PB86214293

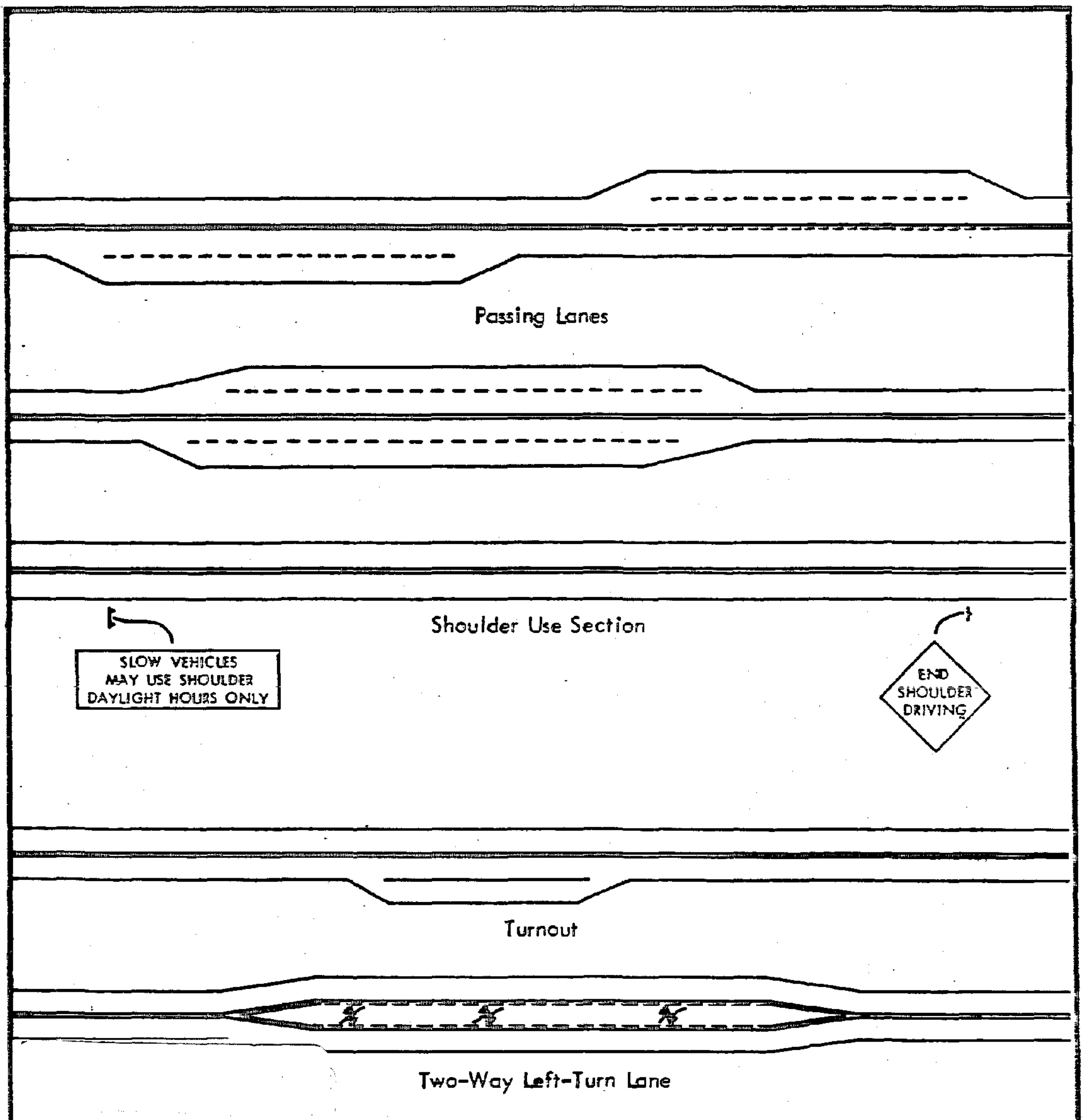
PASSING LANES AND OTHER OPERATIONAL IMPROVEMENTS ON TWO-LANE HIGHWAYS

U.S. Department of Transportation
Federal Highway Administration

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101

Report No.
FHWA/RD-85/028

Technical Report
December 1985



FOREWORD

This report will be of interest to engineers and officials concerned with the planning, design, and operation of rural highways. The five design alternatives examined in the report can be effective in improving the operational and safety performance of two-lane rural highways at relatively low cost compared to conventional four-laning.

Because of the potential economic payoff, further work is being conducted through computer simulation to determine the optimum length, spacing, and cost effectiveness of passing lanes for various terrain and traffic conditions. Following this work, an informational guide will be prepared which could assist the States in the selection, location, and design of cost-effective solutions to operational problems on two-lane rural roads.

The FHWA wishes to express its gratitude to the highway agencies of 13 States that participated in the study. These States are: Arkansas, California, Kansas, Kentucky, Michigan, Mississippi, Nevada, New York, Oklahoma, Oregon, Pennsylvania, Utah, and Washington. Representatives of these State agencies assisted in the study by documenting their current policies and practices, selecting study sites, assembling geometric traffic volume and accident data for the sites, and arranging authorization for MRI to conduct field studies at the sites. This report was critiqued at a workshop attended by representatives of the participating States.

One copy of the report is being sent to each regional office and five copies to each division office. Four of the division office copies should be forwarded to the State highway agency.

Stanley R. Byington

Stanley R. Byington, Director
Office of Safety & Traffic Operations R&D

NOTICE

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/RD-85/028		

4. Title and Subtitle	5. Report Date
PASSING LANES AND OTHER OPERATIONAL IMPROVEMENTS ON TWO-LANE HIGHWAYS	December 1985
	6. Performing Organization Code: 7533-S
7. Author(s): D. W. Harwood and A. D. St. John	8. Performing Organization Report No.
9. Performing Organization Name and Address: Midwest Research Institute, 425 Volker Boulevard, Kansas City, Missouri 64110	10. Work Unit No. (TRAIS): 31M2-682
	11. Contract or Grant No.: DTFH61-82-C-00070
12. Sponsoring Agency Name and Address: Office of Safety and Traffic Operations Research and Development, Federal Highway Administration, 400 Seventh Street, S.W., Washington, D.C. 20590	13. Type of Report and Period Covered: Phase I - Technical Report, October 1982-February 1984
	14. Sponsoring Agency Code: T-0629

15. Supplementary Notes

FHWA Contract Manager: D. L. Warren (HSR-30)

16. Abstract

Many two-lane highways in rural areas experience operational problems related to high traffic volumes, lack of adequate passing opportunities; slow-moving vehicles on grades and in rolling terrain; roadside development that generates turning maneuvers at driveways; and, turning movements at intersections. This report evaluates five operational aids or treatments intended to alleviate these problems including: passing lanes, short four-lane sections, shoulder use sections, turnouts, and two-way left-turn lanes. The report reviews current signing, marking, and design practices used for these operational treatments in 13 States that participated in the study. An operational evaluation of the treatments was based on field data collected at 35 treated sites. Safety evaluation of these treatments was based on accident data collected at 138 treated sites. The study concluded that each of the five types of operational treatments that were evaluated can be effective in improving traffic operations on two-lane highways. Passing lanes, short four-lane sections, and two-way left-turn lanes were also found to improve safety while turnouts and shoulder use sections were found to have no effect on safety.

17. Key Words	18. Distribution Statement
Two-Lane Highway Operations and Safety Passing Lanes Short Four-Lane Sections Shoulder Driving Turnouts Two-Way Left-Turn Lanes	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	138	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

List of Figures

TABLE OF CONTENTS (Continued)

List of Figures (Concluded)

TABLE OF CONTENTS (Continued)

List of Tables

TABLE OF CONTENTS (Continued)

List of Tables (Continued)

TABLE OF CONTENTS (Concluded)

List of Tables (Concluded)

1. INTRODUCTION

Rural two-lane highways constitute the vast majority of the American highway system. While most of the two-lane highway system has relatively low traffic volumes and few major operational problems, there is a significant mileage of two-lane highways that experience traffic operational problems on a regular basis. For example, rural roads serving recreational areas may experience operational problems on a regular basis during peak recreational travel months. Common operational problems on two-lane highways include: overdemand (high volume/capacity ratio); lack of adequate passing opportunities; slow-moving vehicles on grades and in rolling terrain; roadside development that leads to frequent turning maneuvers at driveways; and turning movements at intersections. The common result of these problems is that drivers travel more slowly than they desire, often in platoons of vehicles. Additional undesirable effects may be increased fuel consumption and increased accident potential. Furthermore, such problems can only be expected to increase as traffic volumes grow in the future.

Platoons of vehicles tend to develop on two-lane highways when faster vehicles overtake slow-moving vehicles whose drivers either cannot or do not wish to travel as fast as the overtaking vehicle. The types of vehicles that often become platoon leaders include trucks, buses, and recreational vehicles (RVs), all of which have limited ability to maintain speed on grades. Vehicle speeds may also be limited by narrow lanes, narrow shoulders, narrow bridges, and sharp horizontal curves.

Drivers that are forced to travel more slowly than they desire in a platoon, naturally seek to pass the vehicles ahead of them. However, passing opportunities on two-lane highways are often limited by horizontal and vertical sight distance restrictions and by opposing traffic. As drivers remain trapped behind slow-moving vehicles, they can become frustrated and impatient and may be tempted to pass under limited sight distance conditions or with an inadequate clearance margin to an opposing vehicle. One study of passing behavior found that when forced to follow a slow-moving vehicle for distances of up to 5 miles, almost 25% of the drivers made an illegal pass in a no-passing zone.[9]

Highway agencies have always employed operational improvements or treatments to alleviate such problems at specific sites. However, many highway agencies in the United States have regarded the construction of a four-lane facility as the ultimate and most desirable response to operational problems on two-lane highways. In recent years, as funding constraints have been more severe, there has been increasing interest in operational treatments that can be implemented on two-lane highways without the expense of major reconstruction. Operational treatments intended to increase passing opportunities on two-lane highways include truck climbing lanes on steep grades; passing lanes in rolling or level terrain; short four-lane sections; shoulder use by slow-moving vehicles; and turnouts.

Turning movements at intersections and driveways also create operational problems on two-lane highways. Vehicles slowing or stopping to make a turn, create a potential for rear-end collisions with following vehicles. A vehicle stopping to make a left turn may substantially delay following vehicles and prolong their exposure to the risk of rear-end collisions. Operational treatments that can be used to reduce the delays and accident risk associated with turning maneuvers include intersection turn lanes, paved shoulders (to bypass turning vehicles), and two-way left-turn lanes. In particular, two-way left-turn lanes, which are often thought to be an operational treatment most suited to urban and suburban arterial streets, are finding increasing application at isolated developments in rural areas and in small towns.

This report presents a gross or macroscopic assessment of the traffic operational and safety performance selected operational improvements for two-lane highways based on a combination of field evaluation and accident studies. Further research is planned to evaluate the operational treatments using microscopic computer simulation models.

1.1. Operational Improvements

The research presented in this report addresses operational treatments already in use by highway agencies that offer the potential for improving the level of service on two-lane highways in rural areas and urban fringe areas at relatively low cost. Figure 1 illustrates a typical installation of each of the treatments evaluated in this report.

A passing lane is an added third lane in one direction of a normally two-lane highway to provide opportunities to pass slow-moving vehicles. As illustrated in Figure 1, passing lane sections may be operated either with passing prohibited or permitted for vehicles traveling in the opposing direction. A truck climbing lane on a steep grade is one form of passing lane, but passing lanes are also used in level or rolling terrain. Three-lane highways with an unrestricted center lane, where vehicles traveling in either direction could pass, were common in the United States prior to 1960, but have largely been discontinued for safety reasons. At all of the passing lane sites considered in this study, priority for use of the center lane is assigned to one direction or the other by a yellow center line marking, although some states permit opposing directions vehicles to pass where sight distance is adequate.

A short four-lane section is a section of four-lane highway, generally less than 3 miles (4.8 km) in length, and bounded by two-lane sections at both ends. Short four-lane sections are also intended to increase the passing opportunities on two-lane highways. A short four-lane section is operationally equivalent to two passing lanes located opposite one another. Four-lane sections longer than 3 miles in length operate more like multilane highways than like passing lanes and are not within the scope of this study.

Shoulder use sections include two-lane highways on which the shoulder has been converted to a passing lane and two-lane highways on which the shoulder is used by slow-moving vehicles. In some areas of the country, drivers of slow-moving vehicles move to the shoulder to let faster vehicles pass as a matter of courtesy and local custom; in other areas, shoulder sections have been designated by signs to permit use by slow-moving vehicles.

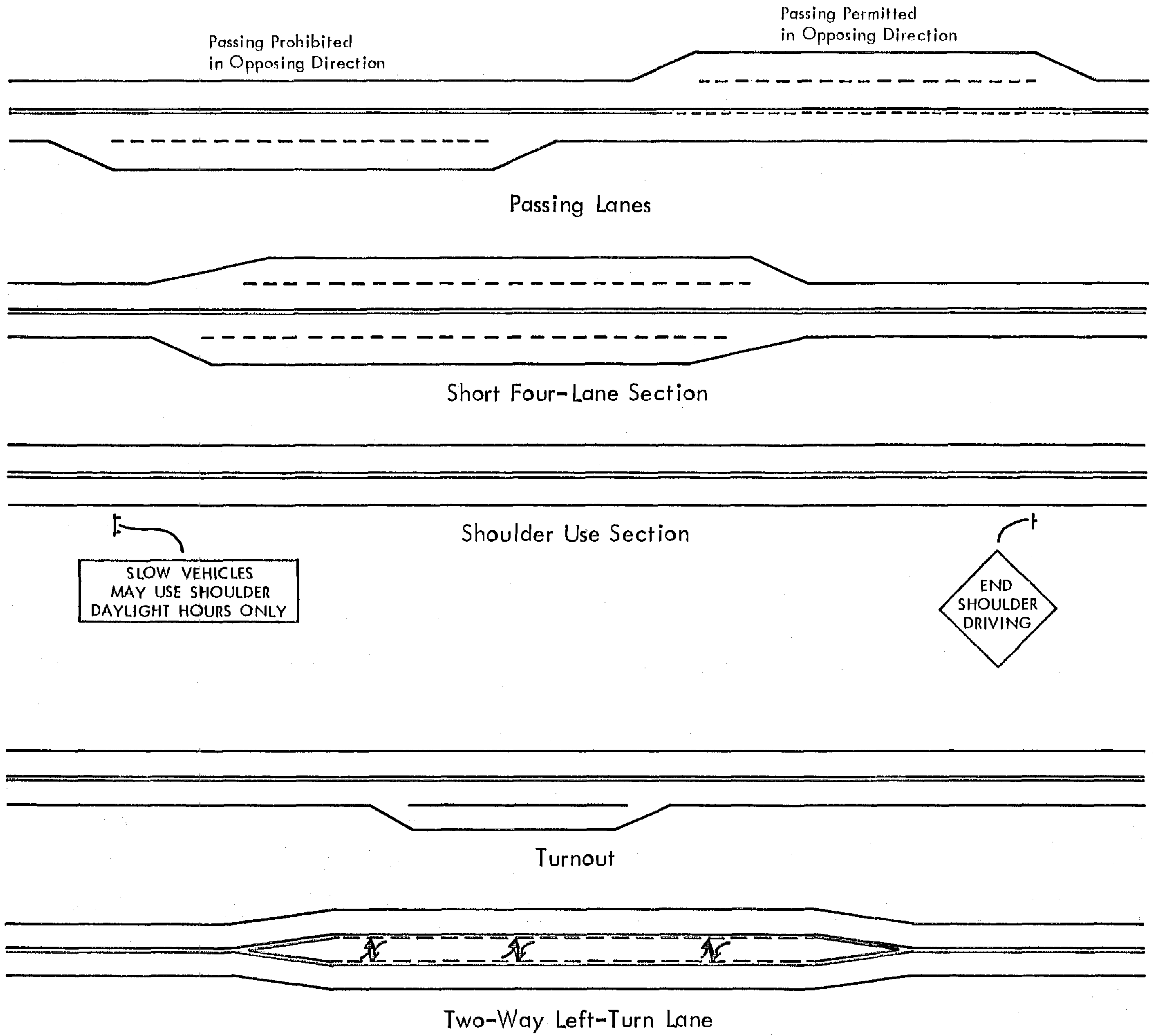


Figure 1 - Typical Operational Treatments Used on Two-Lane Highways

All of these types of shoulder use sections provide opportunities for drivers with higher desired speeds to pass slow-moving vehicles. A shoulder use section designated by signs is illustrated in Figure 1.

A turnout is a widened, unobstructed shoulder area on a two-lane highway provided for slow-moving vehicles to pull out of the through lane and without stopping to allow following vehicles to pass. Turnouts are generally less than 600 ft (190 m) in length.

A two-way left-turn lane is a paved area in the highway median that extends continuously along a highway section and is marked to provide a deceleration and storage area, out of the through traffic stream, for vehicles traveling in either direction to make left turns into intersections and driveways. Two-way left-turn lanes in both rural areas and urban fringe areas were evaluated in this study.

1.2 Research Approach

This section summarizes the general research approach used to evaluate operational treatments in this study. Further details of the data collection and analysis procedures will be found in later sections of the report that address the evaluation of each individual treatment.

The study was conducted with the active participation of the highway agencies of 13 states: Arkansas, California, Kansas, Kentucky, Michigan, Mississippi, Nevada, New York, Oklahoma, Oregon, Pennsylvania, Utah, and Washington. Representatives of these states assisted in the study by documenting their current policies and practices; selecting study sites; assembling geometric, traffic volume and accident data for the sites; and arranging authorization for MRI to conduct field studies at the sites.

1.2.1 Study sites: Table 1 summarizes the number of study sites selected by the participating states for each treatment type. A total of 72 sites were selected for the study, with the majority of these sites being passing lanes. Since many sites contained more than one treatment (e.g., alternating passing lanes), there were a total of 164 treatments studied. Thirty-five (35) of these 164 treatments were evaluated in the field and 138 treatments were included in accident studies. Nineteen (19) of the treated sites selected by the states were not evaluated in either the accident or field studies; these sites were not among those selected for evaluation in the field and did not have adequate accident data available for analysis, usually because the treatments were constructed very recently. Twenty-seven (27) comparable untreated sites were also selected by the participating states. Appendix C provides a complete list of the locations of the treated and comparable untreated sites.

1.2.2 Data collection: Field evaluations of the treatments were performed using a combination of automated and manual methods. Automated traffic data recorders were used to record traffic volumes, traffic mix, vehicle speeds, accelerations, headways and platooning characteristics at several locations upstream, downstream, and within each treatment.

TABLE 1

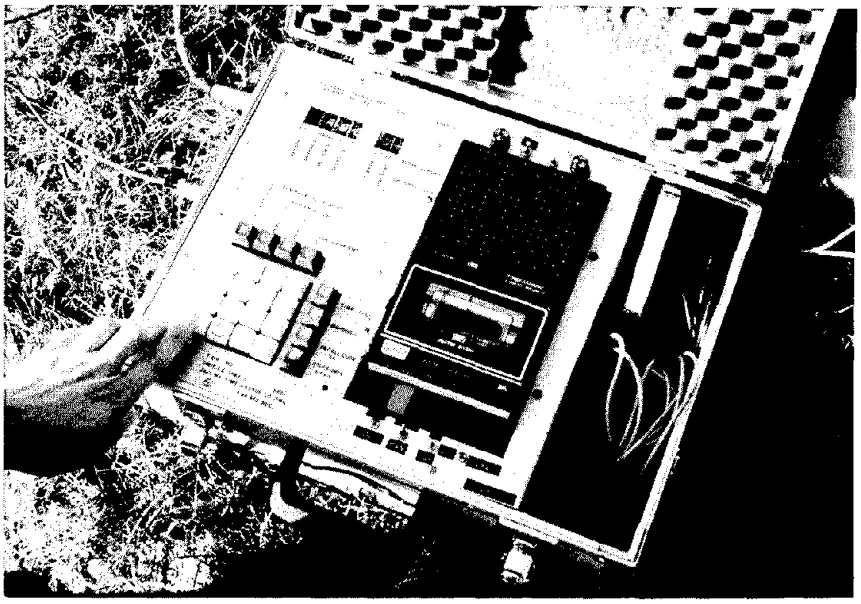
SUMMARY OF AVAILABLE STUDY SITES

Treatment Type	Number of Sites	Number of Treatments	Number of Treatments Used In Field Studies	Number of Treatments Used In Accident Studies	Number of Comparable Untreated Sites
Passing lanes	42	92	12	66	16
Short four-lane sections	8	10	3	10	6
Shoulder use sections	3	4	4	4	1
Turnouts	9	48	9	48	0
Two-way left-turn lanes	10	10	7	10	4
Total	72	164	35	138	27

Manual methods, employing three observers, were used to supplement the recorded data with observations that could not be made automatically including counts of passing maneuvers, traffic conflicts, erratic maneuvers, and driver compliance with legal requirements. The objective of the operational evaluation was to quantify the operational benefits (i.e., improvements in traffic service) provided by the treatments.

The automated traffic data collection made use of piezoelectric cable sensors placed temporarily on the pavement surface. The cable sensors were connected to one of nine automated Traffic Data Recorders (TDRs) built by MRI using a design originally developed at the University of Toronto. Each TDR has the capability to accurately record the time of actuations from up to four sensor cables and to preserve these data on cassette tape. Furthermore, the time clocks of several TDRs can be synchronized so that data collected with different TDRs can be analyzed together. The data recorded on cassette tape can be played back into a computer terminal for subsequent editing, processing, and analysis.

Figure 2 shows a TDR unit and a typical road sensor installation. The usual sensor arrangement for collecting data with a TDR is to place two parallel sensors a known distance apart in each lane of the highway. This sensor arrangement is known as a TDR "trap." From data collected at a TDR trap, a computer program can be used to determine the speed, acceleration, wheelbase, number of axles and time headway for each vehicle passing over the sensors. Vehicle types -- including passenger cars, trucks, and buses -- were distinguished from one another based on wheelbase and number of axles using the same criteria used in the software for FHWA's Traffic Evaluator System (TES).[17] These criteria cannot distinguish recreational vehicles from other vehicle types, because the axle patterns and spacings of various types of RVs are often similar or identical to passenger cars, pickup trucks, trucks, and buses. Therefore, each RV was identified manually in the field by a code entered into the keyboard of one of the TDRs. Appendix B of this report presents examples of the output obtained from the computer processing of TDR data.



Traffic Data Recorder (TDR)

Typical Road Sensor Installation

Figure 2 - Traffic Data Recorder (TDR) and Typical Road Sensor Installation

Traffic operational data were generally collected for 6 hr, including the peak hour, on a typical weekday at each treated site. Appendix A identifies the date and time period of data collection at each site.

1.2.3 Traffic performance measures: The operational measures of effectiveness (MOEs) used to evaluate the treatments included vehicle speeds, percent of traffic platooned and rate of passing maneuvers. These measures of effectiveness were compared between locations upstream of, within, and downstream of each treated section. Traffic operational measures in the untreated or opposing direction of travel were also considered, where appropriate. For ease in summarizing the study findings, the results of the operational evaluation have been expressed as the average value and range of variation in each measure of effectiveness. Multiple linear regression analyses have also been used to illustrate the relationships between the operational MOEs and key site parameters such as flow rate, upstream traffic characteristics, and treatment length.

Safety evaluations were also conducted to determine the effects of the operational treatments on accidents. The objective of the safety evaluation was to make certain that the installation of operational treatments do not cause any reduction in safety and to quantify any safety benefits that result from the treatments.

The measures of effectiveness used in the safety evaluations included both total accident rates and fatal and injury accident rates. The fatal and injury accident rate is considered to be a more reliable safety measure of effectiveness than the total accident rate, because reporting rates for property-damage-only accidents are known to vary widely from jurisdiction to jurisdiction. However, the sample of fatal and injury accidents available for evaluation is often quite small, so the use of the total accident rate provides a larger sample size for analysis.

1.2.4 Accident analysis: Accident data were provided by the participating states for a period of 1 to 5 years for each of the sites studied. In all cases, the study period for the safety evaluation was chosen to be an integer multiple of 12 months in length. The use of accident data for complete years avoids the introduction of seasonal biases in the accident analysis. Generally, the accident data included the entire treated section and 0.5 mile (0.8 km) of untreated two-lane highway at either end. In selected cases, accident data were also available for a period before treatment installation at the same site or for the same time period on a comparable untreated two-lane highway. The participating states provided data in the form of police accident reports or computer listings of accidents. Some states also included collision diagrams or verbal summaries of accidents.

The safety evaluation conducted using these data was based on both formal statistical comparisons and less formal investigations of accident patterns to assure that no indication of a potential safety problem was missed. For all treatment types except passing lanes, the sample sizes of accidents available of the treated sections were relatively small, so greater reliance on judgment was necessary in the interpretation of accident data.

1.3 Organization of This Report

The remainder of this report is organized in four sections that present the evaluation of each type of operational treatment. Section 2 presents the evaluation of passing lane and short four-lane sections. Shoulder use sections are evaluated in Section 3 and turnouts in Section 4. The evaluation of two-way left-turn lanes in rural and urban fringe areas is presented in Section 5. The evaluation of each treatment is organized into three subsections that present the current state policies and practices for use of the treatment; the operational evaluation of the treatment (including the data collection procedures, the measures of effectiveness and the analysis results); and, the safety evaluation. Finally, Section 6 presents the conclusions drawn from the evaluation of all of the treatments.

2. PASSING LANE AND SHORT FOUR-LANE SECTIONS

This section presents the evaluation of two closely related treatments: passing lanes and short four-lane sections. The following discussion presents a description of passing lane and short four-lane sections; a review of current state design and traffic control practices; an operational field evaluation of 12 passing lanes and 3 short four-lane sections; and, a safety evaluation of 66 passing lanes and 10 short four-lane sections.

Figure 3 illustrates a typical passing lane with passing prohibited in the opposing direction, a passing lane with passing permitted in the opposing direction and a short four-lane section.

A passing lane is defined as an added third lane in one direction of a normally two-lane highway to provide opportunities to pass slow moving vehicles where passing opportunities would otherwise be limited by sight distance and opposing traffic. A passing lane is generally introduced with a lane addition taper and terminated with a lane drop taper, although some passing lanes are added and/or dropped at intersections. A passing lane may be used either alone or as part of a series of passing lanes in alternating directions. Where sight distance is adequate, some agencies permit passing by vehicles traveling in the opposing direction to a passing lane, while other agencies prohibit all passing maneuvers by vehicles in the opposing direction.

Passing lanes in level or rolling terrain are a primary focus of the current study, because they have not been evaluated extensively in the United States. However, added lanes of this type are also used extensively on steep grades in hilly or mountainous terrain, where they are generally known as truck climbing lanes. Climbing lanes have been evaluated more extensively than passing lanes in previous research, although most of the research has focused on determining warrants for climbing lanes or establishing design criteria (e.g., position on grade), rather than evaluating their operational benefits.[20] For purposes of this study, only those passing lanes located on grades that are long and steep enough to reduce heavy trucks to crawl speeds have been classified as climbing lanes. The work of Ching and Rooney[1] on grades in California indicates that 5-axle trucks are reduced to steady crawl speeds after 6,000 ft (1,830 m) on a 5% upgrade, after 5,500 ft (1,680 m) on a 6% upgrade and after 5,000 ft (1,525 m) on a 7% upgrade.

A short four-lane section is a section of four-lane highway, generally less than 3 miles in length, and bounded by two-lane sections at both ends. A short four-lane section on a normally two-lane highway could represent the ultimate design for a particular site or could represent the first step in staged construction of a four-lane highway. Whichever purpose a short four-lane section was constructed for, it provides additional passing opportunities and operates essentially as two passing lanes in opposite directions at the same location. A short four-lane section requires greater pavement and right-of-way width than a passing lane, but has the potential advantage that there is no need to permit vehicles traveling in either

Passing Lane with Opposing Direction Passing Prohibited

Passing Lane with Opposing Direction Passing Permitted

Short Four-Lane Section

Figure 3 - Typical Passing Lane and Short Four-Lane Sections Used on Two-Lane Highways

direction to cross the marked centerline in order to pass. Short four-lane sections are usually either undivided or divided with a narrow, flush median, although four-lane divided sections with a raised or unpaved median could operate in a similar manner.

In this study, only those sites where the added lanes begin and end at the same location in both directions, have been classified as short four-lane sections. Sites where the added lanes are offset from one another are classified as alternating passing lanes, even if the overlap of the added lanes is substantial.

2.1 Current Practice

The following discussion presents current state design and traffic control policies and practices for passing lane and short four-lane sections. Also included is an assessment of the practices actually found in the field at the study sites.

2.1.1 Passing lanes: Passing lanes on two-lane highways were found to be in current use in 11 of the 13 states that participated in this study. These states are: Arkansas, California, Kentucky, Michigan, Mississippi, Nevada, Oklahoma, Oregon, Pennsylvania, Utah and Washington. Passing lanes have existed for many years in a number of states (including California), whereas other states (such as Arkansas) have only recently begun to use alternating passing lanes to increase passing opportunities on two-lane highways. All but two of the states participating in the study have installed passing lanes to increase passing opportunities in level and rolling terrain; the remaining two states (Kansas and New York) have installed climbing lanes on steep grades.

The 11 participating states identified above selected 92 passing lane sites for the study. Of these 92 passing lanes, 71 (or 77%) are part of sequences of two or more passing lanes in the same or alternating directions, while the remaining 21 passing lanes (23%) are isolated passing lanes not located near other treatments. While no formal selection procedure was used to assure the representativeness of these sites, they are presumed to be typical of the passing lanes currently implemented in the 11 states.

Passing lane length: Two states were found to have formal criteria concerning the minimum length of passing lanes. Both Oregon and Washington require passing lanes to be at least 1,000 ft (0.19 mile or 310 m) in length. It is interesting to note that since turnouts are generally 600 ft (190 m) or less in length (see Section IV), there is a range of lengths between 600 and 1,000 ft (190 and 310 m) that are not generally used for operational treatments. None of the states had criteria limiting the maximum length of a passing lane.

The lengths of the 92 passing lanes studied range from 0.20 to 3.69 miles (0.32 to 5.94 km). Thus, all of the passing lanes were found to exceed the 1,000-ft (310-m) minimum length criterion recommended by Oregon and Washington. The average passing lane length was found to be 1.10 miles (1.77 km). Table 2 illustrates the distribution of passing lane lengths.

TABLE 2

DISTRIBUTION OF LENGTHS OF PASSING LANE AND SHORT FOUR-LANE SECTIONS

Length (miles)	Number of Passing Lane Sections	Number of Short Four-Lane Sections
< 0.25	1	1
0.25-0.49	18	3
0.50-0.74	19	2
0.75-0.99	13	1
1.00-1.24	8	1
1.25-1.49	7	0
1.50-1.74	8	0
1.75-1.99	7	0
2.00-2.24	4	0
2.25-2.49	2	1
2.50-2.74	2	1
2.75-2.99	1	0
≧ 3.00	2	0
Total	92	10

Note: 1 mile = 1.609 km.

Lane addition tapers: Two states were found to have criteria for the length of the lane addition taper. Oregon recommends a minimum taper length of 400 ft (120 m), while Washington recommends a minimum taper rate of 25:1, which is equivalent to a 300-ft (90-m) taper for a 12-ft (3.7-m) lane.

The lane addition tapers for the passing lanes studied in the field were found to range from 150 to 500 ft (45 to 150 m) in length, measured from the beginning of the taper to the beginning of the broken, white lane line. The average lane addition taper length was 260 ft (80 m).

Lane drop tapers: The Manual on Uniform Traffic Control Devices (MUTCD)[18] recommends in Section 3B-8 that the length (L) of lane reduction transitions (tapers) should be equal to the width of the closed lane (W) multiplied by the 85th percentile speed (S). Most states have either adopted the national MUTCD or have their own state MUTCD that includes this lane reduction transition formula. For a highway with an 85th percentile speed of 60 mph (97 kph) and a lane width of 12 ft (3.7 m), the minimum taper length recommended by the MUTCD formula is 720 ft (220 m).

Three states were found to have design policies that address the minimum length of lane drop tapers specifically for passing lanes. California's passing lane policy incorporates the same formula (L = WS) as the MUTCD. Oregon recommends a 400-ft (120-m) minimum length for the lane drop tapers of passing lanes and Washington recommends a 50:1 taper rate, corresponding to a 600-ft (180-m) taper length for a 12-ft (3.7-m) lane.

The lane drop tapers for the passing lanes studied in the field were found to range from 170 to 900 ft (50 to 275 m) in length with an average length of 540 ft (165 m).

Pavement markings: The pavement markings used by all of the participating states to designate passing lanes are consistent with those presented in Figure 3-2a and 3-2b of the MUTCD.

The major variation in pavement marking policies between the states pertains to whether or not passing is permitted by opposing direction vehicles. Six of the 11 states where passing lanes were studied permit passing in the opposing direction where sight distance is adequate. These states are: Kentucky, Michigan, Oklahoma, Oregon, Utah and Washington. Three states -- Mississippi, Nevada and Pennsylvania -- generally prohibit passing by opposing direction vehicles throughout the entire length of a passing lane. Arkansas generally permits passing by opposing direction vehicles at climbing lane sites, but chose to prohibit passing by opposing direction vehicles at one passing lane site evaluated in this study. The remaining state, California, requires passing to be prohibited in the opposing direction for passing lanes on all highways with ADT over 3,000 vehicles per day; passing in the opposing direction may be permitted on highways with lower traffic volumes.

In those states that permit passing by vehicles traveling in the opposing direction, the sight distance warrants for no-passing zones are generally the same at passing lanes as at other locations. Based on Section 3B-5 of the MUTCD, 1000 ft (305 m) of passing sight distance is typically required in order to permit passing on roads with prevailing speeds of 60 mph (97 kph). One state, Oregon, has a more conservative policy that permits passing by vehicles in the opposing direction only where sight distance exceeds 2,000 ft (610 m).

Signing: The signing practices for passing lanes in 11 states are summarized in the following discussion. This summary is based on formal state policies, where available, or on signing practices actually observed in the field.

There are four general functions and locations for which signing is used at passing lanes. These are:

- Advance notification of passing lane, 2 to 5 miles (3 to 8 km) upstream.
- Advance warning of passing lane addition, within 0.5 mile (0.8 km) upstream.
- Reinforcement of legal requirement for proper passing lane use at beginning of passing lane.
- Advance warning of lane drop at end of passing lane.

Figure 4 summarizes the typical signing practices of the 11 states that have installed pasing lanes.

Only one state (California) has a formal policy to provide more than 0.5-mile (0.8-km) advance notification to drivers of an upcoming passing lane. California uses an advance sign with the legend, PASSING LANE 2 MILES. Advance signing of this type has the potential to reduce the frustration and impatience of drivers following a slow-moving vehicle if they know that they are approaching a dependable passing opportunity. Driver frustration and impatience of this type have been shown to be a potential safety problem on two-lane highways. Hostetter and Seguin[9] found, for example, that when forced to follow a slow-moving vehicle for up to 5 miles, almost 25% of the drivers made an illegal pass in a no-passing zone. The Ontario Ministry of Transportation and Communications in Canada has adopted a policy of 5-mile (8-km) advance signing for passing lanes to reduce the risk of accidents associated with illegal passing maneuvers.[13]

Advance signing within 500 ft (150 m) to 0.5 mile (0.8 km) of the beginning of the passing lane is used by 7 of the 11 states. Advance signs of this type alert drivers that they are approaching a passing lane. Many drivers following slow-moving vehicles will act on this information by moving closer to the leading vehicle, so they are in a better position to pass when they reach the passing lane.

Seven states place a sign at the beginning of the passing lane to reinforce the legal requirement for slow-moving vehicles to use the right lane. The SLOWER TRAFFIC KEEP RIGHT sign is most commonly used for this purpose. The policy of one state (Michigan) recommends that the SLOWER TRAFFIC KEEP RIGHT sign be repeated after intersections or as needed throughout the length the added lane.

To summarize the signing practices upstream of a passing lane:

- 1 state uses a sign 2 miles (3.2 km) in advance of the passing lane, a sign within 0.5 mile (0.8 km) in advance of the passing lane and a sign at the beginning of the passing lane.
- 4 states use a sign within 0.5 mile (0.8 km) in advance of the passing lane and a sign at the beginning of the passing lane.
- 1 state uses a sign 500 ft (150 m) in advance of the passing lane, but no sign at the beginning of the passing lane.
- 1 state uses a sign at the beginning of the passing lane, but no advance signing.
- 3 states use no signing in advance or at the beginning of the passing lane.

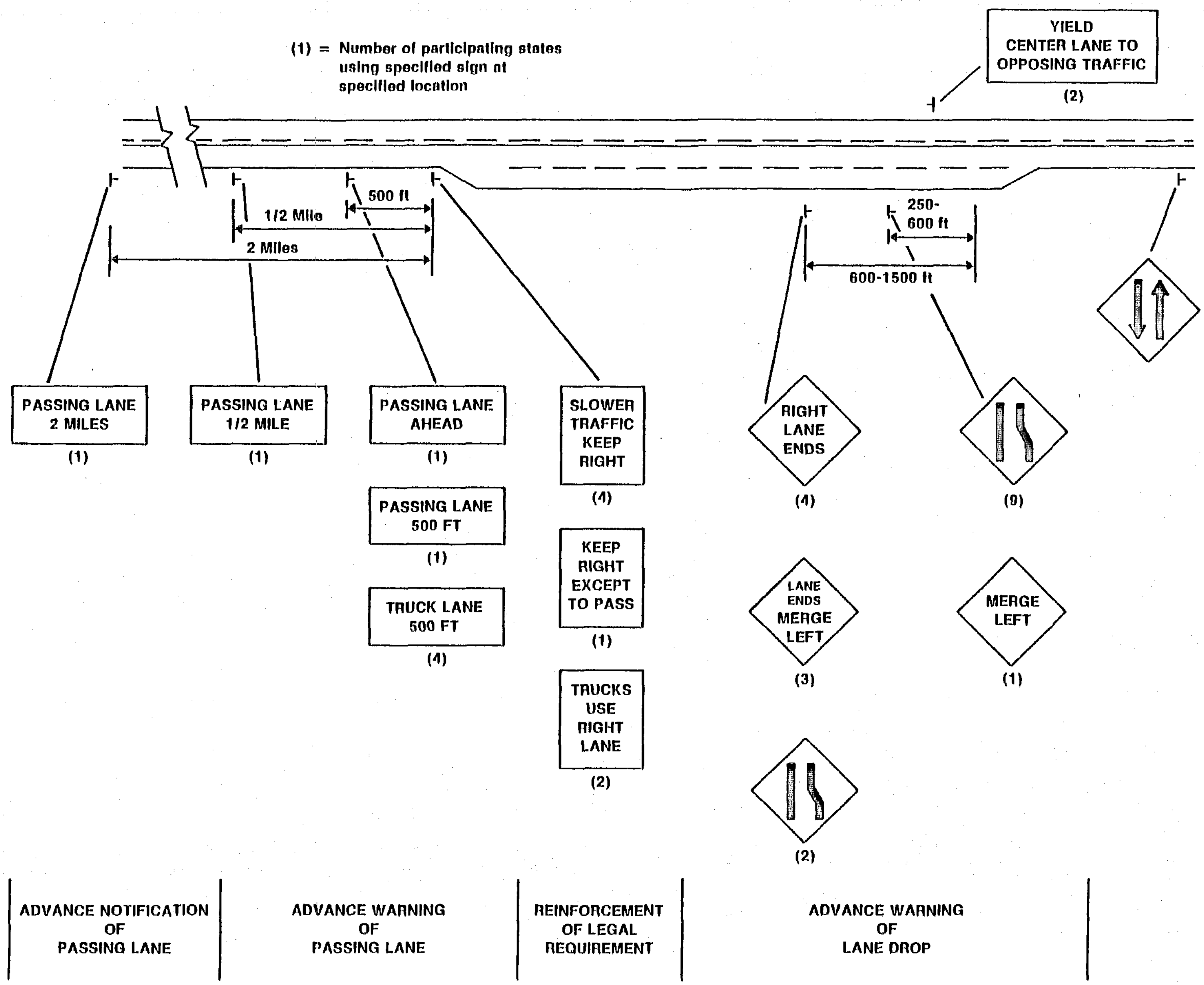


Figure 4 - Typical Signing Practices for Passing Lanes

There is an obvious need for greater uniformity in the signing practices used to introduce passing lanes.

The signing practices used for advance warning of the lane drop at the end of a passing lane are more uniform than the signing used to introduce the passing lane, but there are still considerable variations in the legends and placements of warning signs. Eight of the 11 states use a sequence of two warning signs in advance of the lane drop, one of which is the standard lane drop symbol sign (W4-2).* The other three states use only the lane drop symbol sign. The sign sequences used by the states are shown in Figure 4.

Although not mentioned in any of the passing lane signing policies reviewed, some states use the TWO-WAY TRAFFIC sign (W6-3) immediately downstream from the lane drop taper.

States that prohibit passing by vehicles in the opposing direction at passing lanes, often reinforce the double yellow centerline marking with regulatory DO NOT PASS signs (R4-1) throughout the passing lane area. Two states that allow passing by opposing direction vehicles use signs that read, YIELD TO UPHILL TRAFFIC or YIELD CENTER LANE TO OPPOSING TRAFFIC, to supplement the centerline markings.

2.1.2 Short four-lane sections: Short four-lane sections on two-lane highways were studied in 3 of the 13 participating states. These states are: New York, Oregon, and Washington. These three states selected a total of 10 short four-lane sections for evaluation. The geometric and traffic control practices used by these states for short four-lane sections are generally similar to passing lanes. Many of the other participating states have used short four-lane sections to improve traffic operations on two-lane highways, but did not choose to include any short four-lane sections in this study.

Short four-lane sections are not generally located near other operational treatments. However, a unique feature of two study sites in New York is that pairs of short four-lane sections were used in sequence, separated by sections of conventional two-lane highway less than 0.35 mile (0.55 km) in length. No reason is evident why two separate short four-lane sections were constructed so close to one another, rather than constructing a single, continuous four-lane section.

The distribution of length of the 10 short four-lane sections has been presented with the comparable distribution of passing lane lengths in Table 2. The lengths ranged from 0.15 to 2.56 miles (0.24 to 4.12 km), with an average length of 0.90 mile (1.45 km). There are no explicit state policies regarding the length of four-lane sections.

State policies regarding the length, marking and signing of lane addition and lane drop tapers for short four-lane sections do not differ from the policies applicable to passing lanes, presented above.

* Throughout the report, where appropriate, standard signs are identified by their MUTCD code.

The pavement markings used for short four-lane sections are uniform in all three states. Since there is no need to cross the centerline to perform passing maneuvers, all short four-lane sections have a double, solid yellow centerline marking.

One of the participating states (Oregon) used the same advance signing practice for short four-lane sections as for passing lanes: a PASSING LANE 1/2 MILE sign in advance of the short four-lane section, as well as a SLOWER TRAFFIC KEEP RIGHT sign (R4-3) at the beginning of the lane addition taper. The other two states used only the SLOWER TRAFFIC KEEP RIGHT sign at the beginning of the short four-lane section.

2.2 Operational Evaluation

An operational evaluation was performed for 12 passing lanes and 3 short four-lane sites using traffic performance data collected in the field. The objectives of this evaluation were to determine the effectiveness of these treatments in improving traffic operations on two-lane highways and to determine the influence of traffic volume, geometrics, and treatment length on the operational effectiveness of the treatments. The operational data that were collected have been used here to perform a gross or macroscopic assessment of passing lanes and short four-lane sections and can be used in further research to modify and calibrate microscopic computer simulation models of these operational treatments.

2.2.1 Data collection: The field data collection plan for passing lanes made use of automatic Traffic Data Recorders (TDRs) at six locations and three manual observers. The TDRs were used to record traffic volumes, vehicle mix, speeds, accelerations, headways, and platooning characteristics. The manual observers counted passing maneuvers in both directions in the treated section and traffic conflicts or erratic maneuvers in the lane drop transition area. The manual observers also performed part of the vehicle classification by entering a code for each recreational vehicle into one of the TDRs. Figure 5 illustrates the data collection setup for a typical passing lane, including the location of TDR traps and the observers. Modifications to this general plan were made in the field, as necessary, to adapt the plan to the geometric and traffic characteristics of specific sites.

The data collection plan was structured to determine the effectiveness of passing lanes by a comparison of traffic operational conditions at three key locations: Location 1 (upstream of the passing lane); Location 3 (in the middle portion of passing lane); and Location 5 (downstream). In addition, comparisons between Location 5 and Location 6 (approximately 1 mile (1.6 km) downstream from the passing lane) were intended to determine the rate at which operational benefits of the passing lane are lost downstream. At one site, operational data were also collected at 2 and 3 miles (3.2 and 4.8 km) downstream from a passing lane (Locations 7 and 8, respectively).

Operational data were collected for special purposes at Locations 2 and 4. The data collected at Location 2 (approximately 100 ft (30 m) downstream from the beginning of the lane lines) were intended primarily to indicate the initial lane choice by drivers entering a passing lane for use

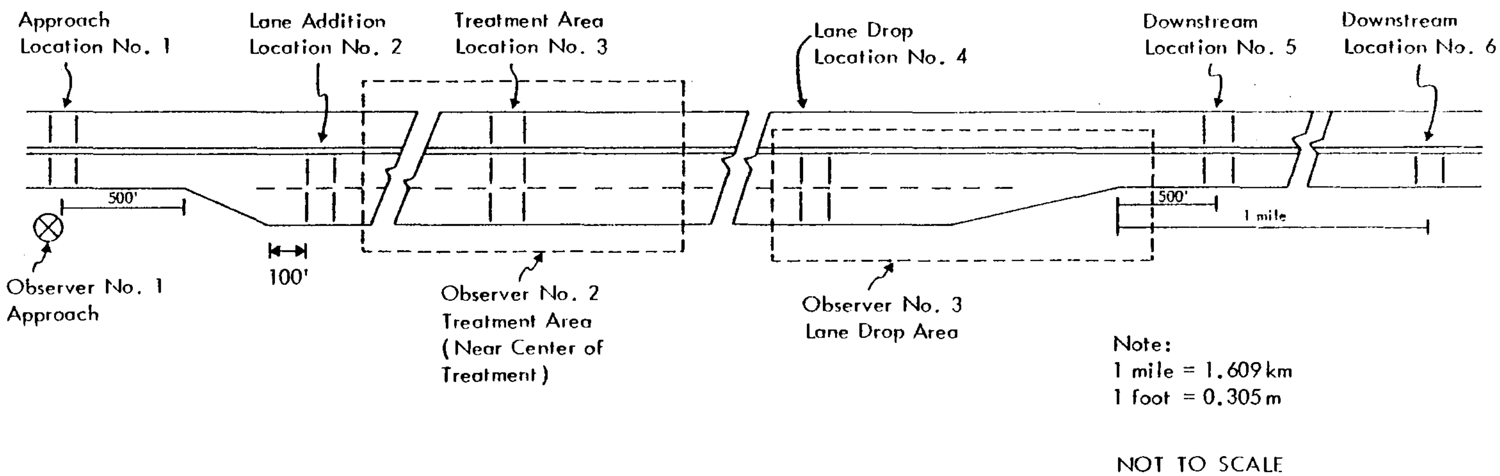


Figure 5 - Locations of TDR Traps and Observers for Data Collection at Passing Lanes

in the development of computer simulation models. The operational data from Location 4 were intended for use, together with manually collected traffic conflict data, to assess the safety performance of lane drop transition areas. The analysis of these data is discussed in Section 2.3 of this report.

The data collected at short four-lane sections were essentially equivalent to that collected at passing lanes. For short four-lane sections, operational data were collected in four lanes rather than three in the middle of the treated section. Data were collected in the lane addition and lane drop areas in one direction of travel only.

2.2.2 Measures of effectiveness: Three primary measures of effectiveness were used in this study to assess the operational benefits of passing lanes and short four-lane sections on two-lane highways. These measures were:

- Traffic speed,
- Percent of vehicles platooned, and
- Passing rate.

The speed of an individual vehicle at any point on a highway is a combined result of the desired speed of the driver, the limitations imposed by roadway geometrics and the limitations imposed by vehicle performance (e.g., inability of trucks and RVs to maintain speeds on grades). The operational data recorded with the TDRs provide the speeds of each individual vehicle passing over each TDR trap. The mean speed and various percentiles of the speed distribution were used as measures of effectiveness. Speed descriptors were obtained separately for passenger cars, trucks and buses, RVs, unimpeded vehicles (free vehicles and platoon leaders), and for the traffic stream as a whole.

Traffic speed has historically been used as the key variable to define the quality of service on two-lane highways. The 1965 Highway Capacity Manual (HCM)[7] defined six levels of service for two-lane highways based on operating speed. Operating speed is defined as the highest overall speed that a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions without at any time exceeding the safe speed of the highway. The 1965 HCM provides a procedure to estimate operating speed, and thus level of service, based upon average highway speed (design speed), volume/capacity ratio, percent of length with adequate passing sight distance, lane width, lateral clearance, terrain and percent trucks.

Experience with the 1965 HCM procedure for two-lane highways has shown the need for measures of service other than traffic speed. Operating speed represents an extreme end of the speed distribution that may be experienced by only a small proportion of the traffic stream. Mean speed is a better overall measure of what drivers actually experience on the highway but, as illustrated below, the mean speed is not very sensitive to variations in flow rate. The measure of service proposed for use in a revised HCM procedure for two-lane highways is the percent of time vehicles are delayed by other traffic.[10] The percent time delayed is the proportion of total travel time represented by vehicles which are following in platoons. Figure 6 illustrates that the percent time delayed is much more sensitive to traffic flow rate than is mean speed. The percent of vehicles that are members of platoons is a spot measure that serves as an approximation to percent time delayed which by definition is measured over a section of highway.

Each vehicle recorded at a TDR trap was classified as a free vehicle, a platoon leader or a platoon member. A vehicle with a time headway of 4 sec or less was classified as a platoon member. A vehicle with a headway greater than 4 sec, but followed by a trailing vehicle with a headway of 4 sec or less, was classified as a platoon leader. A vehicle with no vehicle either ahead or behind within 4 sec was classified as a free vehicle. The percent of traffic platooned, used as a measure of effectiveness in this study, is the percent of the traffic volume constituted by platoon members (i.e., not including either free vehicle or platoon leaders, neither of which are delayed or impeded by other traffic).

The choice of the 4-sec headway criterion to define platooning was made after careful consideration of the criteria used by other researchers. The revised HCM procedures recommend a platoon definition based on a 5-sec headway.[10] Morrall,[11] a Canadian contributor to the revised HCM procedures, used a platoon definition based on a 6-sec headway. Hoban,[8] who has conducted extensive operational research on two-lane highways and passing lanes in Australia, has recently recommended a 4-sec headway criterion. With a 5- or 6-sec headway criterion, traveling at 55 mph (88 kph) can be considered platooned if one vehicle is more than 400 ft (120 m) behind the other. In this study, it was considered critical to avoid classifying a vehicle as platooned unless this was clearly the case. For this reason, we selected the shortest of the criteria frequently cited in the literature -- 4 sec.

Table 3 compares the values of several traffic operation measures in the field data from a passing lane site (Site A01) using the 4-sec headway criterion and two alternative criteria (5 and 6 sec). The table shows that increasing the headway criterion generally increases the percent of vehicles platooned at any given point, but decreases the reduction in the percent of vehicles platooned resulting from a passing lane. The mean speeds of free vehicles and platoon leaders are affected only slightly by the choice of a headway criterion for platooning.

The operational effectiveness of a passing lane is clearly dependent on upstream geometric and traffic conditions which influence the percent of traffic platooned as it enters the passing lane. At any given flow rate, a passing lane would be expected to be most effective where upstream conditions promote the formation of platoons which can be relieved by the passing lane. Passing lanes would be expected to be less effective where relatively few platoons exist (for example, when a previous passing lane exists directly upstream). The percent of traffic platooned on entering the passing lane is one useful measure of this upstream "conditioning."

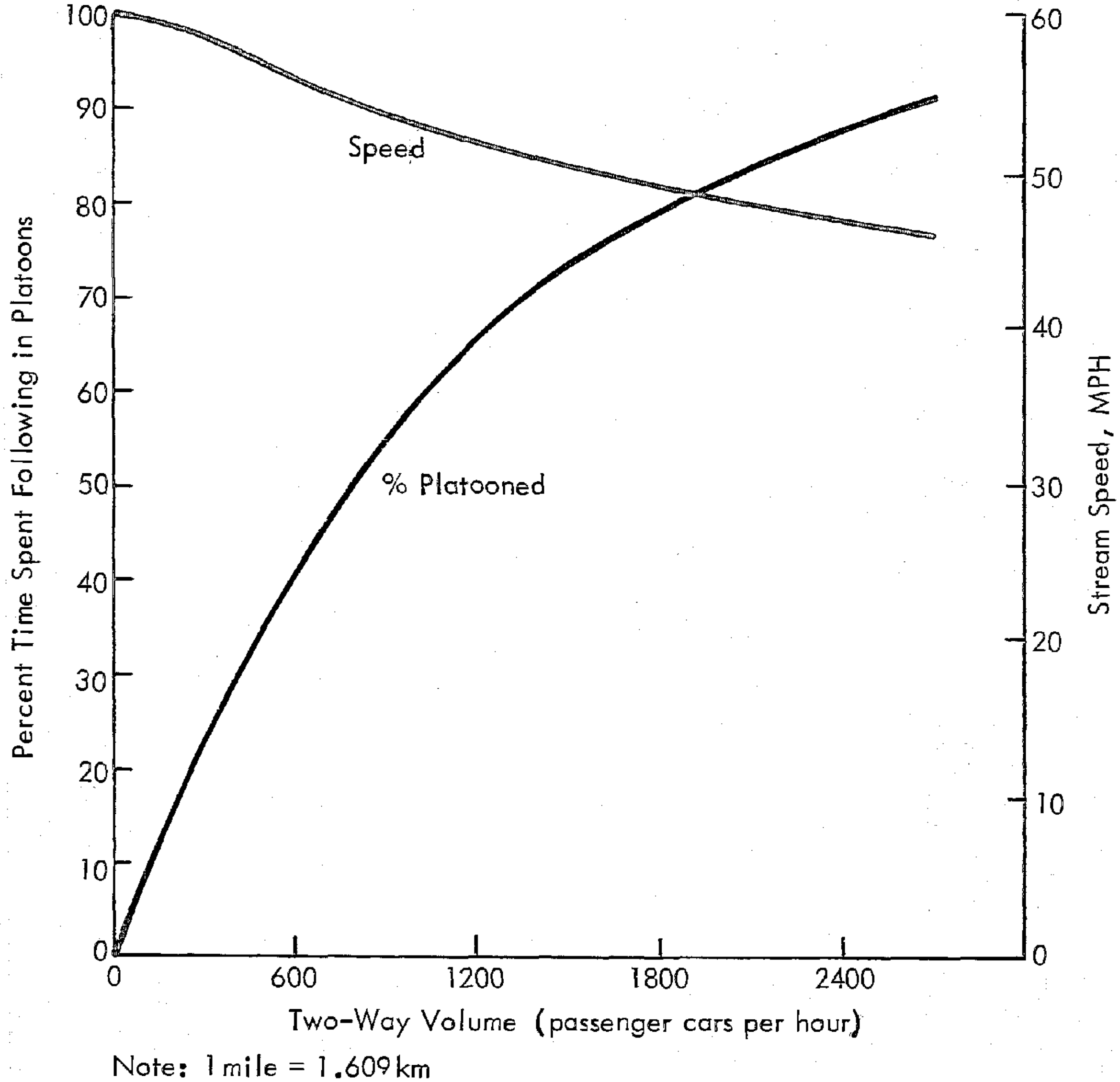


Figure 6 - Relationship Between Total Two-Way Volume on Two-Lane Highways and Average Percent Time Following in Platoons and Average Overall Traffic Stream Speed[10]

TABLE 3

SENSITIVITY OF TRAFFIC OPERATIONAL MEASURES TO PLATOON DEFINITION (4, 5, OR 6 SEC HEADWAY)

Time	Flow Rate (vph)	Percent of Vehicles Platooned[a]			Reduction in Percent of Vehicles Platooned[b]			Mean Speed of Free Vehicles (mph)[a]			Mean Speed of Platoon Leaders (mph)[a]		
		4 sec	5 sec	6 sec	4 sec	5 sec	6 sec	4 sec	5 sec	6 sec	4 sec	5 sec	6 sec
1145-1245	129	27.1	32.6	35.7	6.1	9.2	11.5	53.9	54.0	54.0	51.2	51.4	51.2
1245-1345	140	30.7	32.9	37.9	13.1	11.3	12.9	53.7	53.6	54.2	52.3	52.4	52.2
1345-1445	122	25.4	27.0	27.9	13.3	11.5	8.9	54.6	55.0	55.0	49.8	49.5	49.9
1445-1545	142	26.2	29.1	32.6	5.8	3.9	3.3	55.4	55.5	55.6	51.3	51.2	51.4
1545-1645	147	25.2	27.9	29.9	6.6	5.1	5.1	55.0	55.3	55.4	51.6	51.5	51.5
1645-1745	160	29.4	33.8	35.6	8.2	8.2	4.2	55.0	55.4	55.5	53.1	52.9	52.6
Combined	140	27.4	30.6	33.4	8.7	8.0	7.4	54.6	54.8	55.0	51.6	51.6	51.5

[a] Upstream of passing lane.
[b] Difference between percent of vehicles platooned upstream and downstream of passing lane.

Note: 1 mile = 1.609 km.

The final measure of effectiveness used for the evaluation of passing lanes and short four-lane sections was the passing rate, defined as the number of completed passes per hour per mile in one direction of travel. The passing rate is an appropriate measure of effectiveness because passing lanes are intended to increase the passing rate above that which would occur on a normal two-lane highway.

2.2.3 Operational analysis results: This section presents the results of the operational analysis of 12 passing lanes and 3 short four-lane sections. A combined operational analysis of passing lane and short four-lane sections was conducted. Each direction of travel in the short four-lane sections was treated as a separate passing lane, so the combined data for the operational analysis represent, in effect, 18 passing lanes. Separate analyses of safety issues were performed for passing lanes and short four-lane sections; these safety analyses are found in Section 2.3.

Up to 6 hr of operational data were selected at each study site; a total of 104 hr of operational data are available to evaluate the 18 treatments. However, because of various equipment malfunctions, data were lost for some hours at some locations; since many of the operational analysis involved comparisons of data collected at two locations, such comparisons could not be made if the data for either location are missing. In addition, one site was omitted from the operational analyses for reasons explained below. The number of hours of reliable data which could be used in the following analyses ranged from 77 to 90 hr.

The traffic flow rates observed at the passing lane and short four-lane sites ranged from 26 to 710 vehicles per hour in the treated direction. However, the results reported below are not necessarily valid for flow rates above 400 vehicles per hour, since very little data at flow rates above that level were obtained. All of the conclusions presented below are statistically significant at the 95% confidence level unless otherwise stated.

Percent of vehicles platooned: Passing lanes were found to reduce the percent of vehicles that are members of platoons. Table 4 illustrates the effect of passing lanes on vehicle platooning. The percentage of vehicles platooned decreased, on the average, from 35.1% immediately upstream of a passing lane to 20.7% within the passing lane. Immediately downstream of the passing lane, the percentage of vehicles platooned had increased to 29.2%, on the average, which is still 5.9% lower than the upstream level. This decrease in the percent of vehicles platooned represents a major improvement in traffic service within a passing lane and a small improvement in traffic service downstream of a passing lane.

Table 4 also illustrates, that the operational benefits from the introduction of a passing lane can vary greatly from site to site. These variations are even greater than shown in the table when each hour of data from each site is examined separately. The prediction of these variations as a function of geometric and traffic operational variables is addressed later in this section.

TABLE 4

EFFECT OF PASSING LANE ON PERCENT OF VEHICLES PLATOONED

Site	Average Flow Rate[a] (vph)	Percent of Vehicles Platooned[a] Immediately Upstream	Within Passing Lane[b]	Immediately Downstream	One Mile Downstream	Upstream-Downstream Reduction (ΔPL)
A01	140	27.4	14.6	18.7	23.0	8.7
C05	560	61.9	44.6	57.1	51.5	4.8
C06	120	28.0	11.0	21.7	21.3	6.3
C08	120	43.4	33.3	40.7	41.8	2.7
H11	80	11.7	11.0	8.0	10.7	3.7
N01	150	26.7	13.4	25.5	25.0	1.2
P04	300	41.2	34.4	36.7	40.9	4.5
R02(NB)	410	51.4	31.1	45.8	45.3	5.6
R02(SB)	415	46.1	28.7	42.6	-	3.5
R11	130	34.2	18.5	31.4	25.0	2.8
T01	150	24.1	15.4	22.0	21.6	2.1
T02	35	9.2	2.8	8.0	10.7	1.2
U07	300	49.1	22.2	37.3	41.6	11.8
W01	305	39.0	21.6	44.1	47.2	-5.1
Y03(NB)	280	41.7	24.1	-	-	-
Y03(SB)	330	43.6	24.2	35.4	36.9	8.2
Y04(NB)	340	50.9	22.8	38.4	-	12.5
Y04(SB)	250	36.4	19.6	23.0	30.9	13.4
Average[c]		35.1	20.7	29.2	31.6	5.9

[a] Platooned vehicles include following vehicles that are members of platoons but not platoon leaders.
[b] Combined data for right and left lanes in treated direction near center of passing lane section.
[c] Average of hour-by-hour data, rather than site-by-site data tabulated above.

Table 5 presents the traffic flow characteristics immediately upstream of passing lanes. The table illustrates the variety of vehicle platooning that is found on two-lane highways and the characteristics of platoon leaders. It can be seen in Table 5 that trucks and RVs tend to become platoon leaders more frequently than their proportion in the vehicle population would suggest.

The average length of platoons (including both platoon leaders and platoon members) is also presented in Table 5. As the percentage of vehicles platooned decreased from upstream to downstream of a passing lane, so did the average platoon length, which was approximately 2.9 vehicles upstream of a passing lane and 2.7 vehicles downstream of a passing lane.

An issue of interest to the evaluation of passing lanes is how far downstream the operational benefits of the added lane persist. It is expected, for example, that any reduction in platooning produced by a passing lane would gradually disappear downstream as faster vehicles overtake slower vehicles and are unable to find passing opportunities. Data were collected in the field approximately 1 mile downstream from each passing lane to determine the persistence of the reduction in platooning provided by a passing lane. Table 4 shows that on the average the average percentage of vehicles platooned one mile downstream of a passing lane is still 3.5% lower than upstream of a passing lane (31.6% vs. 35.1%). However, the results obtained from the analysis of these data were inconclusive; as documented below, the persistence of operational benefits from a passing lane appears to be highly dependent on the geometrics and traffic flow conditions in the downstream area.

Fifty-four (54) hours of data were available during which a reduction in the percent platooned was observed over the one-mile segment of two-lane highway just downstream from a passing lane. In 34 of these hours, the percent of vehicles platooned increased in the first mile downstream from a passing lane. The proportion of the operational benefit lost in this one-mile segment varied widely, however, from periods with almost no change in percent platooned to periods where the increase in percent platooned in the first mile downstream was 4.75 times the reduction due to the passing lane itself. During 9 hr of data, the increase in platooning over the first mile downstream was larger than the reduction in platooning provided by the passing lane. On the other hand, during 20 of the 54 hr, the percent of vehicles platooned either stayed the same or continued to decrease in the downstream segment. No general conclusions about the persistence of operational benefits downstream from a passing lane can be drawn from the available data. It is apparent that these benefits can be lost very quickly downstream from a passing lane in some cases, but not in others. It is likely that both the roadway geometrics and the vehicle mix play an important role in this phenomenon.

One of the 18 passing lane sites (site W01) was found to produce consistent increases in platooning, rather than decreases as found at most other sites. The only apparent explanation for the increase in vehicle platooning at this site is a relatively high flow rate (305 vph) combined with lane addition geometrics that encourage motorists to enter the left lane of the passing lane section. Only 28% of motorists at this site

TABLE 5

TRAFFIC FLOW CHARACTERISTICS IMMEDIATELY UPSTREAM OF PASSING LANE SITES

Site	Average Flow Rate (vph)	Percentage of Total Flow: Free Vehicles	Percentage of Total Flow: Platoon Leaders	Percentage of Total Flow: Platoon Members	Average Platoon Length[a]	Percentage of Total Flow: Cars	Percentage of Total Flow: Trucks	Percentage of Total Flow: RVs	Percentage of Platoon Leaders: Cars	Percentage of Platoon Leaders: Trucks	Percentage of Platoon Leaders: RVs
A01	140	53.5	19.1	27.4	2.4	75.4	14.4	10.1	73.8	13.8	12.5
C05	560	14.9	23.2	61.9	3.7	80.2	18.8	1.0	72.2	26.3	1.6
C06	120	54.3	17.7	28.0	2.6	88.2	10.7	1.1	83.7	16.3	0.0
C08	120	37.7	18.9	43.4	3.4	81.1	5.7	13.2	80.0	10.0	10.0
H11	80	78.1	10.2	11.7	2.3	74.4	24.6	1.1	68.8	31.3	0.0
N01	150	57.3	16.0	26.7	2.7	80.8	9.0	10.2	72.9	14.6	12.5
P04	300	36.2	22.6	41.2	2.9	71.4	28.6	0.1	73.2	26.8	0.0
R02(NB)	410	25.3	23.3	51.4	3.2	82.9	11.8	5.2	81.4	12.1	6.6
R02(SB)	415	30.5	23.4	46.1	3.0	84.2	13.0	2.7	81.6	15.1	3.3
R11	130	45.0	20.8	34.2	2.6	69.8	18.3	11.9	62.9	27.4	9.7
T01	150	55.3	20.6	24.1	2.4	72.0	26.5	1.5	68.9	29.5	1.6
T02	35	82.1	8.7	9.2	2.1	60.9	38.6	0.5	33.3	66.7	0.0
U07	300	30.3	20.6	49.1	3.5	77.3	16.8	5.9	64.3	27.6	8.1
W01	305	36.0	25.0	39.0	2.7	77.6	19.1	3.4	74.5	19.4	6.0
Y03(NB)	280	33.8	24.5	41.7	2.7	89.5	10.5	0.0	88.6	11.4	0.0
Y03(SB)	330	35.1	21.3	43.6	3.1	94.1	5.6	0.3	90.3	9.7	0.0
Y04(NB)	340	26.8	22.3	50.9	3.3	92.7	7.0	0.3	91.4	8.1	0.5
Y04(SB)	250	41.8	21.8	36.4	2.7	95.0	4.8	0.2	93.1	6.9	0.0
Average		43.0	20.0	37.0	2.9	80.4	15.8	3.8	75.3	20.7	4.0

[a] Including both platoon leaders and platoon members.

chose the right lane, as opposed to a clear majority of motorists at the other sites studied. It appears that in order to operate effectively at higher flow rates the geometrics of the lane addition should encourage motorists to enter the right lane, so that the motorists using the left lane are a self-selected group with higher desired speeds who are motivated to pass. Because of the unique character of this site, it was classified as an outlier, and omitted from the operational analyses that follow.

While the previous discussion has focused on the average effectiveness of passing lanes in reducing vehicle platooning, it has been emphasized that this effectiveness varies over a range of values. It would be most desirable if these variations in effectiveness could be predicted as a function of geometric and traffic variables. Several predictive models were developed from the available data using multiple regression analysis. A first attempt was made to develop a regression relationship between the upstream-downstream reduction in the percent of vehicles platooned and flow rate, but the correlation between these variables was found to be not statistically significant ($r = 0.086$, $p = 0.44$)*. However, a valid regression model was developed to predict the change in platooning from the upstream percent of vehicles platooned and the passing lane length. This model is:

$$\Delta PL = 3.81 + 0.10\ UPL + 3.99\ LEN \qquad (1)$$

where,

- ΔPL = Difference in percent of vehicles platooned between upstream and downstream of passing lane,
- UPL = Percent of vehicles platooned upstream of passing lane, and
- LEN = Length of passing lane (miles).

This model explains 33% of the variation in the dependent variable (i.e., $R^2 = 0.33$). A positive value of ΔPL represents a reduction in platooning.

The percent of vehicles platooned upstream of a passing lane (UPL) has the strongest correlation with ΔPL of any of the independent variables considered. UPL represents the combined influence of traffic volume, vehicle mix and upstream geometrics on the traffic entering the passing lane. The use of UPL as a predictor of passing lane effectiveness is quite appropriate because, using the revised HCM procedures, UPL can be interpreted directly as the upstream level of service. The positive sign on the regression coefficient of UPL in Equation (1) indicates that the effectiveness of a passing lane increases as the traffic entering the passing lane becomes more congested.

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* Here, and in subsequent discussions, the significance level (p) from a statistical test is cited. A value of p less than 0.05 indicates a result that is statistically significant at the 95% confidence level, a value less than 0.10 corresponds to the significance at the 90% confidence level, etc.

The model presented in Equation (1) also demonstrates that the effectiveness of a passing lane in reducing platooning also increases with passing lane length. The influence of passing lane length has been represented in Equation (1) as a linear term; however, it is expected conceptually that passing lane length has a nonlinear relationship to the effectiveness of a passing lane in reducing platooning, with shorter lanes being more effective per unit length than longer ones. The data currently available are not sufficient to model this nonlinear aspect of passing lane length, but it merits further investigation.

Figure 7 illustrates the predictive model represented by Equation (1) and shows the variation of the reduction in the percent of vehicles platooned as a function of the upstream percent of vehicles platooned and the passing lane length. For example, it can be seen in the figure that a one mile passing lane with 40% of the entering traffic platooned would be expected to reduce platooning by 11.8%.

Equation (1) was intended to be applicable to passing lanes located in level or gently rolling terrain. A subsidiary analysis was conducted to assure that the modeling results obtained were not unduly affected by upgrades located at a few of the sites. Only 4 of the 15 sites on which Equation (1) is based included substantial upgrades; however, one of these 4 sites (Site U07) contains an 8% upgrade and could actually be classified as a climbing lane. The analyses used to develop Equation (1) were repeated without these four sites and comparable results were obtained. Thus, Equation (1) is applicable to passing lanes in level or rolling terrain, but is not necessarily applicable to truck climbing lanes.

Several additional models were tried in an effort to find a model that explained more of the variance in ΔPL than Equation (1). It was found that when flow rate was added to the model presented in Equation (1), the resulting model explained 55% of the variance in ΔPL (i.e., $R^2 = 0.55$). This model is presented in Equation (2):

$$\Delta PL = 7.64 - 0.04\ FLOW + 0.45\ UPL + 4.82\ LEN \qquad (2)$$

for,

$$FLOW \leq 700 \text{ vph}$$

where,

FLOW = Flow rate in treated direction (vph)

and the remaining variables are as previously defined. A conceptual drawback of Equation (2) is that the negative sign of the regression coefficient for flow rate implies an inverse relationship between flow rate and ΔPL, which seems counter-intuitive; however, it should be noted that such an inverse relationship applies only if UPL and LEN are held constant. The unexpected negative sign for the coefficient of the flow rate term results because flow rate and UPL are strongly correlated with one another ($r = 0.89$, $p < 0.0001$). When two variables are so strongly correlated, it is best to use only one of them in a regression model. In this case, UPL is the better predictor of ΔPL and, therefore, Equation (1) is recommended as the best predictive model for ΔPL.

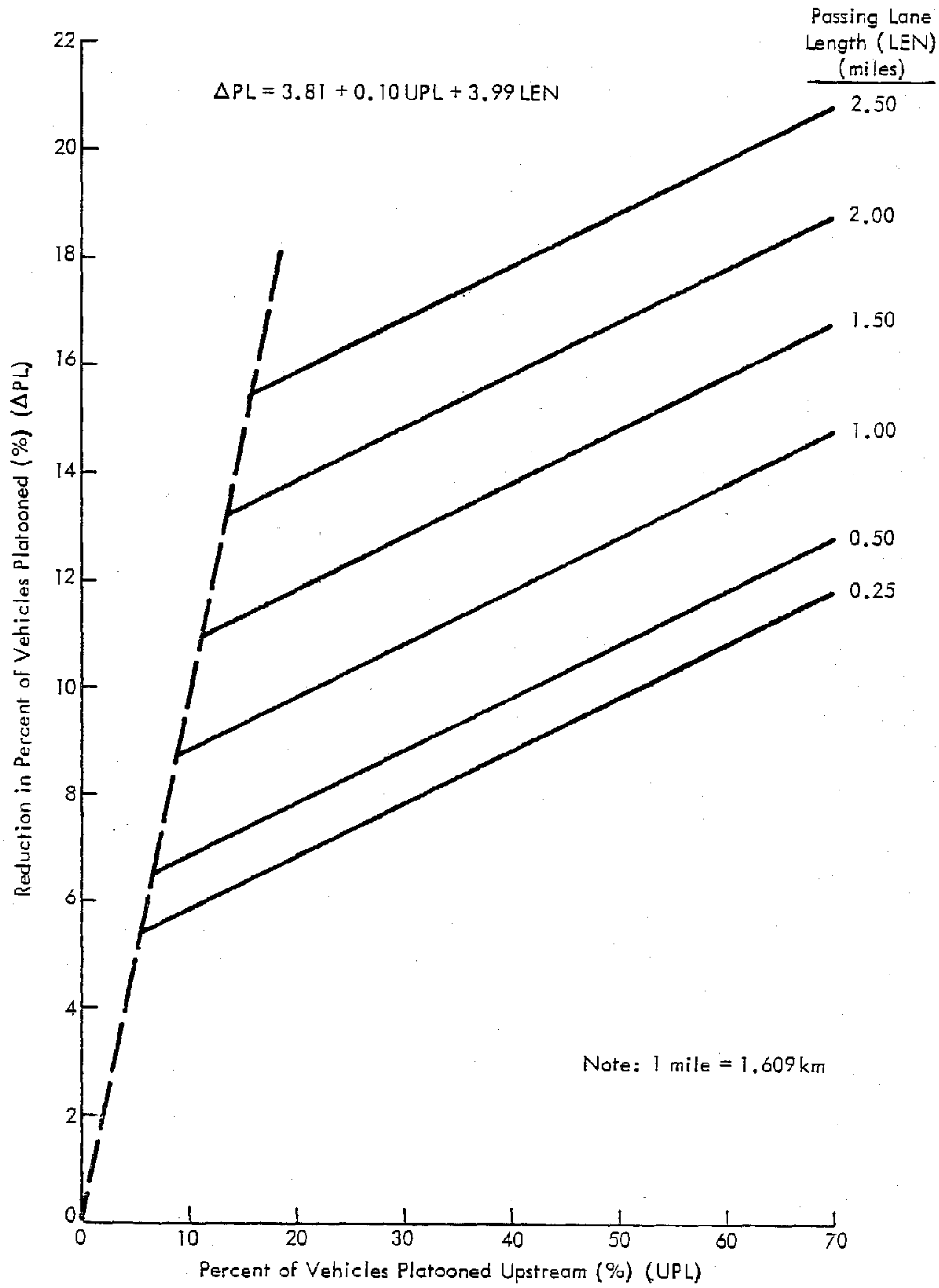


Figure 7 - Relationship to Predict Reduction in Percent of Vehicles Platooned as a Function of Upstream Percent of Vehicles Platooned and Passing Lane Length

Traffic speed: The analysis of traffic speed was based on comparisons between the mean speed immediately upstream of the passing lane, within the passing lane, and immediately downstream of the passing lane. The pattern of changes in mean speeds at passing lanes is shown in Table 6.

Mean speeds were found to be affected, on the average, only slightly by the presence of the passing lane. Mean speeds were found to be approximately 2.2 mph (3.5 kph) higher within a passing lane than upstream of the lane, and 0.9 mph (1.4 kph) higher downstream of a passing lane than upstream of it. These results indicate a small operational benefit in increased speeds due to passing lane, although as suggested in the revised HCM, it appears that vehicle platooning is a more sensitive measure of the effect of passing lanes than is mean speed.

The effect of a passing lane on traffic speed was found to vary widely from site to site. As shown in Table 6, these variations in passing lane effectiveness can range from an increase of 8.3 mph (13.4 kph) in mean speed to a decrease of 6.7 mph (10.8 kph) in mean speed between upstream and downstream of a passing lane. This wide range of differences in upstream-to-downstream speeds suggest that vehicle speeds are influenced more strongly by local geometrics at the upstream and downstream measurement sites than by the presence of a passing lane. Spot speeds are more sensitive to local geometrics than platooning measures because drivers can quickly adjust their speed in response to an external influence, while vehicle platoons require time to develop.

Several attempts were made to model the effect of passing lanes on mean speed, in a manner similar to Equations (1) and (2) for vehicle platooning. However, the relationships obtained from these analyses were considered to be unreliable for predicting the effectiveness of passing lanes, because the underlying data are influenced so strongly by local geometrics.

Tables 7 and 8 illustrate some overall speed measures for all of the passing lane and short four-lane sites taken as a whole. Table 7 compares the mean speeds of all vehicles with the 15th and 85th percentile speed at each measurement location. Table 8 compares the mean speeds of vehicles by vehicle type and platooning status at each measurement location.

The primary finding of the speed analysis is that, as suggested by recent research, changes in traffic service on two-lane highways appear to be better described by corresponding changes in the percent of vehicles platooned rather than by relatively small changes in mean speed.

Passing rate: The rate of completed passes per hour per mile was determined for all or a selected portion of each passing lane and short four-lane section. The following analysis is based on the assumption that, where passing maneuvers were observed for only a portion of an added lane, the portion of the lane studied is representative of the lane as a whole.

TABLE 6

CHANGES IN MEAN VEHICLE SPEEDS AT PASSING LANES

		Mean Speed of All Vehicles (mph)					
						Change in Mean Speed	
Site	Average Flow Rate (vph)	Immediately Upstream	Within Passing Lane	Immediately Downstream	One Mile Downstream	Within-Upstream	Downstream-Upstream
A01	140	53.0	52.7	53.1	53.3	-0.3	0.1
C05	560	56.9	63.1	61.0	57.7	6.2	4.1
C06	120	47.9	54.0	49.8	50.7	6.1	1.9
C08	120	45.9	57.7	53.1	55.4	11.8	7.2
H11	80	58.7	56.6	57.6	57.3	-2.1	-1.1
N01	150	54.6	50.3	55.5	57.3	-4.3	0.9
P04	300	56.8	59.2	55.5	57.5	2.4	-1.3
R02(NB)	410	55.8	58.6	55.9	55.4	2.8	0.1
R02(SB)	415	54.4	58.1	55.6	-	3.7	1.2
R11	130	56.6	59.2	59.0	59.4	2.6	2.4
T01	150	48.0	56.5	56.3	58.2	8.5	8.3
T02	35	54.6	51.1	47.9	50.6	-3.5	-6.7
U07	300	51.9	56.1	55.1	51.6	4.2	3.2
W01	305	53.6	54.9	57.9	58.2	1.3	4.3
Y03(NB)	280	49.4	54.2	-	-	4.8	-
Y03(SB)	330	49.4	55.6	50.9	53.4	6.2	1.5
Y04(NB)	340	55.0	54.9	51.2	-	-0.1	-3.8
Y04(SB)	250	50.7	53.2	53.7	52.5	2.5	3.0
Average[b]		53.8	56.0	54.7	55.2	2.2	0.9

[a] Combined data for left and right lanes in treated direction near center of passing lane.
[b] Average of hour-by-hour data, rather than site-by-site data tabulated above.

Note: 1 mile = 1.609 km.

TABLE 7

MEAN, 15th PERCENTILE AND 85th PERCENTILE SPEEDS (MPH) FOR ALL PASSING LANE SITES COMBINED

Speed Measure	Immediately Upstream	Within Passing Lane	Immediately Downstream	One Mile Downstream
15th Percentile Speed	47	50	48	50
Mean Speed	54	56	55	55
85th Percentile Speed	59	63	61	61

Note: 1 mile = 1.609 km.

TABLE 8

MEAN SPEEDS (MPH) OF ALL VEHICLES, FREE VEHICLES AND PLATOON LEADERS FOR ALL PASSING LANE SITES COMBINED

Vehicle Category	Immediately Upstream	Within Passing Lane	Immediately Downstream	One Mile Downstream
All Vehicles	53.8	56.0	54.7	55.2
Free Vehicles	54.8	55.9	55.2	56.2
Platoon Leaders	52.0	56.8	55.1	54.3

Note: 1 mile = 1.609 km.

Treated direction: The passing rates in the treated direction was found to range from 0 to 219.3 passes per hour per mile or 0 to 136.3 passes per hour per kilometer. The passing rate was found to have a strong relationship to flow rate, represented by the regression model:

$$PR = 13.0 + 0.223\ FLOW \tag{3}$$

for,

$$50\ vph \leq FLOW \leq 400\ vph$$

where,

PR = Passing rate (completed passes per hour per mile) in treated direction, and

FLOW = Flow rate (vph) in treated direction.

This model explains 47% of the variance in the dependent variable (i.e., $R^2 = 0.47$).

Figure 8 compares the passing rate predicted by Equation (3) for passing lanes with a corresponding relationship for one direction of a conventional two-lane highway adapted from a relationship presented in the 1950 Highway Capacity Manual.[12] Although the latter relationship is of questionable value, and was omitted from the 1965 Highway Capacity Manual, the comparison does serve to illustrate that passing lanes provide much higher passing rates than would be possible on a conventional two-lane highway.

An improved regression model for predicting the passing rate in the treated direction was obtained by adding two additional independent variables -- passing lane length and upstream percent of vehicles platooned -- to the model. The revised model for passing rate in the treated direction is:

$$PR = 0.127\ FLOW - 9.64\ LEN + 1.35\ UPL \tag{4}$$

for,

$$50\ vph \leq FLOW \leq 400\ vph$$

where,

PR = Passing rate (passes per hour per mile) in treated direction,

FLOW = Flow rate (vph) in treated direction,

LEN = Passing lane length (miles), and

UPL = Percent of traffic platooned at upstream end of passing lane.

This model explains 83% of the variance in the dependent variable ($R^2 = 0.83$).

The model presented in Equation (4) shows that the passing rate increases with increasing flow rate and with increasing upstream percent of vehicles platooned. The model also shows that the passing rate

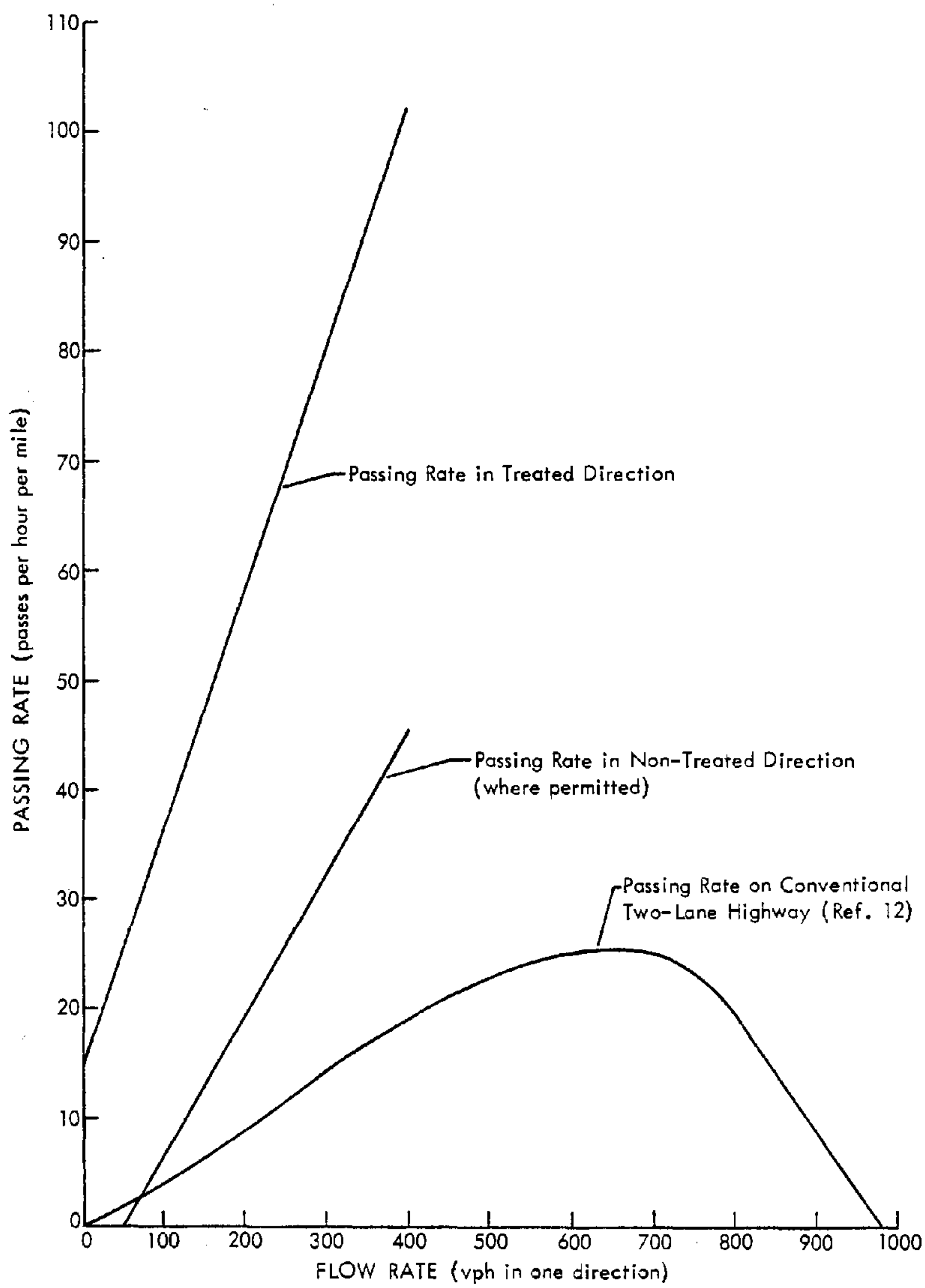


Figure 8 - Passing Rates in Treated and Nontreated Direction of Passing Lanes Compared to Conventional Two-Lane Highways

decreases with increasing passing lane length. This finding tends to confirm the hypothesis that the passing rate is highest near the beginning of a passing lane and decreases to a lower, steady state level at some distance into the lane. The model presented in Equation (4) contains no intercept term, since this term was found to be not statistically significant.

Two brief case studies, in which passing rate data were collected in different portions of the same passing lane, illustrate that the passing rate can change along the length of a passing lane in different ways that are dependent on geometrics and traffic conditions.

Site C08 is a passing lane one mile (1.6 km) in length located on a heavily recreational route, US Route 395 in the Walker Canyon in eastern California. The passing lane is located on a steady 1-3% downgrade, so vertical geometrics do not have a major influence on the traffic operations on this site. However, the approach width upstream of the passing lane is only 10.5 ft (3.2 m) with a narrow 1-ft (0.3-m) shoulder. There are sharp horizontal curves both on the approach and in the passing lane itself. Upstream passing opportunities are very limited. Because of these restrictions, traffic enters the passing lane with a mean speed of only 45.9 mph (73.9 kph) and with 43.4% of the traffic platooned. The flow rate is 120 vph. The passing rate in the first 0.21 mile (0.34 km) of this passing lane is 171.4 passes per hour per mile (106.5 passes per hour per km), despite restricted sight distance due to a horizontal curve. On a 0.34-mile (0.55-km) section later in the passing lane, with no sight distance restriction, the passing rate is only 76.4 passes per hour per mile (122.9 passes per hour per km). Thus, with a large pent-up demand and relatively slow approach speeds, many drivers decide to pass immediately upon entering the passing lane.

By contrast, Site R02 is a short four-lane section, 1.02 miles (1.64 km) in length, located on US Route 97 between the communities of Bend and Redmond in Central Oregon. The northbound approach to the treated section has 12-ft lanes with no significant grades or horizontal curvature. Thus, traffic enters the added lane at a mean speed of 56.2 mph (90.4 kph), higher than for Site C08, although even more vehicles (51.7%) are platooned because of the higher flow rate (410 vph). The first third of the added lane is located on a slight upgrade to a vertical crest that limits the sight distance to the end of the lane; the final two-thirds of the added lane is located on a 2% downgrade with the end of the lane in view, although initially at a considerable distance. The passing rate in the initial third of the lane, prior to crest vertical curve, is 102.7 vehicles per hour per mile (63.8 passes per hour per km). The passing rate increases to 115.7 passes per hour per mile (71.9 passes per hour per km) in the final two-thirds of the lane. However, based on data for the opposing direction of travel, it is estimated that the observed passing rate of 115.7 passes per hour per mile (71.9 passes per hour per km) is the combined result of a passing rate of approximately 184 passes per hour per mile (114.4 passes per hour per km) in the middle third of the added lane and 48 passes per hour per mile (29.8 passes per hour per km) in the final third of the added lane. These data show that with the higher speed of traffic entering the added lane, drivers are less able to complete their passes in the early portion of the added lane, so that the middle portion of the added lane becomes the most heavily used for passing.

The comparison provided by these case studies illustrates the strong influence that upstream geometrics and traffic conditions have on the manner in which a passing lane operates.

Nontreated direction: Passing rates in the nontreated direction were also studied for the 12 passing lane sites. Passing by opposing direction vehicles is permitted at 6 of the 12 passing lane sites and prohibited at the remaining 6.

For passing lanes where passing is permitted in the nontreated direction, the passing rate varied from 0 to 50.0 passes per hour per mile (0 to 31.1 passes per hour per km). At these sites, there is a strong linear relationship between the passing rate and the flow rate in the nontreated direction. The regression model for this relationship is:

$$OPR = -6.97 + 0.13\ OFLOW \qquad (5)$$

for,

$$50\ vph \leq OFLOW \leq 400\ vph,$$

where,

OPR = Passing rate in opposing direction (passes per hour per mile), and

OFLOW = Flow rate in nontreated direction (vph).

This model explains 71% of the variation in the dependent variable (i.e., $R^2 = 0.71$).

The model represented by Equation (5) has a negative intercept, which is conceptually unappealing because it implies that the nontreated direction passing rate could be negative at low flow rates. In practice, however, the model predicts negative passing rates in the nontreated direction only for flow rates below 50 vph. For this reason, the model should be used only for flow rates above 50 vph. The best estimate of the opposing direction passing rate for flow rates below 50 vph is zero.

Figure 8 illustrates that the passing rate in the nontreated direction of a passing lane is substantially less than the passing rate in the treated direction, but is higher than the passing rate for a conventional two-lane highway.

Apparently, more passes occur in the opposing direction of a passing lane than on a conventional two-lane highway because there are more passing opportunities available when the oncoming traffic can use two lanes rather than one.

The prohibition of passing in the opposing direction of a passing lane places that direction of travel at a distinct operational disadvantage. Despite the prohibition, a limited number of passing maneuvers do occur. Thirty-four (34) hours of data were available for the 6 sites with opposing direction passing prohibited. No passing maneuvers by opposing direction vehicles were observed during 21 of these 34 hr and no opposing direction passing maneuvers at all were observed at 2 of the 6 sites. Passing maneuvers in the opposing direction, in violation of the passing prohibition, were observed during 13 of the 34 hr of available data, although in 7 of these hours there was only a single passing maneuver. The passing rates in the opposing direction during these 13 hr ranged from 1.1 to 18.5 passes per hour per mile (0.7 to 11.5 passes per hour per km). No statistically significant relationship was found between opposing direction passing rate and flow rate for passing lanes where opposing direction passing is prohibited.

2.3 Safety Evaluation

A safety evaluation of the effectiveness of passing lanes and short four-lane sections was performed. The purpose of this evaluation was to quantify the safety performance of these treatments in relation to comparable untreated sections and to detect any accident patterns or other safety problems that might limit use of these treatments. Separate safety evaluations were performed for passing lane and short four-lane sections.

2.3.1 Passing lanes: The safety evaluation of passing lanes was based on accident data collected for 66 passing lanes located in nine states. Comparisons of accident rates before and after installation of passing lanes were performed for 22 of the 66 sites and comparisons between passing lanes and comparable untreated locations were performed for 13 passing lanes.

Accident data were requested from the participating states for a period of 5 years. However, it was not always possible to obtain data for a complete 5-year period, either because the required data were not available or because the passing lane was recently constructed. To avoid seasonal effects, only data for complete 12-month periods were used; no partial-year accident samples were included. The average length of the accident study period for the 66 passing lane sites was 3.59 years.

Each accident in a passing lane section was reviewed manually and assigned a code based on the location within the passing lane section at which it occurred. The eight locations coded for passing lanes were:

- Treated direction - upstream
- Treated direction - lane addition
- Treated direction - passing lane
- Treated direction - lane drop
- Treated direction - downstream
- Nontreated direction - upstream
- Nontreated direction - opposite passing lane
- Nontreated direction - downstream

These location codes were assigned, in part, based on the accident coordinates (mileposts) assigned by the state highway agency and, in part on descriptive information provided by police accident reports, accident summaries and collision diagrams. Accidents involving vehicles traveling in opposite directions were assigned to the treated or nontreated direction based on the direction of travel of the offending or "at fault" vehicle. A few accidents where neither vehicle could be identified as "at fault" were assigned to the treated direction.

Comparisons between treated and untreated sites: Table 9 compares the mean accident rates for the treated and nontreated directions of passing lanes and for untreated two-lane highway. The accident rates for passing lanes include only the passing lane section and do not include the upstream approaches or the downstream portions of each site. The table also includes the number of accidents and the exposure (million vehicle-miles of travel) during the study period for the passing lanes and comparable locations, from which the accident rates were computed. The means presented in Table 9 indicate that the accident rates in passing lanes are slightly higher in the treated than in the nontreated direction and that passing lanes have slightly lower accident rates than untreated two-lane highways. However, none of the differences between the means shown in Table 9 is statistically significant.

Table 10 illustrates a matched pair comparison between passing lane sites and comparable untreated sites. The untreated sites were selected by the participating states as comparable to the passing lane sites to which they are matched. Each comparable site is located in the same geographical region of the state and, in most cases, is located on the same highway as its matched passing lane site. Table 10 shows that, in all but two cases, the treated site has a lower accident rate than the comparable untreated site. The passing lane sites were, on the average, 38% less than the comparable sites in total accident rate and 29% less than the comparable sites in fatal and injury accident rate. The observed difference in total accident rate was statistically significant at the 95% confidence level, but the difference in fatal and injury accident rate was not statistically significant.

Comparisons between passing lane sites before and after treatment installation: A more direct comparison of the effect of passing lane installation can be made by comparing the accident rates of passing lane sites before and after installation of the passing lane. Table 11 presents the accident rates before and after installation of passing lanes at 22 of the 70 sites in the current study. On the average, passing lane installation reduced the total accident rate by 8.7% and the fatal and injury accident rate by 17.0%. These differences were not found to be statistically significant, however, because of the high variability in the before-after accident rate differences. Accident rates were observed to increase with passing lane installation at 8 of the 22 passing lane sites. Such high variability in observed results is common in accident studies when short study periods are used; at several sites, including the eight passing lanes at Site A01, only 1 year of accident data was available in the after period.

The likelihood of finding the observed reduction in accident rate to be statistically significant can be increased by lengthening the study period or by increasing the sample size.

TABLE 9

COMPARISON OF ACCIDENT RATES FOR PASSING LANES AND UNTREATED TWO-LANE HIGHWAYS

Type of Location	Number of Sites	Number of Accidents		Exposure (MVM)	Mean Accident Rate[a] (accidents per MVM)	
		Total	Fatal and Injury		Total	Fatal and Injury
Passing Lane - Treated Direction[a]	66	305	133	271.0	1.13	0.49
Passing Lane - Nontreated Direction	66	227	95	242.5	0.94	0.39
Passing Lane - Both Directions Combined	66	532	228	513.5	1.04[b]	0.44[b]
Untreated Two-Lane Highways - Both Directions Combined	13	430	226	273.5	1.57	0.83

[a] Including lane addition and lane drop transition areas.
[b] Based on average of treated and nontreated direction accident rates.

Note: 1 mile = 1.609 km.

TABLE 10

MATCHED PAIR COMPARISON OF ACCIDENT RATES FOR PASSING LANES AND COMPARABLE TWO-LANE HIGHWAYS

Passing Lane Site(s)	Comparable Site	Total Accident Rate (accidents per MVM)			Fatal and Injury Accident Rate (accidents per MVM)		
		Treated	Comparable	Difference	Treated	Comparable	Difference
A01	A11	1.599	1.059	0.540	0.754	0.488	0.266
C05	C15	0.644	0.757	-0.113	0.301	0.451	-0.150
C07/C08	C17	0.989	2.346	-1.357	0.629	1.300	-0.671
C09	C19	1.989	0.747	1.242	1.226	0.560	0.666
H01	H02	1.701	2.248	-0.547	1.701	1.985	-0.284
H03	H04	0.000	1.250	-1.250	0.000	0.000	0.000
H05	H06	0.000	0.846	-0.846	0.000	0.000	0.000
H07	H08	0.000	0.917	-0.917	0.000	0.456	-0.456
H09	H10	0.769	0.913	-0.144	0.769	0.000	0.769
M01	M11	1.369	2.237	-0.868	0.222	0.652	-0.430
N01/N06/N07	N11	1.066	1.776	-0.710	0.215	0.484	-0.269
R11/R13	R12	0.842	1.826	-0.984	0.455	1.494	-1.039
W06/W07	W14	1.421	3.090	-1.669	0.741	2.060	-1.319
Mean Values		0.953	1.539	-0.586 SIG t(12)=-2.64 (p=.02)	0.539	0.764	-0.224 NS t(12)=-1.35 (p=.22)

Note: 1 mile = 1.609 km.

TABLE 11

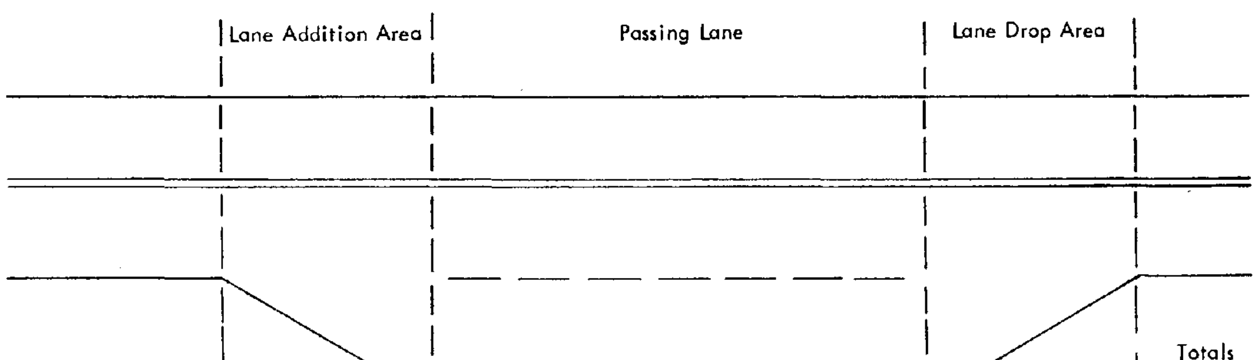
BEFORE/AFTER ACCIDENT RATE COMPARISON OF PASSING LANE INSTALLATION
(data from California study[14] and from current study)

Site Number and Treatment Number	Total Accident Rate (accidents per MVM) Before	After	Difference	% Difference	Fatal and Injury Accident Rate (accidents per MVM) Before	After	Difference	% Difference
CURRENT STUDY								
A01-1	0.787	1.338	0.591	75.1	0.093	0.669	0.576	619.3
A01-2	1.500	2.151	0.651	43.4	0.750	1.075	0.325	43.3
A01-4	1.364	0.000	-1.364	-100.0	0.909	0.000	-0.909	-100.0
A01-5	2.131	4.008	1.877	88.1	1.332	2.004	0.672	-50.4
A01-6	1.938	1.122	-0.816	-42.1	1.118	0.561	-0.557	-49.8
A01-7	1.131	0.493	-0.638	-56.4	1.097	0.493	-0.604	-55.1
A01-8	2.132	1.505	-0.627	-29.4	1.199	0.502	-0.697	-58.1
A01-9	1.066	1.944	0.878	82.3	0.178	0.000	-0.178	-100.0
M01-1	0.712	1.603	0.891	125.1	0.356	0.267	-0.089	-25.0
M01-2	1.196	1.461	0.265	22.2	0.654	0.426	-0.228	-34.9
M01-3	2.563	1.081	-1.482	-57.8	0.641	0.241	-0.400	-62.4
M02-1	2.372	1.334	-1.038	-43.8	1.285	0.148	-1.137	-88.5
M03-1	3.260	1.540	-1.720	-52.8	1.268	0.000	-1.268	-100.0
M03-2	2.537	5.073	2.536	100.0	0.000	4.711	4.711	-
M04-1	6.473	4.549	-1.924	-29.7	2.099	1.255	-0.874	-41.6
M05-1	2.079	0.650	-1.429	-68.8	0.649	0.260	-0.389	-59.9
N01-1	1.454	0.636	-0.818	-56.2	0.726	0.000	-0.726	-100.0
N03-1	3.312	3.283	-0.029	-0.9	1.142	0.657	-0.485	-42.5
N06-1	2.066	1.269	-0.797	-38.6	1.377	0.000	-1.377	-100.0
N07-1	2.066	1.293	-0.773	-37.4	0.345	0.646	0.301	87.2
W11-1	2.114	3.207	1.093	51.7	1.058	0.846	-0.212	-20.0
W11-2	1.056	1.759	0.703	-66.6	0.469	0.821	0.352	-75.1
Mean Values	2.058	1.877	-0.180 NS t(21)=0.70 (p=.25)	-8.7	0.852	0.707	-0.145 NS t(21)=0.55 (p<.25)	-17.0
CALIFORNIA STUDY[14]								
5	5.15	3.93	-1.22	-23.7				
18	3.60	1.71	-1.89	-52.5				
24	7.65	1.33	-6.32	-82.6				
35	2.25	1.87	-0.38	-16.9				
37	3.42	0.21	-3.21	-93.9				
38	0.70	0.93	0.23	32.9				
14A	0.00	2.22	2.22	-				
17A	1.30	1.30	0.00	0.0				
18A	0.00	0.99	0.99	-				
16	2.45	1.03	-1.42	-58.0				
19	1.87	0.93	-0.94	-50.3				
15A	0.99	0.68	-0.31	-31.3				
24A	1.55	1.01	-0.54	-34.8				
Mean Values	2.33	1.47	-0.983	-42.2				
Combined Mean for Both Studies	2.18	1.65	-0.529 SIG t(34)=1.97 (p=0.06)	-24.3				

The latter approach was used by including in the analysis 13 sites from an evaluation of passing lanes conducted by the California Department of Transportation.[14] Only passing lanes identified in the California study as located in level or rolling terrain were used; sites located in mountainous terrain, which are essentially climbing lanes, were excluded. The 13 passing lanes from the California study experienced an average reduction in total accident rate of 42.2% from before to after passing lane installation. The combined data set consisting of 35 passing lanes from the current study and the California study experienced an average reduction in total accident rate of 24.3% with passing lane installation; this reduction was found to be statistically significant at the 90% confidence level.

It is clear from the data collected in this study that the installation of passing lanes does not increase accident rates. The available data strongly suggest, but do not prove conclusively, that the installation of passing lanes may decrease accident rates. The statistical significance of accident rate reductions can be demonstrated when the data collected in this study and the data collected in the previous California study are considered together. It should be noted that the observed accident rate reductions may be the combined result of the construction of an additional lane and other geometric improvements such as lane widening, shoulder widening, and minor changes in superelevation or alignment that often accompany installation of passing lanes.

<u>Lane addition and lane drop transition areas</u>: A separate investigation was made of accidents in the lane addition and lane drop taper areas of passing lanes to determine whether there are any particular safety problems in those areas. Of the 305 accidents that occurred in the treated direction of passing lanes, 48 accidents were found to occur in the first 800 ft (0.15 mile or 0.24 km) of the passing lane and 51 accidents were found to occur in the final 800 ft (0.15 mile or 0.24 km). Figure 9 illustrates the distribution of accidents between different areas of a typical



	Lane Addition Area	Passing Lane	Lane Drop Area	Totals
Number of Accidents in Treated Direction	48	206	51	305
% of Accidents	15.7%	67.6%	16.7%	100.0%
Length (miles)	0.15	0.80	0.15	1.10
% of Total Passing Lane Length	13.6%	72.7%	13.6%	100.0%

1 Mile = 1.6 km

Figure 9 - Distribution of Accidents Along a Passing Lane Section

passing lane. There is no indication that accidents are more likely in one transition area than the other. A slightly greater proportion of accidents occur in the transition areas than would be expected from their relative length alone, but the differences are not large. Thus, there is no indication of any marked safety problem in the lane addition and lane drop transition areas of passing lanes.

Traffic conflict studies were performed at 10 lane drop transition areas. A traffic conflict in a lane drop transition area is a situation in which a vehicle is required to take evasive action, to brake or swerve to avoid an impending collision with another vehicle ahead or alongside. A brake light indication, obvious braking, or swerving by the offended vehicle are indications of a traffic conflict. Figure 10 illustrates the types of traffic conflicts and erratic maneuvers noted by the manual observers at lane drop transition areas. Five types of traffic conflicts were observed in lane drop taper areas: lane-change conflicts, slow-to-merge conflicts, slow-moving vehicle conflicts, stopped-vehicle conflicts, and head-on conflicts. A <u>lane-change conflict</u> is a situation in which a vehicle changes lanes, causing another vehicle to brake or swerve to avoid collision. A <u>slow-to-merge conflict</u> occurs when a vehicle in the dropped lane brakes before changing lanes and then causes a vehicle in the other lane to brake or swerve. A <u>slow-moving vehicle conflict</u> occurs when a vehicle brakes or swerves to avoid a slower vehicle in front of it. A <u>stopped-vehicle conflict</u> occurs when a vehicle approaching the lane drop area encounters a stopped vehicle in the closed lane and brakes, swerves or stops to avoid collision with the stopped vehicle. A <u>head-on collision</u> occurs when a vehicle in the lane drop area or an oncoming vehicle is forced to brake or swerve to avoids collision with the other.

Erratic maneuver counts were also made as part of the lane drop transition studies. Erratic maneuvers are unusual driving actions by a single vehicle. The two types of erratic maneuvers observed for vehicles in lane drop transition areas were <u>centerline encroachments</u> and <u>shoulder encroachments</u>.

Table 12 illustrates the observed conflict and erratic maneuver rates. The most common type of traffic conflict observed in lane drop transition areas was the slow-moving vehicle conflict, which was observed for 0.8% of the vehicles passing through the lane drop area. The total traffic conflict rate for the lane drop areas was 1.3%, while the encroachment rates were 0.4 and 0.3% for centerline and shoulder encroachments, respectively. These conflict and encroachment rates are much smaller than the rates found in lane drop transitions at other locations of the highway system. For example, lane drop transition tapers in work zones can experience conflict rates ranging from 5 to 15%.[4,5]

Although there is no evidence of a safety problem in lane drop transition areas based on the accident, traffic conflict and erratic maneuver studies presented here, it is obvious that such transition areas should be carefully designed to prevent safety problems from developing. Many agencies that use alternating passing lanes either overlap the passing lanes in the opposite direction or provide buffer areas between them to avoid a direct taper transition between passing lanes in opposite directions.

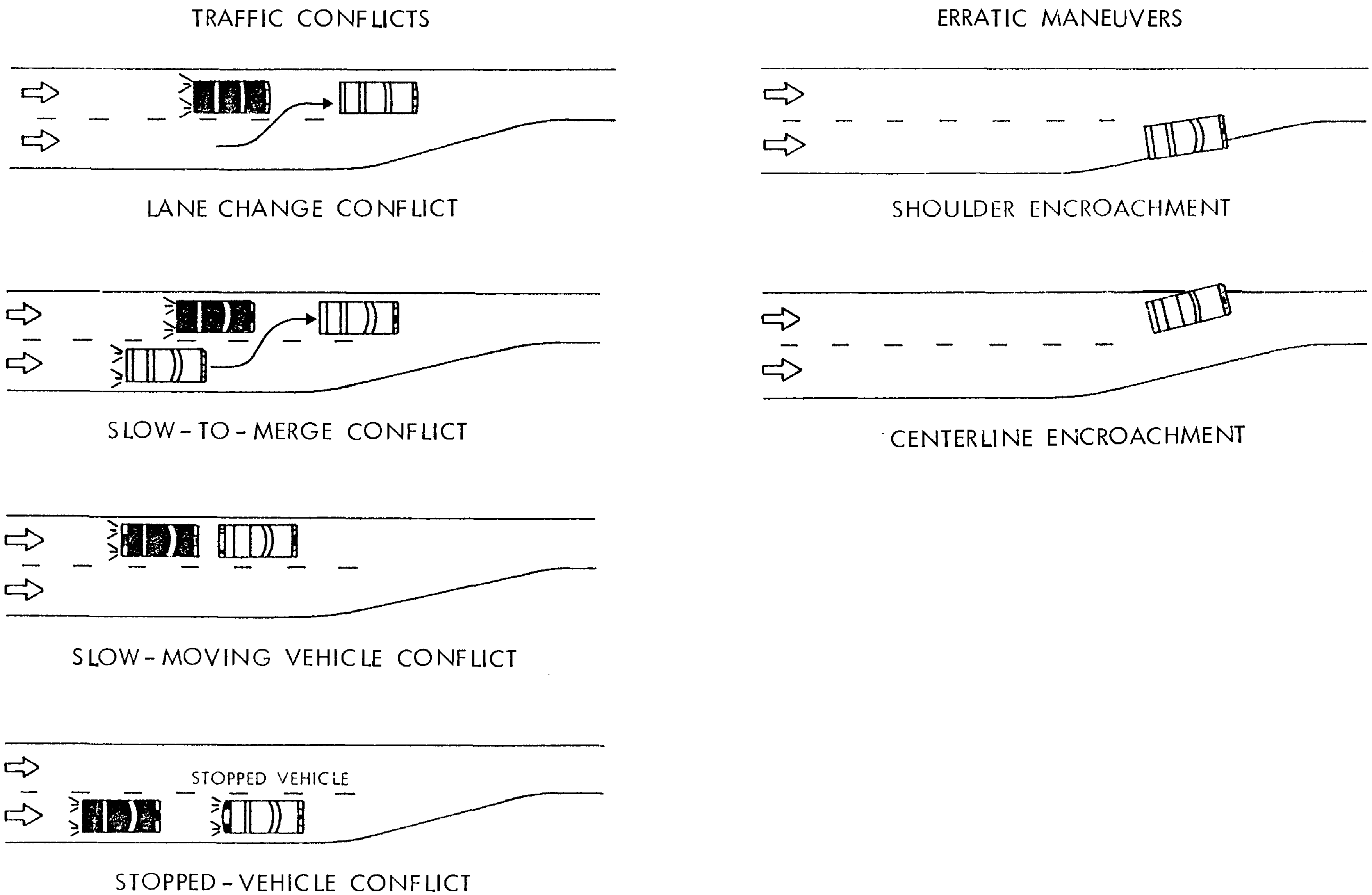


Figure 10 - Traffic Conflict and Erratic Maneuver Types Evaluated in Lane Drop Transition Areas

TABLE 12

TRAFFIC CONFLICT AND ENCROACHMENT RATES IN LANE DROP TRANSITION AREAS FOR PASSING LANES

Site Number	Length of Lane Drop Taper (ft)	Flow Rate (veh/hr)	Conflict Rate (per 100 vehicles) Lane Change	Slow-to-Merge	Slow-Moving Vehicle	Stopped Vehicle	Head-On	Total	Encroahcment Rate (per 100 vehicles) Centerline	Shoulder
A01	370	140	0.00	0.24	0.72	0.00	0.00	0.95	0.00	0.00
C05	700	560	0.15	0.33	0.50	0.00	0.00	0.98	0.21	0.00
C06	430	120	0.00	0.00	1.13	0.00	0.00	1.13	0.00	0.00
H11	665	80	0.40	0.20	0.60	0.20	0.00	1.40	0.20	1.21
P04	300	300	0.50	0.12	1.40	0.00	0.00	2.02	0.17	0.00
R11	600	120	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
T01	270	150	0.54	0.87	1.84	0.00	0.00	3.25	1.41	0.43
T02	650	35	0.48	0.00	0.48	0.00	0.00	0.97	2.42	0.48
U07[a]	900	300	0.82	0.17	0.93	0.00	0.05	1.97	0.44	0.44
Y03[b]	670	330	0.00	0.60	0.36	0.00	0.00	0.42	0.00	0.49
Mean Values			0.29	0.19	0.80	0.02	0.005	1.31	0.48	0.31

[a] Climbing lane site.
[b] Short four-lane site.

Note: 1 ft = 0.305 m.

Distribution of accident types: Table 13 compares the distribution of accident types for passing lanes and comparable two-lane highways. The accident type distribution for passing lanes is further subdivided into distributions for the treated and nontreated directions of travel. In Table 13, and in subsequent presentations of accident distributions in this report, accidents are classified into four categories based on the number of vehicles involved (single vehicle/multiple vehicle) and intersection involvement. The single-vehicle accidents are further classified into eight categories based on the type of object, if any, the vehicle collided with. The multiple-vehicle accidents were subdivided into five categories based on the manner of collision between the vehicles.

The most interesting observation that can be made from Table 13 is the similarity of the accident distributions for passing lanes and comparable untreated sections. None of the dominant accident types differ in their percentage of total accidents by more than a few percent. For both passing lanes and the comparable sections, over 80% of all accidents are not intersection related. The distribution of accident types for single-vehicle, nonintersection accidents on the treated and untreated sections are nearly identical. In the multiple-vehicle, nonintersection accident category, the passing lane sections have a lower percentage of head-on, rear-end and sideswipe accidents and higher percentage of angle accidents. Further discussion of two issues related to the accident types -- cross-centerline accidents (involving vehicles traveling in opposing directions) and left-turn accidents -- will be found in the next two subsections.

Cross-centerline accidents: Some agencies have been reluctant to install passing lanes on two-lane highways because of concern that passing lanes might increase the likelihood of accidents between vehicles traveling in opposite directions, which are generally quite severe. This concern may arise, in part, from adverse accident experience that occurred on three-lane highways that were common before 1960, where passing by vehicles in either direction was permitted in the center lane without a clear assignment of the right-of-way. The passing lanes in use today have two lanes marked for travel in one direction and one lane in the other direction, with a clear assignment of the right-of-way. No safety problem involving vehicles traveling in opposite directions was found for passing lanes, even for sites where passing by opposing direction vehicles is permitted.

Table 14 compares the accident rates for cross-centerline accidents on passing lanes with opposing passing prohibited, passing lanes with opposing passing permitted and comparable untreated sections. Cross-centerline accidents are defined here as all accidents that involve vehicles traveling in opposite directions; such accidents are predominantly head-on and opposing direction sideswipe collisions. No substantial differences in accident rate were found at any severity level between passing lane sections with opposing passing permitted and passing lanes with opposing passing prohibited, but both types of passing lane sections have lower accident rates than untreated two-lane highways. Thus, the provision for passing by vehicles traveling in the opposing direction to a passing lane does not appear to lead to any safety problems at the types of sites and the flow rate levels (up to 400 vph) where it has been permitted by the participating states.

TABLE 13

DISTRIBUTION OF ACCIDENT TYPES FOR PASSING LANES AND COMPARABLE TWO-LANE HIGHWAYS

Accident Type	Number of Accidents (Percent of Total Accidents): Passing Lanes, Treated Dir.		Passing Lanes, Nontreated Dir.		Passing Lanes, Combined		Comparable Two-Lane Highways	
SINGLE-VEHICLE, NONINTERSECTION	156	(51.1)	137	(60.4)	293	(55.1)	244	(53.3)
Collision with Parked Vehicle	2	(0.7)	4	(1.8)	6	(1.1)	5	(1.1)
Collision with Pedestrian	0	(0.0)	1	(0.4)	1	(0.2)	2	(0.4)
Collision with Animal	20	(6.6)	19	(8.4)	39	(7.3)	44	(9.6)
Collision with Fixed Object	94	(30.8)	69	(30.4)	163	(30.6)	131	(28.6)
Collision with Other Object	2	(0.7)	5	(2.2)	7	(1.3)	7	(1.5)
Other Collision	0	(0.0)	0	(0.0)	0	(0.0)	0	(0.0)
Noncollision, Overturning	26	(8.5)	26	(11.5)	52	(9.8)	46	(10.0)
Other Noncollision	13	(4.3)	13	(5.7)	26	(4.9)	9	(2.0)
MULTIPLE-VEHICLE, NONINTERSECTION	105	(34.4)	67	(29.5)	172	(32.3)	146	(31.9)
Head-on	21	(6.9)	6	(2.6)	27	(5.1)	27	(5.9)
Rear-end	20	(6.6)	25	(11.0)	45	(8.5)	47	(10.3)
Sideswipe	22	(7.2)	13	(5.7)	35	(6.6)	38	(8.3)
Angle	32	(10.5)	19	(8.4)	51	(9.6)	18	(3.9)
Other	10	(3.3)	4	(1.8)	14	(2.6)	16	(3.5)
SINGLE-VEHICLE, INTERSECTION	23	(7.5)	6	(2.6)	29	(5.5)	22	(4.8)
Collision with Parked Vehicle	1	(0.3)	0	(0.0)	1	(0.2)	0	(0.0)
Collision with Pedestrian	1	(0.3)	0	(0.0)	1	(0.2)	0	(0.0)
Collision with Animal	2	(0.7)	2	(0.9)	4	(0.8)	1	(0.2)
Collision with Fixed Object	11	(3.6)	1	(0.4)	12	(2.2)	10	(2.2)
Collision with Other Object	0	(0.0)	0	(0.0)	0	(0.0)	0	(0.0)
Other Collision	0	(0.0)	0	(0.0)	0	(0.0)	0	(0.0)
Noncollision, Overturning	5	(1.6)	2	(0.9)	7	(1.3)	11	(2.4)
Other Noncollision	3	(1.0)	1	(0.4)	4	(0.8)	0	(0.0)
MULTIPLE-VEHICLE, INTERSECTION	21	(6.9)	17	(7.5)	38	(7.1)	46	(10.0)
Head-on	0	(0.0)	0	(0.0)	0	(0.0)	3	(0.7)
Rear-end	8	(2.6)	3	(1.3)	11	(2.1)	4	(0.9)
Sideswipe	2	(0.7)	3	(1.3)	5	(0.9)	2	(0.4)
Angle	11	(3.6)	10	(4.4)	21	(3.9)	37	(8.1)
Other	0	(0.0)	1	(0.4)	1	(0.2)	0	(0.0)
TOTAL	305	(100.0)	227	(100.0)	532	(100.0)	458	(100.0)

TABLE 14

COMPARISON OF CROSS-CENTERLINE ACCIDENT RATES FOR PASSING LANES AND COMPARABLE UNTREATED SECTIONS

Accident Severity Level	Passing Lane Sections - Opposing Passing Prohibited			Passing Lane Sections - Opposing Passing Permitted			Comparable Untreated Sections		
	Number of Accidents	Exposure (MVM)	Accident Rate (per MVM)	Number of Accidents	Exposure (MVM)	Accident Rate (per MVM)	Number of Accidents	Exposure (MVM)	Accident Rate (per MVM)
Fatal	6	234.7	0.026	5	278.8	0.018	7	273.5	0.026
Injury	15	234.7	0.064	12	278.8	0.043	39	273.5	0.143
PDO	10	234.7	0.043	14	278.8	0.050	28	273.5	0.102
TOTAL	31	234.7	0.133	31	278.8	0.111	74	273.5	0.271

Note: 1 mile = 1.609 km.

Left-turning accidents: Accidents involving left-turning vehicles are a potential safety problem on passing lane sections. A vehicle turning left into an intersection or driveway from the treated direction of a passing lane section is in an exposed position if it must slow or stop in the left lane, which is normally the higher-speed lane, and yield to opposing traffic before completing a turn. The Pennsylvania Department of Transportation has experienced both left-turn accident problems on higher-volume highways with passing lanes and complaints from motorists who feel uncomfortable when stopped in the left or high-speed lane to make a left turn. Consequently, Pennsylvania has converted the center lane of a number of passing lane sections to a two-way left-turn lane. This type of conversion reduces the potential for left-turn accidents, but also eliminates the traffic operational benefits of the passing lane.

No data were available from Pennsylvania to characterize or quantify the problem noted above. The data provided by the remaining states were examined for any evidence of a similar problem. It was found that only 8 accidents on the 66 passing lane sections involved vehicles turning left from the treated direction. These accidents were not very severe; they included no fatal accidents, two injury accidents, and six property-damage-only accidents. Two of the eight accidents involved intersections and the remaining six were presumably driveway-related. On the other hand, the sample of untreated two-lane highways experienced 29 left-turn accidents of which none were fatal accidents, 18 were injury accidents and 18 were property-damage-only accidents. The untreated sections experienced virtually the same total travel as the treated direction of the passing lane sections (273.5 and 271.0 million-vehicles-miles of travel, respectively), so the overall exposure of the two types of sections is comparable. Unfortunately, no complete data on left-turn volumes or the number of driveways and intersections are available to permit more precise exposure measures to be used. However, on the basis of the available data, there does not appear to be a safety problem associated with left-turn accidents in passing lane sections.

2.3.2 Short four-lane sections: The safety evaluation of short four-lane sections was based on accident data collected for nine short four-lane sections in three states -- New York, Oregon, and Washington. Three of these nine sections were included in the operational field studies described earlier in this report. Accident data were also available for six untreated two-lane highway sections located near all but one of the nine treated sections.

Comparison between treated and untreated sites: Table 15 compares the overall accident experience for the treated and untreated sites. The accident rate for short four-lane sections is approximately 34% less than for the untreated sections in total accident rate and 43% less in fatal and injury accident rate, although these differences are not statistically significant. The accident rates for short four-lane sections and untreated sections presented in Table 15 are of comparable magnitude, respectively, to the accident rates for passing lanes and untreated sections presented in Table 9.

A matched pair comparison of accident rates for short four-lane sections and comparable untreated sections is presented in Table 16. In all but one case, the short four-lane section had a lower accident rate

TABLE 15

COMPARISON OF ACCIDENT RATES FOR SHORT FOUR-LANE SECTIONS AND COMPARABLE TWO-LANE HIGHWAYS

Type of Location	Number of Sites	Number of Accidents: Total	Number of Accidents: Fatal and Injury	Exposure (MVM)	Accident Rate (per MVM): Total	Accident Rate (per MVM): Fatal and Injury
Short four-lane section	9	106	69	89.6	1.18	0.77
Comparable two-lane highways	6	250	189	139.4	1.79	1.36

Note: 1 mile = 1.609 km.

TABLE 16

COMPARISON OF ACCIDENT RATES FOR SHORT FOUR-LANE SECTIONS AND COMPARABLE TWO-LANE HIGHWAYS

Short four-Lane Site	Comparable Site	Total Accident Rate (accidents per MVM): Treated	Total Accident Rate (accidents per MVM): Comparable	Total Accident Rate (accidents per MVM): Difference	Fatal and Injury Accident Rate (accidents per MVM): Treated	Fatal and Injury Accident Rate (accidents per MVM): Comparable	Fatal and Injury Accident Rate (accidents per MVM): Difference
R02	R01	0.549	0.371	0.178	0.274	0.186	0.088
Y02[a]	Y12	0.313	1.834	-1.521	0.313	1.542	-1.229
Y03	Y13	1.968	2.082	-0.114	1.475	1.511	-0.036
Y04	Y14	1.460	2.402	-0.942	1.211	1.781	-0.570
Y07[a]	Y17	0.000	0.917	-0.917	0.000	0.611	-0.611
Y08	Y18	0.000	1.441	-1.441	0.000	1.200	-1.200
Mean Values		0.715	1.508	-0.793	0.546	1.139	-0.593

[a] Combined rates for two short four-lane sections located near one another within the same site.
Note: 1 mile = 1.609 km.

than its comparable untreated section. Because of the small number of sites available, the mean difference in accident rates, although substantial, is not statistically significant for either total accidents or fatal and injury accidents.

Comparison between short four-lane sites before and after treatment installation: Accident data before and after construction of a short four-lane section were available for only one location (Site W08). At this site, the total accident rate decreased from 2.16 accidents per MVM (1.34 accidents per MVkm) before improvement to 1.3 accidents per MVM (0.84 accidents per MVkm) after improvement. A similar decrease in fatal and injury accident rate, from 1.06 to 0.45 accidents per MVM (0.66 to 0.28 accidents per MVkm) was observed.

Distribution of accident types: Table 17 compares the distribution of accident types for the short four-lane sites and comparable untreated sites. The distributions of single vehicle accidents are very similar for the short-four lane and untreated sites. The distributions of multiple-vehicle accidents differ markedly, however, with the short four-lane sections having a larger percentage of multiple-vehicle, intersection accidents and a smaller percentage of multiple-vehicle, nonintersection accidents. It is notable that the short four-lane sections, where passing in opposing lanes is not necessary, have a lower percentage of nonintersection head-on accidents than the untreated sections. There is no apparent reason for the higher percentage of multiple-vehicle, intersection accidents in short four-lane sections, but it should be noted that despite this higher percentage the overall accident rate for short four-lane sections is lower than for the untreated sections.

Cross-centerline accidents: Table 18 illustrates that the rates for cross-centerline accidents on short four-lane sections are generally less than half of the rates for the same type of accidents on the comparable untreated sections.

2.4 Summary

Passing lanes and short four-lane sections were found to provide substantial operational benefits when used as an operational treatment on two-lane highways. Both types of added lanes increase the passing rate in the treated direction to several times the passing rate that would occur on a conventional two-lane highway. Passing rates in passing lanes and short four-lane sections can be predicted as a function of flow rate, length of treated section and upstream percent of vehicles platooned using Equation (4) on page 33.

The percentage of vehicles platooned is reduced by nearly half (from 35.1% of vehicles following in platoons to 20.7% of vehicles following in platoons) within a passing lane. The percentage of vehicles platooned immediately downstream of a passing lane is 6% less than its upstream value (29.2% vs. 35.1%); the persistence of these downstream benefits is variable and highly dependent on the characteristics of particular sites.

TABLE 17

DISTRIBUTION OF ACCIDENT TYPES FOR PASSING LANES AND UNTREATED TWO-LANE HIGHWAYS

Accident Type	Number of Accidents (percent of total accidents): Short Four-Lane Sections	Comparable Two-Lane Highways
SINGLE-VEHICLE, NONINTERSECTION	65 (37.6)	96 (38.4)
Collision with parked vehicle	0 (0.0)	1 (0.4)
Collision with pedestrian	0 (0.0)	4 (1.6)
Collision with animal	16 (9.2)	15 (6.0)
Collision with fixed object	35 (20.2)	74 (9.6)
Collision with other object	1 (0.6)	0 (0.0)
Other collision	2 (1.2)	0 (0.0)
Noncollision, overturning	10 (5.8)	2 (0.8)
Other noncollision	1 (0.6)	0 (0.0)
MULTIPLE-VEHICLE, NONINTERSECTION	36 (20.8)	91 (36.4)
Head-on	4 (2.3)	12 (4.8)
Rear-end	19 (11.0)	32 (12.8)
Sideswipe	7 (4.0)	36 (14.4)
Angle	4 (2.3)	6 (2.4)
Other	2 (1.1)	5 (2.0)
SINGLE-VEHICLE, INTERSECTION	15 (8.7)	15 (6.0)
Collision with parked vehicle	0 (0.0)	1 (0.4)
Collision with pedestrian	0 (0.0)	1 (0.4)
Collision with animal	2 (1.2)	0 (0.0)
Collision with fixed object	10 (5.8)	11 (4.4)
Collision with other object	0 (0.0)	0 (0.0)
Other collision	2 (1.2)	2 (0.8)
Noncollision, overturning	1 (0.6)	0 (0.0)
Other noncollision	0 (0.0)	0 (0.0)
MULTIPLE-VEHICLE, INTERSECTION	57 (32.9)	48 (19.2)
Head-on	1 (0.6)	2 (0.8)
Rear-end	29 (16.8)	23 (9.2)
Sideswipe	10 (5.8)	8 (3.2)
Angle	15 (8.7)	14 (5.6)
Other	2 (1.2)	1 (0.4)
TOTAL	173 (100.0)	250 (100.0)

TABLE 18

COMPARISON OF CROSS-CENTERLINE ACCIDENT RATES FOR SHORT FOUR-LANE AND COMPARABLE UNTREATED SECTIONS

Accident Severity Level	Short Four-Lane Sections Number of Accidents	Exposure (MVM)	Accident Rate per (MVM)	Comparable Untreated Sections Number of Accidents	Exposure (MVM)	Accident Rate per (MVM)
Fatal	3	89.6	0.033	1	139.4	0.007
Injury	10	89.6	0.112	45	139.4	0.323
PDO	4	89.6	0.045	10	139.4	0.072
Total	17	89.6	0.190	56	139.4	0.402

These results imply that at 250 vph, a typical flow rate for a passing lane, if 90 vehicles are following platoons upstream of a passing lane during a given hour, then only 50 vehicles will be following in platoons within the passing lane and only 75 vehicles willbe following in platoons immediately downstream of the passing lane. The operational benefits of passing lanes can persist for several miles downstream from the treated section.

The reduction in platooning from upstream to downstream of a passing lane can be predicted as a function of the upstream percent of vehicles platooned and the length of the added lane using Equation (1) on page 27. Further research is needed to better define the influence of passing lane length on the passing rates and the reduction of vehicle platooning observed in passing lanes.

A safety evaluation found that the installation of a passing lane on a two-lane highway does not increase accident rate and, in fact, probably reduces accident rate. No unusual safety problems were found to be associated with either lane addition or lane drop transition areas. The rate of accidents involving vehicles traveling in opposite directions was found to be the same or lower on passing lane sections than on untreated two-lane highways at all severity levels, even for passing lanes where passing by opposing direction vehicles is permitted. One state has reported a rear-end accident problem associated with vehicles stopped in the center lane of a passing lane section to make a left turn; however, this problem was not evident at the sites evaluated in this study.

A substantially lower accident rate was found for short four-lane sections than for comparable untreated two-lane highways. The accident rates involving vehicles traveling in opposite directions on short four-lane sections were generally less than half of the rates found on comparable untreated sections. Because of the small sample size available for short four-lane sections the statistical significance of these conclusions could not be demonstrated.

It is recommended that states adopt more consistent policies for advance signing of passing lanes and short four-lane sections. In particular, advance signs from 2 to 5 miles (3 to 8 km) in advance of the added lane may help reduce the risk of accidents caused by driver frustration and impatience with lack of passing opportunities.

3. SHOULDER USE SECTIONS

This section presents an evaluation of the increasingly common practice of encouraging slow-moving vehicles to drive on the shoulder so that higher-speed vehicles can pass without delay. Two contrasting types of shoulder use sites were studied at four locations in the States of Washington and New York.

3.1 Current Practice

The primary purpose of the shoulder on a two-lane highway is to provide a stopping and recovery area for disabled or errant vehicles. However, recent research by Downs and Wallace[2] has catalogued up to 21 other uses for shoulders, most of which are applicable to two-lane highways. One of these alternatives is for the shoulder to be used by slow-moving vehicles so that higher-speed vehicles are able to pass.

In some parts of the country there is a long-standing custom, where adequate paved shoulders are provided, for slow-moving vehicles to move to the shoulder when another vehicle approaches from the rear and to return to the travel lane when that vehicle has passed. In extensive travel while collecting data for this study during the Summer of 1983, this practice was observed in many Western states, but was not observed east of the Mississippi River. In some areas, shoulders are used by drivers of slow-moving vehicles even though driving on the shoulder is technically illegal.

The practice of driving on paved shoulders is probably more prevalent in Texas than in any other state. Recent research by Fambro, et al.[3] evaluated this practice in Texas and provided the only operational evaluation of shoulder driving found in the literature. The results of the operational and safety studies conducted by Fambro are discussed later in this section.

The State of Washington has implemented a number of locations where shoulder driving is encouraged by signing. Shoulder driving sections of this type are generally employed on upgrades where turnout construction is infeasible and traffic volumes are too low for construction of a passing lane to be cost-effective.

Figure 11 illustrates the signing used in Washington State to introduce and terminate a shoulder driving section. A black-on-white regulatory sign is used to introduce the shoulder driving section and a black-on-yellow warning sign is used to terminate it. The lengths of shoulder driving sections were found to range from 0.50 mile (0.8 km) to nearly 3 miles (4.8 km) Shoulder driving sections of this type generally require that the shoulder be strengthened before the signing is put in place; trucks were found to constitute 40 to 75% of the vehicles using the shoulder.

The shoulder driving sections in Washington State were authorized by a specific change in state law. Until the late 1970's driving on the shoulder of a roadway was illegal in Washington. A state law was enacted authorizing the State DOT to determine portions of two-lane highway on which

SLOW VEHICLES
MAY USE SHOULDER
DAYLIGHT HOURS ONLY
(black on white)

END SHOULDER DRIVING
(black on yellow)

Figure 11 - Signing Used in Washington State to Introduce and Terminate a Shoulder Driving Section

drivers of slow-moving vehicles may safely drive onto improved shoulders for the purpose of allowing overtaking vehicles to pass and to indicate the beginning and end of such zones by appropriate signs. Where signs are in place to define a shoulder driving zone, the driver of a slow-moving vehicle may drive onto and along the shoulder within the zone, but only for the purpose of allowing overtaking vehicles to pass and then shall return to the roadway. Signs erected to define a shoulder driving zone are considered to take precedence over pavement markings for the purpose of allowing the movements just described.

Highway agencies have also tried converting shoulders to travel lanes, so that an existing two-lane highway can function as a passing lane or a four-lane undivided section. This approach has been used extensively in Texas, and has been referred to there as a "poor-boy" conversion. Converted shoulders of this type are usually marked as multilane sections with broken white lane lines between the original roadway and the shoulder and solid white edge lines at the right edge of the shoulder. The strength of the shoulder structure must be equivalent to a travel lane, since the data collected by Fambro[3] in Texas indicate that more than two-thirds of drivers will typically use the right or "shoulder" lane. Shoulder conversions provide many of the operational advantages of a passing lane or four-lane section, but reduce operational flexibility and potentially reduce safety, by eliminating the presence of a shoulder outside the traveled way.

New York State has experimented with the construction of a temporary lane for use as a passing lane during a special event (the 1980 Winter Olympics) and the subsequent conversion of this temporary lane to a shoulder that can be used by slow-moving vehicles.

Paved shoulders can also provide other operational benefits on two-lane highways, such as providing space for through vehicles to bypass left-turning vehicles at driveways and intersections, but such additional uses of shoulders have not been evaluated in this report.

3.2 Operational Evaluation

This section presents an operational evaluation of shoulder use sections. The discussion is organized into four subsections. The first subsection presents the operational benefits from shoulder driving without specific signs which is a common practice in Texas. The second subsection evaluates the practice of signing to designate specific sections for shoulder driving, as used in Washington State. The third subsection addresses the conversion of shoulders to travel lanes, based on the results obtained in Texas. The final section presents the evaluation of the New York site at which a temporary travel lane was converted to a shoulder.

The operational data collected in the field at the sites in Washington and New York are essentially equivalent to the data collected for passing lanes and short four-lane sections, as presented in Figure 5 of this report. Operational data were collected with the TDR both upstream, downstream, and within each shoulder use section. At the locations within the shoulder use sections, TDR traps were located in both the normal travel lanes and on the shoulder. Manual observers were also used to count the number of vehicles using the shoulder within the treated section and the number of passing maneuvers resulting from shoulder use by slow-moving vehicles. The measures of effectiveness for the shoulder use sections are the same as for passing lanes and short four-lane sections -- mean speed, percent of vehicles platooned, and passing rate. Since only three locations were evaluated in Washington and New York, these results should be interpreted more as case study examples, than as a general evaluation of shoulder use.

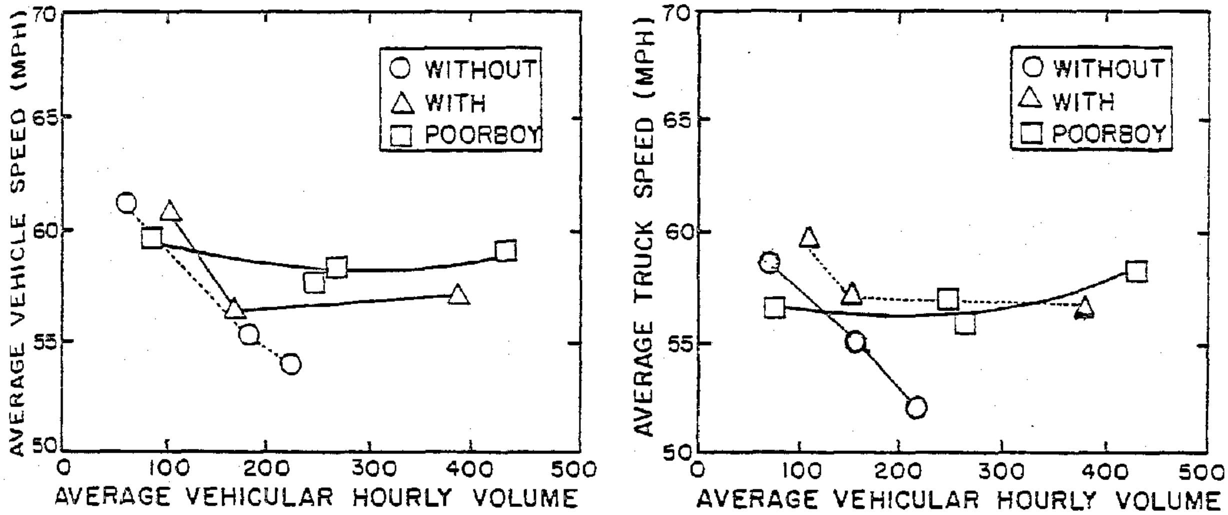
3.2.1 Shoulder driving without specific signing: The operational benefits of shoulder driving without specific signing was evaluated in Texas by Fambro.[3] Drivers in Texas, by long-standing custom, will move to the shoulder to let faster vehicles pass. Thus, no signing is necessary to encourage this practice, which can be observed almost anywhere a paved shoulder is provided.

Fambro evaluated the effectiveness of shoulder driving on two-lane highways by comparing operational conditions on highways with paved shoulders, with operational conditions on highways without paved shoulders, where shoulder driving was minimal. The results of the Fambro study are presented in Figure 12. In addition to the operational comparison of highways with and without shoulders, Figure 12 also presents data on "poor-boy" conversions of shoulders to travel lanes which will be discussed in a later subsection.

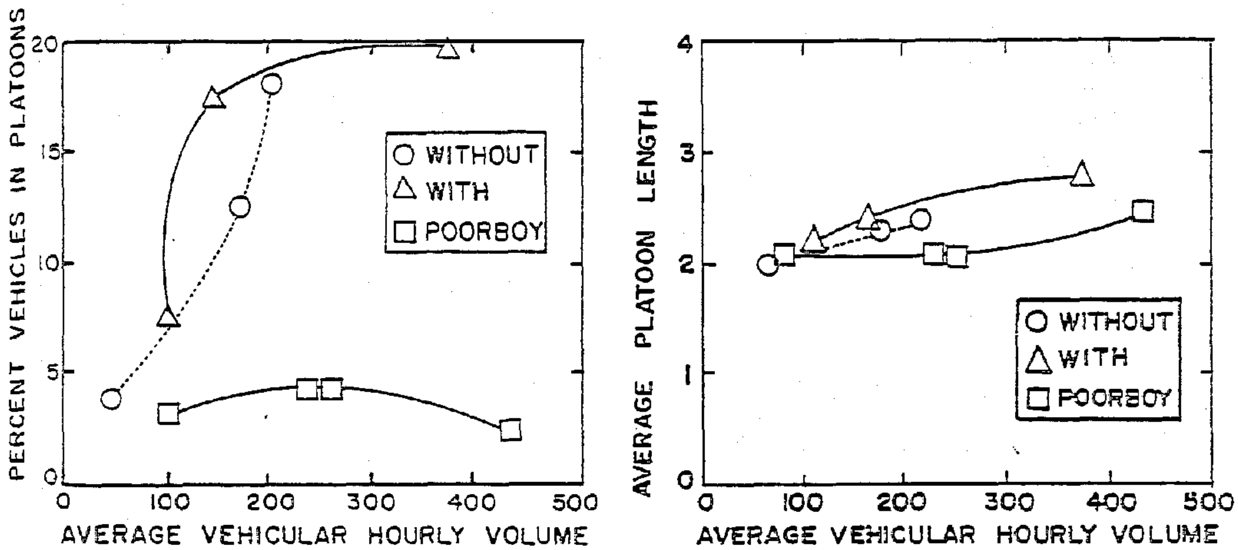
The Fambro study found that the operational benefits from providing a full-width paved shoulder increase as the volume increases. These benefits are minimal at volumes below 200 vehicles per hour; however, at volumes greater than 200 vehicles per hour, a paved shoulder will increase the average speed on the roadway by at least 10%. This increase in speed is undoubtedly due in part to shoulder driving, but part of the increase is probably due to the presence of the paved shoulder, even when it is not used for shoulder driving. The percentage of traffic in platoons was generally lower, and the platoons were generally shorter, on roadways with paved shoulders. At any location, however, only about 5% of the traffic actually used the shoulder. Based on these findings, it was recommended that it should be legal for a motorist to pull onto a paved shoulder in order to let faster vehicles pass, but that this should not be a requirement. To obtain the operational benefits from shoulder driving, it was concluded that paved shoulders should probably be added to all two-lane roads in Texas with traffic volumes in excess of 200 vehicles per hour.

3.2.2 Shoulder driving sections designated by signs: Three shoulder driving sections designated by signs were evaluated in field operational studies in Washington State. Each driving section was intended to provide opportunities to pass slow-moving vehicles. Two of the three shoulder driving sections were located on opposite sides of the road at a crest vertical curve, while the third was located on a sustained upgrade.

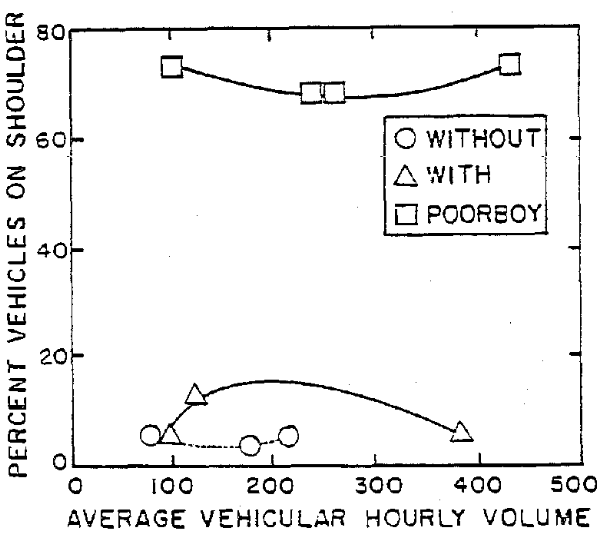
Site W03 is located at a relatively short crest vertical curve with 5% upgrades approaching the crest from both directions. Shoulder driving sections of 0.5 to 0.7 mile (0.8 to 1.1 km) in length were



AVERAGE SPEED FOR DIFFERENT ROADWAY TYPES



FREQUENCY AND LENGTH OF PLATOONS



SHOULDER USAGE

Figure 12 - Operational Effects of Paved Shoulders on Texas Highways[3]

designated in each direction of travel on the upgrade, over the crest and part or all of the way down the grades on the other side. The average flow rate during a 6-hr study period was 155 vph in the northbound direction and 175 vph in the southbound direction. The traffic stream was composed of approximately 10% RVs and 35% trucks, including a substantial number of logging trucks.

Table 19 presents the shoulder usage in each direction at this site, including the number of vehicles using the shoulder expressed as a percentage of the total flow and as a percentage of the platoon leaders. The percentage of vehicles observed to drive on the shoulder at some point in the shoulder driving section was 5.3% in the northbound direction and 8.4% in the southbound direction.

Table 20 presents the measures of effectiveness for the shoulder driving sections. The trends in mean speed at Site W03 from the upstream TDR trap, to the TDR trap located within the treated area (at the crest), to the TDR trap downstream of the treatment are not very informative, because they show exactly the pattern one might expect for traffic proceeding over a crest, even without a shoulder driving section. Traffic enters the site with a mean speed of about 55 mph (88 kph), slows down markedly ascending the grade to the crest and speeds up again on the downgrade.

The benefits of the shoulder driving section are demonstrated more clearly by the observed changes in the percent of vehicles platooned at Site W03, also presented in Table 20. In each direction of travel, the percent of vehicles platooned decreases from the upstream location to the crest, where vehicles may drive on either the travel lane or the shoulder. The percent of vehicles platooned then increases from the crest to the downstream location, but the percent of vehicles platooned downstream of the shoulder driving section is still smaller than at the upstream location. The decrease in percent of vehicles platooned due to the shoulder driving section was just over 2%, which is quite notable because the percent of vehicles platooned would normally be expected to increase at a crest, as the speeds of trucks and RVs would be reduced by the upgrade. Thus, the shoulder driving sections provide measurable operational benefits, even at a flow rates below 200 vph. On the other hand, the shoulder driving sections at Site W03 provide only about one-fifth of the reduction in the percent of vehicles platooned that would be expected from passing lanes of the same length at this site, based on Equation (1) in Section 2.2.

The passing rates observed at Site W03 were 14.5 passes per hour per mile (9.0 passes per hour per km) in the northbound direction and 29.9 passes per hour per mile (18.6 passes per hour per km) in th southbound direction. These passing rates are substantially higher than would be found on a typical two-lane highway, but are still only 20 to 50% of the passing rate that would be expected in a passing lane at this site, based on Equation (3) in Section 2.2.

Under Washington State law, described earlier in the current practice section, drivers are supposed to move to the shoulder only when overtaken by a faster vehicle. Slow-moving vehicles are not supposed to use the designated shoulder, as they would use a climbing lane, when other vehicles are not present. Both types of operation were observed in the field, however.

TABLE 19

SHOULDER USAGE ON SECTIONS DESIGNATED FOR SHOULDER DRIVING IN WASHINGTON STATE

Site Number	Length (miles)	Time of Day	Flow Rate (vph)	Number of Platoon Leaders	Shoulder Usage: % of Total Flow	Shoulder Usage: % of Platoon Leaders
W03 (NB)	0.70[a]	0930-1030	127	24	5.5	29.2
		1030-1130	177	41	6.8	29.3
		1130-1230	158	34	2.5	11.8
		1230-1330	156	38	8.3	34.2
		1330-1430	149	32	6.7	31.3
		1430-1530	154	29	1.9	10.3
		Average			5.3	24.3
W03 (SB)	0.59[a]	0930-1030	142	26	9.2	50.0
		1030-1130	163	34	10.4	50.0
		1130-1230	186	44	7.0	29.5
		1230-1330	162	31	6.2	32.2
		1330-1430	189	27	6.9	48.1
		1430-1530	205	48	10.7	45.8
		Average			8.4	42.6
W10 (SB)	0.58[b]	0845-0945	81	12	1.3	8.3
		0945-1045	73	9	6.8	55.5
		1045-1145	118	16	3.4	25.0
		1300-1400	75	4	1.3	25.0
		1400-1500	104	14	0.0	0.0
		1500-1600	80	13	2.5	15.4
		Average			2.6	21.5

[a] Entire length of shoulder driving section.
[b] Selected portion of a 2.47-mile shoulder driving section.

Note: 1 mile = 1.609 km.

TABLE 20

MEASURES OF EFFECTIVENESS FOR DESIGNATED SHOULDER DRIVING SECTIONS IN WASHINGTON STATE

Site Number	Flow Rate (vph)	Mean Speed (mph)			Percent of Vehicles Platooned			Passing Rate (passes per hour per mile)
		Upstream	Within Treatment	Downstream	Upstream	Within Treatment	Downstream	
W03 (NB)	155	54.6	53.0	56.0	38.5	34.6	36.4	14.5
W03 (SB)	175	55.4	48.2	56.1	33.0	11.2	30.6	29.9
W10 (SB)	90	53.3	49.5	46.1	16.5	19.0	23.4	5.2

Note: 1 mile = 1.609 km.

Site W10, the other designated shoulder driving section in Washington State, had an average flow rate of only 90 vph, with 15% RVs and 10% trucks. The shoulder driving section at Site W10 is approximately 2.5 miles (40 km) in length and is located on a continuous 3 to 4% upgrade with many horizontal curves. Two portions of the shoulder driving section, totaling 0.58 mile (0.93 km) in length, were observed. The shoulder was much less used at Site W10 than at Site W03. Tables 19 and 20 summarize the operational data for Site W10. Only 2.6% of the total volume was observed to use the shoulder within these areas, resulting in a passing rate of 5.2 passes per hour per mile (3.2 passes per hour per km). There is no discernible effect of the shoulder driving section on the mean speed and platooning data presented in Table 20, which change in exactly the manner one might expect on a sustained grade. It was concluded that the benefits of shoulder driving sections for volumes below 100 vph are minimal.

3.2.3 Conversion of shoulder to travel lane: The Fambro study of shoulder driving in Texas examined the conversion of shoulders to travel lanes on two-lane highways, or "poor-boy" conversion. Measures of operational performance for "poor-boy" conversions are compared to two-lane highways with and without paved shoulders in Figure 12. Fambro concluded that the conversion of the shoulder to an additional travel lane offers no apparent operational benefits at volumes below 150 vph. This conclusion apparently applies only to average vehicle speeds, because Figure 12 shows clearly that the conversion of a shoulder to a travel lane reduces the percent of vehicles platooned at all of the volume levels studied.

Fambro found that, unlike paved shoulders, which are used by only 5% of all vehicles, more than two-thirds of all vehicles used the right or "shoulder" lane on shoulder conversion sections. Thus, it appears that drivers treat a converted shoulder as they would any other multilane section.

3.2.4 Conversion of temporary travel lane to shoulder: A unique shoulder conversion project in New York State (Site Y05) was evaluated as part of the current study. This site is located on a highway that was used as the major access to the 1980 Winter Olympic Games held at Lake Placid. The pavement was widened from 2 to 3 lanes for a 3.50-mile (5.6-km) section and was operated as a conventional passing lane immediately before and during the Olympics. After the Olympics, when traffic returned to normal, the section was restriped as two lanes with a 10-ft (3-m) paved shoulder. This site is illustrated in Figure 13.

The unique feature of the site is that it is currently marked with a solid white line between the shoulder and the travel lane, as well as at the right edge of the shoulder. Though designated as a shoulder, it is treated by drivers as a travel lane. The shoulder lane is added at an intersection and dropped with a taper and conventional lane drop signing.

Table 21 presents the measures of effectiveness obtained in the operational field study of Site Y05. These measures of effectiveness were obtained upstream, downstream, and at two locations within the treated section. One of the data collection locations within the site was located on a level tangent near the center of the 3.50-mile (5.6-km) section and the other was located on an 8% grade 1 mile (1.6 km) further downstream.

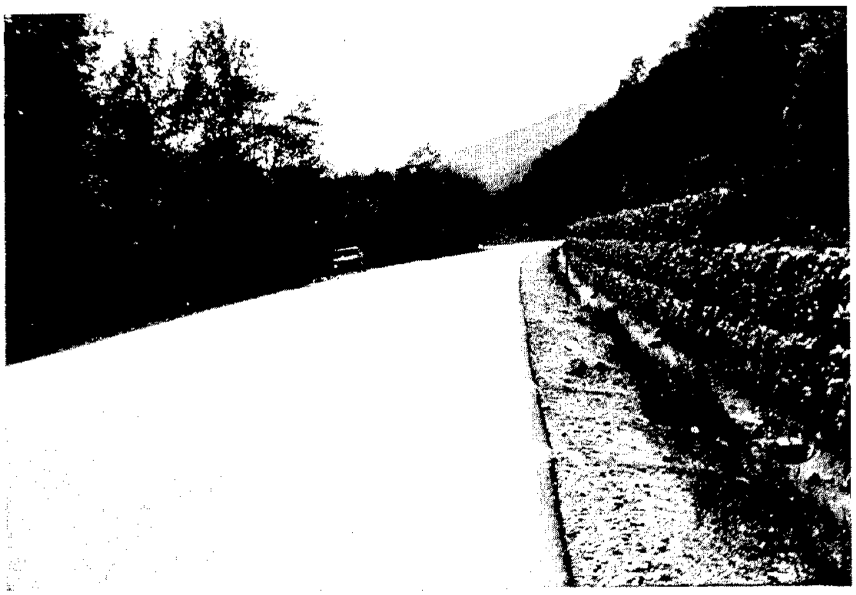


Figure 13 - Study Site with Temporary Travel Lane Converted to Shoulder in New York State

TABLE 21

MEASURES OF EFFECTIVENESS FOR SHOULDER CONVERTED TO PASSING LANE IN NEW YORK STATE
(Site Y05, Length = 3.50 Miles, Flow Rate = 70 vph)

Measure	Upstream	Within Treatment (on level tangent) Shoulder Lane	Within Treatment (on level tangent) Left Lane	Within Treatment (on level tangent) Combined	Within Treatment (on 8% grade) Shoulder Lane	Within Treatment (on 8% grade) Left Lane	Within Treatment (on 8% grade) Combined	Downstream
Mean Speed (mph)	45.0	53.2	56.0	55.3	47.9	46.0	47.4	52.9
Percent of Traffic Flow by Lane	-	23.6	76.4	100.0	76.7	23.3	100.0	-
Percent of Traffic Flow by Platooning Status								
Free Vehicles	65.9	98.0	86.0	88.8	81.0	91.3	83.4	77.8
Platoon Leaders	15.5	1.0	6.5	5.2	8.6	4.9	7.7	10.3
Platoon Members	18.6	1.0	7.5	6.0	10.4	3.9	8.9	12.0
Passing Rate (passes per hour per mile)	-	-	-	4.3	-	-	11.6	-

Note: 1 mile = 1.609 km.

The site experienced a very low volume, 70 vph, during the 6-hr study period. Table 21 shows that the percent of traffic platooned decreased from 18.6% upstream of the treatment to between 6.0 and 9.0% within the treatment and increased to 12.0% downstream of the treatment.

Motorists do not use this New York section in the manner that motorists in Texas and Washington use shoulder driving sections; i.e., motorists do not move to the shoulder briefly to allow faster vehicles to pass. The motorists using the right lane at the New York site tend to behave in a manner similar to motorists using the right lane of a conventional passing lane. However, at the level tangent location within the treatment, only 25% of the traffic used the right or shoulder lane. By contrast, about 75% of the traffic uses the right lane in a typical passing lane section. The mean speeds of traffic were higher in the left lane than in the right lane, but not markedly so (56.0 mph vs. 53.2 mph).

Further downstream, on the 8% upgrade, drivers seem to choose their lane more nearly like a conventional passing or climbing lane. About 75% of the drivers used the right lane, although the mean speeds in the right and left lanes are nearly identical (47.4 mph).

There are several possible explanations of the observed differences in lane use on the level tangent and on the 8% grade. It is possible that drivers avoid the right lane in the level tangent section because of the lack of a paved shoulder, but that this lack of a shoulder does not concern them on the upgrade. It is also possible that some drivers that use the left lane in the level tangent section and use the right lane on the upgrade consider themselves to be slow-moving in the latter case but not in the former.

3.3 Safety Evaluation

An evaluation of the safety of shoulder use sections has been based on both the literature and on the accident experience of the sites in Washington and New York evaluated in the current study.

The Fambro study of shoulder driving in Texas compared the safety records of two-lane highway sections with and without paved shoulders and of highway sections before and after installation of paved shoulders. The study concluded that two-lane highways with paved shoulders have lower accident rates than two-lane highways without paved shoulders, in the ADT range from 1,000 to 7,000 vehicles per day. However, since the practice of driving on paved shoulders is widespread in Texas, this finding represents the combined effect of providing a paved shoulder and permitting shoulder driving. The Fambro study does not address the safety effects of encouraging shoulder driving on highways that already have paved shoulders.

Table 22 presents the accident rates of the shoulder use sections in Washington and New York, before and after treatment installation. The before-after accident rate comparison for the shoulder driving sections in

TABLE 22

BEFORE-AFTER ACCIDENT RATE COMPARISONS OF SHOULDER USE SECTIONS

Site Number	Treatment Type	Before Treatment: Number of Accidents	Before Treatment: Exposure (MVM)	Before Treatment: Accident Rate (per MVM)	After Treatment: Number of Accidents	After Treatment: Exposure (MVM)	After Treatment: Accident Rate (per MVM)
W03	Designated Shoulder Driving Section	0	1.86	0.00	2	1.90	1.06
W10	Designated Shoulder Driving Section	7	1.80	3.88	3	1.98	1.52
Y05	Shoulder Conversion to Travel Lane[a]	7	3.23	2.17	3	1.85	1.62
Y15	Comparable Untreated Section	11	8.66	1.27	10	4.97	2.01

[a] Excludes a 12-month period during which the site was marked as a conventional passing lane.

Note: 1 mile = 1.609 km.

Washington State designated by signs shows little change in accident experience due to shoulder driving. The treated portions of Site W03 experienced no accidents in a 2-year period before treatment and two accidents in a comparable period after treatment; Site W10 experienced seven accidents in the treated direction in a 2-year period before treatment and three accidents in a comparable period after treatment. No statistically valid conclusions can be drawn from such a small sample. It is worth noting, however, that all of the accidents at both sites -- both before and after treatment -- involved only a single vehicle. Thus, there is no evidence of any safety problem related to collisions between vehicles using the shoulder and vehicles using the travel lanes.

Table 22 includes the accident rate for the travel lane conversion project in New York State discussed earlier and a comparable two-lane highway section on the same route, for 2-year periods both before and after construction of the paved shoulder. While there are not enough data to draw statistically valid conclusions, it is notable that the accident rate of the treated site decreased from before-to-after construction, while the accident rate of the comparable untreated site increased.

Fambro concluded that "poor-boy" shoulder conversion projects in Texas generally had accident rates between the accident rates for two-lane highways with and without paved shoulders. Furthermore, it was found that the accident rates for roadways with converted shoulders were less sensitive to traffic volume than either of the other types of roadways. The conversion of a shoulder to an additional travel lane was found to result in fewer total accidents only at volume levels greater than 3,000 vehicles per day. It was noted that "poor-boy" roadways had unusually high nighttime accident rates.

3.4 Summary

The use of shoulders by slow-moving vehicles at sites designated by signs was found to provide operational benefits on two-lane highways with flow rates over 100 vph in one direction. The percentage of vehicles platooned was reduced by 2% as a result of shoulder driving on a two-lane highway with a flow rate approximately 165 vph at a site where an increase in platooning would normally be expected without a shoulder use section. Five to 8% of all vehicles and 25 to 40% of platoon leaders used the shoulder at this site. However, the operational benefits provided by this shoulder use section were estimated to be only one-fifth of the benefits that would be provided by a passing lane of comparable length at the same site. Minimal operational benefits were found from a shoulder use section on an extended grade with a flow rate less than 100 vph.

Shoulder use sections designated by signs did not produce a substantial change in accident rate. Although the available accident sample size was very small, it is worth noting that neither of the shoulder use sites experienced any multiple-vehicle accidents associated with slow-moving vehicles moving onto the shoulder, driving on the shoulder, or returning to the travel lane.

One site was evaluated at which a shoulder had been converted to a passing lane. This site had a flow rate less than 100 vph during the study period. The percent of vehicles platooned was reduced by the treatment, even at this low flow rate, although the level of service would have been high even without the treatment. The shoulder conversion project reduced accident rates, as well, although the available sample size is too small to draw statistically significant conclusions.

Shoulders designated for use by slow-moving vehicles or converted to travel lanes must be strengthened to support heavy vehicles. Slow-moving vehicles using designated shoulders include a substantial proportion of trucks. Drivers tend to treat a converted shoulder as a normal travel lane, so the converted shoulder usually carries a majority of the traffic flow. However, drivers may be discouraged from using the shoulder lane, especially at low traffic flows if it is narrower than the left lane or provides inadequate lateral clearance.

4. TURNOUTS

This section presents an operational and safety evaluation of slow-moving-vehicle turnouts on two-lane highways. The discussion includes a description of the purpose of slow-moving-vehicle turnouts; a review of state design and traffic control practices; an operational field evaluation of nine turnouts; and a safety evaluation of 47 turnouts located in five states.

Figure 14 illustrates typical slow-moving-vehicle turnouts on two-lane highways in several states. A turnout is a widened, unobstructured shoulder area provided for slow-moving vehicles to pull out of the through lane and, without stopping, allow following vehicles to pass. It is intended that the driver of a slow-moving vehicle should remain in the turnout only long enough for the following vehicle(s) to pass and that the driver should then return to the through lane. Thus, turnouts can be relatively short, generally less than 600 ft (180 m) in length. Rooney found that when turnouts longer than 500 ft (150 m) are used, drivers tend to treat them as they would a passing or climbing lane and remain in the turnout after the followers have passed.[15,16] On the other hand, a turnout must not be too short, or a driver may be forced to stop in the turnout before all of the followers have passed or, worse still, the driver may reenter the through lane prematurely and create the potential for collision with a following vehicle.

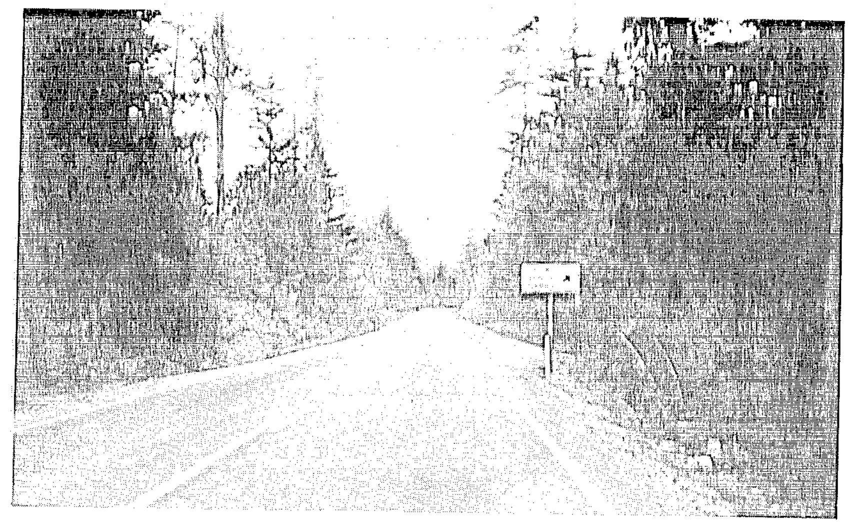
Turnouts can be employed successfully both on sustained grades and in level or rolling terrain. A series of turnouts can provide a portion of the operational benefits that would be provided by passing or climbing lane. Turnouts are especially well suited to mountainous terrain where the cost of providing an added lane or even a continuous paved shoulder may be prohibitive. Since turnouts are relatively short, they can be located where they are easy and inexpensive to construct.

4.1 Current Practice

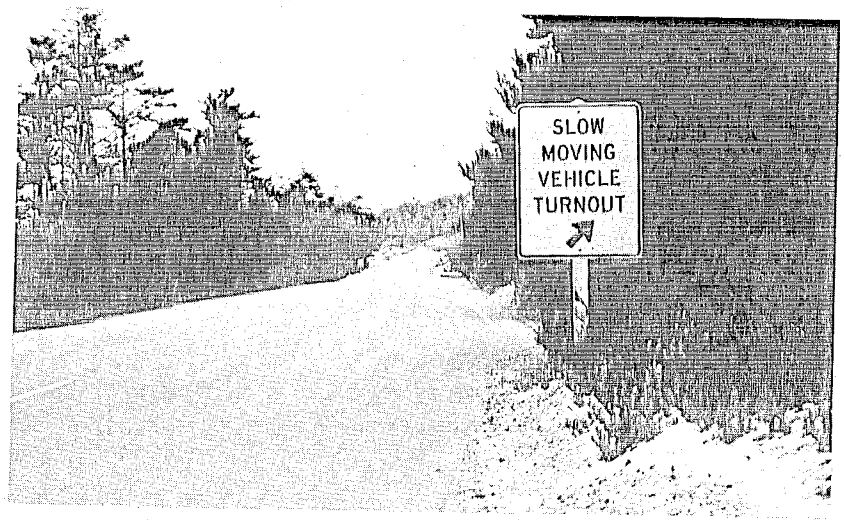
The use of slow-moving-vehicle turnouts as an operational aid on two-lane highways is found primarily in the Western states. Of the 13 participating states in the current study, only three -- California, Oregon, and Washington -- use turnouts that are officially designated for use by slow-moving vehicles. The designated turnouts in these states are paved and have permanent signing to inform drivers of their location and the legal requirements for their use. Two additional states -- New York and Utah -- have identified unsigned shoulder areas for this study that are occasionally used as turnouts by slow-moving vehicles. These locations are not officially designated as turnouts and there is no legal requirement that they be used as such.

4.1.1 Legal requirements for turnout use: The legal requirements for turnout use differ among the three Western states in which turnouts were studied. In Oregon, a slow-moving vehicle is required by law to use a turnout if it is overtaken or followed by even a single vehicle. In California

California



Washington



Oregon

Figure 14 - Typical Turnout Sites on Two-Lane Highways

and Washington, a slow-moving vehicle is required to use a turnout only if five or more following vehicles are present. It should be noted that, while slow-moving vehicles followed by fewer than five vehicles are not required to use turnouts, they are not prohibited from doing so. California and Washington laws define a slow-moving vehicle as one which is proceeding at a rate of speed less than that of the normal flow of traffic at the particular time and place.

4.1.2 Turnout location and geometrics: The Oregon Department of Transportation was found to have a more explicit design policy concerning turnouts than any of the other participating states. Their policy states that slow-moving vehicle turnouts should meet the following requirements:

1. The turnout is not on a curve that restricts sight distance in both directions.

2. The turnout is preceded by alignment that provides a clear view for a distance of at least 1,000 ft (310 m).

3. The turnout is a minimum width of 16 ft (5 m) measured from the edge of the travel lane.

4. The turnout is a minimum of 150 ft (50 m) and a maximum of 600 ft (180 m) in length.

5. The turnout must be level and smooth so house trailers and camper pickups need not be afraid to pull off in the turnout at 30 to 40 mph (48 to 64 kph).

Oregon recommends the following minimum turnout lengths based on approach speed:

Approach Speed (mph)	Recommended Minimum Length of Turnout (ft)[a]
25	200
30	200
40	250
50	375
55	450
60	535

a Including entry and exit tapers.

Note: 1 mile = 1.602 km.
1 ft = 0.305 m.

These minimum length requirements were established to provide adequate stopping distance in the turnout, assuming that vehicles slow 5 mph (8 kph) before entering the turnout and decelerate at a constant rate of 7 ft/sec^2 (2.1 m/sec^2) in the turnout.

The turnout design policy of the Washington State Department of Transportation requires a minimum turnout width of 10 ft (3.1 m) with a 12-ft (3.7 m) width desirable. A minimum turnout length of 100 ft (310 m) is required, in addition to an entrance taper with a 25:1 taper rate and an exit taper with a 50:1 taper rate. Thus, the required minimum turnout length, including both tapers and assuming a 12-ft (3.7-m) turnout width, is 1,000 ft (310 m). A constant cross-slope is required across both the through lane and the turnout.

The 43 paved turnouts studied in California, Oregon, and Washington were found to vary in length from 300 to 1,200 ft (90 to 370 m). The average turnout length was 570 ft (170 m). Ten of the turnouts studied (or 23%) were shorter in length than the 450 ft (140 m) criterion suggested by Oregon to provide adequate stopping distance for an approach speed of 55 mph (88 kph). The unpaved turnouts studied in New York and Utah ranged in length from 200 to 500 ft (60 to 150 m).

The widths of paved turnouts ranged from 9 to 24 ft (2.7 to 7.3 m). Seventeen (or 40%) of the 43 paved turnouts had paved widths less than 12 ft (3.7 m). In some cases, there were wide unpaved areas outside of the paved turnout that were occasionally used for parking, but in other cases cut slopes and/or curbs provided for drainage purposes constrained the turnout width, making it narrow and potentially unattractive to potential users.

4.1.3 Pavement markings: Alternative pavement marking practices for turnouts are illustrated in Figure 15. The marking practice used by the California Department of Transportation, shown in the upper portion of the figure, uses an 8-in. (20-cm) solid white line to separate the turnout from the through lane. The white line is dropped through the entry and exit taper areas so that there is a gap between the edge line and the turnout marking. Oregon uses a similar marking practice, illustrated in the lower portion of Figure 15. The turnout is separated from the through lanes by a 4-in. (10-cm) solid white line. A 4-in. (10-cm) solid white line is also used to delineate the right edge of the turnout. A series of dots spaced at 15 ft (4.5 m) center-to-center are used to connect the edge line and the turnout marking through the entry and exit taper areas. (The use of these dots was not observed at the turnouts studied in the field in Oregon. However, this may be because they tend to wear quickly in the areas where vehicles enter and exit the turnout areas.)

4.1.4 Signing: Figure 16 illustrates the sequences of advanced signing used for turnouts in California, Oregon, and Washington. Four specific functions for turnout signing are illustrated by the signs in Figure 16. Turnout signing is intended to:

- Notify drivers of an upcoming series of turnouts,
- Notify drivers of a specific turnout,
- Remind drivers of the legal requirements for turnout use, and
- Identify the beginning of a specific turnout.

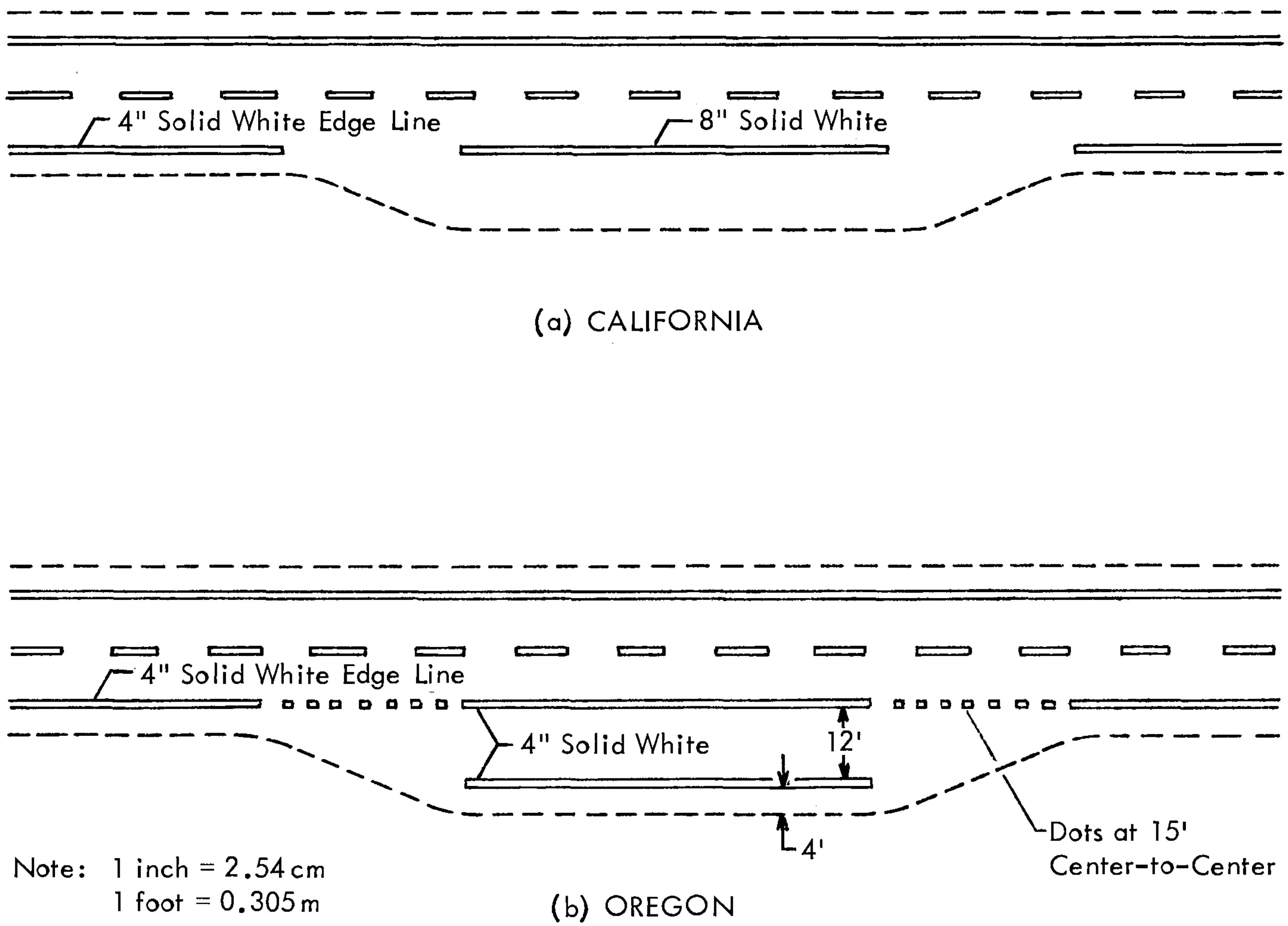


Figure 15 - Turnout Marking Practices in California and Oregon

Function/Location of Sign	CALIFORNIA	OREGON	WASHINGTON
First Notification of Series of Turnouts	SLOWER TRAFFIC USE TURNOUTS	None	SLOW VEHICLES USE TURNOUTS NEXT 5 MILES DELAY OF 5 VEHICLES ILLEGAL MUST USE TURNOUTS
First Notification of a Specific Turnout	TURNOUT 1/4 MILE	SLOW MOVING VEHICLE TURNOUT 1/2 MILE	None
Reinforcement of Legal Requirement for Turnout Use (500' in advance)	None	LAW REQUIRES SLOW VEHICLES TO USE TURNOUT	None
At Beginning of Turnout	TURNOUT	SLOW MOVING VEHICLE TURNOUT	SLOW VEHICLE TURNOUT

Figure 16 - Turnout Signing Practices in California, Oregon, and Washington

The signing sequence used in each state fulfills some, but not all of the functions.

California and Washington both provide advance notification to the driver that a series of turnouts is upcoming. Washington reinforces this notification with the distance covered by the turnouts (e.g., NEXT 5 MILES) and reminds drivers of the legal requirement for turnout use. No advance notification of a series of turnouts is used in Oregon.

California and Oregon both provide notification to the driver of each turnout in a series within 1/4-mile or 1/2-mile in advance. In Washington, the sign located at the beginning of a turnout is the only information the driver receives concerning that specific turnout. Oregon provides a reinforcement of the legal requirement for turnout use (LAW REQUIRES SLOW VEHICLES TO USE TURNOUTS) 500 ft in advance of the turnout.

All three states place signs at the beginning of the turnout to assure that approaching drivers see the turnout. The signs used by Oregon and Washington provide the greatest degree of positive guidance since they include an upward-sloping arrow indicating that slow-moving vehicles are to move to the right.

All four functions of advance turnout signs appear important and the use of a signing sequence incorporating all four functions is recommended.

4.2 Operational Evaluation

An operational evaluation was conducted in the field at 7 signed turnouts in California, Oregon and Washington and 2 unsigned turnouts in New York. Two unsigned turnouts in Utah were not evaluated in the field because the highway on which they are located was temporarily closed at the time of the field study.

The purpose of the operational evaluation of turnouts was to determine the extent of turnout usage, the resulting operational benefits and the potential for increasing turnout usage through public education or increased enforcement. The differences in legal requirements for turnout usage were also examined.

4.2.1 Data collection: Figure 17 illustrates the typical data collection setup for turnout studies. Two TDR traps were located upstream and and two TDR traps downstream of each turnout. One TDR trap was located 800 to 1,500 ft (240 to 460 m) upstream of the turnout and another was located 100 ft (30 m) in advance of the turnout. The upstream TDR traps supplied the most important automated data collected at turnouts, since they established the platooning characteristics of traffic approaching the turnout. TDR traps were also located 100 ft (30 m) and 800 to 1,500 ft (240 to 460 m) downstream of each turnout. A TDR trap was also located in the turnout lane itself. This data collection plan was adapted in the field, as necessary, to fit the geometric and traffic characteristics of each site and to enable evaluation of more than one turnout at the same site.

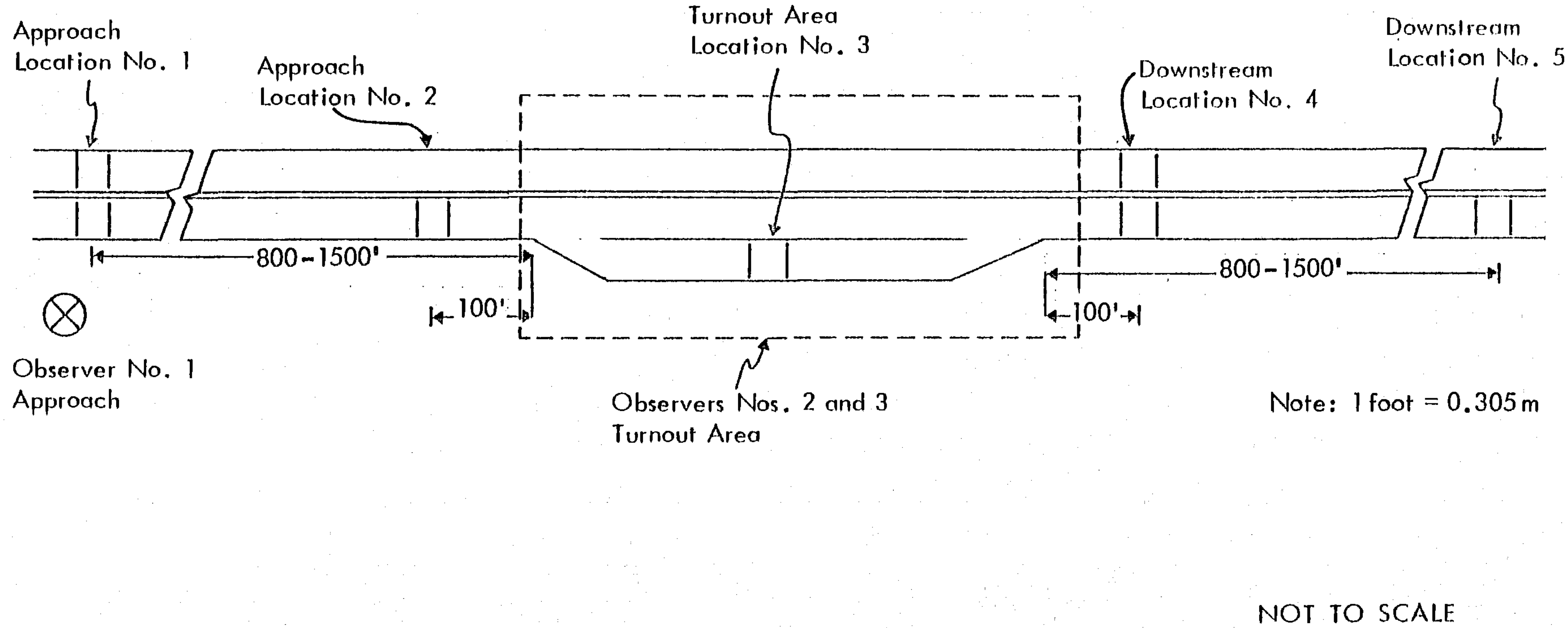


Figure 17 - Locations of TDR Traps and Observers for Data Collection at Turnouts

Data were also collected manually on each vehicle that used a turnout. The manually observed data included:

- Time of entry.
- Vehicle type and description.
- Number of trailing vehicles.
- Number of trailing vehicles able to pass the turnout vehicle.
- Was vehicle forced to stop because the turnout was too short?
- Did vehicle reenter traffic prematurely and delay trailing vehicles?
- Did premature reentry result in a traffic conflict?
- Did vehicle use turn signal when entering turnout? When reentering highway?
- Did vehicle stop in turnout for some reason other than allowing trailing vehicles to pass? Reason? Length of stop?
- Did a vehicle traveling in the opposing lane turn left into a turnout?
- Did a traffic conflict result from a left turn?

The manual data were intended to establish the extent of turnout use, the frequency of erratic maneuvers or misuse of turnouts by drivers, and the frequency of traffic conflicts that may indicate potential safety problems in turnouts.

The data collection period at each turnout was typically 6 hr in length. However, at three turnouts where no usage was observed, the study was stopped after 2.5 to 3 hr.

4.2.2 Measures of effectiveness: The operational measures of effectiveness for turnout usage include:

- Number of turnout users per hour.
- Percent of platoon leaders using turnout.
- Number of passes per hour.
- Percent of followers able to pass.
- Percent of users forced to stop in turnout.

- Change in percent of vehicles platooned from upstream to downstream of turnout.

4.2.3 Operational analysis results: Table 23 summarizes the geometric and traffic control characteristics of the 9 turnouts that were evaluated in the field. For each turnout, the table includes the flow rate and percent of vehicles platooned during the study period; the turnout length and width; the horizontal and vertical alignment at the turnout site; and comments concerning the character of traffic at the site and any upstream geometric features that influence vehicle platooning and/or turnout operations. The turnouts evaluated in the field were located on highways with traffic volumes ranging from 1,300 to 6,300 vehicles per day.

The observed measures of effectiveness for the turnouts are presented in Table 24. Four of the turnouts studied were located on highways with flow rates between 200 and 300 vph in the treated direction while the remaining five turnouts were located on highways with flow rates under 100 vph. It is apparent that turnouts contribute very little to improved traffic operations at flow rates under 100 vph. At such low flow rates, two turnouts were used by an average of only 1.5 vehicles per hour and three turnouts (including the two unsigned turnouts in New York) received no use at all.

Turnouts on highways with flow rates over 200 vph were used by 5.9 to 15.3 vehicles per hour. Turnout usage appears to be influenced by flow rate, percentage of vehicles platooned and vertical alignment. The highest frequency of turnout usage was observed at Site C21-2, which has the highest flow rate, the highest percentage of vehicles platooned and the steepest grade of any of the high-volume turnout sites.

At the turnouts that were used, the percentage of platoon leaders using the turnout ranged from 9.5 to 29.5%. These results are in agreement with the range of turnout usage (2.8 to 36%) observed by Rooney in California.[16]

The vehicles using turnouts were almost exclusively passenger cars, recreational vehicles and single unit trucks. Drivers of combination trucks probably consider most turnouts too short and/or too narrow for them to use effectively. The only substantial use of a turnout by combination trucks was observed at Site C21-2, at which the turnout is 750 ft (230 m) long and is located on a 5% grade where the average speed of trucks was reduced to 43.6 mph (70.2 kph). Therefore, it is recommended that turnouts be used primarily on recreational routes; operational benefits from turnout are combination trucks should be anticipated only on steep grades where a turnout of adequate length can be provided.

The number of passing maneuvers resulting from turnout usage at flow rates over 200 vph ranged from 12.4 to 52.3 passes per hour. Based on Figure 6 presented in Section 2.2 of this report, a conventional two-lane highway with a flow rate of 265 vph would be expected to experience about 15 passes per hour per mile (9.5 passes per hour per km). Thus, 3 of the 4 high-volume turnouts enable more passes to occur at one point than would be expected in one mile of a conventional two-lane highway.

TABLE 23

GEOMETRIC AND TRAFFIC CONTROL CHARACTERISTICS OF TURNOUTS EVALUATED IN THE FIELD

Site Number and Treatment Number	Flow Rate[a] (vph)	% of Vehicles Platooned	Turnout Length (ft)	Turnout Width (ft)	Horizontal Alignment	Vertical Alignment	Comments
C21-1	261	39.3	600	9	Tangent	2% upgrade	Recreational route 16% RVs, 6% trucks
C21-2	265	52.1	750	10	Curve to left	5% upgrade	Recreational route 16% RVs, 6% trucks 3 miles downstream from turnout C21-1
R04-1	211	38.4	600	12	Tangent	1% upgrade	Recreational route 21% RVs, 14% trucks
R04-2	208	44.0	600	12	Tangent	1% down-grade	Recreational route 17% RVs, 14% trucks Directly opposite turnout R04-1
R07-1	94	26.7	475	25	Tangent	3% down-grade	Recreational route 19% RVs, 19% trucks 2 miles downstream from long steep (8%) upgrade
W04-2	78	24.7	500	12	Tangent	Level	Recreational and logging route 13% RVs, 18% trucks Immediately downstream of 5% grade
W04-3	76	27.0	400	16	Tangent	Level	Recreational and logging route 13% RVs, 18% trucks 1.4 miles downstream from turn-out W04-2
Y06-2	53	21.8	200	25+	Tangent	8% upgrade	Unsigned, unpaved turnout Recreational route 4% RVs, 16% trucks
Y06-3	48	13.4	200	18	Tangent	8% down-grade	Unsigned, paved turnout Recreational route 4% RVs, 7% trucks 700 ft from turnout Y06-2 in opposing direction

[a] Treated direction only.

Note: 1 mile = 1.609 km.
1 foot = 0.305 m.

TABLE 24

OPERATIONAL EFFECTIVENESS OF TURNOUT USAGE

Site Number and Treatment Number	Flow Rate (vph)[a]	% of Vehicles Platooned	Number of Turnout Users per Hour	% of Platoon Leaders Using Turnout	Number of Passes per Hour	% of Followers Able to Pass Turnout User	% of Users Forced to Stop in Turnout	% of Users Creating a Reentry Conflict	% of Turnout Users Employing Turn Signals	
									Entering Turnout	Exiting Turnout
C21-1	261	39.3	5.9	12.4	12.4	80.0	10.3	10.3	72.4	55.2 (71.4)[b]
C21-2	265	52.1	15.3	26.5	52.3	89.2	9.0	9.0	35.1	20.8 (33.3)
R04-1	211	38.4	9.8	21.9	16.5	84.6	11.9	11.9	25.4	23.5 (55.6)
R04-2	208	44.0	13.0	29.5	31.5	91.3	24.4	5.1	32.4	28.2 (0.0)
R07-1	94	26.7	0.0	0.0	0.0	-	-	-	-	- (-)
W04-2	78	24.7	1.5	9.8	3.8	100.0	22.2	0.0	55.6	33.3 (-)
W04-3	76	27.0	1.5	9.5	3.7	91.7	50.0	33.3	66.7	33.3 (100.0)
Y06-2	53	21.8	0.0	0.0	0.0	-	-	-	-	- (-)
Y06-3	48	13.4	0.0	0.0	0.0	-	-	-	-	- (-)

[a] Treated direction only.
[b] Number in parenthesis is percentage of turnout vehicles using turn signal leaving turnout when not all followers have passed.

The high volume turnouts resulted in up to 72% of the number of passes that would occur in a one mile (1.6 km) passing lane in level terrain at the same volume level. It should be noted, however, that the highest passing rate for turnouts was observed at a site on a 5% grade. A passing lane located on the same grade might produce more passing of slow-moving vehicles than would occur in level terrain. In general, a single turnout should be expected to provide from 20 to 50% of the number of passes that would occur in a 1-mile (1.6-km) passing lane in level terrain.

It should be recognized, however, that a pass completed because of a turnout maneuver may not provide as much operational benefit as a pass completed in a passing lane. In a passing lane, the passing vehicles represent self-selected drivers with higher desired speeds than their immediate platoon leader. By contrast, at a turnout, the passed vehicles (turnout users), rather than the passing vehicles, are self-selected and the passing drivers may or may not have higher desired speeds. The passing vehicles at a turnout may simply continue downstream as a new platoon leader. Thus, it is expected that a turnout may not provide as much reduction in platooning per passing maneuver as a passing lane.

At every turnout, over 80% of the following vehicles in the platoon immediately behind the turnout user were able to pass the turnout user. In most cases, vehicles with only 1 or 2 followers were able to use the turnout without stopping, while vehicles with 4 or more followers were forced to stop, to slow to nearly a crawl speed, or to reenter the traffic stream before all of the following vehicles had passed. At the higher volume turnouts, from 9 to 25% of the turnout users were forced to stop in the turnout to let the entire following platoon pass them. Another 14 to 24% turnout users chose not to stop, but reentered the through traffic stream prematurely without allowing all of the following vehicles to pass. Between 35 and 60% of these premature reentries resulted in a traffic conflict (typically, braking by a a following vehicle). Between 5 and 11% of all turnout users caused a conflict when they reentered the traffic stream.

The percentage of vehicles platooned decreased by 0 to 3.5% from 100 ft (30 m) upstream to 100 ft (30 m) downstream of a turnout. The average decrease in vehicle platooning was 2.0%. There is no major decrease in vehicle platooning from immediately upstream to downstream of a turnout, since a platoon leader that uses a turnout often joints the rear of the platoon it formerly led after using a turnout.* In level terrain with good alignment, an additional 1 to 4% decrease in vehicle platooning was observed between 100 and 1,500 ft (30 and 450 m) downstream as the platoon begins to disperse. However, when the highway downstream from a turnout is located on a grade or has a sharp horizontal curve vehicle platooning may remain constant or increase downstream from a turnout.

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* It should be noted that platoons were defined in this study based solely on a 4-sec headway criterion; some additional benefit from turnout usage might be seen if the platoon definition also included a relative speed criterion.

The use of vehicle turn signals can be an important aid to enable turnouts to operate efficiently and safely. The use of a right turn signal by a driver entering a turnout warns following vehicles that he may be slowing down and prepares following drivers to accelerate past the vehicle while it is in the turnout. Use of a left turn signal on leaving a turnout warns any vehicles that were not able to pass the turnout vehicles so that potential rear-end and side-swipe accidents can be avoided. It appears from the data in Table 19 that drivers employ their turn signals more often when entering than when leaving a turnout. On the average, 38.6% of the drivers use their turn signals when entering a turnout and 28.5% use their turn signals when leaving a turnout. This comparison is misleading, however, because it was found that drivers do not often use their turn signals to reenter the highway if all of the following vehicles have passed. In fact, 45.4% of drivers used their turn signals to reenter the highway if some of the following vehicles had not yet passed.

Table 25 presents a more detailed breakdown of turnout usage than was shown earlier. The number of vehicles using each turnout is given in the table in the form of a usage rate expressed as a percentage of the total number of platoon leaders and the number of "potential users" (the estimated number of platoon leaders who should have used the turnout, as defined below). Turnout usage is tabulated for all platoons and for platoons of six or more vehicles.

The turnout usage rate based on the number of potential users provides a refinement of the overall usage rate through recognition that not all platoon leaders can reasonably be expected to use a turnout. Some platoon leaders are going too fast to be considered slow-moving vehicles and others are going too fast to use a relatively short turnout. The upper portion of Figure 18 illustrates for one turnout (Site R04-1) that platoon leaders who chose to use turnouts were generally traveling more slowly than platoon leaders who chose not to use the turnout. It is also apparent in the lower portion of Figure 18 that most platoon leaders that used the turnout are traveling at or below the median speed of platoon leaders as a whole. A platoon leader approaching a turnout was classified as a potential user of a turnout only if:

- its speed is less than the median speed for all platoon leaders; and,
- its speed is less than the maximum safe entry speed for the turnout, defined as a function of turnout length.

The maximum safe entry speed for a turnout is estimated as follows:

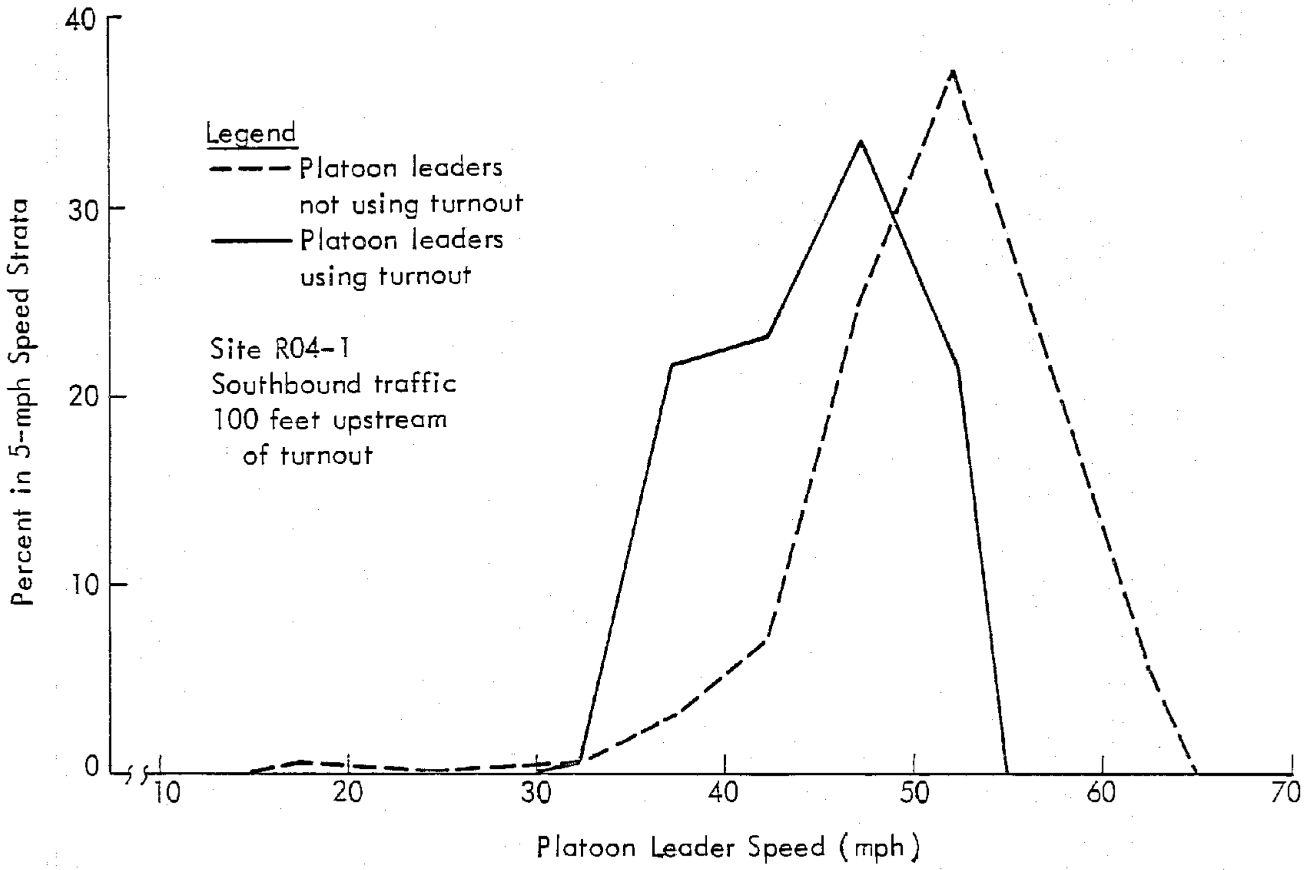
Turnout Length (ft)	Maximum Safe Entry Speed (mph)
200	28.9
300	36.8
400	43.1
500	48.6
600	53.5
700	57.9

Note: 1 mile = 1.609 km.
1 foot = 0.305 m.

TABLE 25

TURNOUT USAGE BY PLATOON LENGTH

Site Number and Treatment Number	Number of Turnout Users	Number of Platoon Leaders	% Usage Based on Platoon Leaders	Number of Potential Users	% Usage Based on Potential Users
			PLATOON LENGTH ≧ 2 VEHICLES		
C21-1	29	233	12.4	117	24.8
C21-2	89	336	26.5	164	54.3
R04-1	59	269	21.9	133	44.3
R04-2	78	264	29.5	132	59.1
R07-1	0	51	0.0	4	0.0
W04-2	9	92	9.8	13	69.2
W04-3	9	94	9.5	18	50.0
Y06-2	0	19	0.0	3	0.0
Y06-3	0	14	0.0	0	-
			PLATOON LENGTH ≧ 6 VEHICLES		
C21-1	3	14	21.4	8	37.5
C21-2	32	48	66.7	25	128.0
R04-1	5	5	100.0	4	125.0
R04-2	14	14	100.0	13	107.7
R07-1	0	1	0.0	0	0.0
W04-2	2	2	100.0	2	100.0
W04-3	2	2	100.0	2	100.0
Y06-2	0	0	-	0	-
Y06-3	0	0	-	0	-



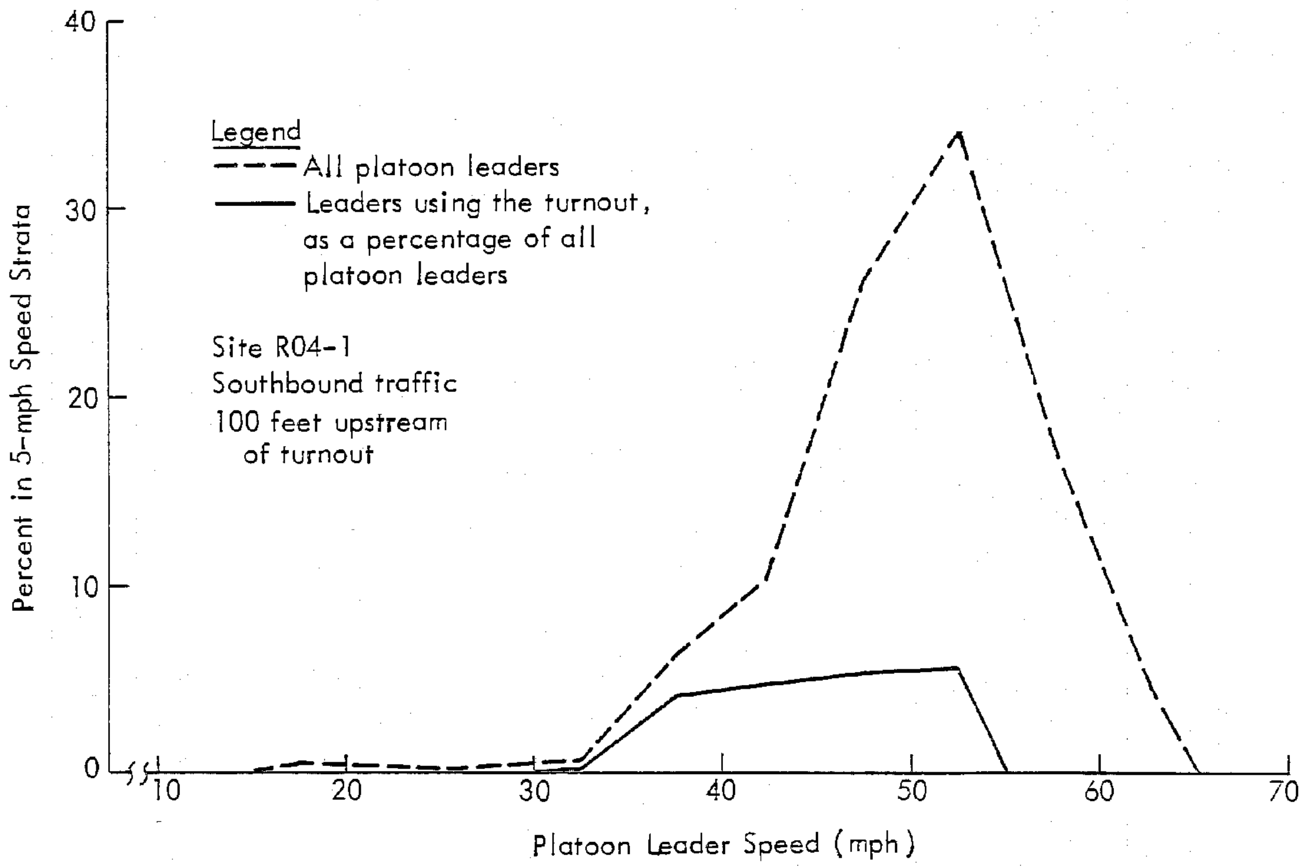


Figure 18 - Comparison of Speed Distribution for Platoon Leaders Using Turnout to Platoon Leaders Not Using Turnout and to All Platoon Leaders

The derivation of these values for the maximum safe entry speed is described in Appendix A. The maximum safe entry speed is that which will allow a vehicle that finds it cannot reenter the through lanes without creating a conflict to stop safely in the downstream half of the turnout. The recommended speeds are in good agreement with the actual maximum observed speeds in turnouts.

Table 25 shows that at turnouts that were used, turnout users constituted from 9.5 to 29.5% of all platoon leaders. Higher turnout usage rates ranging from 24.8 to 69.2%, were observed for platoon leaders traveling slowly enough to be classified as potential users. At 4 of the 6 turnouts that were used, more than half of the potential users actually used the turnout.

The turnout usage data indicate that the leaders of longer platoons are more likely to use turnouts than platoon leaders delaying only a single vehicle. Leaders of longer platoons not only travel more slowly than leaders of two-vehicle platoons, but they appear either to recognize that they are delaying more vehicles or else they tend to be a type of driver who is more willing to use a turnout. Turnout usage by leaders of platoons of six or more vehicles ranged from 21.4 to 100.0%. At 5 of the 6 turnouts that were used at all, the number of platoon leaders using the turnout equaled or exceeded the number of platoon leaders classified as potential users. A turnout usage rate greater than 100% for potential users implies that some leaders of longer platoons tend to use turnouts even when they are traveling above the median speed of platoon leaders.

Three turnouts received no use at all during the study period. One signed turnout that received no usage (Site R07) was 475 ft (145 m) in length and was located on a 3% downgrade about 2 miles (3.2 km) downstream from a long 8% upgrade. Of the 51 platoon leaders that passed the turnout during the 3-hr study period, only 4 vehicles were traveling slowly enough to be classified as potential users (below 47.3 mph or 25.7 kph). The speed distribution for platoon leaders at this site is such that the turnout would have to be 700 ft (215 m) long in order for 50% of the platoon leaders to be considered potential users. Drivers also appeared reluctant to use this turnout because the roadway alignment was favorable for traveling at or near their desired speeds for the first time after an extended upgrade.

Two unsigned turnouts that were evaluated in New York received no usage during a 2.5-hr study period. These two turnouts were located near one another on opposite sides of the roadway; one turnout was located on an 8% upgrade and the other on an 8% downgrade. It should be noted, however, that both turnouts were only 200 ft (60 m) in length and thus, had a maximum safe entry speed (29.8 mph or 47.9 kph) that made them unattractive for use by most drivers. Because of this relatively low safe entry speed, there were only 2 potential users for the upgrade turnout and no potential users for the downgrade turnout. A school bus was observed to use the upgrade turnout before the study period began. The driver of the school bus stopped in the turnout to allow 3 following vehicles to pass. Based on the limited available data, it was concluded that unsigned turnouts may receive some usage by drivers who are familiar with a particular highway, but that such turnouts generally provide minimal operational benefits.

It was mentioned in the discussion of current practice that some states require that any vehicle that is delaying another must use a turnout, while other states require only vehicles that are delaying five or more following vehicles to use turnouts. Substantial operational benefits are obtained from turnouts in the latter states only because many vehicles that are not required by law to use turnouts choose to do so. Table 26 shows that slow-moving vehicles delaying five or more following vehicles (i.e., platoons of six or more vehicles) are relatively rare at the traffic volume levels considered in this study. At the sites with flow rates over 200 vph, only 9.7% of all platoons were composed of six or more vehicles; at the low volume sites, less than 1% of all platoons were composed of six or more vehicles. Table 26 suggests strongly that the five-vehicle-delay requirement is unrealistic for most two-lane highways and use of the one-vehicle-delay requirement should be encouraged. Turnout use by vehicles with only a single following vehicle should also be encouraged because there is less likelihood than with multiple followers that a vehicle using the turnout will be forced to stop or will create a conflict with a following vehicle when reentering the highway. Reinforcement of the legal requirement with appropriate signing, such as the LAW REQUIRES SLOW VEHICLES TO USE TURNOUTS sign used in Oregon, is also recommended.

TABLE 26

DISTRIBUTION OF PLATOON LENGTHS ON TURNOUT APPROACHES

Site Number and Treatment Number	Percentage of All Platoons by Platoon Length				
	≧ 2 veh.	≧ 3 veh.	≧ 4 veh.	≧ 5 veh.	≧ 6 veh.
HIGH FLOW RATE (200-300 vph)					
C21-1	100.0	48.1	23.1	10.8	6.6
C21-2	100.0	55.3	32.1	22.3	22.1
R04-1	100.0	42.2	18.9	9.6	3.3
R04-2	100.0	50.8	25.6	12.4	6.8
Mean Value	100.0	49.1	24.9	13.8	9.7
LOW FLOW RATE (under 100 vph)					
R07-1	100.0	27.5	13.7	2.0	2.0
W04-2	100.0	29.5	11.5	6.4	1.3
W04-3	100.0	33.3	11.1	4.9	1.2
Y06-2	100.0	42.1	5.2	0.0	0.0
Y06-3	100.0	14.3	0.0	0.0	0.0
Mean Value	100.0	29.3	8.3	2.7	0.9

Several instances of undesirable maneuvers were observed at turnouts. Seventeen (17) vehicles or 0.2% of all vehicles stopped in the turnouts for reasons other than to allow following vehicles to pass. These stops lasted from 25 sec to 24 min. The apparent reasons that motorists stopped in turnouts included checking tires, rearranging load, changing drivers, reading a map and eating lunch. Stopping was particularly prevalent at the turnouts at Site W03 where no shoulders or other suitable stopping places were available elsewhere along the road. Nearly 2% of the motorists at Site W03 chose to stop in one of the two turnouts.

Nine (9) following vehicles made passes in the vicinity of the turnout when the leaders they were trailing chose not to use the turnout. All but one of these passing maneuvers occurred at one site (Site R07) with nearly level tangent alignment throughout the turnout area. No traffic conflicts were created by these passing maneuvers, but they are an indication that turnout usage is not as high as the drivers of some following vehicles would like.

No vehicles entered the turnouts by making left turns from the opposing direction. However, three vehicles traveling in the treated direction used the turnouts to make a U-turn.

4.3 Safety Evaluation

The safety evaluation of slow-moving vehicle turnouts shows that they operate safely. Accident data were obtained for 42 signed turnout sites for an average of 4.37 years at each site. Table 27 shows that the accident rates at turnouts are nearly identical in the treated and nontreated directions.* The "at turnout" accident and exposure data include the turnout itself and the through lanes from 150 ft (50 m) upstream to 150 ft (50 m) downstream of the turnout area. Thus, there is no indication that the presence of the turnout increases either the total or the fatal and injury accident rates.

The turnout sites, in both directions, have lower accident rates than the adjacent sections of two-lane highway. To make the comparison fair, only nonintersection accidents have been considered for the adjacent sections, since turnouts are not usually located near intersections. The adjacent sections extend from 0.20 to 0.50 miles (0.30 to 0.80 km) in each direction from each turnout, but exclude the "at turnout" site described above. It should be noted that both the turnout sites and their adjacent sections tend to have higher accident rates than passing lanes and their comparable untreated sections, respectively. This difference may occur because the turnouts are more likely to be located in mountainous terrain than passing lanes.

* It should be noted in Table 27 that the "at turnout" exposure is higher in the treated than in the nontreated direction because some turnouts are located opposite one another and, at such locations, there is no nontreated direction.

TABLE 27

COMPARISON OF NONINTERSECTION ACCIDENT RATES FOR TURNOUTS AND ADJACENT UNTREATED SECTIONS

Type of Location	Number of Sites	Number of Accidents		Exposure (MVM)	Accident Rate (accidents per MVM)	
		Total	Fatal and Injury		Total	Fatal and Injury
At turnouts - treated direction	43	37	15	22.27	1.66	0.67
At turnouts - nontreated direction	43	28	16	17.46	1.60	0.92
Adjacent untreated sections	-	201	115	84.57	2.38	1.36

Note: 1 mile = 1.609 km.

Accident data for periods before and after turnout construction were obtained from the participating states for four turnout sites, which contained a total of eight turnouts. The before and after study periods for each site were 2 years in length. There were only two accidents in 2 years at the eight turnout sites in the before period, and only two accidents in 2 years at the same sites after installation of the turnouts.

Table 28 compares the accident type distributions for the 37 accidents that occurred at turnouts with the 201 nonintersection accidents that occurred on the adjacent untreated sections. As might be expected, the turnouts have a higher proportion of multiple vehicle accidents and the adjacent sections have a higher proportion of single vehicle accidents.

TABLE 28

DISTRIBUTION OF ACCIDENT TYPES FOR NONINTERSECTION ACCIDENTS AT TURNOUTS AND AT ADJACENT UNTREATED SECTIONS

	Number of Accidents (percent of total accidents)	
Accident Type	At Turnout - Treated Direction	Adjacent Untreated Sections
SINGLE-VEHICLE, NONINTERSECTION	21 (56.8)	141 (70.1)
Collision with parked vehicle	0 (0.0)	0 (0.0)
Collision with pedestrian	0 (0.0)	0 (0.0)
Collision with animal	3 (8.1)	15 (7.5)
Collision with fixed object	12 (32.4)	87 (43.2)
Collision with other object	1 (2.7)	5 (2.5)
Other collision	0 (0.0)	0 (0.0)
Noncollision, overturning	3 (8.1)	32 (15.9)
Other noncollision	2 (5.4)	2 (1.0)
MULTIPLE-VEHICLE, NONINTERSECTION	16 (43.2)	60 (29.4)
Head-on	2 (5.4)	8 (4.0)
Rear-end	4 (10.8)	14 (7.0)
Sideswipe	7 (18.9)	24 (11.9)
Angle	2 (5.4)	10 (5.0)
Other	1 (2.7)	4 (2.0)
Total	37 (100.0)	201 (100.0)

The 16 multiple vehicle accidents that occurred at turnouts sites are of greatest interest. Of these 16 accidents:

- 3 involved sideswipe collisions between vehicles using the turnout and vehicles in the through lanes;

- 6 involved rear-end or sideswipe collisions with a vehicle stopped in the turnout; and

- 7 involved collisions in which a vehicle crossed the centerline into the opposing lanes.

The latter accidents were apparently not directly related to the turnout; possibly these accidents involved a vehicle attempting to pass another vehicle which did not use the turnout.

The available data do not indicate a persistent pattern of accidents related to turnouts. The data in Table 27 indicate that a typical turnout experiences less than one accident every 5 years. Based on the usage rate observed at the seven turnouts evaluated in the field (Sites C21, R04, R07, and W04), these seven turnouts experienced about one accident per 400,000 turnout users. This rate is even lower than the rate of one accident per 80,000 turnout users estimated by Rooney in California.[16]

The operational data obtained in the field suggest that between 5 and 10% of turnout users create a traffic conflict with a following vehicle when they return to the through lane. Given this conflict rate, the accident experience associated with the turnout exit maneuver (3 sideswipe collisions) is surprisingly low. This contrast in traffic conflict and accident rates implies that following drivers anticipate the possible return of the turnout vehicle to the through lanes and that their braking is a controlled response that does not indicate the likelihood of a collision.

None of the five unsigned turnouts evaluated in New York and Utah experienced any accidents related to the turnouts. However, the operational data collected in the field suggest that most unsigned turnouts receive very little use.

4.4 Summary

Turnouts can be effective in increasing the opportunities to pass slow-moving vehicles on two-lane highways. The provision of turnouts is most appropriate in mountainous terrain and on recreational routes in level or rolling terrain. Combination trucks do not generally use turnouts except on steep grades where their speeds are substantially reduced.

A single turnout can provide an opportunity for between 20 and 50% of the passing maneuvers that would occur in a 1-mile (1.6-km) passing lane in level terrain. However, passing maneuvers resulting from turnout usage may not provide as much operational benefit as passing maneuvers at passing lanes, because the passing vehicles at a turnout do not necessarily have higher desired speeds than the turnout users. Turnout usage can reduce the percentage of vehicles platooned by 2% immediately downstream of the turnout. Further decreases in platooning may occur downstream in the absence of steep grades and sharp horizontal curves.

There is a maximum safe entry speed for turnouts that varies with turnout length. Platoon leaders traveling above the maximum safe entry speed will generally not use a turnout. At turnouts of adequate length, between 9 and 30% of platoon leaders will use turnouts. The likelihood of a platoon leader using a turnout increases with decreasing speed and increases with increasing platoon length. Over 80% of the following vehicles in platoons immediately behind a turnout user are generally able to pass the turnout user. However, turnouts of inadequate length may be used only infrequently.

Legal requirements for turnout use vary from state to state. It is recommended that any slow-moving vehicle that is delaying another vehicle be required by law to use a turnout if one is provided. This legal requirement should be reinforced by signing (e.g., LAW REQUIRES SLOW VEHICLES TO USE TURNOUTS) in advance of every turnout. Public education and enforcement should be used to increase the proportion of slow-moving vehicles that use turnouts.

Turnouts were found to operate safely. A typical turnout experiences only one accident every 5 years. The accident rates of turnouts were found to be lower than the accident rates of adjacent untreated highway sections. Turnouts whose usage rates were evaluated in the field were found to experience only one accident per 400,000 turnout users. Although between 5 and 10% of turnout users caused a traffic conflict when reentering the highway from a turnout, the accident experience associated with this maneuver was minimal.

Unsigned and unpaved turnouts, not officially designated as turnouts by the state highway agency, were found to receive minimal use.

5. TWO-WAY LEFT-TURN LANES

A two-way left-turn lane (TWLTL) is a paved area in the highway median that extends continuously along a highway section and is marked to provide a deceleration and storage area, out of the through traffic stream, for vehicles traveling in either direction to make left turns into intersections and driveways. Figure 19 illustrates a typical TWLTL. Although TWLTLs are most commonly found on urban and suburban arterial streets, the emphasis of this report is on their application on two-lane highways in rural areas and urban fringe areas.

5.1 Current Practice

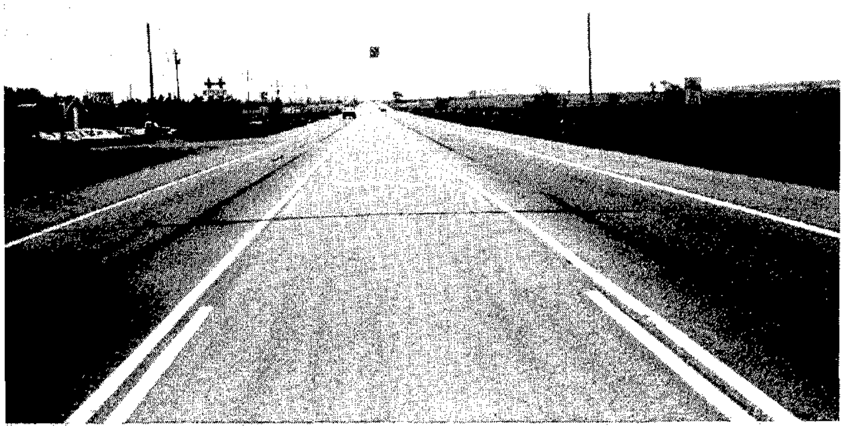
This section summarizes current state practice for the use of TWLTLs in rural and fringe areas including the current uses of TWLTLs; the characteristics of the sites studied; and state signing and marking practices.

5.1.1 Current uses of two-way left-turn lanes: TWLTLs have been used for many years on urban and suburban arterial streets. A TWLTL is often the most appropriate median treatment for arterials with strip commercial development, where left-turn demands are nearly continuous in space and time. Because they remove left-turning vehicles from the through traffic lanes, TWLTLs have been found to reduce stops and delays to through traffic and to reduce traffic accident rates by approximately 35% on urban and suburban arterials.[6]

Highway agencies have recently begun to use TWLTLs to obtain these same types of traffic operational and safety benefits on rural two-lane highways. TWLTLs are typically employed in rural areas at locations with some development (often roadside businesses) that have higher speeds and lower traffic flow rates than the TWLTL sections used in urban areas. TWLTLs have also been employed on two-lane highways in urban and small town fringe areas, which often have the potentially hazardous combination of dense development, frequent turning maneuvers, and high approach speeds.

Two other uses of TWLTLs in the rural setting are closely related to passing lanes. Some agencies that have experienced left-turn accident problems or complaints from residents who must make left-turns into their driveways in passing lane sections, have converted the center lane of the passing lane to a TWLTL. Such use of TWLTLs is obviously desirable at locations with documented left-turn accident problems, but these TWLTL sections may also extend into undeveloped areas with no left-turn demands. The use of a TWLTL in undeveloped areas may be detrimental operationally, without a corresponding safety benefit, since the presence of a TWLTL prevents legal passing maneuvers at locations with adequate sight distance where passing would otherwise be permitted. Furthermore, drivers have been observed to make illegal passing maneuvers in the TWLTL at such sites. TWLTLs have also been used as buffer areas between alternating passing lanes in opposite directions of travel.

California



Kansas

Figure 19 - Typical Two-way Left-turn Lane Sites on Two-lane Highways

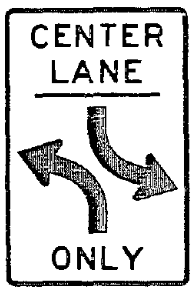
5.1.2 Characteristics of study sites: The current study evaluated 10 TWLTL sections in rural and fringe areas. These 10 sections were located in 8 states: Arkansas, California, Kansas, Kentucky, Nevada, Oregon, Pennsylvania and Utah.

Seven of the 10 TWLTL sections were located in rural areas with 55-mph (88-kph) speed limits. The length of these TWLTL sections ranged from 0.19 to 1.10 miles (0.31 to 1.77 km), with an average length of 0.54 miles (0.87 km). The driveway densities in these sections ranged from 5 to 30 driveways per mile (3 to 19 driveways per km). The traffic volumes at these 7 sites ranged from 2,800 to 8,800 vehicles per day. The width of the TWLTLs ranged from 12 to 16 ft (3.7 to 4.9 m).

Three of the 10 study sites (Sites P05, P07 and T04) were classified as fringe area sites located near urban areas or small towns. These sites have higher traffic volumes and lower traffic speeds than the rural area sites. The fringe area sites had traffic volumes ranging from 8,000 to 14,000 vehicles per day. The speed limits of the fringe area sites ranged from 35 to 45 mph (56 to 72 kph); in two of the three cases, the TWLTL site contained a transition from a 55-mph (80-kph) speed limit to a lower speed limit as the traffic entered the developed area. The driveway density in the developed portion of each fringe area TWLTL was over 40 driveways per mile (25 driveways per km).

5.1.3 Pavement markings: The pavement markings used to designate TWLTLs were uniform in all of the 8 participating states and follow in the guidelines of MUTCD Sections 3B-1 and 3B-2.[18] As shown in Figure 3-5a of the MUTCD, the TWLTL is marked on each side with a double yellow line -- a solid line on the side adjacent to the travel lane and a broken line on the side adjacent to the TWLTL.

5.1.4 Signing: The signing used for TWLTLs is nearly uniform among the states although placement practices vary. The states evaluated all used the regulatory (black-on-white) MUTCD symbol sign (R3-9b):



or a nonsymbol sign with an equivalent message (e.g., CENTER LANE LEFT TURN ONLY).

Most states use these signs mounted on posts on the right shoulder in each direction of travel at the beginning of and within the TWLTL section. One state employed overhead signs mounted on span wires at the beginning of the TWLTL in each direction. These signs were used in conjunction with a standard KEEP RIGHT sign (R4-7) post-mounted in the median at the beginning of the TWLTL. The latter sign became both a maintenance problem and a potential safety hazard, as it was struck repeatedly by vehicles entering the TWLTL.

5.2 Operational Evaluation

An operational evaluation of seven TWLTL sections -- four in rural areas and three in fringe areas -- was performed in the field. The following discussion presents the data collected as part of this evaluation, the measures of effectiveness used and the results obtained.

5.2.1 Data collection: Figure 20 summarizes the general data collection plan for each TWLTL site. TDR traps were placed on the roadway at three locations in each direction of travel -- upstream, downstream, and within the TWLTL. Variations from this plan were made where required in the field to adapt the plan to the geometrics of each site. For example, the TDR traps at one end of the site or the other had to be omitted at some locations, because of special circumstances such as a traffic signal or a passing lane adjacent to the TWLTL section. At other locations, two TDR traps rather than one were used within the TWLTL because of variations in the speed limit or degree of development (driveway density) within the site.

Operational data were also collected by manual observers at each site. The data collected manually included:

- Number of vehicles making right turns into driveways;
- Number of vehicles making left turns into driveways;
- Waiting time for left-turn vehicles (sec); and,
- Number of vehicles that would have been delayed by each left-turn vehicle if there were no TWLTL.

At TWLTLs less than 0.50 miles (0.80 km) in length, the manual data were collected for the entire TWLTL section. At longer sites, the manual data were collected for a selected portion of the TWLTL, usually the portion with the highest driveway density and/or the highest observed left-turn volumes. The length of the portion selected for evaluation at longer TWLTL sites ranged from 0.35 to 0.50 miles (0.56 to 0.80 km).

The length of the study period at each site was 6 hr, including the peak hour.

5.2.2 Measures of effectiveness: Two measures of effectiveness were used in the evaluation of TWLTLs. These measures are: potential delay due to left-turn vehicles and percent of traffic platooned.

There is no direct method to measure the reduction in delay resulting from installation of a TWLTL unless operational field studies are conducted both before and after TWLTL installation. Since before and after

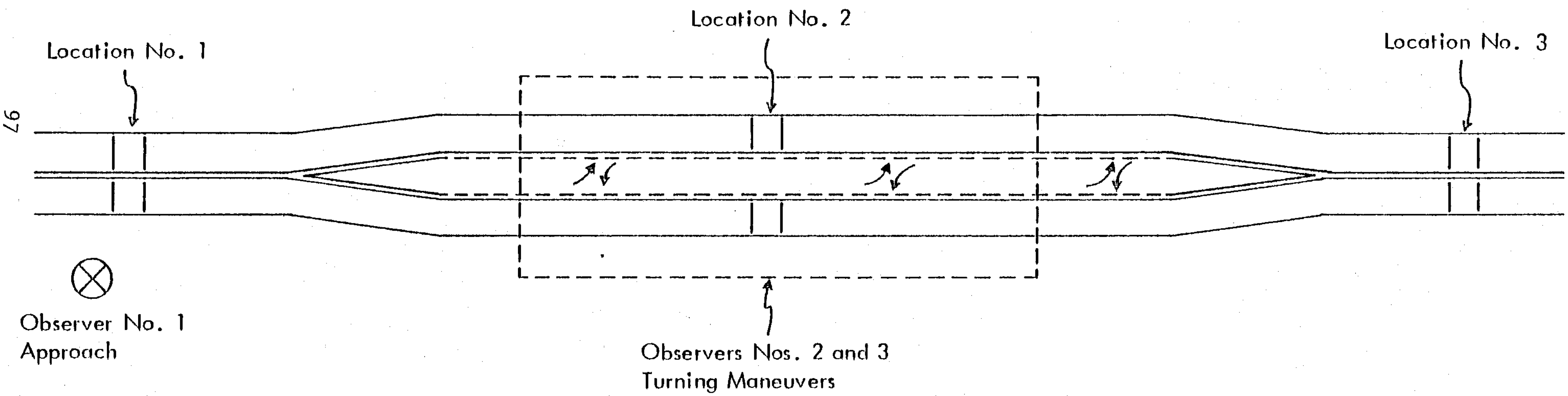


Figure 20 - Locations of TDR Traps and Observers for Data Collection at Turnouts

studies were not possible at existing TWLTL sites, a simple field method was developed to estimate the additional delay that would occur if the TWLTL were not there.

There are two types of delays that result from left-turns on two-lane highways: delay to left-turn vehicles and delay to vehicles following the left-turn vehicles. Left turn vehicles are often required to stop and are delayed while they wait for an adequate gap in opposing traffic to complete their turn. Since installation of a TWLTL does not alter the opposing traffic volume, there should be no major effect of TWLTL installation on left-turn waiting times. (There are two potential effects of TWLTLs that could influence left-turn waiting times. First, drivers of left-turn vehicles may wait for a longer gap to turn left at a TWLTL, since they can wait at a location less exposed to the risk of rear-end accidents. Second, the TWLTL eliminates some gaps in opposing traffic that would otherwise be created by left turns in the opposing direction. However, no data are available to quantify these effects).

The more important component of delay reduced by TWLTL installation is the delay to through vehicles following left-turn vehicles. Without a TWLTL, when a left-turning vehicle stops to wait for a gap in opposing traffic, vehicles following immediately behind must also stop and any further vehicles that arrive while the left-turn vehicle is waiting must join the queue. (It is assumed here that there is not a paved shoulder on the right wide enough to allow following vehicles to bypass the left-turning vehicle.) Delays of this sort are virtually eliminated if a TWLTL is provided. Thus, a reasonable estimate of the potential delay to following vehicles if there were no TWLTL can be made by noting the waiting times of left-turn vehicles and the number of following vehicles that pass them while they are waiting. The potential delay due to a left-turn maneuver was estimated in this study as:

$$PD = (N_I)\ (WT) + \frac{(N_L)(WT)}{2} \qquad (5)$$

where,
- PD = Potential delay to followers (veh-sec);
- N_I = Number of vehicles in platoon immediately behind left-turn vehicles;
- N_L = Number of vehicles passing left-turn vehicles after the initial platoon; and
- WT = Left-turn wait time (sec).

This expression assumes that all vehicles in the platoon immediately behind the left-turn vehicle are delayed for the entire left-turn wait time and that vehicles arriving after the initial platoon are delayed, on the average, for half of the left-turn wait time. The sum of the potential delays for all left-turn vehicles in a specified time period provides an estimate of the total delay reduced by a TWLTL.

A comparison of the percentage of vehicles platooned upstream and downstream of a TWLTL also provides a measure of TWLTL effectiveness. If there were no TWLTL present, the percentage of vehicles platooned would be expected to increase due to platoons formed by vehicles slowing and stopping to make left turns. Thus, if the percentage of vehicles platooned does not increase from upstream to downstream of a TWLTL, this is an indication that the TWLTL is working effectively.

5.2.3 Evaluation results: The operational evaluation included four rural TWLTL sites and three fringe area TWLTL sites. Effects of TWLTLs on both potential delay and percent of traffic platooned were considered. Since 6 hr of operational data were evaluated at each of the seven sites, and each direction of travel was evaluated separately, a total of 84 hr of data were available for the operational analysis. All flow rates in the following discussion are flow rates for one direction of travel only. The operational data collected at TWLTLs are summarized in Appendix D.

The traffic flow rates at the four rural TWLTL sites ranged from 60 to 310 vehicles per hour. The left-turn volumes ranged from 0to 62 vph in each direction, which represents a range from 0 to 27% of the total traffic volume. Flow rates and left-turn volumes were generally higher at the fringe area sites. The flow rates observed at these sites ranged from 375 to 1,100 vehicles per hour. Left-turn volumes ranged from 2 to 65 left-turns per hour, corresponding to a range from 1 to 11% of the total traffic volume.

Potential delay due to vehicles turning left: Very little potential delay was observed at rural TWLTL sites, especially those with flow rates below 200 vph. In fact, in 28 of the 48 hr observed at rural TWLTL sites there was no potential delay at all. During these 28 hr, either no vehicles turned left, no delay due to opposing traffic was experienced by vehicles that did turn left or no following vehicles were present that could potentially have been delayed. The highest level of potential delay observed at a rural TWLTL site was 212 vehicle-seconds for 62 vehicles that turned left during 1 hr. This level of potential delay corresponds to an estimate that each left-turn vehicle would cause an average of 3.4 seconds of delay if there were no TWLTL. Thus, it was concluded that the operational benefits from installation of TWLTLs in rural areas at flow rates less than 300 vph are minimal.

Substantially more potential delay was observed at the higher volume fringe area sites. There was no potential delay observed in only two of the 36 hr during which data were collected at the fringe sites. The lack of potential delay for those 2 hr was the result of very low left-turn volumes (2 and 4 vehicles per hour, respectively). Higher levels of delay were observed during most of the study period at fringe area sites. The highest level of potential delay observed in any hour was 5,317 veh-sec for 66 turning vehicles or 80.6 sec per left-turn vehicle. This maximum delay occurred during the hour between 1:00 and 2:00 p.m. with a flow rate of 525 vehicles per hour in the direction studied and 400 vehicles per hour in the opposing direction. These conditions were observed at Site T04, a highway with strip commercial development entering a small town in a recreational area. Thus, it was concluded that TWLTLs can provide substantial operational benefits under the higher volume conditions typical in fringe areas.

It would be desirable to be able to predict the potential delay in a TWLTL section as a function of key traffic flow variables such as left-turn volume, through volume, opposing volume and/or opposing percent of vehicles platooned. All of these predictor variables were found to have statistically significant correlations with potential delay. However, it was not possible to construct a regression model containing all of these variables, because each is highly correlated with the others. For example, the flow rate in one direction in any given hour has a correlation coefficient of 0.79 with the opposing direction during the same hour. The most useful predictor model for delay that was developed predicts potential delay per left-turning vehicle as a function of opposing flow rate. This model is presented in Equation (6):

$$\mathrm{PDPLT} = -\,6.87 + 0.058\ \mathrm{OFLOW} \tag{6}$$

where PDPLT = Potential delay per left-turning vehicle (sec); and

OFLOW = Opposing flow rate (vehicles per hour).

This model explains 32% of the variation in the dependent variable (i.e., $R^2 = 0.32$). Figure 21 presents this model and the data points used in its development.

The model presented in Equation (6) has a negative intercept term which although not physically meaningful, indicate that the operational benefits of a TWLTL are minimal at low flow rates. The model predicts that there is no delay reduction due to a TWLTL at flow rates below 120 vph and less than 10 sec potential delay per left-turn vehicle at flow rates below 300 vph. Because of the strong correlation between the flow rates in opposite directions, it is recommended that Equation (6) be applied only to sites where the flow rates in opposite directions are approximately equal.

Percent of vehicles platooned: The percent of vehicles platooned was essentially unchanged from upstream to downstream of TWLTL sites. At rural sites, the percent of traffic platooned decreased, on the average, from 25.3% upstream to 25.0% downstream of the TWLTL. In fringe areas, the percent of traffic platooned decreased from 53.9% upstream to 53.1% downstream of the TWLTL. Both of these changes in platooning are not statistically significant ($p=0.52$ and $p=0.47$ for rural and fringe areas, respectively). Thus, it appears that the TWLTLs evaluated meet the objective of preventing increased platooning due to delays caused by left-turning vehicles.

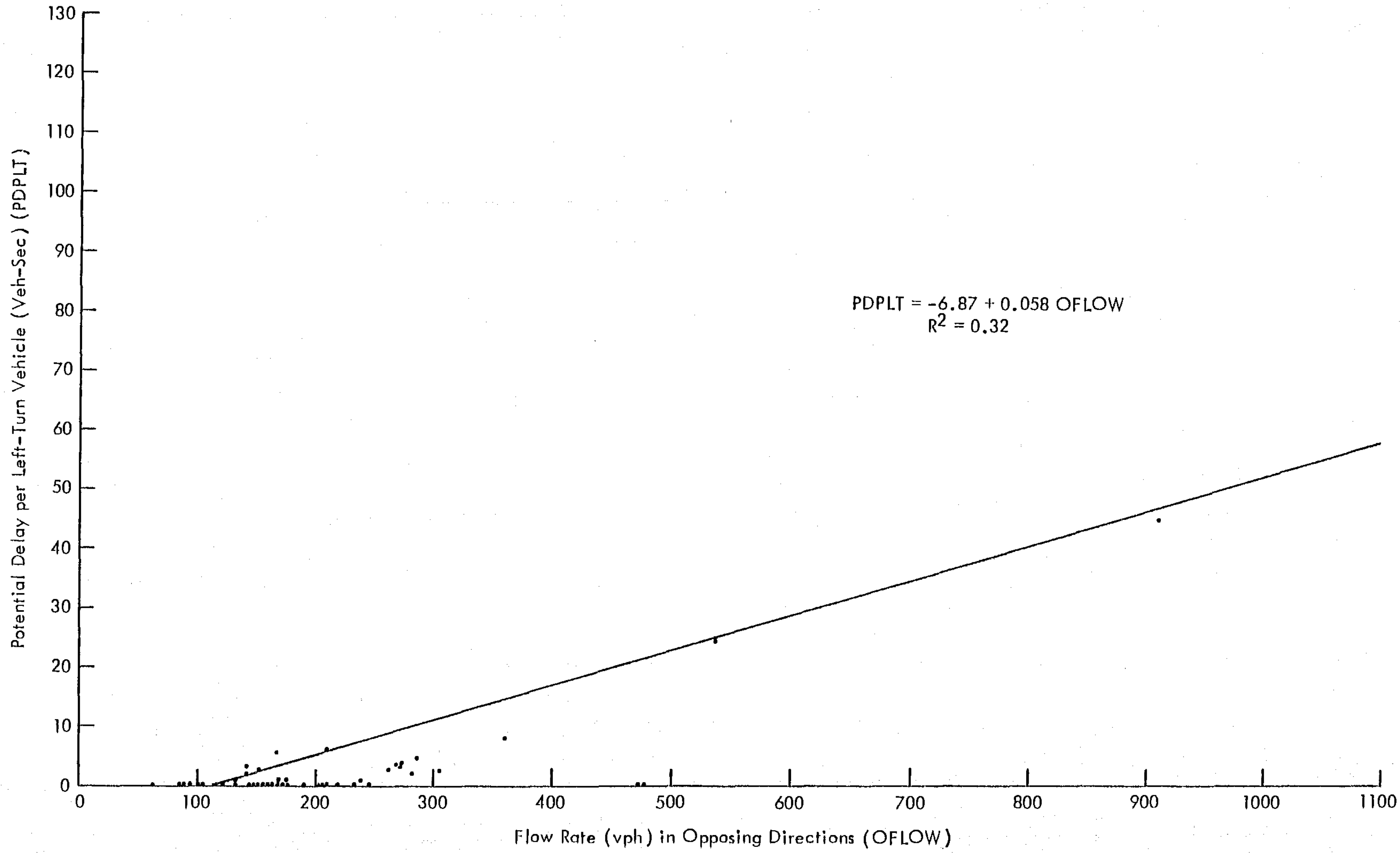


Figure 21 - Relationship to Predict Potential Delay Reduced by Two-way Left-turn Lanes on Two-lane Highways

The percent of traffic platooned within the TWLTL can vary considerably as vehicles make turns off of and onto the through lanes. This is particularly true of rural TWLTL sites with a single major traffic generator, such as a rural grocery store. At one such location (Site C01), the percent of traffic platooned was reduced by 4 to 6% from just upstream to just downstream of the store as platoon leaders turned off of the highway. Platooning was higher in the TWLTL section than on the upstream approach, in about 75% of the cases observed, and lower in the remaining 25%.

5.3 Safety Evaluation

A safety evaluation was conducted to quantify the effectiveness of TWLTL sections on rural two-lane highways. Seven TWLTL sites in rural areas were used in this evaluation, along with four untreated two-lane sites selected by the participating states as comparable to four of the TWLTL sites.

The time period for which accident data were available was 5 years for four of the TWLTL sites and 1 year for the remaining three sites. The accident study period was relatively short at the latter three sites, because the TWLTLs were recently installed. Data for 3 to 4 years prior to TWLTL installation were also available at these sites. The accident study period was 5 years for all but one of the comparable untreated sites, which had a 3 year study period.

Accident data were available for only one of the three fringe area TWLTL sites evaluated in the field (Site T04). It was decided to omit this one site from the safety analyses described below, so that the evaluation would address TWLTL effectiveness in the rural setting, with lower volumes and higher speeds than the urban and suburban settings where TWLTLs have often been evaluated before.*

5.3.1 Comparisons between treated and untreated sites: Table 29 presents the observed accident rates for TWLTL sites and comparable untreated two-lane highways. The four comparable untreated sites were selected by the participating states and are roughly comparable to four of the treated sites in traffic volume and degree of development (driveway density). Table 29 shows that the TWLTL sites experienced total accident rates and fatal and injury accident rates that are both less than 30% of the accident rates on the comparable untreated sections.

Table 30 makes this comparison more explicit. The table shows the differences in accident rates for each TWLTL site and its comparable untreated site.

* It is interesting to note, however, that the installation of the TWLTL at Site T04 reduced the total accident rate by 34.7%, which is virtually identical to the accident reduction effectiveness estimates for TWLTLs reported in the literature.[19]

TABLE 29

COMPARISON OF ACCIDENT RATES FOR TWLTL SITES AND COMPARABLE UNTREATED SITES

Type of Location	Number of Sites	Number of Accidents		Exposure (MVM)	Accident Rate (accidents per MVM)	
		Total	Fatal and Injury		Total	Fatal and Injury
TWLTL	7	23	12	26.82	0.856	0.447
Comparable Untreated	4	88	48	28.04	3.138	1.712

Note: 1 mile = 1.609 km

TABLE 30

COMPARISON OF ACCIDENT RATES FOR TWLTL SITES AND COMPARABLE UNTREATED SITES

TWLTL Site	Comparable Site	Total Accident Rate (accidents per MVM) TWLTL	Total Accident Rate (accidents per MVM) Comparable	Total Accident Rate (accidents per MVM) Difference	Fatal and Injury Accident Rate (accidents per MVM) TWLTL	Fatal and Injury Accident Rate (accidents per MVM) Comparable	Fatal and Injury Accident Rate (accidents per MVM) Difference
C01	C11	1.230	7.030	-5.800	0.000	2.636	-2.636
K01	K02	0.789	3.302	-2.513	0.197	1.501	-1.304
N05	N15	0.000	2.379	-2.379	0.000	1.190	-1.190
R10	R09	0.263	2.950	-2.687	0.263	2.023	-1.760
Mean Values		0.571	3.915	-3.345 SIG (p=0.03)	0.115	1.838	-1.723 SIG (p=0.01)

Note: 1 mile = 1.609 km

In every case, the total accident rate and the fatal and injury accident rate for the TWLTL site are substantially lower than for the comparable untreated site. The differences in both total accident rate and fatal and injury accident rate shown in Table 30 are statistically significant (p=0.03 and p=0.01, respectively).

5.3.2 Comparison between TWLTL sites before and after TWLTL installation: Similar indications of the safety effectiveness of TWLTLs are found in the comparison of accident rates before and after TWLTL installation in Table 31. The table shows that, at the four sites for which before and after data are available, total accident rate was reduced 85% and fatal and injury accident rate was reduced 67% by TWLTL installation. The reduction in total accident rate is statistically significant at the 90% confidence level (p=0.07), while the reduction in fatal and injury accident rate is not statistically significant (p=0.44). The latter result is nearly inevitable, because very small numbers of fatal and injury accidents are involved and two of the four sites experienced no fatal and injury accidents in either the before or the after period.

TABLE 31

COMPARISON OF ACCIDENT RATES FOR TWLTL SITES
BEFORE AND AFTER TWLTL INSTALLATION

Site	Total Accident Rate (accidents per MVM)			Fatal and Injury Accident Rate (accidents per MVM)		
Number	Before	After	Difference	Before	After	Difference
A01	2.561	0.000	-2.651	0.000	0.000	0.000
K01	2.884	0.789	-2.095	1.202	0.197	-1.005
N05	1.461	0.000	-1.461	0.000	0.000	0.000
R10	0.264	0.264	0.000	0.176	0.264	0.088
Mean Values	1.793	0.263	-1.552 SIG (p=0.07)	0.345	0.115	0.229 NS (p=0.44)

Note: 1 mile = 1.609 km.

5.3.3 Distribution of accident types: The distributions of accident types in TWLTL sections and comparable two-lane highways are compared in Table 32. In interpreting Table 32, it should be kept in mind that only 23 accidents occurred in the TWLTL sections, so that the sample size of TWLTL accidents is really too small to fully define the accident distribution. Nevertheless, the comparison is striking when one considers that the comparable untreated sites, where 88 accidents occurred, had only about 5% more vehicles-miles of exposure than the TWLTL sites.

TABLE 32

DISTRIBUTION OF ACCIDENT TYPES FOR TWO-WAY LEFT-TURN LANES AND COMPARABLE TWO-LANE HIGHWAYS

Accident Type	Number of Accidents (percent of total accidents)			
	TWLTL Sites		Comparable Untreated Sites	
SINGLE-VEHICLE, NONINTERSECTION	3	(13.0)	16	(18.2)
Collision with Parked Vehicle	0	(0.0)	1	(1.1)
Collision with Pedestrian	0	(0.0)	2	(2.3)
Collision with Animal	0	(0.0)	1	(1.1)
Collision with Fixed Object	2	(8.7)	7	(8.0)
Collision with Other Object	0	(0.0)	0	(0.0)
Other Collision	0	(0.0)	0	(0.0)
Noncollision, overturning	0	(0.0)	1	(1.1)
Other noncollision	1	(4.3)	4	(4.5)
MULTIPLE-VEHICLE, NONINTERSECTION	6	(26.1)	49	(55.7)
Head-on	0	(0.0)	4	(4.5)
Rear-end	1	(4.3)	26	(29.5)
Sideswipe	0	(0.0)	7	(8.0)
Angle	4	(17.4)	12	(13.6)
Other	1	(4.3)	0	(0.0)
SINGLE-VEHICLE, INTERSECTION	1	(4.3)	2	(2.3)
Collision with Parked Vehicle	0	(0.0)	0	(0.0)
Collision with Pedestrian	0	(0.0)	0	(0.0)
Collision with Animal	0	(0.0)	1	(1.1)
Collision with Fixed Object	1	(4.3)	1	(1.1)
Collision with Other Object	0	(0.0)	0	(0.0)
Other Collision	0	(0.0)	0	(0.0)
Noncollision, overturning	0	(0.0)	0	(0.0)
Other noncollision	0	(0.0)	0	(0.0)
MULTIPLE-VEHICLE, INTERSECTION	13	(56.5)	21	(23.9)
Head-on	1	(4.3)	0	(0.0)
Rear-end	4	(17.4)	10	(11.4)
Sideswipe	1	(4.3)	0	(0.0)
Angle	7	(30.4)	11	(12.5)
Other	0	(0.0)	0	(0.0)
Total	23	(100.0)	88	(100.0)

The TWLTL sites experienced fewer nonintersection accidents -- both single-vehicle and multiple-vehicle -- than the comparable untreated sites, in both absolute numbers and as a percentage of total accidents. In particular, the TWLTL sites experienced substantially less of all of the major types of multiple-vehicle, nonintersection accidents -- head-on, rear-end, sideswipe and angle -- than the comparable sites. There is no indication, either in the data collected for this study or in the literature, of any problem associated with head-on accidents on TWLTLs. Table 33 shows the number of rear-end and angle accidents involving left-turns. It is apparent from the table that left-turn accidents, particularly nonintersection left-turn accidents, appear to be much less frequent at the TWLTL sites than at the untreated sites.

TABLE 33

FREQUENCY OF REAR-END AND ANGLE ACCIDENTS INVOLVING LEFT-TURNS

	TWLTL Sites	Comparable Untreated Sites
Nonintersection	3	10
Intersection	6	17
Total	9	27

5.3.4 Traffic conflicts and erratic maneuvers: A field study of traffic conflicts and erratic maneuvers in TWLTLs was conducted as an adjunct to the safety study.

The collection of traffic conflict data was intended to identify any potential safety problems not apparent in accident data. Figure 22 illustrates the three types of traffic conflicts that were considered in TWLTLs -- rear-end conflicts, opposing left-turn conflicts and entering-from-driveway conflicts. As shown in Table 34, very few conflicts were observed. The most frequently observed conflict type was the opposing left-turn conflict, which occurred in 5 (or 0.7%) of the 664 left-turn maneuvers observed. It is notable, however, that all 5 opposing left-turn conflicts were observed at the same site, which had the highest flow rate of any of the rural TWLTL sites. Nevertheless, there is no indication of any potential safety problem in TWLTLs based on the results of the traffic conflict study.

The erratic maneuver study was intended to identify any potential problems related to lack of driver compliance with the traffic control devices (signing and marking) that define the TWLTL. Four types of erratic maneuvers were considered -- left-turn from the through lane (not using the TWLTL), driving in the TWLTL, passing in the TWLTL and making a U-turn across the TWLTL. These maneuvers are illustrated in Figure 22. Table 34 indicates that passing in the TWLTL was the only one of these maneuvers observed with any frequency. Although this violation was committed by only 30 (or 0.4%) of the 8,290 vehicles that passed through the sites during the field study period, it was the only type of traffic conflict or erratic maneuver that was observed at each of the four sites.

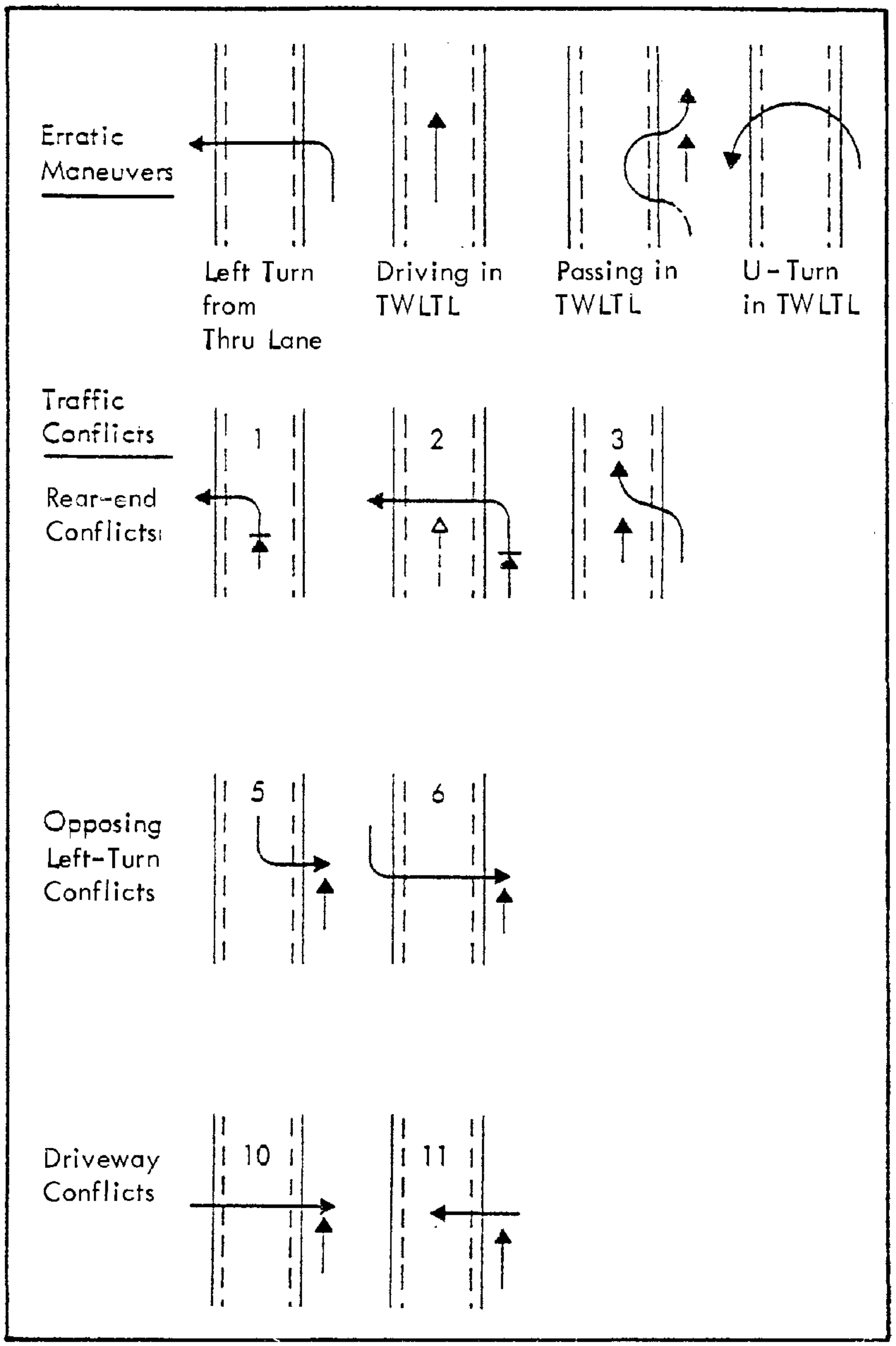


Figure 22 - Traffic Conflict and Erratic Maneuver Types Evaluated in Two-way Left-turn Lanes

TABLE 34

TRAFFIC CONFLICTS AND ERRATIC MANEUVERS AT RURAL TWLTL SITES

			Number of Traffic Conflicts			Number of Erratic Maneuvers			
Site Number	Total Volume[a]	Left-Turn Volume[a]	Rear End	Opposing Left Turn	Entering from Driveway	Left Turn from Thru Lane	Driving in TWLTL	Passing in TWLTL	U-turn in TWLTL
C01	2,063	86	0	0	0	2	0	10	0
K01	1,208	45	0	0	0	0	5	6	0
R14	3,118	316	0	5	0	2	1	3	1
U01	1,901	217	0	0	1	0	1	11	1
Totals	8,290	664	0	5	1	4	7	30	2

[a] In both directions of travel, during 6 hr study period.

Since some drivers will pass illegally in TWLTLs, a careful evaluation is recommended of a proposed TWLTL installation that would eliminate an existing passing zone.

5.4 Summary

The operational benefits of TWLTLs in rural areas with flow rates below 300 vph are minimal. However, rural TWLTLs can reduce accident rates up to 85%. There is no indication, in either the data collected for this study or in the literature, of any problem related to head-on accidents in TWLTLs. The use of TWLTLs in rural areas is recommended at sites where a pattern of left-turn accidents susceptible to correction by a TWLTL is found. The installation of a TWLTL in rural areas should be carefully reviewed if it would eliminate an existing passing zone, to assure that this operational disbenefit is accompanied by an expected safety benefit.

TWLTLs will provide both operational and safety benefits in fringe areas with flow rates over 300 vph. The delay reduced per left-turn vehicle by installation of a TWLTL can be estimated using Equation (6) on page 100. The estimated reduction in accident rate resulting from TWLTLs in fringe areas is 35%.

6. CONCLUSIONS

Each of the five operational treatments for two-lane highways evaluated in this study were found to improve traffic operations on two-lane highways. Passing lanes, short four-lane sections and two-way left-turn lanes were found to improve safety as well. Traffic volumes at the treated sites ranged from 1500 to 15,000 vehicles /day. The specific conclusions reached in the study are presented below.

6.1 Passing Lanes

The evaluation of passing lanes concluded that:

1. The percentage of vehicles platooned is reduced to nearly half of its upstream level within a passing lane (from 35.1% of vehicles following in platoons to 20.7% of vehicles following in platoons, on the average).

2. The percentage of vehicles platooned immediately downstream of a passing lane is, on the average, 6% less than upstream of a passing lane (29.2% vs. 35.1%), and can be as much as 20% less during some hours at some sites. This reduction in vehicle platooning can be predicted as a function of the upstream percent of vehicles platooned and the length of the passing lane using Equation (1) on page 27.

3. The reduction in the percent of vehicles platooned may persist for several miles downstream from a passing lane, or may disappear within the first mile, depending on vehicle mix and geometrics.

4. Vehicle speeds within a passing lane are, on the average, about 2.2 mph (3.5 kph) higher than speeds upstream of the passing lane and 1.4 mph (2.3 kph) higher than speeds downstream of the passing lane. These speed differences are, however, strongly influenced by local geometrics.

5. The passing rate in the treated direction of a passing lane can be predicted as a function of flow rate, passing lane length and upstream percent of vehicles platooned using Equation (4) on page 33. Prohibition of passing by opposing direction vehicles at a passing lane eliminates passing opportunities at locations that might otherwise be good passing zones. If passing by opposing direction vehicles is permitted, the resulting passing rates for these vehicles may be higher than that found on conventional two-lane highways.

6. The installation of a passing lane on a two-lane highway does not increase the accident rate. In fact, accident rates are probably reduced by passing lanes.

7. The rate of cross-centerline accidents, involving vehicles traveling in opposite directions, was found to be the same or lower on passing lane sections than on untreated two-lane highways at all severity levels, even for passing lanes where passing by opposing direction vehicles is permitted. Passing lanes with passing by opposing direction vehicles permitted can be operated safely at flow rates up to at least 400 vph in each direction.

It is recommended that States adopt more uniform policies for advance signing of passing lanes. In particular, advance signs from 2 to 5 miles (3 to 8 km) in advance of a passing lane may help reduce the risk of accidents caused by driver frustration and impatience with the lack of passing opportunities.

Further research on passing lanes is recommended to better define the effect of passing lane length on the operational effectiveness of passing lanes; to determine how far downstream the operational benefits of a passing lane persist; and to refine the effectiveness estimates presented in this report through computer simulation modeling that will help to eliminate the effects of local geometrics on the measures of effectiveness.

6.2 Short Four-Lane Sections

The evaluation of short four-lane sections concluded that:

1. Short four-lane sections operate in a similar manner to passing lanes and provide the same traffic operational benefits as passing lanes in both directions of travel at the same location.

2. Short four-lane sections had substantially lower accident rates than comparable untreated two-lane highways.

3. Accident rates for cross-centerline accidents, involving vehicles traveling in opposite directions on short four-lane sections, were found to be less than half of the rates found on comparable untreated sections.

The additional research on the operational effects of passing lanes recommended above should provide results that are also applicable to short four-lane sections.

6.3 Shoulder Use Sections

The evaluation of shoulder use sections concluded that:

1. Slow-moving vehicles will use shoulders designated by signing for their use. Up to 8% of the total traffic volume and 40% of platoon leaders were observed to use the shoulder in such sections.

2. Shoulders designated for use by slow-moving vehicles provide minimal operational benefits at flow rates below 100 vph.

3. At flow rates above 100 vph, shoulder use by slow-moving vehicles does provide traffic operational benefits, but these benefits are estimated to be only one-fifth of the traffic operational benefits provided by a passing lane of comparable length.

4. There is no indication of any safety problem associated with designation of a shoulder section for use by slow-moving vehicles.

5. Conversion of a shoulder to a travel lane ("poor-boy" conversion) provides operational benefits similar to a passing lane. Drivers will treat the shoulder as a normal travel lane unless it is narrower than the adjacent lane or has limited lateral clearance.

Further investigation of shoulder use sections is recommended to determine the operational benefits that would be obtained if all slow-moving vehicles used the shoulder.

6.4 Turnouts

The evaluation of turnouts concluded that:

1. Turnouts can be effective in increasing the opportunities to pass slow-moving vehicles on two-lane highways. A single turnout can provide an opportunity for between 20 and 50% of the passing maneuvers that would occur in a 1-mile (1.6-km) passing lane in level terrain. However, passing maneuvers resulting from turnout usage may not provide as much operational benefit as passing maneuvers at passing lanes, because the passing vehicles at a turnout do not necessarily have higher desired speeds than the turnout users.

2. Turnout usage can reduce the percentage of vehicles platooned by 2% immediately downstream of the turnout. Further decreases in platooning may occur downstream in the absence of steep grades and sharp horizontal curves.

3. There is a maximum safe entry speed for turnouts that varies with turnout length. For example, the maximum safe entry speed for a 500-ft (150-m) turnout is 48.6 mph (72.8 kph). Platoon leaders traveling above the maximum safe entry speed will generally not use a turnout.

4. Turnouts operate safely. A typical turnout experiences only one accident every 5 years.

It is recommended that any slow vehicle that is delaying another vehicle be required by law to use a turnout if it can safely do so. This legal requirement should be reinforced by signing (e.g., LAW REQUIRES SLOW VEHICLES TO USE TURNOUTS) in advance of every turnout. Additional public education and enforcement may be needed to increase the proportion of slow-moving vehicles that use turnouts.

Further research concerning turnouts is recommended to investigate the downstream operational benefits of turnout usage on steep grades and in level terrain; to determine the benefits that could be achieved if turnout usage were increased; and, to develop specific guidelines for locating turnouts effectively so that they will be used.

6.5 Two-Way, Left-Turn Lanes

The evaluation of two, left-turn lanes (TWLTLs) concluded that:

1. The operational benefits of TWLTLs in rural areas with flow rates below 300 vph are minimal. However, the installation of TWLTLs in rural areas can reduce accident rates up to 85%.

2. TWLTLs in urban fringe areas with flow rates over 300 vph provide both operational and safety benefits. The delay reduction per left-turn vehicle by installation of a TWLTL can be estimated using Equation (6) on page 100. The estimated reduction in accident rate resulting from a TWLTL in a fringe area is 35%.

The installation of TWLTLs on highways with flow rates below 300 vph should be based primarily on safety considerations. At higher volume levels, both operational and safety benefits should be evaluated. The likelihood of illegal passing in the TWLTL should be considered carefully in the design and location of new TWLTL sections, especially if the TWLTL would eliminate an existing passing zone.

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APPENDIX A

ESTIMATION OF MAXIMUM SAFE ENTRY SPEED FOR TURNOUTS

The traffic operations at two turnouts were examined in detail to provide guidance for evaluations of turnouts in general. The insights gained in this investigation were used to estimate the maximum safe entry speed for turnouts. The results will also be useful in the development of computer simulation models of turnout usage.

The detailed examination of turnout operations was made with printouts summarizing individual vehicle passages in both directions upstream of the turnout, at the turnout threshold, midway in the turnout lane, and immediately downstream of the turnout. In most cases, it was possible to identify and trace the vehicles using the turnout and the platoons involved.

The results indicated that:

- Almost all vehicles entered the turnouts with speeds between 34 and 55 mph (55 and 88 kph). All speeds of turnout users were less than or equal to 55 mph (88 kph) and only 4 vehicle speeds were beteen 27 and 34 mph (43 and 55 kph).

- Turnout users that did not reduce their speed before the turnout midpoint were passed by either 0 or 1 following vehicles or, in a very few cases, 2 vehicles. None of these turnout users stopped.

- For turnout users that reduced speed by 3 to 14 mph (5 to 22 kph) before the turnout midpoint, there were increasing numbers of users with 1, 2, or 3 following vehicles able to pass without the turnout users stopping. There was one case of 4 followers passing without forcing the turnout user to stop.

- For turnout users that were forced to stop in the turnout, the speed reduction observed in the turnout midpoint varied from 3 to 28 mph (5 to 45 kph). At the turnout midpoint, all vehicles that eventually stopped had speeds between 15 and 41 mph (24 and 66 kph).

- Turnout users were forced to stop in all cases but one where 4 to 8 followers passed.

The speeds of turnout users and nonusers have been summarized in Figure 14 in Section IV-B of this report.

Based on the observations described above, a simple analytical model of turnout use was developed and evaluated. The goal of the model was to determine how turnouts of different lengths should be expected to operate. The model results provide a maximum turnout entry speed that will permit both operational benefits to followers and safety for turnout users. Here, the requirement for operational benefits means the opportunity for one or two followers to pass while the turnout makes a moderate speed reduction at midpoint and then proceeds without being forced to stop. The safety constraint on entry speed requires that the turnout user quickly reduces speed by 3 mph (5 kph), coasts at constant speed to the midpoint of the turnout and then is able to stop, if necessary, in the remaining half of the turnout length using a maximum deceleration of 10 ft/sec^2 (3 m/sec^2). The effective length of each turnout was approximated as 60 ft (18 m) less than the total length including tapers.

Tests with the model indicated that both the operational and safety requirements for turnouts, described above, can be met with the same maximum turnout entry speed. The resulting equation for maximum safe entry speed is:

$$V_{max} = 3.4 + \sqrt{4.65\ (L-60)} \qquad (7)$$

where:

V_{max} = maximum safe entry speed (mph), and

L = turnout length (ft)

Equation (7) was used to derive the maximum safe entry speeds for turnouts tabulated in Section IV-B of this report.

APPENDIX B

TRAFFIC OPERATIONAL MEASURES OBTAINED FROM ANALYSIS OF TDR FIELD DATA

This appendix presents examples of the output obtained from computer analysis of the traffic operational data collected in the field with MRI's Traffic Data Recorder (TDR) system. Two computer programs have been used to analyze these data. The first program processes the raw TDR data collected in the field and creates a file containing the number of axles, wheelbase, vehicle type, speed, acceleration and headway of each vehicle. A sample of the vehicle-by-vehicle output obtained from the first program is presented in Table 35.

The second program processes the output file created by the first program and determines for specified time periods the traffic flow rate; vehicle type distribution (passenger car/truck/RV); platooning status distribution (free vehicle/platoon leader/follower); speed parameters, including mean speed, standard deviation of speed, and various percentiles of the speed distribution, for the total traffic flow; the same speed parameters by vehicle type and by platooning status; and the distribution of platoon lengths and lead vehicle types. The program also has the capability to trace RVs identified at one location by a TDR code and to identify those same vehicles when they pass another station further upstream or downstream. A sample of the output from the second program is presented in Table 36.

TABLE 35

EXAMPLE OF PRINTED OUTPUT FROM FIRST ANALYSIS PROGRAM

TDR OUTPUT FILE

IVEHNO	NO. OF AXLES	TRANSIT TIMES(HMS) UPSTREAM	DOWNSTREAM	HEADWAY (SEC)	SPEED (FT/SEC)	ACCELERATION (FT/SEC**2)	WHEELBASE (FT)	LATERAL PLACEMENT (FT)	VEH TYPE	TIME TO COLLISION (SEC)	FOLLOWING DISTANCE (FT)
1	2	11.45.16.121	11.45.16.344	-99.000	67.164	0.0	11.08	-99.00	5	-99.00	-99.00
2	4	11.47.11.289	11.47.11.500	115.168	70.922	-0.531	36.77	-99.00	10	60.00	5000.00
3	2	11.48. 8.617	11.48. 8.867	57.331	60.133	0.542	11.23	-99.00	5	-99.00	4023.27
4	2	11.48.50.129	11.48.50.320	41.508	77.798	1.291	10.25	-99.00	3	60.00	2478.79
5	-99	11.49. 3.738	11.49. 4.141	13.611	-99.000	-99.000	-99.00	-99.00	13	-99.00	1042.64
6	3	11.49.54.590	11.49.54.777	50.853	80.812	-1.018	28.33	-99.00	13	-99.00	-99.00
7	2	11.50.14.496	11.50.14.695	19.905	75.630	0.0	10.34	-99.00	5	-99.00	1574.24
8	2	11.50.16.641	11.50.16.852	2.144	71.146	0.0	7.83	-99.00	1	-99.00	145.83
9	2	11.52.40.965	11.52.41.152	144.325	80.717	0.0	10.83	-99.00	5	60.00	5000.00
10	5	11.53.24.105	11.53.24.297	43.140	78.947	0.0	50.39	-99.00	10	-99.00	3465.32
11	2	11.54.32.445	11.54.32.645	68.338	74.206	-1.306	8.59	-99.00	1	-99.00	5000.00
12	2	11.55. 8.277	11.55. 8.520	35.832	61.644	0.0	9.09	-99.00	2	-99.00	2644.40
13	2	11.55.10.906	11.55.11.168	2.631	57.809	0.532	10.17	-99.00	3	-99.00	147.08
14	2	11.55.18.008	11.55.18.215	7.099	72.289	0.0	9.76	-99.00	3	27.22	394.23
15	2	11.55.38.961	11.55.39.160	20.956	75.314	0.0	9.48	-99.00	2	60.00	1499.12
16	2	11.55.40.656	11.55.40.855	1.693	74.689	0.0	11.14	-99.00	5	-99.00	112.05
17	2	11.55.45.230	11.55.45.441	4.577	71.146	0.0	11.26	-99.00	13	-99.00	324.68
18	2	11.55.56.902	11.55.57.082	11.668	82.723	-1.695	9.16	-99.00	2	60.00	812.89
19	2	11.56.23.973	11.56.24.172	27.072	75.000	0.0	9.19	-99.00	2	-99.00	2224.35

TABLE 36

EXAMPLE OF PRINTED OUTPUT FROM SECOND ANALYSIS PROGRAM

A01A1 NORTHBOUND TREATED DIRN

RV DATA BASED ON RVS IDENTIFIED AT THIS LOCATION
PLATOON DEFINITION BASED ON MAXIMUM HEADWAY OF 4.00 SECONDS

PERIOD	DURATION (MIN)	NO. OF VEHS	LOST TIME	USABLE TIME	NO. OF VEHICLES BY 15-MIN PERIODS			
1145-1245	60.	125	0.0	60.00	23	27	42	33
1245-1345	60.	133	0.0	60.00	25	36	43	29
1345-1445	60.	121	0.0	60.00	35	22	33	31
1445-1545	60.	135	0.21	59.79	35	38	34	28
1545-1645	60.	142	0.0	60.00	35	34	42	31
1645-1745	60.	152	0.0	60.00	33	39	54	26
COMB	360.	808	0.21	359.79				

TRAFFIC VOLUME AND COMPOSITION

PERIOD	FLOW RATE (VPH)	% CARS	% TRUCKS	% RVS	% FREE	% LEADERS	% FOLLOWERS
1145-1245	125	73.6	14.4	12.0	55.2	20.0	24.8
1245-1345	133	66.9	21.8	11.3	53.4	19.5	27.1
1345-1445	121	69.4	18.2	12.4	58.7	16.5	24.8
1445-1545	135	64.4	23.0	12.6	58.5	18.5	23.0
1545-1645	142	73.9	17.6	8.5	60.6	17.6	21.8
1645-1745	152	78.3	14.5	7.2	59.2	16.4	24.3
COMB	135	71.3	18.2	10.5	57.7	18.1	24.3

SPEED SUMMARY (MPH)

PERIOD		MEAN SPEED	STD DEV	SPEED PERCENTILES 85%ILE	50%ILE	15%ILE	MIN SPEED	MAX SPEED
1145-1245	TOTAL	52.7	5.76	58	53	47	39	68
1245-1345	TOTAL	52.7	6.92	59	53	48	21	70
1345-1445	TOTAL	52.6	5.62	58	53	47	37	68
1445-1545	TOTAL	53.7	5.94	60	53	49	31	74
1545-1645	TOTAL	53.6	4.77	58	54	49	36	73
1645-1745	TOTAL	53.4	5.85	59	54	48	37	76
	COMB	53.2	5.84	59	53	48	21	76
1145-1245	CARS	53.0	5.94	59	53	47	39	68
1245-1345	CARS	52.6	6.57	59	53	48	32	68
1345-1445	CARS	52.5	5.89	58	53	45	37	68
1445-1545	CARS	54.0	6.36	61	54	49	31	72
1545-1645	CARS	53.4	4.94	58	54	49	36	73
1645-1745	CARS	53.6	5.57	59	54	49	40	67
	COMB	53.2	5.86	59	53	48	31	73

TABLE 36

EXAMPLE OF PRINTED OUTPUT FROM SECOND ANALYSIS PROGRAM (continued)

1145-1245	TRCKS	51.2	4.52	56	52	48	39	58
1245-1345	TRCKS	53.2	8.82	59	55	48	21	70
1345-1445	TRCKS	52.6	4.88	57	53	47	45	65
1445-1545	TRCKS	52.9	4.58	58	53	49	40	61
1545-1645	TRCKS	53.6	4.50	58	54	50	45	65
1645-1745	TRCKS	52.7	7.40	60	52	47	40	76
	COMB	52.8	6.06	58	53	48	21	76
1145-1245	RVS	53.0	5.99	64	55	47	46	64
1245-1345	RVS	52.4	4.84	59	53	48	42	59
1345-1445	RVS	53.1	5.34	62	53	48	46	63
1445-1545	RVS	53.7	6.09	56	53	51	47	74
1545-1645	RVS	55.3	3.67	59	56	52	50	64
1645-1745	RVS	53.3	5.90	57	55	53	37	58
	COMB	53.4	5.33	58	54	48	37	74
1145-1245	FREE	53.8	5.63	59	54	48	39	68
1245-1345	FREE	53.9	7.14	59	54	49	21	70
1345-1445	FREE	54.6	5.63	60	56	49	37	68
1445-1545	FREE	55.1	6.60	61	55	49	31	74
1545-1645	FREE	54.8	5.17	59	55	50	36	73
1645-1745	FREE	55.0	5.69	61	55	49	37	76
	COMB	54.6	5.98	60	55	49	21	76
1145-1245	LEAD	51.2	5.55	57	51	47	39	62
1245-1345	LEAD	51.6	6.98	59	52	46	34	69
1345-1445	LEAD	49.5	4.42	56	50	45	39	56
1445-1545	LEAD	51.7	3.97	55	53	51	40	58
1545-1645	LEAD	51.7	3.16	55	52	49	45	56
1645-1745	LEAD	52.5	4.26	57	54	49	40	59
	COMB	51.4	4.91	56	52	47	34	69

PLATOONING SUMMARY

	FREE	NO. OF PLATOONS BY LENGTH										MEAN	LEAD VEHICLE TYPE		
PERIOD	VEHS	2	3	4	5	6	7	8	9	>=10	TOT	LENGTH	% CARS	% TRUCKS	% RVS
1145-1245	69	21	2	2	0	0	0	0	0	0	25	2.2	56.0	32.0	12.0
1245-1345	71	19	6	0	0	1	0	0	0	0	26	2.4	61.5	26.9	11.5
1345-1445	71	10	10	0	0	0	0	0	0	0	20	2.5	65.0	10.0	25.0
1445-1545	79	19	6	0	0	0	0	0	0	0	25	2.2	56.0	36.0	8.0
1545-1645	86	22	2	0	0	1	0	0	0	0	25	2.2	84.0	12.0	4.0
1645-1745	90	19	4	0	1	0	1	0	0	0	25	2.5	76.0	8.0	16.0
COMB	466	110	30	2	1	2	1	0	0	0	146	2.3	66.4	21.2	12.3

APPENDIX C

LIST OF STUDY SITES

This appendix identifies the sites that were used in the study. Table 37 identifies the location of each study site for each treatment type and Table 38 identifies the locations of the comparable treated sites. For each site, the tables show the state, site number, treatment number, county, route, boundaries in mileposts or stations, length and approximate ADT. The table also identifies the treatments that were selected for field evaluation.

TABLE 37

LOCATIONS OF TREATED STUDY SITES

State	Site Number	Treatment Number	County	Route	Mileposts	Length (miles)	Approx. ADT (veh/day)	Field Study Site
				PASSING LANE SITES				
Arkansas	A01	1	Boone	US-65	0.40-2.65 (SB)	2.25	3,640	No
		2	Boone	US-65	3.04-4.44 (NB)	1.40	3,640	No
		4	Boone	US-65	4.85-5.87 (SB)	1.02	3,140	No
		5	Boone	US-65	6.21-7.08 (NB)	0.87	3,140	No
		6	Boone	US-65	7.08-10.19 (SB)	3.11	3,140	No
		7	Boone	US-65	10.66-12.43 (NB)	1.77	3,140	Yes
		8	Boone	US-65	13.02-14.76 (SB)	1.74	3,140	No
		9	Boone	US-65	15.30-17.21 (SB)	1.91	3,950	No
California	C05	1	Solano	SR-37	R0.65-R1.50 (WB)	0.85	15,600	No
		2	Solano	SR-37	R1.50-R3.50 (EB)	2.00	15,600	Yes
		3	Solano	SR-37	R3.90-R5.60 (WB)	1.70	15,600	No
		4	Solano	SR-37	R5.50-R6.80 (EB)	1.30	15,600	No
	C06[a]	1	Calaveras	SR-12	13.32-13.86 (EB)	0.54	5,250	Yes
	C07	1	Mono	US-395	97.50-98.30 (SB)	0.80	3,600	No
	C08	1	Mono	US-395	104.10-105.10 (NB)	1.00	3,600	Yes
	C09	1	San Luis Obispo	SR-46	57.20-57.55 (EB)	0.35	3,590	No
		2	San Luis Obispo	SR-46	57.30-57.70 (WB)	0.40	3,590	No
		3	San Luis Obispo	SR-46	58.70-59.10 (EB)	0.40	3,590	No
		4	San Luis Obispo	SR-46	58.95-59.35 (WB)	0.40	3,590	No
	C10	1	San Diego	SR-67	10.61-11.09 (NB)	0.48	7,700	No
Kentucky	T01	1	Perry	US-15	20.90-22.04 (NB)	1.14	7,070	Yes
		2	Perry	US-15	21.69-23.66 (SB)	1.97	7,070	No
	T02	1	Laurel	US-25	22.50-23.23 (NB)	0.73	1,390	No
		2	Luarel	US-25	20-92-22.45 (SB)	1.53	1,390	Yes
Michigan	M01	1	Osceola	SR-115	2.20-3.10 (WB)	0.90	5,700	No
		2	(C.S. 67051)	SR-115	2.30-3.60 (EB)	1.30	5,700	No
		3	(C.S. 67051)	SR-115	4.00-5.00 (EB)	1.00	5,700	No
	M02	1	Wexford (C.S. 83052)	SR-115	6.60-8.80 (EB)	2.2[2]	4,200	No
	M03	1	Clare	SR-115	8.30-9.10 (EB)	0.80	4,200	No
		2	(C.S. 18011)	SR-115	8.60-9.20 (WB)	0.60	4,200	No
	M04	1	Baraga (C.S. 07012)	US-41	14.30-16.10 (NB)	1.80	2,900	No
	M05	1	Kalkaska (C.S. 40002)	SR-72	14.50-16.45 (WB)	1.90	3,700	No

TABLE 37 (continued)

State	Site Number	Treatment Number	County	Route	Mileposts		Length (miles)	Approx. ADT (veh/day)	Field Study Site
Mississippi	S01	1	Lawrence	US-84	0.97-2.10	(EB)	1.13	2,850	No
		2	Lawrence	US-84	2.54-3.07	(EB)	0.53	2,850	No
	S02	1	Marion	US-98	264+00-285+00	(EB)	0.40	3,700	No
		2	Marion	US-98	271+00-309+00	(WB)	0.72	3,700	No
	S03	1	George	US-98	204+00-237+00	(EB)	0.63	3,950	No
		2	George	US-98	352+50-386+00	(EB)	0.64	3,950	No
Nevada	N01	1	Douglas	US-395	5.32-5.97	(NB)	0.65	3,310	Yes
	N03	1	Douglas	US-395	32.38-33.19	(NB)	0.81	10,300	No
	N06	1	Douglas	US-395	11.66-12.32	(SB)	0.66	3,270	No
	N07	1	Douglas	US-395	14.20-15.50	(SB)	1.30	3,260	No
Oklahoma	H01	1	Ellis	US-60	8.65-9.00	(WB)	0.35	1,150	No
	H03	1	Harper	US-183	8.90-9.80	(SB)	0.90	800	No
	H05	1	Woods	US-64	0.90-1.25	(WB)	0.35	1,250	No
	H07	1	Woods	US-281	12.50-13.40	(WB)	0.90	980	No
	H09	1	Woods	US-287	2.42-3.08	(NB)	0.66	1,350	No
	H11	1	Major/ Woodward	US-183	0.80 Major- 0.80 Woodward	(NB)	1.40	3,200	No
		2	Major/ Woodward	US-183	1.00 Major- 1.30 Woodward	(SB)	1.70	3,200	No
		3	Woodward	US-183	1.60-2.40	(NB)	0.80	3,200	No
		4	Woodward	US-183	4.50-7.00	(NB)	2.50	3,200	No
		5	Woodward	US-183	4.70-5.70	(SB)	1.00	3,200	No
		6	Woodward	US-183	6.60-8.00	(SB)	1.40	3,200	No
		7	Woodward	US-183	8.40-8.90	(NB)	0.50	3,200	No
		8	Woodward	US-183	14.90-17.80	(NB)	2.90	3,200	No
		9	Woodward	US-183	16.00-17.80	(SB)	1.80	3,200	No
		10	Woodward	US-183	18.80-19.50	(SB)	0.70	3,200	No
		11	Woodward	US-183	20.10-22.60	(NB)	2.50	3,200	No
		12	Woodward	US-183	21.50-22.80	(SB)	1.30	3,200	No
		13	Woodward	US-183	24.10-26.30	(NB)	2.20	3,200	Yes
Oklahoma	H12	1	Beaver	US-270	1.20-1.70	(EB)	0.50	1,500	No
		2	Beaver	US-270	1.60-2.00	(WB)	0.40	1,500	No
		3	Beaver	US-270	2.40-4.00	(EB)	1.60	1,500	No
		4	Beaver	US-270	3.10-4.20	(WB)	1.10	1,500	No
		5	Beaver	US-270	5.30-5.80	(WB)	0.50	1,500	No
		6	Beaver	US-270	6.70-6.90	(WB)	0.20	1,500	No

TABLE 37 (continued)

LOCATIONS OF TREATED STUDY SITES

State	Site Number	Treatment Number	County	Route	Mileposts		Length (miles)	Approx. ADT (veh/day)	Field Study Site
Oregon	R03	1	Deschutes	US-97	147.65-149.13	(SB)	1.51	8,600	No
	R05	1	Douglas/Coos	US-101	215.74-217.37	(SB)	1.63	6,560	No
		2	Douglas/Coos	US-101	217.78-218.08	(NB)	0.30	6,560	No
		3	Douglas/Coos	US-101	220.15-221.21	(NB)	1.06	6,560	No
	R11	1	Douglas	SR-38	20.84-21.51	(EB)	0.67	2,970	Yes
		2	Douglas	SR-38	21.68-22.16	(WB)	0.48	2,970	No
		3	Douglas	SR-38	23.04-23.59	(EB)	0.55	2,970	No
		4	Douglas	SR-38	24.09-24.57	(WB)	0.48	2,970	No
		5	Douglas	SR-38	25.05-25.53	(EB)	0.48	2,970	No
		6	Douglas	SR-38	26.13-26.55	(WB)	0.42	2,970	No
	R13	1	Douglas	SR-38	45.00-45.51	(EB)	0.51	2,200	No
		2	Douglas	SR-38	45.15-45.73	(WB)	0.58	2,200	No
Pennsylvania	P04	4	Snyder	US-11/15	16+750-25+520	(NB)	1.66	12,000	Yes
Utah	U04	1	Utah	US-6	180.96-181.93	(EB)	0.97	5,840	No
	U05	1	Utah	US-6	204.78-205.45	(EB)	0.67	4,300	No
	U06	1	Utah	US-6	217.77-218.56	(WB)	0.79	4,325	No
	U07[b]	1	Summit/Wasatch	US-40	5.05-7.12	(WB)	2.07	6,100	Yes
Washington	W01	1	King	SR-18	6.60-7.09	(EB)	0.49	12,300	No
		2	King	SR-18	9.68-10.07	(EB)	0.39	12,300	No
		3	King	SR-18	9.89-10.31	(WB)	0.42	12,300	Yes
	W02	1	King	SR-18	20.20-23.89	(EB)	3.69	6,800	No
		2	King	SR-18	23.44-25.72	(WB)	2.28	6,800	No
	W06	1	Clallam	US-101	218.24-219.17	(EB)	0.93	2,800	No
		2	Clallam	US-101	219.03-220.93	(WB)	1.90	2,800	No
	W07	1	Clallam	US-101	239.54-241.73	(EB)	2.19	4,400	No
	W11	1	King	SR-169	19.69-20.41	(NB)	0.49	9,000	No
		2	King	SR-169	21.38-22.18	(SB)	0.80	9,000	No
				SHORT FOUR-LANE SITES					
New York	Y02	1	Cayuga	SR-5	3.37-3.62		0.25	6,490	No
		3	Cayuga	SR-5	3.88-4.15		0.27	6,490	No
	Y03	1	Albany	US-9W	4.65-5.48		0.83	6,710	Yes
	Y04	1	Dutchess	US-9	11.55-13.95		2.40	7,350	Yes
	Y07	1	Hamilton	SR-30	78.15-78.30		0.15	1,630	No
		3	Hamilton	SR-30	78.65-79.20		0.55	1,630	No
	Y08	1	Hamilton	SR-30	80.80-81.10		0.30	1,630	No
	Y09	1	Cayuga	SR-5	2.35-3.02		0.67	8,600	No

TABLE 37 (continued)

State	Site Number	Treatment Number	County	Route	Mileposts	Length (miles)	Approx. ADT (veh/day)	Field Study Site
Oregon	R02	1	Deschutes	US-97	131.58-132.60	1.02	9,800	Yes
Washington	W08	1	Clallam	US-101	252.70-255.26	2.56	10,650	No
				SHOULDER USE SITES				
New York	Y05	1	Essex	SR-73	52.90-56.40 (NB)	3.50	1,450	Yes
Washington	W03	1	Grays Harbor	US-101	99.90-100.60 (NB)	0.70	4,020	Yes
		2	Grays Harbor	US-101	100.12-100.71 (SB)	0.59	4,020	Yes
	W10	1	Jefferson	US-101	296.90-299.37 (SB)	2.47	1,800	Yes
				TURNOUT SITES				
California	C03	1	Trinity	SR-299	45.70-45.78 (EB)	400	1,390	No
		2	Trinity	SR-299	46.00-46.11 (EB)	570	1,390	No
		3	Trinity	SR-299	48.40-48.60 (WB)	1,200	1,390	No
		4	Trinity	SR-299	49.16-49.24 (WB)	450	1,390	No
		5	Trinity	SR-299	49.35-49.50 (WB)	800	1,390	No
		6	Trinity	SR-299	50.55-50.65 (WB)	550	1,390	No
	C21	1	Madera	SR-41	12.55-12.67 (NB)	600	6,250	Yes
		2	Madera	SR-41	15.15-15.30 (NB)	750	6,250	Yes
		3	Madera	SR-41	15.70-15.82 (NB)	600	6,250	No
		4	Madera	SR-41	22.10-22.20 (SB)	500	4,700	No
		5	Madera	SR-41	23.70-23.80 (NB)	500	4,700	No
		6	Madera	SR-41	25.35-25.52 (NB)	850	4,700	No
		7	Madera	SR-41	25.40-25.50 (SB)	500	4,700	No
		8	Madera	SR-41	26.10-26.21 (NB)	550	4,700	No
		9	Madera	SR-41	29.09-29.30 (SB)	1,100	4,700	No
		10	Madera	SR-41	29.70-29.81 (SB)	530	4,700	No
		11	Madera	SR-41	29.94-30.00 (SB)	300	4,700	No
		12	Madera	SR-41	30.50-30.64 (NB)	700	4,700	No
		13	Madera	SR-41	31.00-31.10 (NB)	500	4,700	No
		14	Madera	SR-41	31.04-31.20 (SB)	800	4,700	No
		15	Madera	SR-41	31.20-31.26 (NB)	300	4,700	No
		16	Madera	SR-41	32.19-32.25 (SB)	300	4,700	No
		17	Madera	SR-41	32.50-32.62 (NB)	600	4,700	No
		18	Madera	SR-41	33.20-33.27 (NB)	350	4,700	No

TABLE 37 (continued)

LOCATIONS OF TREATED STUDY SITES

State	Site Number	Treatment Number	County	Route	Mileposts	Length (miles)	Approx. ADT (veh/day)	Field Study Site
				TURNOUT SITES (Concluded)				
		19	Madera	SR-41	34.03-34.15 (SB)	600	3,000	No
		20	Madera	SR-41	38.50-38.60 (NB)	500	4,700	No
		21	Madera	SR-41	39.70-39.84 (NB)	700	3,000	No
		22	Madera	SR-41	40.75-40.90 (NB)	750	3,000	No
		23	Madera	SR-41	40.78-40.90 (SB)	600	3,000	No
		24	Madera	SR-41	42.85-42.91 (NB)	300	3,000	No
		25	Madera	SR-41	43.60-43.68 (NB)	400	3,000	No
		26	Madera	SR-41	44.40-44.52 (NB)	600	3,000	No
		27	Madera	SR-41	44.28-44.40 (SB)	600	3,000	No
New York	Y06[c]	1	Essex	SR-73	43.37-43.41 (NB)	200	1,200	No
		2	Essex	SR-73	43.51-43.55 (SB)	200	1,200	Yes
		3	Essex	SR-73	43.73-43.77 (NB)	200	1,200	Yes
Oregon	R04	1	Douglas	US-101	201.00-201.11 (SB)	600	4,400	Yes
		2	Douglas	US-101	201.00-201.11 (NB)	600	4,400	Yes
	R07	1	Lane	SR-58	58.50-58.59 (EB)	475	2,300	Yes
Utah	U14[c]	1	Utah	US-6	199.87-199.93 (EB)	300	4,300	No
		2	Utah	US-6	200.49-200.60 (EB)	550	4,300	No
Washington	W04	1	Grays Harbor	US-101	141.05-141.11 (SB)	300	2,000	No
		2	Grays Harbor	US-101	141.54-141.63 (NB)	500	2,000	Yes
		3	Grays Harbor	US-101	142.89-142.96 (NB)	400	2,000	Yes
	W09	1	Clallam	US-101	261.43-261.53 (EB)	500	10,100	No
	W10	2	Jefferson	US-101	296.06-296.14 (NB)	400	2,200	No
		3	Jefferson	US-101	296.29-296.44 (SB)	800	2,200	No
		4	Jefferson	US-101	299.59-299.71 (SB)	600	2,200	No

TABLE 37 (Concluded)

State	Site Number	Treatment Number	County	Route	Mileposts	Length (miles)	Approx. ADT (veh/day)	Driveways per Mile	Field Study Site
TWO-WAY LEFT-TURN LANE SITES									
Arkansas	A01	3	Boone	US-65	4.44-4.85	0.41	3,640	-	No
California	C01	1	Humboldt	SR-299	40.36-40.66	0.30	2,970	23	Yes
Kansas	K01	1	Cloud	US-81	100+00-152+67	1.00	2,780	5	Yes
Kentucky	T04	1	Floyd	US-23	17.30-18.10	0.80	8,010	55	Yes
Nevada	N05	1	Washoe	SR-341	21.50-21.69	0.19	7,120	-	No
Oregon	R10	1	Polk	SR-18	20.14-21.24	1.10	6,300	30	No
	R14	1	Lane	SR-58	3.65-4.14	0.49	8,800	8	Yes
Pennsylvania	P05	1	Dauphin	US-22/322	44+290-66+680	4.24	12,000	13[d]	Yes
	P07	1	Lancaster	US-30	22+715-30+790	1.63	14,000	41	Yes
Utah	U01	1	Duchesne	US-40	106.27-106.57	0.30	3,750	17	Yes

[a] A passing lane was constructed at site C06 after the end of the accident study period. This site was considered an untreated (control) site in the accident analysis, but the treated site was studied in the field.

[b] Climbing lane site.

[c] Unsigned turnouts.

[d] Over 40 driveways per mile in southermost 0.5-mile of site.

TABLE 38

LOCATIONS OF COMPARABLE UNTREATED STUDY SITES

State	Site Number	Treatment Number	County	Route	Mileposts	Length (miles)	Approx. ADT (veh/day)	Corresponding Treated Site
COMPARABLE UNTREATED SITES-PASSING LANES								
Arkansas	A11		Boone/ Marion	US-62	Boone 0.00-9.74 Marion 0.00-8.45	18.19	3,650	A01
Caifornia	C06		Calaveras	SR-12	12.80-14.37	1.57	5,250	None
	C15		Sonoma	SR-37	3.80-R5.98	2.18	15,600	C05
	C16		Calaveras	SR-12	0.00-5.00	5.00	4,740	None
	C17		Mono	US-395	98.80-103.60	4.80	3,600	C07, C08
	C19		SanLuis Obispo	SR-46	R17.00-R19.30	2.30	2,550	C09
	C20		San Diego	SR-67	14.90-18.50	3.60	8,540	C10
Michigan	M11		Osceola (C.S. 67501)	SR-115	8.00-10.10	2.10	3,500	M01
Mississippi	S11		Lincoln	US-84	14.79-18.48	3.69	2,870	S01
Nevada	N11		Douglas	US-395	0.00-2.56	2.56	3,310	N01, N06, N07
Oklahoma	H02		Ellis	US-60	10.30-11.50	1.20	1,150	H01
	H04		Harper	US-183	5.90-7.27	1.37	800	H03
	H06		Dewey	SR-34	4.83-6.63	1.80	900	H05
	H08		Dewey	US-183	17.10-19.16	2.06	1,450	H07
	H10		Dewey	SR-34	6.95-9.20	2.25	1,000	H09
Oregon	R12		Douglas	SR-38	41.00-42.50	1.50	2,200	R11, R13
Washington	W14		Clallam	US-101	192.15-193.80	1.65	4,300	W06, W07
COMPARABLE UNTREATED SITES-SHORT FOUR-LANE								
New York	Y12		Cayuga	SR-5	4.65-5.50	2.90	6,490	Y02, Y09
			Onondaga	SR-5	0.00-2.05			
	Y13		Albany	US-9W	2.05-4.05	2.00	6,710	Y03
	Y14		Dutchess	US-9	5.65-9.25	3.60	7,350	Y04
	Y17		Hamilton	SR-30	64.45-66.65	2.20	1,630	Y07
	Y18		Hamilton	SR-30	66.95-68.35	1.40	1,630	Y08
Oregon	R01		Jefferson	US-97	110.00-112.00	2.00	5,900	R02

TABLE 38 (Concluded)

State	Site Number	Treatment Number	County	Route	Mileposts	Length (miles)	Approx. ADT (veh/day)	Corresponding Treated Site
COMPARABLE UNTREATED SITES-SHOULDER USE								
New York	Y15		Essex	SR-73	56.95-61.65	4.70	1,450	Y05

State	Site Number	Treatment Number	County	Route	Mileposts	Length (miles)	Approx. ADT (veh/day)	Driveways per Mile	Corresponding Treated Site
COMPARABLE UNTREATED SITES-TWO-WAY LEFT-TURN LANES									
California	C11		Humboldt	SR-299	39.34-39.55	0.21	2,970	52	C01
Kansas	K02		Marshall	US-36	32+60-125+00	1.75	3,130	25	K01
Nevada	N15		Washoe	SR-341	20.30-21.00	0.70	6,580	-	N05
Oregon	R09		Lincoln	SR-18	4.75-6.00	1.25	5,200	30	R10

☆U.S. GOVERNMENT PRINTING OFFICE:1986—620—753-

FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH, DEVELOPMENT, AND TECHNOLOGY

The Offices of Research, Development, and Technology (RD&T) of the Federal Highway Administration (FHWA) are responsible for a broad research, development, and technology transfer program. This program is accomplished using numerous methods of funding and management. The efforts include work done in-house by RD&T staff, contracts using administrative funds, and a Federal-aid program conducted by or through State highway or transportation agencies, which include the Highway Planning and Research (HP&R) program, the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board, and the one-half of one percent training program conducted by the National Highway Institute.

The FCP is a carefully selected group of projects, separated into broad categories, formulated to use research, development, and technology transfer resources to obtain solutions to urgent national highway problems.

The diagonal double stripe on the cover of this report represents a highway. It is color-coded to identify the FCP category to which the report's subject pertains. A red stripe indicates category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, and green for category 9.

FCP Category Descriptions

1. Highway Design and Operation for Safety

Safety RD&T addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act. It includes investigation of appropriate design standards, roadside hardware, traffic control devices, and collection or analysis of physical and scientific data for the formulation of improved safety regulations to better protect all motorists, bicycles, and pedestrians.

2. Traffic Control and Management

Traffic RD&T is concerned with increasing the operational efficiency of existing highways by advancing technology and balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, coordinated signal timing, motorist information, and rerouting of traffic.

3. Highway Operations

This category addresses preserving the Nation's highways, natural resources, and community attributes. It includes activities in physical maintenance, traffic services for maintenance zoning, management of human resources and equipment, and identification of highway elements that affect the quality of the human environment. The goals of projects within this category are to maximize operational efficiency and safety to the traveling public while conserving resources and reducing adverse highway and traffic impacts through protections and enhancement of environmental features.

4. Pavement Design, Construction, and Management

Pavement RD&T is concerned with pavement design and rehabilititation methods and procedures, construction technology, recycled highway materials, improved pavement binders, and improved pavement management. The goals will emphasize improvements to highway performance over the network's life cycle, thus extending maintenance-free operation and maximizing benefits. Specific areas of effort will include material characterizations, pavement damage predictions, methods to minimize local pavement defects, quality control specifications, long-term pavement monitoring, and life cycle cost analyses.

5. Structural Design and Hydraulics

Structural RD&T is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highway structures at reasonable costs. This category deals with bridge superstructures, earth structures, foundations, culverts, river mechanics, and hydraulics. In addition, it includes material aspects of structures (metal and concrete) along with their protection from corrosive or degrading environments.

9. RD&T Management and Coordination

Activities in this category include fundamental work for new concepts and system characterization before the investigation reaches a point where it is incorporated within other categories of the FCP. Concepts on the feasibility of new technology for highway safety are included in this category. RD&T reports not within other FCP projects will be published as Category 9 projects.