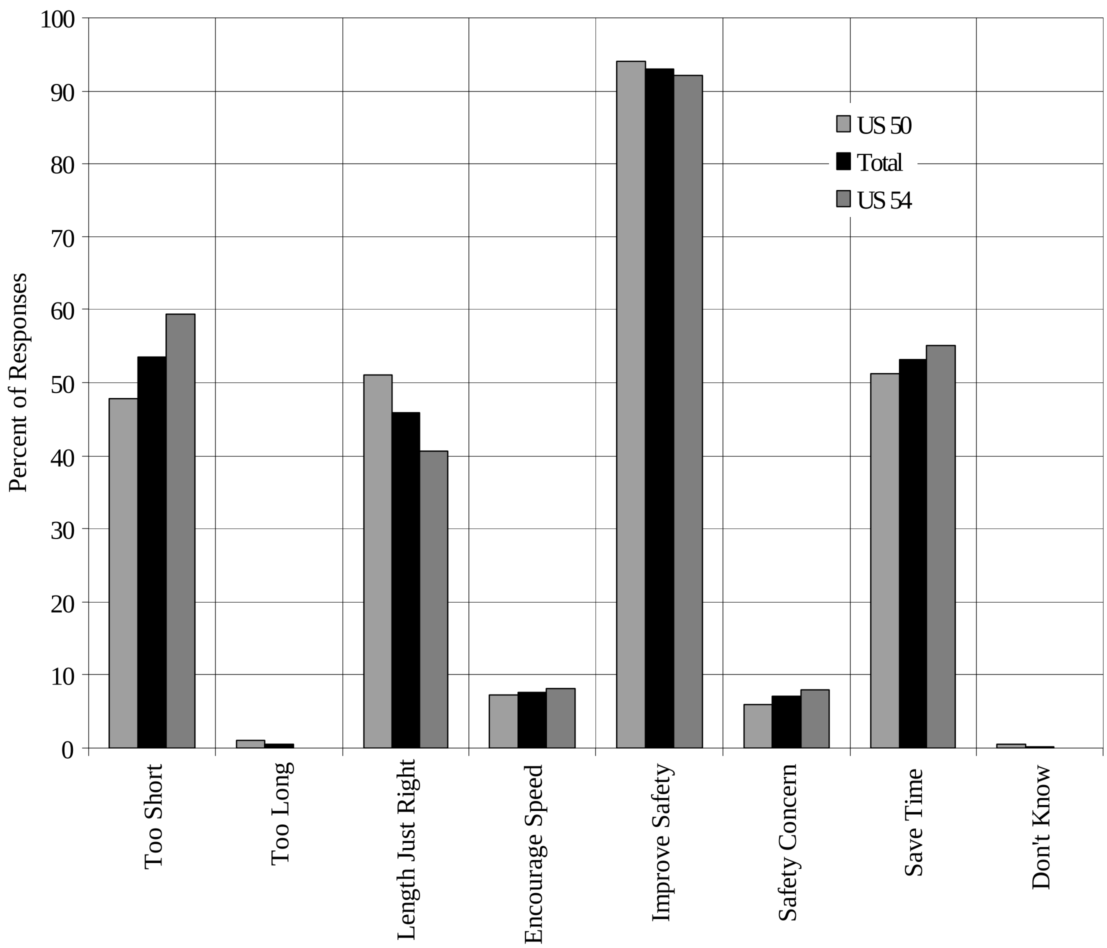


Figure 4.12: Passing Lane Attributes

4.6.5.1 Length

Passing lane lengths on US 50 range from 3,200 to 4,594 feet (975 - 1,400 m) with an average of 4,034 feet (1,230 m), while those on US 54 range from 3,880 to 5,800 feet (1,181 - 1,768 m) with an average of 4,512 feet (1,375 m). Drivers are equally divided regarding the length of passing lanes. While 53 percent think passing lanes are too short, 46 percent think the length is just right. It is surprising to find out that drivers rate passing lanes as too short despite the fact that the length provided is within the suggested optimum length found in the literature review (Harwood and St. John (1985)). The response distributions for US 50 and US 54 are significantly different (p-value of 0.048), with drivers on US 50 being more satisfied with the length than those on US 54. Although passing lanes on US 50 are shorter than those on US 54, a higher proportion of drivers on US 54 think the length is too short compared to the drivers on US 50.

4.6.5.2 Speed

Only eight percent of drivers think that passing lanes encourage speeding, and there was no significant difference between the percentages from each highway (p-value of 0.711).

4.6.5.3 Safety

Safety received the highest rating, with 93 percent of drivers thinking that passing lanes improve safety. There was no significant difference between the proportions of drivers from each highway (p-value of 0.564).

4.6.5.4 Time Saving

About 53 percent of respondents think that passing lanes save time, with no difference between the two highway response distributions (p-value of 0.378). Although engineers provide passing lanes with the primary objective of dispersing platoons and hence saving time, drivers in Kansas consider safety as a greater benefit of passing lanes than the saving of time.

4.6.6 Drivers' Residence

Surveyed drivers were asked whether they live in the state of Kansas or other states. Table 4.22 and Figure 4.13 show the distribution of drivers' state of residence. The proportion of drivers on US 54 from states other than Kansas is significantly higher than that on US 50. This might be due to the fact that US 54 is closer to the state border than is US 50. In Section 4.6.2 it was postulated that there is likely a high correlation between out-of-state drivers and less frequent drivers. Fifty-two percent (34 out of 62) of drivers for both highways who reside in other states travel the sections at a lower frequency (less than once per month). Also, 54 percent (34 out of 63) of drivers for both highways who have lower frequency of travel, reside in states other than Kansas.

Table 4.22: Drivers' State of Residence

Highway	Kansas	Other
	Number(%)	Number(%)
US 50	183(87.98)	25(12.02)
US 54	157(80.10)	39(19.90)
Total	340 (84.16)	64 (15.84)
p-value	0.0302	

NOTE: p-value for testing the significance difference in the response distributions between US 50 and US 54 highways.

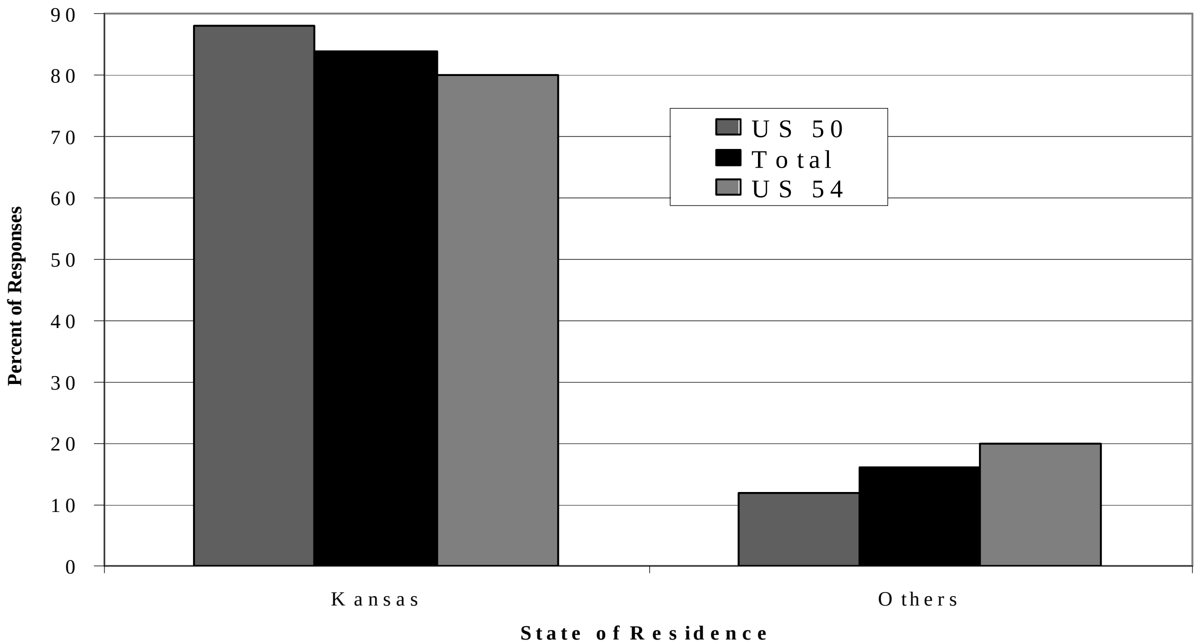


Figure 4.13: Drivers' State of Residence

4.6.7 Drivers' Comments

On US 50, 108 drivers and on US 54, 109 (total of 217 respondents) responded to question six with comments. Many respondents made more than one comment. Only 227 comments providing additional information were analyzed, i.e., the same or similar comments were not repeated.

Eighty-eight comments (or 39 percent) suggested some sort of improvement for some specific locations/sections or for the general road network. Out of these 86 comments, 38 suggested building four-lane highways. While six comments were negative in nature, 82 comments were positive about passing lanes and KDOT's road network. Of interest were the comments suggesting building four-lane highways. Out of 38 comments in this category, half or 19 comments came from US 54 respondents. Eleven comments were specific, suggesting that a four-lane highway is needed from west of Kingman city to the border state.

Eleven comments complained about the “ignorance” of drivers on passing lanes, while 14 comments observed inappropriate lane usage at the passing lane sections. Three comments mentioned the problem of drivers merging back to the main stream at the end of the passing lane, and there was a comment that passing lanes on US 50 were better than those on US 54 in terms of length and signing.

Passing lanes on US 54 have two advance signs, the first at two miles (3.2 km), and the second at ¼ mile (400 meters) before reaching the passing lane. At the beginning of the lane-drop section of the passing lane they are signed **SLOWER TRAFFIC KEEP RIGHT**. Passing lanes on US 50 have only one advance sign, at ¼ mile (400 meters) before reaching the passing lane, and at the beginning of the lane-drop section of the passing lane they are signed **KEEP RIGHT EXCEPT TO PASS**. Both highways have a symbolic lane reduction transition sign near the beginning of the lane-drop taper. The lane reduction transition sign is defined by the Manual on Uniform Traffic Control Devices (MUTCD) as W4-2. All passing lanes are marked with double yellow lines restricting passing in opposite directions.

On signing, drivers complained about the small size of signs (five comments), motorists not properly obeying signs and markings (four comments), recommendations to use **KEEP RIGHT EXCEPT TO PASS** sign on US 54 (six comments), more advance notification signs on US 50 (two comments), the need for a sign showing the distance to the end of passing lane, the need for better and consistent signing, and an opinion about “wrong” signing on US 54 (probably referring to the “slower traffic keep right” sign).

4.7 COMPARISON OF PASSING LANE CONFIGURATIONS

Using percent time delay as the measure of effectiveness, there is no significant difference between side-by-side and head-to-head configurations. Other configurations are significantly different. The best arrangement, in descending order is: side-by-side, head-to-head, tail-to-tail, at ends of the sections, and without passing lane. Table 4.23 shows the results of the simulation on the existing highway section. The ranking of passing lane configurations is similar to those obtained for a hypothetical highway section (not shown here).

Table 4.23: Percent Time Delay for Simulation of Different Passing Lane Configurations for Existing Highway Section

Passing Lane Arrangement and Configuration	**Two Way Hourly Volume**		
	800	**1600**	**2400**
Without Passing Lanes	42.56(5)	61.94(5)	70.5 (5)
At ends only[a]	36.39(4)	52.89(2)	63.44(4)
Tail-to-Tail[b]	34.75(3)	52.92(3)	63.18(3)
Side-Side[c]	34.74(2)	53.04(4)	62.92(2)
Head-to-Head[d]	34.64(1)	52.67(1)	62.92(1)

[a] Two passing lanes one at the beginning of each direction
[b] Two passing lanes at the middle section with a tail-to-tail configuration
[c] Two passing lanes at the middle section with a side-by-side configuration
[d] Two passing lanes at the middle section with a head-to-head configuration
() Ranking

The problem of platooning vehicles when they encounter the opposing passing lane does not appear to create large differences in the configurations. It has been cited in the literature (Morrall and Werner 1990b) that percent time delay is not as sensitive to traffic flow conditions as is the number of passes.

This could contribute to the marginal differences found using percent time delay. The number of passes was not used to differentiate between different arrangements because the number of passes from the TWOPAS model, simulating sections with passing lanes, has been found to overestimate the number of passes observed in the field (Harwood and St. John (1986)). The overall conclusion is that passing lanes reduce percent time delay, however different passing lane configuration seem to differ only marginally.

Chapter 5

WARRANTS DEVELOPMENT

Different fundamental models cited in the literature, or used for the justification of the provision of passing lane(s), were reviewed and compared. These models include: supply-demand models, benefit-cost ratio models, maximum queue models, and HCM level-of-service models. Warrants suitable for Kansas conditions were developed. Also, guidelines for location of a passing lane in relation to side-road intersections are proposed.

It is recommended that the decision to provide passing lanes should be a two-level process. At the first or “higher” level (Network Level), highway segments needing improvements from an operational standpoint; e.g, congestion, are identified for the entire state, two-lane, rural highway network. The analysis tool at this level should employ simple and relatively fewer input data. It is expected that KDOT already has in place data for use in identification of these sections. At the second “lower level” (Project Level), highway sections, or projects selected at the network level, are ranked on the basis of their need for passing lanes. The number of projects to be implemented from the prioritized list will depend on the funding level. At the project level, detailed economic analysis using different passing lane lengths, spacings, and configurations can be evaluated to develop an optimum program.

5.1 RECOMMENDED WARRANTS AT THE NETWORK LEVEL

A highway segment on which the analysis is performed is defined as a highway segment that will have insignificant interruptions due to side roads and/or urban influence. For the purpose of this study, most low-volume roads intersecting Kansas state highways may be considered insignificant. The definition of "highway segment" is chosen to match the properties of the segment that can be simulated using the TWOPAS program later at the project level. Therefore, the boundaries of these highway segments should be major, high volume intersecting roads and or urban boundaries.

Analysis at the network level would be done using the HCM level-of-service model. The passing opportunity model is not recommended because of difficulty in relating Net Passing Opportunities (NPO) threshold values and the quality of traffic flow. The maximum queue model is not recommended because it does not consider opposing volume as a passing constraint. Benefit-cost ratio is not recommended for this level for two reasons: 1) it is not convenient for the first-level selection of segments needing improvements from an operational point of view using simple and easily available input data; and 2) it is more appropriate for project level analysis. The passing ratio model is not recommended because of the weakness of TWOPAS (needed in its application) in predicting the number of passes on a section with a passing lane, and the need of computer simulation for its implementation.

5.1.1 HCM Level-of-Service Warrants

The HCM level of service procedure for two-lane highways offers three types of analysis: 1) operational analysis of general terrain segments; 2) operational analysis of specific grades; and 3) planning analysis. A general terrain segment analysis for level and rolling terrain is recommended for this level. Tables 5.1 and 5.2 indicate threshold AADT values for different terrain types where passing lane systems would be recommended. Tables 5.1 and 5.2 were constructed using the HCM procedure. These tables do not include recreational vehicles and busses, and thus conform to the way KDOT summarizes its traffic data into two categories; i.e., total vehicles and percentage of commercial vehicles. Also, the proportion of these vehicles is low (in the order of five percent maximum). The analysis segment should be longer than two miles for HCM procedures to be valid. Values corresponding to LOS “B” could be used for principal arterial highways, while those values for LOS “C” could be used for minor arterial highways.

5.2 WARRANTS AT PROJECT LEVEL

Once the highway segments in need of improvement are identified at the network level, detailed analysis can be performed to prioritize and select individual projects. The purpose of the detailed analysis is to rank the selected sections based on some desired criterion. If possible, use traffic computer simulation, such as TWOPAS program, to rank projects based on expected costs and expected benefits; i.e., a benefit-cost analysis. TWOPAS can also be used to choose passing lane locations to minimize percent time delay.The TWOPAS computer program is in public domain and can be obtained from Federal Highway Administration (FHWA) by contacting them at the following contact information:

Highway Research Engineer
FHWA T303
Tuner-Fairbank Hwy. Research Ctr.
6300 Georgetown Pike
McLean, VA 221-1-2296
Telephone Number: (202) 493-3318.

If TWOPAS is needed at the project level analysis, the input at this stage would be data needed to run TWOPAS, value for time savings, accident costs, and predicted accident reduction factors due to the provision of passing lanes. The use of TWOPAS may be used to determine the optimum number of passing lanes within the highway segment and their associated configuration.

Table 5.1: Suggested Minimum AADT for Rural Two-Lane Highways for LOS B and C in Level Terrain that would Warrant Passing Lane(s)

		Projected Design Year ADT									
% Trucks		**10**		**15**		**20**		**30**		**40**	
LOS		**B**	**C**	**B**	**C**	**B**	**C**	**B**	**C**	**B**	**C**
% "no-passing" zones	**0%**	3900	6200	3700	5890	3520	5600	3210	5110	2950	4690
	20%	3460	5630	3290	5340	3130	5080	2850	4630	2620	4260
	40%	3030	5190	2880	4930	2740	4690	2500	4280	2290	3930
	60%	2740	4900	2600	4660	2480	4430	2260	4040	2080	3710
	80%	2450	4760	2330	4520	2220	4300	2020	3920	1860	3600
	100%	2310	4620	2190	4380	2090	4180	1900	3800	1750	3490

Assumptions: K=0.15, directional split = 60/40, PHF=0.92, Lane width   12 ft, shoulder width   6ft

Table 5.2: Suggested Minimum AADT for Rural Two-Lane Highways for LOS B and C in Rolling Terrain that would Warrant Passing Lane(s)

		Projected Design Year ADT									
% Trucks		**10**		**15**		**20**		**30**		**40**	
LOS		**B**	**C**	**B**	**C**	**B**	**C**	**B**	**C**	**B**	**C**
% "no-passing" zones	**0%**	3000	4850	2630	4240	2340	3770	1910	3090	1620	2610
	20%	2660	4500	2320	3940	2070	3500	1690	2870	1430	2430
	40%	2190	4040	1920	3540	1710	3140	1400	2570	1180	2180
	60%	1960	3690	1720	3230	1530	2870	1250	2350	1060	1990
	80%	1730	3460	1520	3030	1350	2690	1100	2210	940	1670
	100%	1500	3230	1320	2830	1170	2520	960	2060	810	1740

Assumptions: K=0.15, directional split = 60/40, PHF=0.92, Lane width 12 ft, shoulder width 6ft

5.3 PASSING LANE LOCATION RELATIVE TO INTERSECTING ROADS

This section develops guidelines for locating passing lanes with relation to the locations of side road intersections from the safety point of view. During a passing maneuver on a two-lane highway, either on a conventional, standard section or within a passing lane section, motorists are engaged in diverging, merging and passing activities. The level of these activities is considered higher on the passing lane section than on a standard section due to relatively more passes executed in the passing lane section. Because of this, the literature suggests avoiding side road intersections within a passing lane section, in particular within the lane-addition and lane-drop sections where diverging and merging activities take place. Others suggest avoiding high-volume, side-road intersections, but the decision of what constitutes "high volume" is left to the judgment of the highway design engineer. All these guidelines are geared to ensure that the safety of the passing lane section provided is not degraded. Indeed, avoiding side road and driveway intersections as much possible was one of KDOT's criteria for locating the existing passing lanes. However, because there are some intersections immediately after the passing lane, they are very close to the terminal of the passing lane where the effect of merging is still present.

One of the objectives of this study was to develop guidelines on the location of passing lanes with relation to side road and driveway intersections. This is important because with the high density of cross roads and driveway intersections in Kansas, it would be difficult to avoid intersections in any passing lane section.

5.3.1 Passing Lane Guidelines in Relation to Intersections

When choosing the location of a passing lane, the following guidelines in relation to intersections are suggested:

- Side road or driveway intersections should be avoided within and immediately after the passing lane section if possible, especially the high-volume intersections. (A suggested definition of high-volume side road intersection is presented in the following section);
- Where a low-volume side road or driveway intersection cannot be avoided within the passing lane, it should be located close to the middle of the passing lane rather than in merging, diverging, or immediately after the passing lane areas.

5.3.2 Suggested Definition of Low-Volume Side Road/Driveway Intersection

Side road/driveway intersections along the main highway are considered as Two-Way-Stop-Controlled (TWSC). T-intersections with a stop sign on a perpendicular approach are also considered as TWSC intersections because the operation is similar to that on complete TWSC, except that the number of turning movements are reduced by half. Even where stop signs are not physically posted, as long as traffic on the main highway always has the right-of-way, the intersection will operate as a TWSC intersection.

Vehicles turning left from the main highway into the side road will directly impede the following "through" vehicles within the same lane and, depending on traffic level, may cause the impedance to spill over to the adjacent lane in the same direction if there is one. Interaction between the left-turning and through vehicles within the passing lane requires a prior knowledge of the distribution of traffic between the two available lanes in the same direction. The HCM asserts that (TRB 1994 p 2-20):

> "when two or more lanes are available for traffic in a single direction, lane distribution varies widely depending on traffic regulations, traffic composition, speed and volume, the number and location of access points, the origin-destination patterns of drivers, development environment, and local driver habits."

Researchers who have attempted to assign the lane distribution for multi-lane highways, have used locally observed parameters. When Tanweer and Stokes (1996) developed guidelines for right-turn lanes at unsignalized intersections and driveways in Kansas, they used an average distribution of 32/68 for inner/outside lane respectively for four-lane, two-way highways. These values were observed from the field.

It would have been reasonable to use the same lane distribution as used in the Tanweer and Stokes study, since both studies were conducted in the same geographical area. However, it can be argued that the lane distribution at the passing lane is likely to differ from that of a four-lane highway because motorists on a passing lane section do not perceive a passing lane the same way as a four-lane highway. With the passing lane advance sign, or familiarity with highway, drivers know that the double lanes section provided at the passing lane is a temporary one, and therefore they may react differently than they would on a continuous four-lane facility. Traffic distribution by lanes at the passing lane section has been found to vary with traffic volume (Emoto and May 1988), passing lane entry design (Staba et al.1991; Fong and Rooney 1990), and the distance from the beginning of the passing lane location along the passing lane, as supported by the Morrall and Blight (1984) study, and "field data" from this study. These variations make it difficult to come up with a single value representing traffic distribution by lane within a passing lane, hence difficult to analyze the interaction of the turning and through vehicles at an intersection located within a passing lane.

Because of the difficulty of assigning a single value for lane distribution within a passing lane section, as explained above, the approach used to assess the safety of a side road intersection within a passing lane was to carry out the assessment as if the side road intersects a standard section on the main highway; i.e, the presence of the passing lane is ignored.

A side road was considered high-volume if a left-turn lane on the main highway is warranted; i.e., where left-turn vehicles from the main highway required a separate lane for efficient and safety. If such intersection is within the passing lane, the inner lane of the passing lane will essentially act as *defacto* left-turn lane.

5.3.2.1 Warrants for a Left-Turn Lane

Probability Based Criterion

KDOT uses the AASHTO green book guidelines to determine the need of a left-turn lane at an unsignalized intersection on two-lane highways. The guidelines were developed by Harmerlink (1967) and use hourly volume for both approaches of the main highway, the percentage of left-turn in the approach, and the opposing hourly volume. The left-turn lane is warranted at the volume where a certain probability is exceeded. This is the probability of one or more through vehicles in the approach being caught in the queue formed by a left-turning vehicle waiting for an appropriate gap in the opposing volume. Threshold probabilities were set by a panel of engineers and depend on the speed on the main highway. The probability model used to develop these warrants evaluates left-turn conditions from a single approach of the main highway, with the following assumptions:

- right-turns from the approach under consideration are considered as through vehicles, and
- right- and left-turns on the opposite approach are considered as through vehicles.

Kikuchi and Chakroborty (1991) found and corrected two errors in Harmelink's modeling, resulting in new volume warrants presented in Table 5.3.

Level-of-Service Criterion

Level-of-service criterion as described in HCM (TRB 1994) is defined by the average total delay per vehicle. HCM recommends using these procedures for analysis of rural intersections despite the fact that some parameters used in these procedures; e.g., critical gap, were obtained from urban

Table 5.3: Volume Combinations Justifying a Left-Turn Lane on the Basis of the Modified Harmelink's Model

Opposing Volume	Advancing Volume[a]			
	5% Left Turns	**10% Left Turns**	**20% Left Turns**	**30% Left Turns**
	40-mph Operating Speed			
800	434	300	219	189
600	542	375	272	234
400	682	472	343	293
200	863	600	435	375
100	946	679	493	424
	50-mph Operating Speed			
800	366	257	185	162
600	460	320	234	202
400	577	403	294	255
200	735	513	373	324
100	830	576	424	365
	60-mph Operating Speed			
800	294	207	154	146
600	365	259	187	165
400	461	324	238	206
200	586	414	303	263
100	663	468	344	297

[a] Total approach volume/LOS for LT/probability of no queue

[b] probability that one or more through vehicles are present in queues formed by left-turning vehicles waiting for the crossing gap.

Source: Kikuchi and Chakroborty (1991)

intersections with speed limits of not greater than 30 mph. Hamed et al. (1997) concluded that among the factors that affect a driver's critical gap at a TWSC is the speed on the main highway. Also, the threshold values of delay for different LOS may not be true for rural intersections where motorists with relatively longer journey times might tolerate higher delays than their counterparts in an urban environment.

HCM procedures are very powerful and can be used to determine operating conditions for a single movement (capacity and delay), but at the "cost" of requiring detailed input data. For example, for a four-leg TWSC intersection the following information is needed:

- traffic volume for all 12 movements,
- grades for all four approaches,
- number of through lanes on major approaches, and
- the presence of left-turn lanes on major approaches.

Table 5.4 shows the volume warrants of Table 5.3 along with the operating LOS obtained using the HCM procedure. From the LOS point of view, the main highway approaches seem to operate at a satisfactory level of service; i.e., B or A.

Capacity Criterion

From the capacity point of view, an intersection would be considered to operate satisfactorily if all the vehicles arriving at an intersection within a given period of time are discharged through the intersection within specified limits of delay. This is to say that vehicles turning into and from the side road gets a gap greater than or equal to the critical gap. Consider the volume, V_s, on a single approach of a side road, and one-way flow (adjacent to V_s) on a two-lane main highway, V_m. All the turning movements which make up the side road volume (except three movements) will require gaps from the main highway flow, V_m. The three exceptions are: (1) right-turn from the main highway, which needs no gap in V_m; (2) left-turn from the side road which needs gap in V_m and opposing flow of V_m (i.e., two-way flow of main highway); and (3) through movement from and to the side road which will need gaps in the two-way flow of the main highway.

Table 5.4: Volume Combinations Justifying a Left-turn Lane and Associated Operating Characteristic

Opposing Volume	Advancing Traffic Operating Characteristic[a]/LOS[b]			
	5% Left Turns	10% Left Turns	20% Left Turns	30% Left Turns
	40-mph Operating Speed			
800	434/B	300/B	219/B	189/B
600	542/A	375/A	272/A	234/A
400	682/A	472/A	343/A	293/A
200	863/A	600/A	435/A	375/A
100	946/A	679/A	493/A	424/A
	50-mph Operating Speed			
800	366/B	257/B	185/B	162/B
600	460/A	320/A	234/A	202/A
400	577/A	403/A	294/A	255/A
200	735/A	513/A	373/A	324/A
100	830/A	576/A	424/A	365/A
	60-mph Operating Speed			
800	294/B	207/B	154/B	146/B
600	365/A	259/A	187/A	165/A
400	461/A	324/A	238/A	206/A
200	586/A	414/A	303/A	263/A
100	663/A	468/A	344/A	297/A

[a] Total approach volume/LOS for LT.
[b] Letters indicate LOS.

Since they don't restrict any other turning movement, right-turns from the main highway have two effects: 1) reducing the number of gaps needed by side road traffic, and 2) increasing the number of gaps available in main-line flow V_m. The left-turns and through turns from the side road have the effect of reducing the number of usable gaps from the main-line V_m, since some of the gaps available in main-line flow might not be used because of the lack of similar gaps in the flow opposite the main-line. It can then be assumed that the demand for more gaps by left-turning and through vehicles is compensated for by right-turning vehicles. Therefore, one could compare the number of gaps in the one-way flow of the main highway (supply), with the single approach side road flow (demand), to assess the ability of the intersection to handle side road volume. Figure 5.1 shows the number of headways between vehicles that are equal to or greater than 6.5 seconds that would be available in the traffic stream with volume up to 600 vph. An example follows:

Assume V_m is 300 vph, the number of gaps greater or equal to 6.5 seconds[6] and hence the side road flow that can be served is equal to:

$$V_s \quad V_m e^{\frac{V_m \times t}{3600}} \quad 300 \times e^{\frac{300 \times 6.5}{3600}} \quad 175$$

Where:

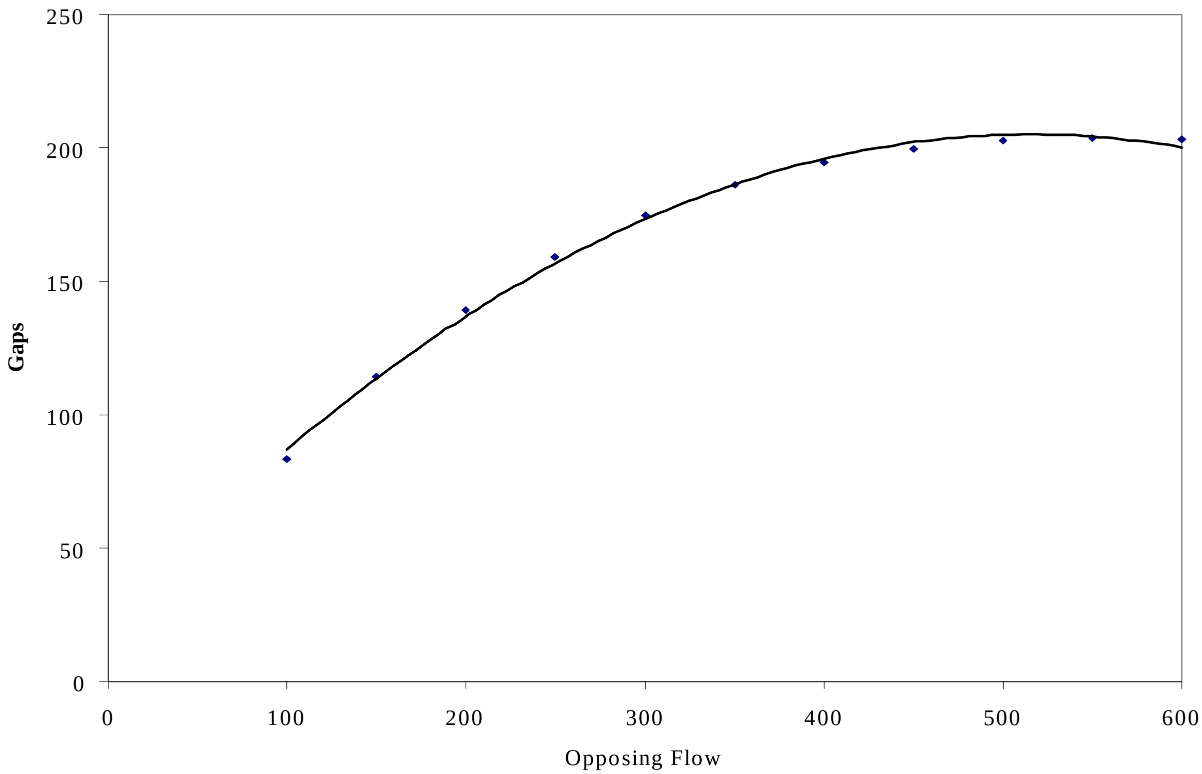


Figure 5.1: Number of Gaps Greater than 6.5 seconds

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[6] The critical gaps for different movements at urban unsignalized intersection ranges between 5 to 6.5 seconds (TRB 1994).

V_s = hourly side road volume that can be accommodated,

V_m = hourly main-line volume,

t = critical gap, and

e = natural logarithm.

Such a side road is considered a high volume. None of the existing intersections experience a capacity problem based on the capacity criterion.

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based on the results of this study.

6.1 CONCLUSIONS

The following conclusions can be drawn from the analysis of the videotape data of passing lane operations:

- Passenger cars are less likely to move to the right lane than are medium and heavy vehicles.
- With the current pavement markings for passing lane-addition at both highways US 50 and US 54, most vehicles stay in the inner (passing) lane rather than being channeled to the outer (shoulder) lane.
- Overall, the percentage of vehicles using the right lane, and the keep right compliance rate, is higher at passing lanes on US 54 than on US 50, despite the fact that US 50 uses the **KEEP RIGHT EXCEPT TO PASS** sign while US 54 uses the **SLOWER TRAFFIC KEEP RIGHT** sign.
- Based on the results of this study, and other studies reviewed in the literature, it seems that pavement markings which channel vehicles to the right, are more effective than signing for moving vehicles to the outer lane.

- At passing lanes on level or rolling terrain, passenger cars perform more passes than was expected, while non-passenger cars perform fewer passes than was expected.
- On the average, the proportion of illegal passes executed in the opposing lane was insignificant (less than 0.5 percent).
- The more motorists comply with the “keep right” sign, the better the passing lane functions.
- All sites studied had similar relationships between the number of passes and traffic volume, and between pass rates and traffic volume.
- The proportion of vehicles traveling in platoons at all sites was within a narrow range (32 to 40 percent).

Based on the data obtained in the traffic count and speed studies the following conclusions were drawn:

- The percentage of vehicles at the downstream location of the passing lane with speeds higher than the mode speed was found to be higher than the corresponding percentage at the upstream location.
- By using five-seconds as a minimum time headway in defining a vehicle to be traveling in a platoon, the time headway data (obtained from traffic counters) were not sufficient to detect any difference in the proportion of vehicles traveling in platoons between the upstream and downstream ends of a passing lane.
- The percentages of vehicles with time headway less than two seconds decreased from the upstream to downstream location of the passing lanes. This indicates the ability of

passing lanes to break up groups of vehicles traveling together.

- The speed data collection method used resulted in speed data which were not suitable for statistical testing of significance between speed at the upstream and downstream ends of a passing lane.

From the traffic conflict studies at lane-drops, it is concluded that no significant differences in conflicts exist for the range of taper lengths used by KDOT. However, it should be noted that sample sizes were relatively small and only large differences would have been detected.

Three major conclusions from the intersection traffic conflict study are:

- Through and left-turn traffic from the side road do not appear to create a potential accident risk to the major-highway through traffic. Left-turn traffic from the major highway appears to create the highest potential accident risk.
- Intersections located within passing lanes do not necessarily present a potential risk to major highway traffic. In fact, the data suggest that intersections located within passing lane sections have significantly fewer traffic conflicts than those located outside the passing lane section.
- The comparison of conflicts between intersections located immediately after the passing lanes and those located some distance from the passing lane was inconclusive. The data did not suggest any significant difference in conflicts. However, this could have been due to the small sample size which would find only large differences to be significant.

From the analysis of the accident data, two conclusion were drawn:

- The two-lane highway sections with passing lanes had fewer accidents than the state average for two-lane highways of similar type.
- The accident data were insufficient to detect any significant difference in any accidents before and after the construction of passing lanes.

The following conclusions were drawn from the opinion survey:

- The survey results indicate that the public supports KDOT's passing lane program.
- Respondents indicated that many slower drivers fail to move to the right. Use of **KEEP RIGHT EXCEPT TO PASS** sign and better, positive channelization could reduce this problem.
- Many respondents at the US 54 site were out-of-state or infrequent drivers. This emphasizes the importance of consistent highway design and operating practice to clearly sign passing lanes and to enhance safety and operational efficiency.
- Although the lengths of passing lanes are within suggested optimum lengths, respondents were equally divided on whether the lengths are too short or just right;
- Drivers view safety as the main benefit of passing lanes.

From the traffic simulation study it was concluded that:

- 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, they ranked better than other configurations. The difference in percent time delay among different configurations as predicted by the TWOPAS model appears to differ only marginally.

6.2 RECOMMENDATIONS

The recommendations presented here are based on the results from this study, findings from the literature survey and engineering judgement.

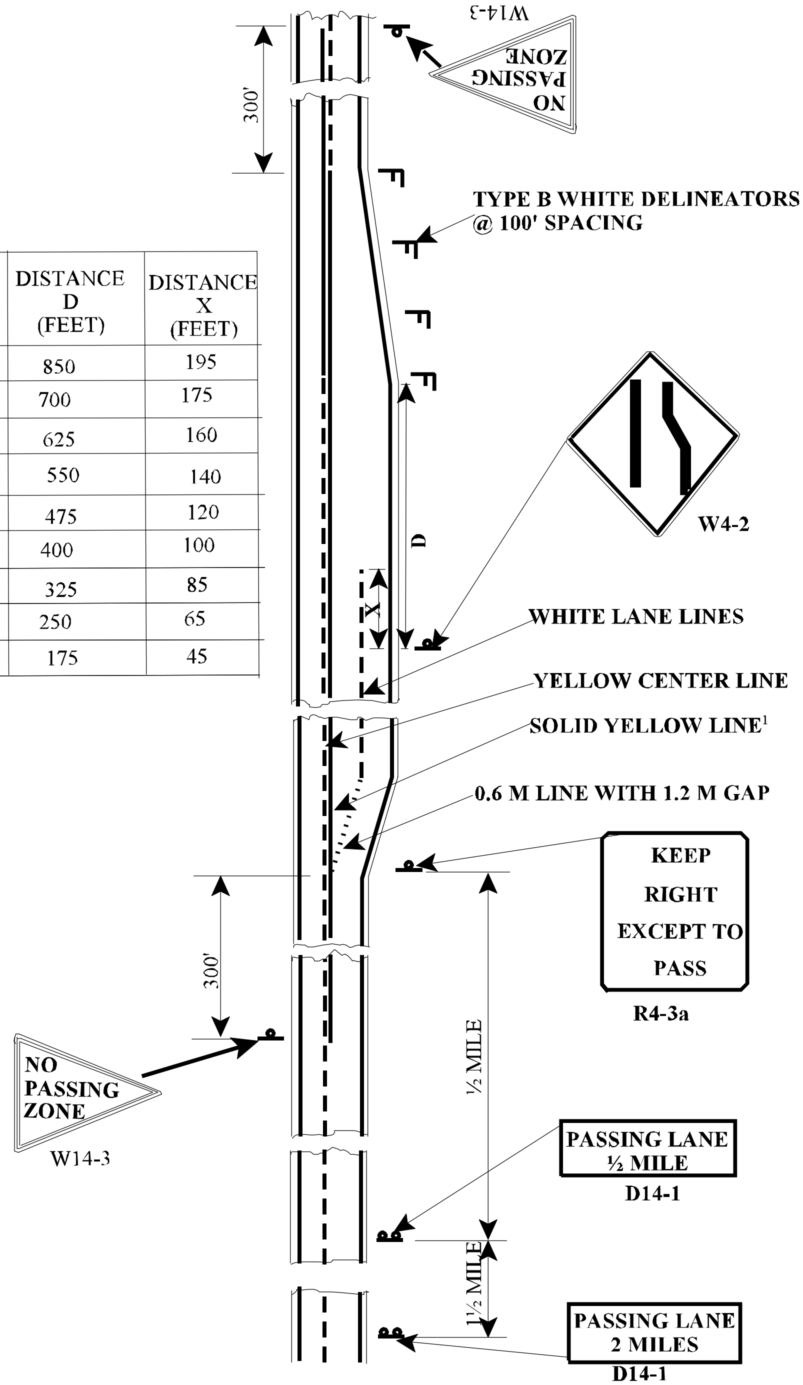
6.2.1 Guidelines for Identifying Passing Lane Sites

Determining candidate locations for a passing lane should accomplished by a two-level process. At the Network Level, two-lane, rural highway segments that are planned for operational improvements are identified. At the Project Level, those segments identified at the network level are ranked by assessing the benefits and costs of providing passing lanes in a particular segment. The number of passing lane projects to be implemented will depend on the funding level. Because there is relatively more diversity at the network level, a few, easily available parameters should be sufficient for selecting segments. The HCM level-of-service warrants shown in Tables 6.1 and 6.2 are recommended for identifying candidate passing lane sites at the network level.

At the project level, a detailed economic analysis of different passing lane length, spacing, and configuration can be undertaken to obtain optimum passing lane parameters. Computer simulation using TWOPAS is recommended at the project level.

6.2.2 Signing

Refer to Figure 6.1 for a typical signing and marking for a passing lane. Of the two types of signing used by KDOT at the beginning of a passing lane, i.e., **KEEP RIGHT EXCEPT TO PASS** and **SLOWER TRAFFIC KEEP RIGHT**, the former is recommended. This recommendation is the result of the users'



POSTED SPEED (MPH)	DISTANCE D (FEET)	DISTANCE X (FEET)
65	850	195
55	700	175
50	625	160
45	550	140
40	475	120
35	400	100
30	325	85
25	250	65
20	175	45

[1]*If side-by-side configuration, center line is marked by double yellow lines*

Figure 6.1: Typical Signing and Marking for Passing Lanes

survey and engineering judgement. Standardization is also desirable and only one type of the sign should be used.

It is recommended that there be at least two advance signs on the approach to the passing lanes. The two signs, one at two miles (3.2 km) and one at ½ mile (0.8 km), as recommended by KDOT, should be regarded as a minimum requirement.

6.2.3 Pavement Markings

There are two recommendations regarding pavement marking (Refer to Figure 6.1): 1) the lane-addition should be marked so that all traffic is channeled to the right lane; 2) the whole passinglane should be marked by double yellow lines to prohibit passing in the opposing lanes when one-way hourly volume is greater than 400[7] or when there are sight distance restrictions.

6.2.4 Location of Passing Lanes

It is recommended that crossroad intersections be avoided within a passing lane section if possible, especially along high traffic volume segments. 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. High volume side roads should be avoided. High volume side roads are defined as those crossroads where left-turn volume from the main highway would warrant a separate left-turn lane on a conventional two-lane section. Side roadintersections within lane-drops and lane-additions should be avoided. Further, right turn lanes are recommended at high volume

[7] Based on the observational study by Harwood and St. John (1985).

crossroads. These turn lanes act as a bypass lane for through traffic in the event that the through lane is occupied by left turning vehicles waiting for a suitable gap in the opposing traffic.

It is recommended that passing lanes be constructed leading away from rather than into areas of traffic congestion, and into points where significant traffic may end their trips or leave the highway system. Traffic congestion areas includes sections with significant no-passing zones, in/or adjacent to communities where the speed limit of the highway is reduced, etc. Points where significant traffic may end their trips or leave the highway system includes urban areas, major intersections, recreation areas, etc. Leading into areas of congestion will thwart the benefits (which normally extends some distance downstream) gained at the passing lane.

6.2.5 Implementation Plan

The results of this study should be reviewed for possible incorporation into the appropriate KDOT design manual(s). (See the Executive Summary for proposed design guidelines). At aminimum, it is suggested that the department's passing lane planning and design criteria and policies address the following issues.

1. Make all future passing lanes consistent in regard to signing and pavement markings. Specific recommendations include:

    a) channelize traffic to the outer lane;

    b) advance signs should be placed at two miles and ½ mile before the passing lane;

    c) a symbolic merge sign should be located at the end taper; and

    d) use double yellow lines between side-by-side passing lanes.

2. Implement written policy regarding a) the need for additional passing lanes on the two-lane highway network based on the traffic volumes shown in Tables 5.1 and 5.2 of Chapter five, and b) the use of benefit/cost analysis to rank candidate passing lanes sites.

3. Implement developed written design guidelines regarding a) minimum length of a passing lane, b) minimum lengths of tapers, c) avoiding major intersections where the left-turn movement from the major road is such that a left-turn lane would be warranted, d) locating minor intersections near the middle of the passing lane, and e) providing right-turn lanes at crossroad intersections.

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APPENDIX

SURVEY CARD SAMPLE

PASSING LANE STUDY
KANSAS DEPARTMENT OF TRANSPORTATION

Dear Motorist:

The Kansas Department of Transportation (KDOT) needs your help in a special study of PASSING LANES on Kansas highways. The goal of the study is to assess the operations, safety and public opinion of passing lanes in Kansas. A PASSING LANE is an added lane in one or both directions of travel on a two-lane highway to improve passing opportunities.

To identify possible improvements in the design of passing lanes and to determine whether more passing lanes should be constructed, we need to know how individual drivers feel about these lanes. Your answers to the attached questions will provide valuable information on the use and operation of these special lanes.

NO POSTAGE
NECESSARY
IF MAILED
IN THE
UNITED STATES

BUSINESS REPLY MAIL
FIRS CLASS MAIL PERMIT NO. 18 MANHATTAN, KS 66502

POSTAGE WILL BE PAID BY ADDRESSEE

Departement of Civil Engineering
119 Seaton Hall
Kansas State University
1700 Anderson Avenue
Manhattan, Kansas 66502 - 9908

PASSING LANE STUDY
KANSAS DEPARTMENT OF TRANSPORTATION

Your comments concerning the operation, safety and design of passing lanes are important to us. Detach and mail the completed portion of the pre-addressed questionnaire at your earliest convenience.

Information you provide will be kept confidential. Only a summary of the results will be available for review.

In appreciation for filling out and returning the attached postage paid postcard, we would like to send you a free State of Kansas highway map. To receive your map, complete all the survey questions and provide your mailing address at the bottom of the survey form.
Your cooperation is appreciated.

PLEASE ANSWER ALL QUESTIONS AND DROP IN MAIL
NO STAMP REQUIRED

1. How often do you travel this section of highway? ☐ daily ☐ once per week ☐ once per month ☐ other(please specify)
2. Type of vehicle? ☐ passenger car ☐ pickup ☐ van ☐ semi ☐ other vehicle (please specify)__________________
3. Which of the following do you think is true of passing lanes in Kansas (CHECK ALL THAT APPLY)? ☐ too short ☐ too long ☐ length is just right ☐ encourage speeding ☐ improve safety ☐ cause safety concern ☐ save time ☐ I don't know what a passing lane is
4. Should KDOT build more passing lanes? ☐ yes ☐ no ☐ not sure
5. In what state do you live? ☐ Kansas ☐ Other (Specify) _________
6. Comments?____

☐ *Yes send me a highway map.* Name____

____

Address ____