Low-Cost Methods For Improving Traffic Operations On Two-Lane Roads

• Informational Guide

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FOREWORD

This report contains guidance on the use of low-cost methods for improving traffic operations on two-lane rural roads to relieve passing restrictions, and to reduce delay caused by turning movements. These recommendations are of interest to highway engineers and designers, maintenance engineers, and road supervisors who have a responsibility for relieving traffic congestion and delay, and preserving safety on two-lane rural roads.

Special recognition is made to the contributions of Dr. Chris Hoban of the Australian Road Research Board for his participation in the development of this guide. Through a series of workshops presented by Dr. Hoban, Mr. Douglas W. Harwood of the Midwest Research Institute, Dr. Ramey O. Rogness of North Dakota State University, and Dr. Carroll J. Messer of the Texas Transportation Institute, the contents of this Guide were reviewed by highway practitioners from highway agencies representing a wide variety of roadway types.

As output from the workshop presentations, training kits consisting of this Guide, Instructors' Notes, student handouts, and visual aids are being prepared for distribution to Technology Transfer centers throughout the country.

Distribution of the Guide is being made to each Federal Highway Administration Region and Division Office. A limited number of additional copies may be obtained from the Offices of Research, Development, and Technology, HRD-11, McLean, Virginia 22101-2296.

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16. Abstract

This report is an Informational Guide for highway agencies on the use of low-cost improvements to alleviate operational problems on two-lane highways. The Guide addresses both passing and turning improvements that can be constructed for a lower cost than construction of a continuous four-lane highway. The passing improvements presented in the Guide include passing lanes, climbing lanes, short four-lane sections, turnouts, shoulder driving, and shoulder use sections. The turning improvements included are intersection turn lanes, shoulder bypass lanes, and two-way left-turn lanes. Design and traffic control guidelines presented for these improvements include location and configuration guidelines, geometric design criteria, recommended signing and marking practices, and operational and safety effectiveness estimates. Guidelines for planning operational improvements over an extended section of two-lane highway are provided. The Guide also reviews evaluation procedures for passing and turning improvements, with particular emphasis on the application of the 1985 Highway Capacity Manual procedures for two-lane highways. A case study involving the evaluation of passing lane effectiveness on two-lane highways is also presented.

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1. INTRODUCTION

The vast majority of the American highway system is made of two-lane rural roads. Most of these roads carry relatively low traffic volumes, and have few operational problems. However, a substantial mileage of two-lane roads experiences operational and safety problems on a regular basis. These problems can often be related to three basic causes:

- Inadequate road geometry (e.g., width, grades, alignment, and sight distances), either at specific locations or over long sections;

- Lack of passing opportunities, due either to limited sight distance or heavy oncoming traffic volumes; and

- Traffic conflicts due to turns at intersections and driveways.

Many of these operational and safety problems could be overcome by extensive highway realignment, construction of four-lane divided highways, provision of access control, and major intersection improvements. Indeed, many highway agencies in the United States have regarded the construction of a four-lane divided highway as the most desirable response to operational problems on two-lane highways. Such improvements are expensive, however, and funds for road upgrading are very limited in many jurisdictions. This has led to a growing backlog of rural roads requiring improvement. In this situation it is possible to provide major upgrading on only a small proportion of the network, while many other candidate road sections continue to operate unsatisfactorily.

An alternative approach is to provide low-cost improvements on existing two-lane rural roads, thus covering a much larger proportion of the roads in need of upgrading. Research and experience have shown that the provision of passing lanes, turning lanes, localized alignment improvements, and other relatively low-cost measures can be highly cost-effective in improving both traffic operations and safety on existing two-lane rural roads. These options are also appropriate for roads with lower traffic volumes which do not warrant major improvement projects and on recreational or other routes with high seasonal demand.

This Guide presents information on a range of low-cost road improvements, with particular emphasis on passing and turning improvements. Section 2 of the Guide describes methods for assessing the quality of traffic operations on an existing highway, and for deciding what types of improvement alternatives may be appropriate.

Section 3 discusses methods for improving passing opportunities. Detailed guidelines are provided for the assessment, design, and location of passing lanes, climbing lanes, and short four-lane road sections. In some States, short turnouts and continuous paved shoulders are also used to improve passing opportunities. All of these passing improvements are reviewed in Section 3 and compared with "traditional" improvements such as realignment and extensive four-laning.

The safety and operational assessment of turning treatments is discussed in Section 4. These include left- and right-turn lanes at intersections, shoulder bypass lanes at T-intersections, and continuous two-way left-turn lanes in rural and urban fringe areas. The Guide does not consider signalized intersections, small towns, or urbanized areas.

Section 5 discusses the integration of minor improvements to achieve a consistent overall road standard as it is seen by the driver. This requires a review of improvement needs for relatively long highway sections and planning for long-term improvement objectives. The aim should be to ensure that the driver is provided with consistent geometrics and adequate information about the road ahead at each stage in the implementation of operational improvements.

Two appendices to the Guide present useful background information. Appendix A discusses traffic operations on two-lane highways and the applicable capacity and level of service evaluation procedures. Appendix B presents a case study that illustrates many of the principles discussed in this Guide.

2. OPERATIONAL PROBLEMS AND ALTERNATIVE IMPROVEMENTS

To set the stage for the consideration of low-cost improvement alternatives in this guide, it is useful to examine the typical causes of operational problems on two-lane highways, the range of improvement alternatives available, and the methods for evaluating alternative improvements.

2.1 OPERATIONAL PROBLEM ASSESSMENT

Operational problems on two-lane highways are mainly concerned with delays. Drivers may be prevented from travelling at their desired speeds by slower vehicles, turning traffic, highway alignment, or roadside development, possibly accompanied by reduced speed limits. Such delays result in lost time, driver frustration, and conflicts between fast and slow vehicles. Even when time delays are relatively small, driver perception of the problem may be larger if speed changes are frequent or if a large proportion of travel time is spent following other vehicles. Figure 1 illustrates a two-lane highway with heavy platooning resulting in substantial delays to motorists.

Figure 1 - Two-Lane Highway with Heavy Platooning Resulting in Substantial Delays to Motorists

Delay problems may be localized at a grade, curve, or intersection, or may be experienced over an extended section of highway due to lack of passing opportunities. Delays are much greater where there is a mixture of vehicle types (including trucks or recreational vehicles) or a mixture of trip types and purposes (such as long-distance/local, business/recreational/agricultural, or through/turning traffic).

Operational delays and conflicts can also lead to safety problems. In many cases the operational delays at a particular location may have only a small effect on overall traffic performance, but the mix of fast and slow vehicles could contribute to an unacceptably high accident rate. Delays due to turning vehicles, for example, are rarely sufficient to justify special treatment on two-lane highways unless traffic volumes are high or safety problems also exist.

2.2 IMPROVEMENT ALTERNATIVES

Numerous alternative methods and strategies are available to the highway engineer for upgrading traffic operations and safety on rural two-lane highways. These may be considered in three broad categories: high-cost improvements; passing and turning improvements; and other low-cost improvements.

2.2.1 High-Cost Improvements

The "traditional" approaches to upgrading sections of two-lane rural highways having traffic operational and safety problems include:

- Extensive realignment to higher design speed standards;
- Widening to four or more lanes;
- Improved access control; and
- Grade separation of intersections.

All of these may be considered as high-cost options compared with the approaches emphasized in this Guide.

Extensive realignment is most appropriate on highways with poor or inconsistent geometrics, high traffic volumes, and long-distance trips. The major benefits of realignment are reductions in vehicle operating costs and in travel times for free vehicles. Trucks benefit most from grade reductions whereas cars benefit more from horizontal curve improvements.[1] However, the benefits from realignment are relatively small on highways which already have moderately good design speeds.

In many cases, however, operational problems are caused not so much by poor geometrics, as by interactions between fast and slow vehicles.

In such cases, it is generally more cost-effective to improve passing opportunities by providing passing and climbing lanes than to reconstruct the existing road. Improving road alignment is not usually an efficient method for improving passing opportunities; in fact, the lengthening of horizontal and vertical curves may reduce the availability of long tangents having good passing sight distance.

Highway sections with poor alignment are often found where reconstruction would be difficult or expensive. For example, highways in mountainous areas often have narrow roadways, steep grades, and sharp horizontal curves because design to higher standards would be very costly or, perhaps, impractical. Lower design speeds may be acceptable on scenic routes or roads carrying primarily local or recreational traffic. These factors should be considered in evaluating roads for extensive alignment improvements. Selective localized alignment improvements are discussed further in Section 2.2.3.

Widening from two to four lanes over extended sections of highway is an expensive but effective method for improving traffic operations and safety. Further improvements may be achieved through access control and grade separation of intersections.

Extensive realignment and widening to four lanes may be appropriate on high-volume roads, but in some cases may not be practical due to sensitive roadside environments or local opposition. Where highway funds are limited, there is often a backlog of highway sections awaiting major improvements, and many projects may be unlikely to be funded in the foreseeable future. In these circumstances low-cost alternatives should be considered, either as short-term improvements or as stages of development towards the ultimate design.

Access control and grade separation of intersections are sometimes used with two-lane roads, particularly where multilane roads are planned for the future. These provide for a high-quality operations on two-lane highways, and may be appropriate where traffic volumes are high, but should be subjected to careful economic analysis.

2.2.2 Lower-Cost Passing and Turning Improvements

A number of methods for improving passing opportunities and for reducing problems caused by turning vehicles are discussed in detail in the following sections of this guide. The passing improvements include passing lanes in level and rolling terrain, climbing lanes on sustained grades, short four-lane sections, turnouts, and continuous paved shoulders to encourage shoulder driving. The turning improvements include left-turn and right-turn lanes at intersections, turn bypass lanes at T-intersections, and continuous two-way left-turn lanes. These alternatives provide some of the benefits of the major realignment and widening improvements discussed above, at much lower costs, as indicated in Table 1. They may, therefore, be appropriate at lower traffic volumes, or where funds or other constraints prevent the construction of major improvements in the near future.

TABLE 1

COMPARISON OF OPERATIONAL AND SAFETY IMPROVEMENT ALTERNATIVES FOR TWO-LANE HIGHWAYS

Improvement Alternative	Type of Improvement (passing or turning)	Relative Cost	Operational and Safety Effects		
			Travel Time	Traffic Platooning	Traffic Accidents
Four-lane divided	Both	High	Reduction	Reduction	Reduction
Realignment	Neither	High	Variable effect	Variable effect	Reduction
Access control	Turning	High	Small reduction	No effect	Reduction
Grade separation	Turning	High	Small reduction	No effect	Reduction
Passing lanes	Passing	Low-med	Reduction	Reduction	Reduction
Turnouts	Passing	Low	Small reduction	Small reduction	No effect
Shoulder driving	Both	Low	Small reduction	Reduction	No effect
Left-turn lanes	Turning	Low	Small reduction	No effect	Reduction
Right-turn lanes	Turning	Low	Small reduction	No effect	Reduction
Shoulder bypass lanes	Turning	Low	Small reduction	No effect	Reduction
Two-way left-turn lanes	Turning	Low	Small reduction	Possible increase	Reduction

TABLE 2

ADVANTAGES OF LOW-COST IMPROVEMENT STRATEGIES

Allocate Funds to More Small Projects

- Many roads improved
- Quick response to need
- Can allow for local constraints
- Warranted at lower volumes

Staging of Major Upgrading

- Little excess capacity
- Expensive structures deferred until fully warranted
- Less risk due to errors in projected traffic growth
- More efficient allocation of funds

Improvement strategies that consider passing lanes, turn lanes, and other low-cost improvement alternatives would lead to widespread improvements on many roads rather than complete reconstruction of the total length of a few roads. This approach will provide a better and more uniform quality of service over the rural road network, and expenditures that provide excess capacity will be minimized. In addition, improvement strategies based on passing and turning improvements can be more flexible, can avoid expensive structures, and can offer a means for quick response to perceived needs. The risk of poor investment decisions due to errors in future traffic projections would also be reduced. Some advantages of low-cost improvement strategies are summarized in Table 2.

Figure 2 illustrates an important advantage of lower-cost improvements in comparison to extensive four-laning -- lower-cost improvements result in little excess roadway capacity. By contrast, four-laning may introduce a large excess capacity on one section of highway and leave the capacity of the remainder of the highway unimproved. In the example, the existing road provides a maximum level-of-service (LOS) C flow rate of 800 veh/hr, but the design hour volume is 1,000 veh/hr. The objective is therefore to improve the road to provide at least LOS C operations for the design hour flow.

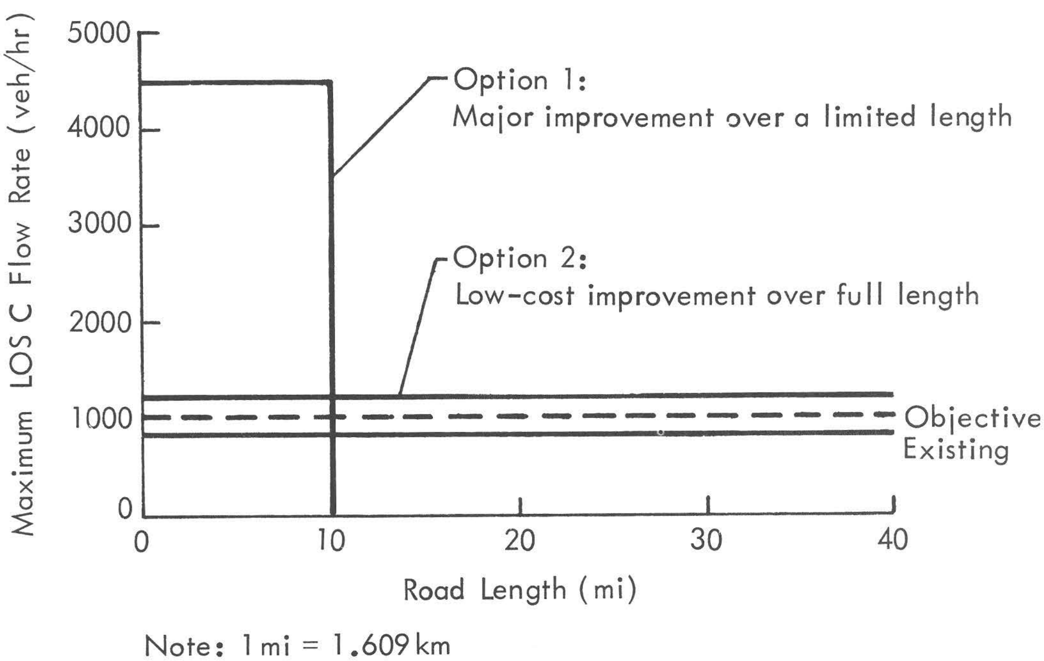

Figure 2 - Comparison of Excess Capacity Provided by High-Cost and Lower-Cost Improvements

The first option illustrated in Figure 2 is to provide a four-lane divided road, but limited funds will only allow for four-lane construction over 10 mi (16 km) of the 40-mi (64-km) road section. This provides a huge increase in LOS C service flow rate to 4,500 veh/hr on the improved length, but no improvement over the remaining 30 mi (48 km). The second option calls for passing lanes at regular intervals along the entire 40-mi (64-km) road section. This option provides a LOS C service flow rate of 1,200 veh/hr over the entire section, at lower cost than even the short length of divided highway.

2.2.3 Other Lower-Cost Improvements

A number of other lower-cost road improvement alternatives are not discussed in detail in this Guide, but are briefly described below.

Localized alignment improvements can be highly cost-effective in improving safety at specific problem locations, such as high-accident locations, isolated sharp curves, and intersections with poor sight distance for approaching drivers. Upgrading specific roadside features should also be considered as a method for reducing accidents if major improvements are not undertaken.

Improved road signs and markings can be used to inform drivers about the road ahead, delineate curves, and warn drivers of other unexpected changes in geometry or traffic operating conditions.

The quality of service experienced by the driver can also be enhanced by improved parking or rest areas, scenic viewpoints, and signs that give tourist information and that help the driver in making travel and route choice decisions.

2.3 EVALUATION METHODS

There are several methods available for assessing the effectiveness of proposed highway improvements, and determining whether such improvements are warranted for a given road and traffic volume. These can be considered in four groups: operational criteria, level of service criteria, cost-effectiveness analysis and benefit-cost analysis.

2.3.1 Operational Criteria

Operational criteria are direct measures of the effectiveness of a proposed improvement, such as the percent reduction in vehicle platooning, travel time, or accidents. These are important measures for evaluating alternatives and determining appropriate design characteristics. They are used extensively in the operational and safety assessments in the following sections. However, they do not provide enough information for deciding if the improvement is warranted or for comparing different types of improvements with different costs.

2.3.2 Level of Service Criteria

Levels of service on two-lane roads are defined in Chapter 8 of 1985 Highway Capacity Manual[2] (HCM) in terms of the percentage of travel time spent delayed, i.e., travelling in platoons behind other vehicles. These criteria are illustrated by Table 3. The percent time delay was chosen as the level of service criterion for the 1985 HCM because it is more sensitive to variation in flow rate than other candidate measures, such as vehicle speeds.[3] On steep grades, the average upgrade speed serves as an additional criterion to define the levels of service. It should be noted that the cutoff points between the levels of service are not directly related to the levels defined in the 1965 HCM.[4]

TABLE 3

LEVEL OF SERVICE CRITERIA FOR TWO-LANE HIGHWAYS[2]

Level of Service	Percent Time Delay on General Segments	Average Upgrade Speed (mi/hr) on Specific Grades
A	≤ 30	≥ 55
B	≤ 45	≥ 50
C	≤ 60	≥ 45
D	≤ 75	≥ 40
E	> 75	≥ 25-40
F	100	< 25-40

Note: 1 mi = 1.609 km

The level of service concept provides a set of uniform operational criteria for assessing existing conditions, comparing improvement alternatives, and setting targets for desired operating conditions on a given highway network. The use of standard definitions also allows comparisons across a State or between States. However, this approach still does not consider the cost of achieving the target level of service which, in some cases, may be very expensive. The lack of economic criteria also makes it difficult to compare low-cost and high-cost alternatives.

A more detailed discussion of traffic operations on two-lane highways and the applicable capacity and level of service evaluation procedures is presented in Appendix A.

2.3.3 Cost-effectiveness Analysis

Cost-effectiveness analysis considers the cost of achieving a given level of operational or safety improvement. The analysis is done by calculating a ratio, such as percent accident reduction per thousand dollars of expenditure. Such ratios can be used to compare different types of investment, and to examine the incremental or marginal effects (i.e., the added benefits vs the added costs) of different designs. The differences between short and longer passing lanes, for example, can be compared in this way. Where a proposed improvement would be particularly expensive (e.g., a climbing lane on a difficult grade), its cost-effectiveness ratio can be compared with those of other candidate projects to see if the money could be better spent elsewhere.

2.3.4 Benefit-cost Analysis

Benefit-cost analysis provides a more accurate and detailed method for taking into account the economics of highway expenditures. Benefit-cost analysis procedures attempt to measure operational and safety improvements in dollars, using estimated values of time saved, accidents avoided, and reductions in fuel consumption and vehicle operating costs. These benefits can then be compared with the construction and maintenance costs for the project, to give an overall economic evaluation. Some care is required in the use of these procedures and the interpretation of the results. A particular problem concerns the valuation of small travel time savings, especially for nonwork trips. These are sometimes considered to have little economic value, yet they reflect improved traffic operations and reduced driver frustration which are considered desirable by the driving public. A recommended procedure for benefit-cost evaluation is provided by the AASHTO "Manual on User Benefit Analysis of Highway and Bus-Transit Improvements."[5]

2.3.5 Safety Evaluations

Safety evaluation procedures may make use of operational, cost-effectiveness, or benefit-cost analysis. The first objective is to identify high-accident locations. This is usually based on monitoring accident records and applying subjective criteria to select sites needing improvement. Problem locations may also be identified from public complaints about near-miss incidents, and research studies which pinpoint particular road features needing attention.

The second objective is to estimate accident reductions which may be expected to result from proposed road improvements. These are generally determined from research studies such as those discussed in Sections 3 and 4 of this Guide.

Most state highway agencies and some local agencies have established accident evaluation procedures, which are closely related to funding requirements and procedures.

3. PASSING IMPROVEMENTS

This section addresses improvements to increase the availability of passing opportunities on two-lane highways. It begins with a discussion of the need for passing improvements on two-lane highways and summarizes the types of improvement alternatives that should be considered. Specific guidance is then provided for the use of passing lanes on two-lane highways and for the use of minor passing improvements such as short turnouts and shoulder use sections.

3.1 NEED FOR PASSING IMPROVEMENTS

The need for passing improvements on two-lane highways is discussed in this section. The discussion addresses passing demand and supply on two-lane highways, the determination of the need for passing improvements, and field studies of traffic platooning.

3.1.1 Passing Demand and Supply

The need for passing opportunities on a two-lane road arises when the demand for passing opportunities exceeds their supply. The factors affecting passing demand and supply are reviewed in Appendix A. It should be noted that the demand for passing opportunities can vary considerably with the mix of traffic characteristics on a road.

The supply of passing opportunities on a two-lane road depends on the availability of passing sight distance and gaps in the opposing traffic stream. It is common to characterize passing supply by the percentage of the road length where passing is permitted and by the percentage of road length with passing sight distance greater than 1,500 ft (460 m). Criteria for marking no-passing zones on two-lane highways are set by the Manual on Uniform Traffic Control Devices for Streets and Highways[6] (MUTCD). For a 60-mi/hr (96-km/hr) design speed, a no-passing zone is warranted where the passing sight distance falls below 1,000 ft (305 m). This requirement assures that passing is prohibited where sight distance is inadequate and passing would be unsafe. However, passing zones as short as 400 ft (120 m) can occur between no-passing zones, and such short zones do not provide effective opportunities to pass other than very slow-moving vehicles. Engineers should be aware that roads may appear to provide a high percentage of length in passing zones, but in practice allow few passing opportunities and experience high levels of platooning. The lack of passing opportunities may be further increased by high traffic volume levels that limit the frequency of adequate gaps in opposing traffic.

Traffic platoons develop and grow as faster vehicles catch up with slower ones and are unable to pass. The percentage of traffic following in platoons reflects the extent to which passing demand exceeds supply, and hence the extent of delay to drivers caused by inadequate passing opportunities. The percentage of travel time spent delayed by other vehicles is the measure of effectiveness used by the 1985 Highway Capacity Manual (HCM)[2] to define the level of service on two-lane highways.

3.1.2 Determination of Need for Passing Improvements

The determination of need for passing improvements is usually based on a level of service analysis. For an existing two-lane highway, the level of service can be determined from the procedures in Chapter 8 of the 1985 HCM. These are based on two criteria: the maximum percent time delay over a length of road; and, the minimum average upgrade speed on steep grades. Both of these criteria are illustrated in Table 3. However, the HCM procedures do not take account of the effects of passing and turning improvements. The effects of these improvements are discussed in the following sections of this Guide, and overall level of service analysis procedures are presented in Appendix A. These procedures can be used to evaluate all types of passing and turning improvements on a common basis.

Determination of need can also be based on other methods of analysis. For example, AASHTO[7] provides warrants and design guidelines for climbing lanes which are not entirely compatible with the HCM procedures. Alternative analysis methods based on benefit-cost criteria and the need for "assured passing opportunities" that are currently used in Australia and Canada, respectively, are presented in Section 3.4 of this Guide.

3.1.3 Field Studies of Traffic Platooning

Field studies of traffic platooning give the engineer useful local information on passing problems and needs. Such field studies are particularly useful if the engineer suspects that traffic characteristics or passing opportunities vary substantially along the road, or are quite different from those assumed in the HCM. While percent time delay over a length of road is not easy to measure in the field, spot platooning is very easy to measure. Spot platooning or percent following is defined as the percentage of vehicles with headways (or time gaps) of 5 sec or less as they pass a given point on the road. This measure of spot platooning provides an estimate of percent time delay.*

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* Theoretically, the probability of platooning at a given spot on the highway is proportional to the percent distance delayed, rather than to the percent time delayed. Since a vehicle travels farther in a given time when unimpeded than when following another vehicle, spot measures will always underestimate the percent time delayed. Consider, for example, a vehicle which travels 5 miles at its free speed of 60 mi/hr (time = 5 min) and 5 miles at a following speed of 50 mi/hr (time = 6 min). This vehicle spends 50 percent of its travel distance delayed in platoons, but its percent time delayed is 54.5 (6/11*100). Spot field measures will tend to reflect the lower value.

A field study of traffic platooning can consist of a simple manual count of the percentage of vehicles traveling with headways of 5 sec or less. The intervals between vehicles can be timed with a stopwatch or estimated visually depending upon the precision desired. Some commercially available traffic counting equipment can also be adapted to count both the total traffic volume and the number of vehicles following at headways of 5 sec or less, as well as the total traffic volume.

If spot studies of traffic platooning are made at several locations, their average result can be used to estimate the overall level of service (i.e., percent time delayed), while the location of higher platooning levels can be used to identify potential improvement sites and improvement needs. In this assessment, the following points may be noted:

- Drivers will generally tolerate a higher level of platooning for some distance, provided speeds do not fall too low. It may be satisfactory, for example, for spot platooning to exceed the percent time delay criteria in Table 3 for a few miles, provided that the overall percent time delay over a longer road section, perhaps 10 mi (16 km) in length, is satisfactory. The minimum spot speeds of Table 3 should also be satisfied.

- Where improved passing opportunities are needed, the field results should indicate whether the need is localized, or occurs over a longer highway section. Isolated improvements are best located where the traffic platooning is high, to assure that the passing demand is available to make best use of the added passing opportunities provided.

- In some cases where speeds are relatively uniform, large high speed platoons may be observed. In such cases, some engineering judgment may be needed to decide whether drivers are content to follow at this speed or feel some constraint due to the inability to select their own speeds.

3.2 PASSING IMPROVEMENT ALTERNATIVES

There are two basic methods available for improving passing opportunities on a two-lane highway: modifying the highway alignment or widening the roadway to add more lanes. Alignment modifications are rarely as inexpensive or effective as added lanes, since the improved sight distance can be used only when no oncoming traffic is present, and since drivers generally leave large safety margins in passing decisions. Additional passing opportunities can be provided through roadway widening in a number of ways:

- Widening to four lanes over extended highway sections. A four-lane highway provides continuous passing opportunities and a major increase in level of service compared with a two-lane road.

- Adding passing lanes. Passing lanes are defined here as any form of lane added in one or both directions of travel over part of a road length to increase passing opportunities. Passing lanes may be located in level and rolling terrain, or sustained grades, and this term is defined to include truck climbing lanes on grades and short four-lane road sections up to 3 mi (5 km) in length.

- Encouraging slow drivers to pull over onto paved shoulders or into short turnouts so that platooned vehicles may pass.

Short passing lanes are generally more highly utilized and more cost-effective per unit length in improving traffic performance than extended sections of four-lane highway for two reasons. First, the traffic entering the passing lane from a normal two-lane section is more highly platooned, and thus "primed" to make the most of the extra lane. Second, the benefits of platoon break-up in the passing lane carry over as reduced delay on the downstream two-lane highway, until new platoons form over a number of miles. A road with regular passing lanes thus has a cyclic pattern of platooning, with zones of build-up, passing, and improved downstream operations. This cycle makes best use of a relatively small highway investment, and provides an intermediate quality of traffic operations between those of a two-lane and four-lane highway.

The effect of a passing lane on traffic operations on a two-lane road is illustrated by Figure 3. The solid line in this figure shows the normal fluctuation of platooning on a two-lane highway with the availability

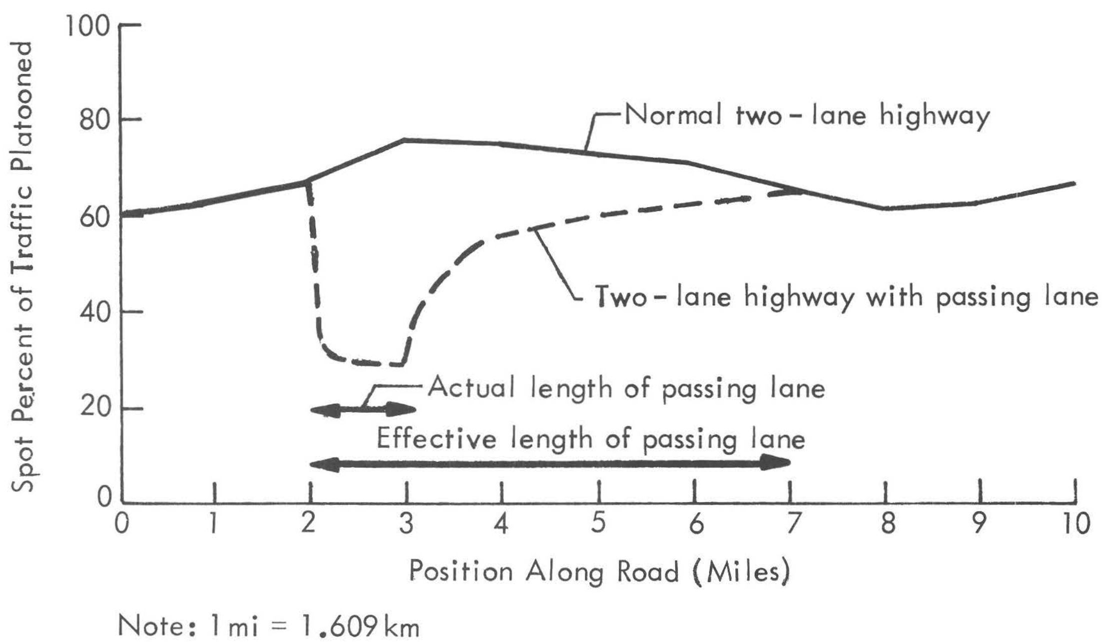

Figure 3 - Example of the Effect of a Passing Lane on Two-Lane Highway Traffic Operations

of passing sight distance. When a passing lane is added, the percentage of vehicles following in platoons falls dramatically and stabilizes at about half the value for the two-lane road. Because platoons are broken up in the passing lane, its "effective length" extends for a considerable distance downstream of the passing lane. The use of passing lanes to improve two-lane highway operations is discussed further in Section 3.3.

The use of shoulders and turnouts by slow vehicles can provide some of the benefits of a passing lane. These treatments are discussed in Section 3.4.

3.3 PASSING LANES

A passing lane is an added lane provided in one or both directions of travel on a conventional two-lane highway to improve passing opportunities. This definition includes passing lanes in level or rolling terrain, climbing lanes on grades, and short four-lane sections. The length of the added lane can vary from 1,000 ft (305 m) as much as 3 mi (5 km). Figure 4 illustrates a plan view of a typical passing lane section. Figure 5 presents photographs of some typical examples of passing lanes.

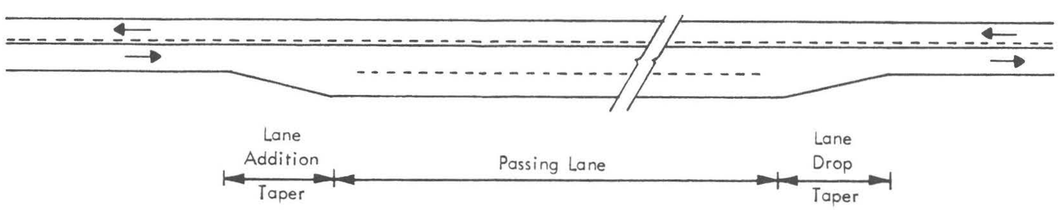

Figure 4 - Plan View of Typical Passing Lane Section

On two-lane rural roads, passing lanes have two important functions:

- To reduce delays at specific bottleneck locations, such as steep upgrades where slow-moving vehicles are present; and
- To improve overall traffic operations on two-lane highways by breaking up traffic platoons and reducing delays caused by inadequate passing opportunities over substantial lengths of highway.

The first function, to reduce delays at bottleneck locations, has been recognized for some time, and guidelines for the provision of climbing lanes on grades have been established. The second function, to improve overall traffic operations, has evolved more recently, particularly as a result of the lack of funds for major road improvements. In practice, many passing lanes perform both functions, and it is often difficult to draw a clear traffic operational distinction between the two. The distinction is important, however, in planning and design. The evaluation of a climbing lane

Passing Lane with Opposing Direction Passing Prohibited

Passing Lane with Opposing Direction Passing Permitted

Short Four-Lane Section

Figure 5 - Typical Passing Lane and Short Four-Lane Sections Used on Two-Lane Highways

considers only the bottleneck location, with the objective of improving traffic operations at the bottleneck to at least the same quality of service as adjacent road sections. For passing improvements, on the other hand, the evaluation should consider traffic operations for an extended road length, typically 5 to 50 mi (8 to 80 km). Furthermore, the location of the passing improvement can be varied and the selection of an appropriate location is an important design decision.

3.3.1 Location and Configuration

When passing lanes are provided at an isolated location, their function is generally to reduce delays at a specific bottleneck, and the location of the passing lane is dictated by the needs of the specific traffic operational problem encountered. Climbing lane design guidelines, for example, usually call for the added lane to begin before speeds are reduced to unacceptable levels and, where possible, to continue over the crest of the grade so that slower vehicles can regain some speed before merging. Requirements for sight distance and taper lengths further define the location of such lanes. In some cases, construction of a climbing lane over the full length of a grade may be too expensive, and the use of shorter lanes over part of the grade may be considered. Recent research at the University of California[8] suggests that a single short climbing lanes approximately 1,500 ft (460 m) in length near the midpoint of the grade, or two such lanes at the one-third and two-thirds points, are cost-effective methods for providing passing opportunities on long sustained grades. The location of a climbing lane drop on an upgrade section has been found to produce no adverse safety problems, provided sight distance is adequate.[9]

When passing lanes are provided to improve overall traffic operations over a length of road, they are often constructed systematically at regular intervals. The designer can choose from a number of alternative configurations,[10] as illustrated in Figure 6. The choice of configuration, and the location of the added lanes, may vary with particular local needs and constraints, so there is no single "correct" answer. The following factors should be considered in choosing the location and configuration for passing lanes:

Location

- A primary objective in choosing the location for a passing lane should be to minimize construction costs, subject to other constraints. Data from several States indicates that the cost of constructing a passing lane can vary from $200,000 to $750,000 per mile, depending upon terrain. Climbing lanes in mountainous terrain can cost up to $1,800,000 per mile, depending on terrain. Thus, the choice of a suitable location for a passing lane may be critical to its cost-effectiveness.

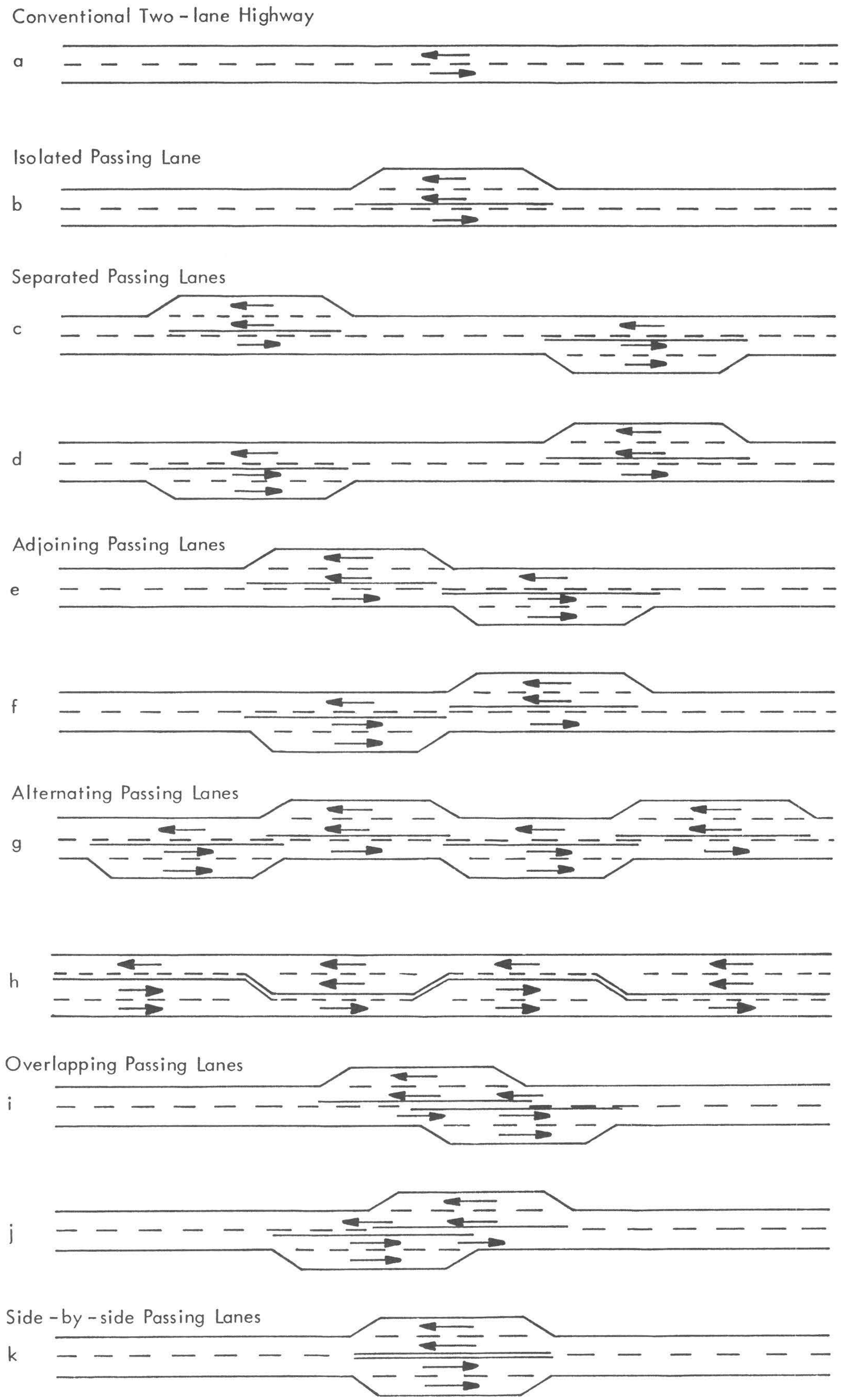

Figure 6 - Alternative Configurations for Passing Lanes[10]

- The passing lane location should appear logical to the driver. The value of passing lanes is more obvious to the driver at locations where passing sight distance is restricted than on long tangent sections which already provide good passing opportunities. In some cases, a passing lane on a long tangent may encourage slow drivers to speed up, thus reducing the passing lane effectiveness. At the other extreme, highway sections with low speed curves should be avoided, since they may not be suitable for passing.

- The passing lane location may be on a sustained grade or on a relatively level section. If delay problems on the grade are severe, the grade will usually be the preferred location for a passing lane. However, if platooning delays exist for some distance along a road, locations other than upgrades should also be considered. While speed differences are often greater on upgrades, particularly if heavily loaded trucks are present, construction costs and constraints may be greater at such locations. Some types of slow vehicles are not slowed by upgrades as dramatically as heavy trucks, so passing lanes in rolling terrain may provide opportunities to pass such vehicles that are just as good as on upgrades. Passing lanes are also effective in level terrain where the demand for passing opportunities exceeds supply.

- The choice of passing lane location should take into account the need for adequate sight distance at the lane addition and lane drop tapers. This is discussed further in Section 3.3.3.

- The location of major intersections and high-volume driveways should be considered in selecting passing lane locations, to minimize the volume of turning movements on a road section where passing is encouraged. Low-volume intersections and driveways do not usually create problems in passing lanes. Where the presence of higher-volume intersections and driveways cannot be avoided, special provisions for turning vehicles should be considered. The prohibition of passing by vehicles traveling in the opposing direction should also be considered on passing lane sections with higher-volume intersections and driveways.

- Other physical constraints, such as bridges and culverts, should be avoided if they restrict the provision of a continuous shoulder.

- Passing lanes can also be constructed as part of a realignment of a road segment with safety problems.

Configuration

- Separated or adjoining passing lanes (shown as (c) through (f) in Figure 6) are often used in pairs, one in each direction, at regular intervals along a two-lane highway.

- Where pairs of adjoining passing lanes are used and passing by opposing direction vehicles is prohibited, the use of the "head-to-head" configuration (shown as (e) in Figure 6) has the advantage of building platoons before the passing lane, whereas the reverse configuration tends to rebuild platoons more quickly after the passing lane. The "head-to-head" configuration is also preferable because the lane drop areas of the opposing passing lanes are not located adjacent to each other.

- Transitions between passing lanes in opposing directions should be carefully designed; intersections, bridges, two-way left-turn lanes or painted medians can often be used effectively to provide a buffer area between opposing passing lanes.

- Alternating passing lanes (shown as (g) and (h) in Figure 6) are sometimes appropriate where a wide pavement is already available. However, the provision of passing lanes over 50 percent of the road length is probably excessive. Drivers may also feel unduly constrained when passing is prohibited on the other 50 percent of the road length if sight distance is good and traffic volumes are low.

- Short four-lane sections, both divided and undivided, are particularly appropriate where this is the ultimate design for the highway. Construction of short four-lane sections at the least expensive locations can provide a substantial proportion of the benefits of the ultimate design for a relatively small proportion of the total cost, particularly if major bridge work or right-of-way acquisition can be avoided. This staged four-laning will generally return a high marginal benefit-cost ratio, while the economic justification for the remaining stages will increase with increasing traffic volumes in future years. Where the ultimate design is uncertain or the need for it is many years away, however, the use of lower cost options should be considered.

- Overlapping passing lanes (shown as (i) and (j) in Figure 6) are often used at crests where a climbing lane is provided on each upgrade.

3.3.2 Length and Spacing

Two important elements of passing lane design are the length of passing lanes and the spacing between them.

Length

Passing lanes should rarely be shorter than 1,000 ft (305 m) in length, not including the lengths of the lane addition and lane drop tapers except possibly for very low-speed roads in hilly terrain. This minimum length is needed to assure that delayed vehicles have an opportunity to complete at least one pass in the passing lane. Where passing or climbing lanes are provided to reduce delays at a specific bottleneck, the length of the passing lane may be set by the length of the bottleneck. Where passing lanes are provided to improve overall traffic operations on a two-lane highway, the passing lane should be long enough to provide a substantial reduction in traffic platooning. Passing lanes of 0.25 mi (0.4 km) or less in length are not very effective in reducing traffic platooning. The optimal length of a passing lane to reduce platooning is usually 0.5 to 1.0 mi (0.8 to 1.6 km). As the length of a passing lane increases above 1.0 mi (1.6 km), passing lanes generally provide diminishing reductions in platooning per unit length. The issue of passing lane length is addressed further in Section 3.3.5.

Spacing

The spacing of passing lanes will depend primarily on the magnitude of improvements needed to achieve satisfactory traffic operations (see Section 3.2.1). The operational benefits of a passing lane typically carry over in reduced traffic platooning for 3 to 8 mi (5 to 13 km) downstream, depending on traffic volumes and passing opportunities (see Section 3.3.5). Beyond this distance, traffic will experience normal levels of traffic platooning for two-lane highways until the next passing lane is encountered. Advance signs for the next passing lane should reduce driver frustration and risky passing maneuvers over this distance.

On a highway which needs only a moderate improvement in passing opportunities, a good strategy may be to construct passing lanes initially at fairly large spacings, such as 10 to 15 mi (16 to 24 km). Where the need for improved passing opportunities is greater, or grows with increasing traffic volumes, more passing lanes may be added to reduce spacings as low as 3 to 5 mi (5 to 8 km). In many locations, a mix of grade and level locations may be used. The spacing between passing lanes must be flexible to permit selection of suitable and inexpensive passing lane locations.

3.3.3 Geometrics

Geometric design of passing lanes should consider lane and shoulder widths and the lane addition and lane drop taper designs.

The lane widths in a passing lane section should not normally be narrower than the lane widths on the adjacent sections of two-lane highway. Lane widths of 12 ft (3.7 m) are desirable. It is desirable for passing lane sections to have a minimum shoulder width of 4 ft (1.2 m) on either side of the highway. Whenever possible, the shoulder width in a passing lane section should not be narrower than the shoulder width on the adjacent sections of two-lane highway.

The lane addition and lane drop transition areas at the beginning and end of a passing lane should be designed with care to encourage safe and efficient traffic operations. Many highway agencies have used relatively short lane addition and lane drop tapers at passing lanes. However, the use of a longer tapers should be encouraged to minimize traffic conflicts and to get the greatest operational benefit from the investment in passing lanes.

The lane drop taper at the downstream end of a passing lane should be designed in accordance with the requirements for lane reduction transitions set by MUTCD Section 3B-8. The recommended geometric configuration is to terminate the right lane with a lane drop taper and merge the traffic from both lanes into a single lane. In a few cases, such as with alternating passing lanes on a three-lane pavement of constant width, dropping the left lane is appropriate. The lane drop taper length should be computed from the formula L = WS, where L is the taper length in feet, W is the width of the dropped lane in feet, and S is the off-peak 85th percentile speed in miles per hour. At the termination of a 12-ft (3.7-m) lane, the required taper length for a 60-mi/hr (96-km/hr) design speed is 720 ft (220 m). A wide shoulder is desirable at the lane drop taper to provide a recovery area should drivers encounter a merging conflict.

There is no MUTCD requirement for the length of the lane addition taper at the upstream end of a passing lane. The diverge maneuver does not require as much length as the merge maneuver, but a good lane addition transition design is needed for effective passing lane operations. The recommended length for a lane addition taper is half to two-thirds of the length of a lane drop taper, or 360 to 480 ft (115 to 150 m) in the example of the 60-mi/hr (96-km/hr) design speed presented above.

Passing lanes work most effectively if the majority of drivers enter the right lane at the lane addition transition and use the left lane only when passing a slower vehicle. Little or no operational benefit is gained from passing lanes if most drivers continue directly into the left lane. The geometric design of the lane addition transition area should encourage drivers to enter the right lane, and this intent should be reinforced through appropriate signing and marking, which is discussed in Section 3.3.4.

Safe and effective passing lane operations require adequate sight distance on the approach to both the lane addition and lane drop tapers. Lack of sight distance in advance of the lane addition taper may result in lack of readiness by vehicles wishing to pass, so that some of the length of the passing lane is wasted. When sight distance approaching the lane drop taper is limited, vehicles may merge too early or too late, resulting in erratic behavior and poor utilization of the passing lane. Therefore, a minimum sight distance of 1,000 ft (305 m) on the approach to each taper is recommended.

3.3.4 Signing and Marking

The signing and marking of passing lanes is partially addressed in the MUTCD, which indicates the appropriate centerline markings for passing lanes and the signing and marking of lane drop transitions areas. The following discussion extends the MUTCD criteria to provide a consistent set of traffic control devices for use at passing lanes, as illustrated in Figure 7.

Signing

Signing is needed to convey information to drivers at three locations at passing lane sites:

- In advance of the passing lane;
- At the lane addition; and
- In advance of the lane drop.

Advance signing: A sign with the legend PASSING LANE 1/2 MILE should be placed 0.5 mi (0.8 km) in advance of each passing lane. This sign provides advance notification of the passing lane to the drivers of both slow-moving vehicles and following vehicles so that they can prepare to make effective use of the passing lane. Additional advance signs are desirable 2 to 5 mi (3 to 8 km) in advance of a passing lane. Such advance signing may reduce the frustration and impatience of drivers following a slow-moving vehicle because they know they will soon have an assured passing opportunity. Driver frustration and impatience when following slow-moving vehicles has been shown to be a potential safety problem on two-lane highways. Hostetter and Seguin[11] found, for example, that when forced to follow a slow moving vehicle for up to 5 mi (8 km), almost 25 percent of drivers made an illegal pass in a no-passing zone.

Lane addition signing: A black-on-white regulatory sign with the legend KEEP RIGHT EXCEPT TO PASS should be placed at the beginning of the lane addition taper. This sign, in conjunction with the geometrics and pavement markings at the lane addition taper, informs drivers of the beginning of the passing lane and encourages them to enter the right lane unless they are immediately behind a vehicle they wish to pass. An acceptable alternative legend for this sign is SLOWER TRAFFIC KEEP RIGHT, although this legend is not preferred because it provides less definite instructions to drivers. Sign legends that refer specifically to trucks, such as TRUCKS USE RIGHT LANE, are not recommended because they appear to exclude other vehicle types, such as slow-moving recreational vehicles and passenger cars, which should also be encouraged to use the right lane.

Lane drop signing: The MUTCD requires a black-on-yellow warning sign, either a symbol sign (W4-2)* or a text sign (W9-1 or W9-2), in advance of a lane drop. According to MUTCD Table II-1, for a design speed of 60 mi/hr

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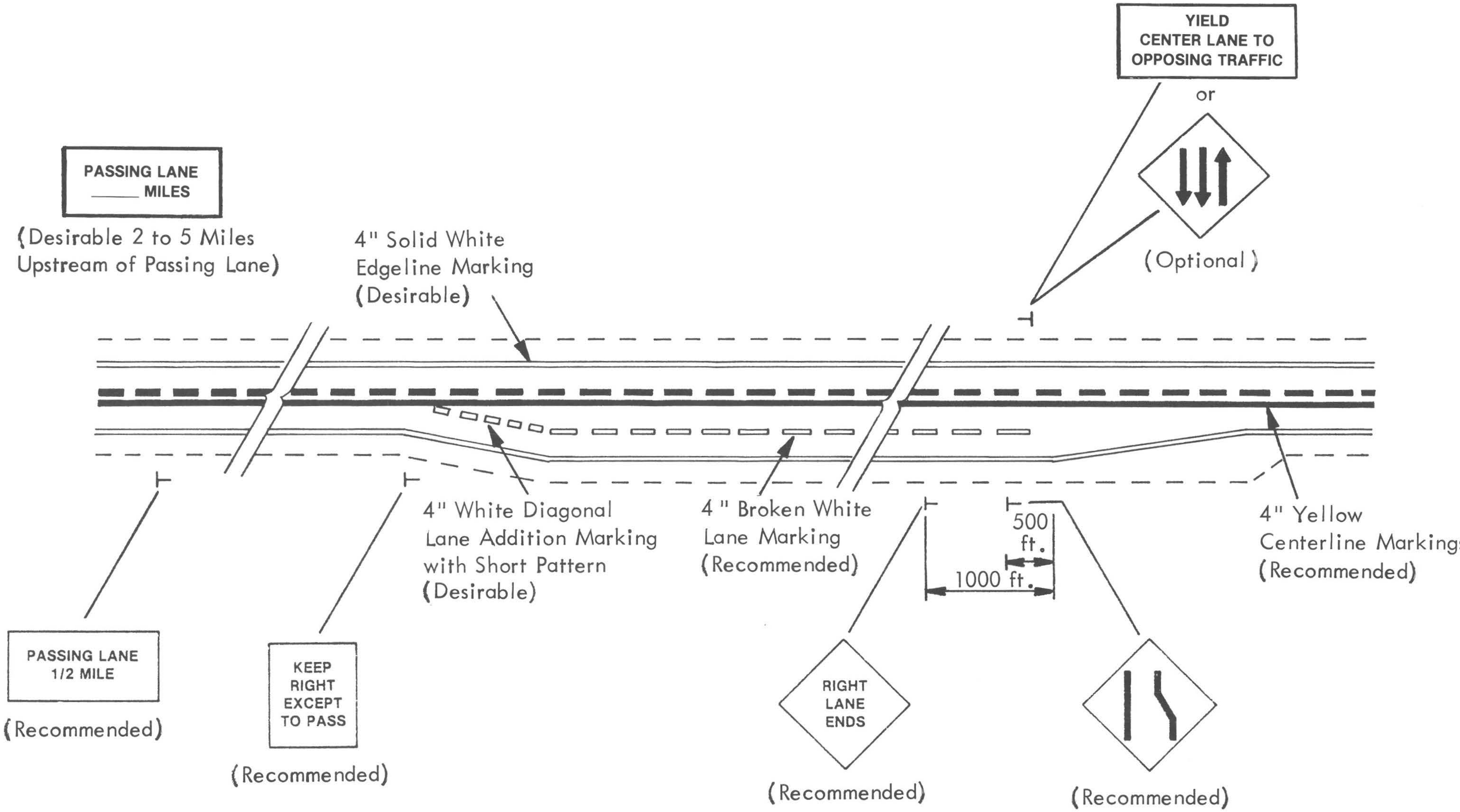
* Sign numbers in parentheses refer to codes used in the MUTCD.



Note: 1 mi = 1.609 km
1 ft = 0.305 m
1 in = 2.54 cm

Figure 7 - Recommended Signing and Marking Practices for Passing Lanes

(96 km/hr), a single warning sign should be placed 775 ft (240 m) in advance of a decision point that requires a high degree of judgment, such as a lane drop merging maneuver. Many highway agencies use two warning signs in advance of the lane drop transition areas of passing lanes, and this practice is recommended. The first advance warning sign with the legend RIGHT LANE ENDS, should be located 1,000 ft (305 m) in advance of the lane drop taper. This sign may carry a supplemental distance plate (e.g., 1,000 FEET) below the sign. The second advance warning sign should be the lane reduction transition symbol sign (W4-2) and should be located 500 ft (150 m) in advance of the lane drop taper.

Signing for opposing traffic: Highway agencies that generally provide signing for passing and no-passing zones on conventional two-lane highways, including the DO NOT PASS sign (R4-1), the PASS WITH CARE sign (R4-2), and the pennant-shaped NO PASSING ZONE sign (W14-3), usually continue this practice in the opposing direction of travel at passing lane sites. Where passing by vehicles traveling in the opposing direction is permitted, some agencies use a regulatory sign specifically appropriate to passing lanes, such as YIELD CENTER LANE TO OPPOSING TRAFFIC, in place of the PASS WITH CARE sign. An alternative sign for use in the opposing direction to a passing lane is the three-arrow sign used in Australia, which is illustrated in Figure 7. This sign does not identify whether passing by vehicles traveling in the opposing direction is permitted or prohibited, but it does alert drivers that there are two lanes of oncoming traffic.

Marking

Marking a passing lane section with two lanes in one direction of travel and one lane in the opposite direction of travel should be marked in accordance with MUTCD Figure 3-2. A yellow centerline marking should be used to separate the lanes normally used by traffic moving in opposite directions. A broken white lane line is used to separate traffic in lanes normally moving in the same direction of travel. Pavement edge lines are desirable on both sides of the highway in passing lane sections to guide drivers and to delineate the boundary between the pavement and shoulder.

Passing by vehicles traveling in the opposing direction to a passing lane may be either permitted or prohibited, as illustrated in Figure 5 of this Guide and in MUTCD Figure 3-2. A study by Harwood and St. John[12] found no difference in cross-centerline accident rates between passing lane sections where passing in the opposing direction was prohibited and passing lane sections where passing in the opposing direction was permitted where adequate sight distance was available. Therefore, passing by opposing direction vehicles may be allowed where sight distance is adequate. No-passing zones should be marked for the opposing direction of travel where warranted by the same criteria used in marking normal two-lane highways, specified in MUTCD Section 3B-5. For a 60-mi/hr (96-km/hr) design speed, a no-passing zone is warranted in the opposing direction of travel where sight distance is less than 1,000 ft (305 m).

It is not a desirable practice to prohibit passing by vehicles traveling in the opposing direction at all passing lane sites, because this unnecessarily reduces the level of service in that direction of travel. Prohibition of passing in the opposing direction at all passing lanes, regardless of sight distance, may be counterproductive to improved safety, since some drivers traveling in the opposing direction may be tempted to pass despite the prohibition in areas of good sight distance that would otherwise be excellent passing zones. Some agencies may choose to institute a site-by-site review of passing lanes and prohibit opposing direction passing at particular sites on the basis of unusual geometrics, roadside development, high traffic volumes, or similar factors, in addition to limited sight distance. The prohibition of passing by vehicles traveling in the opposing direction is particularly appropriate at sites with roadside development that generates frequent left-turn movements from the left lane of the treated direction in the passing lane section.

Lane addition markings: The MUTCD does not provide any specific guidance for marking a lane addition transition area. The recommended pavement marking scheme is illustrated in Figure 7. The use of a pavement edge marking in the lane addition transition area is recommended. A white diagonal marking across the left lane immediately prior to the beginning of the lane line is recommended. Several highway agencies have found this marking to be effective in guiding most drivers into the right lane. Drivers who desire to pass immediately upon entering the passing lane are permitted to cross the diagonal marking.

Lane drop markings: Pavement markings in the lane drop transition area should be provided in accordance with MUTCD Section 3B-8, as illustrated in MUTCD Figure 3-10. For a 60-mi/hr (96-km/hr) design speed, the broken white lane line should be discontinued 580 ft (175 m) prior to the beginning of the lane drop taper. The use of a pavement edge marking in the lane drop transition area is recommended.

Climbing Lanes and Short Four-Lane Sections

The signing and marking of climbing lanes on sustained grades should be identical to that for passing lanes in level or rolling terrain. The signing and marking of short sections of undivided four-lane roadway should also be identical to passing lanes, except that a double yellow centerline or a narrow flush painted median can be used throughout the length of the four-lane section. On short sections of four-lane divided highway, the recommended signing is similar to that for passing lanes except the legend DIVIDED HIGHWAY should be used in preference to PASSING LANE in advance signing. The two-way traffic warning sign (W6-3) may be used downstream of the four-lane divided section to remind drivers that they have returned to a two-way roadway. The pavement markings for a short section of four-lane divided roadway are identical to those for a four-lane undivided roadway, except that pavement edge lines are required throughout the divided roadway. The edge line adjacent to the outside shoulder in each direction of travel should be white and the edge line adjacent to the median should be yellow.

3.3.5 Operational Effectiveness

The operational effectiveness of passing lanes on two-lane highways has been evaluated extensively in Australia, Canada and the United States. The results of these evaluations are summarized in the following discussion to provide guidance on where passing lanes should be used and what operational benefits should be expected.

Field evaluation and computer simulation modeling have found passing lanes to be effective in improving traffic operations on two-lane highways in the United States. Field evaluations conducted by Harwood and St. John[12] at 12 passing lanes and three short four-lane sections located in 10 States found that passing lanes increase passing rates and reduce vehicle platooning. More recently, simulation modeling of passing lanes has been conducting with a computer model known as TWOPAS,[13] which is a modified version of the TWOWAF model used in the development of Chapter 8 of the 1985 HCM.

Effective Length of a Passing Lane Used for Analysis

Figure 8 illustrates the effects of passing lanes of various lengths on traffic platooning within a passing lane and downstream of a passing lane for flow rates of 400 and 700 veh/hr in one direction of travel. Figure 8 is based on the percentage of vehicles delayed in platoons at specific spot locations on the highway. It can be seen in Figure 8 that the level of traffic platooning within a passing lane is less than half of the level observed upstream of the passing lane. Traffic platooning remains at a reduced level downstream of a passing lane. For a flow rate of 400 veh/hr, the effects of passing lanes can still be substantial 7 mi (11 km) downstream of the beginning of the passing lane, especially for longer passing lanes. At the higher flow rate of 700 veh/hr, nearly all of the operational benefits of the passing lane are gone within 5 mi (8 km), although there is a small residual effect even at 7 mi (11 km) downstream. The length of the passing lane has a strong influence on the improvement in traffic operations immediately downstream of the passing lane, but this differential between passing lane lengths largely disappears further downstream.

The results in Figure 8 indicate that the effective length of a passing lane can vary from 3 to 8 mi (5 to 13 km) depending on passing lane length, traffic flow and composition, and downstream passing opportunities.

The concept of effective length is needed for analysis purposes to determine the overall effect of a passing lane on level of service over an extended highway section. For most cases, effective length can be estimated from Figure 8, with adjustments for factors which might hasten or slow the downstream overtaking or catch-up process. If the two-lane highway downstream of the passing lane has few passing opportunities, for example, the effective length determined from Figure 8 should be reduced.

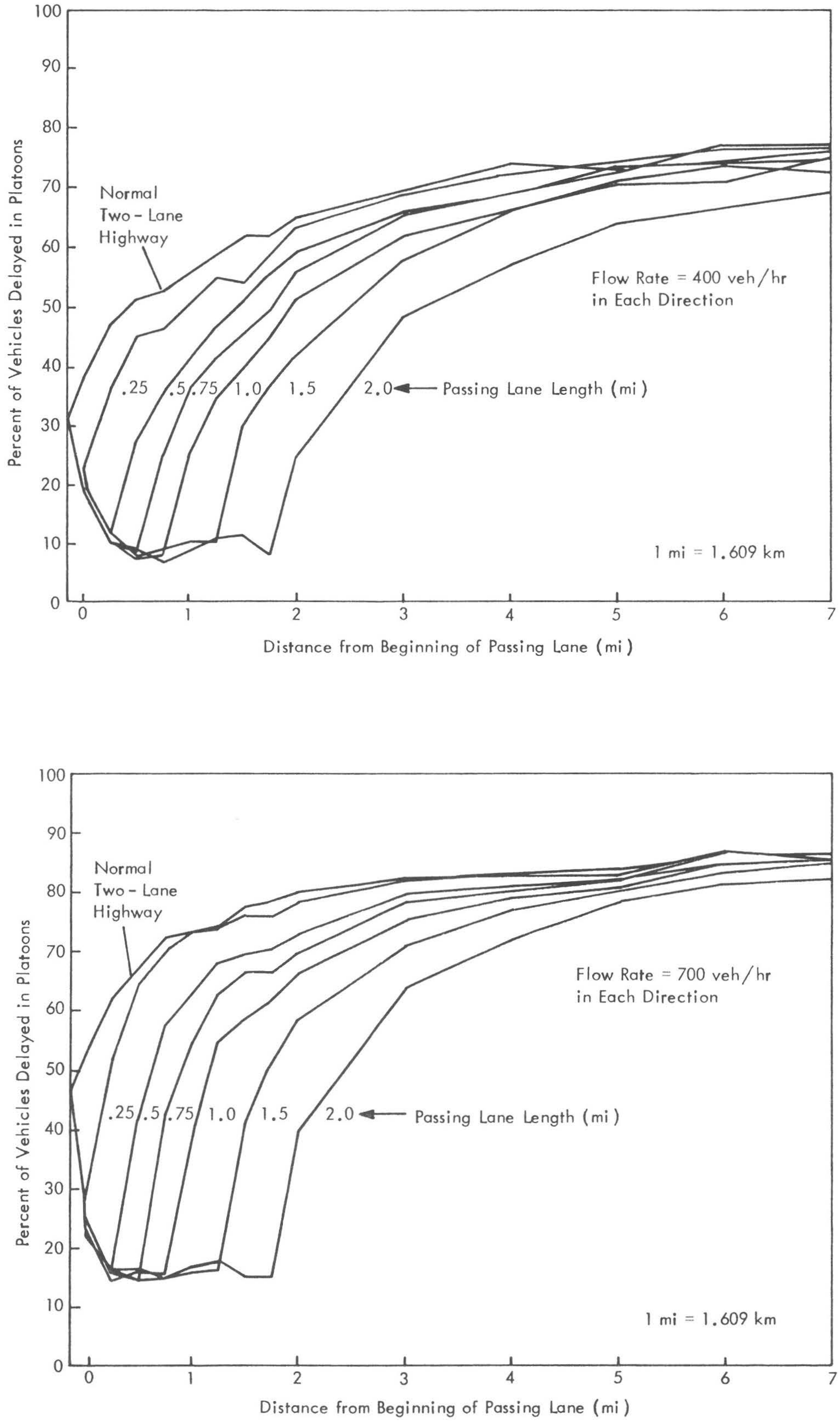


Figure 8 - Gradual Increase in Percentage of Vehicles Delayed in Platoons Downstream of Passing Lanes[13]

In some cases, the effective length of a passing lane is constrained by other road features, such as small towns, four-lane sections, or additional passing lanes a few miles downstream. In these situations, the distance to the downstream constraint should be used as the effective length for analysis purposes, if this is less than that estimated from Figure 8.

Effectiveness Over an Extended Road Section

Figure 9 illustrates the effectiveness of passing lanes of various lengths in improving traffic operations on two-lane highways, based on results obtained with the TWOPAS simulation model. The curves presented in Figure 9, for passing lanes of varying lengths, represent their effectiveness in increasing traffic speeds and decreasing the percent of time vehicles spend delayed in platoons on a two-lane highway in moderately rolling terrain. The vehicle speed and platooning measures in Figure 9 are averages over an 8-mi (13-km) highway section with the passing lane located at the beginning; thus, these curves represent the combined effects of improved traffic operations in the passing lane and downstream of the passing lane. Figure 9 illustrates that passing lanes produce relatively small increases in vehicle speeds, but can dramatically decrease vehicle platooning. Further simulation of passing lanes with the TWOPAS model has enabled the development of the predictive equations for passing lane effectiveness presented below.

An 8-mi (13-km) highway section is used in Figure 9 because the "effective length" of a passing lane includes both the passing lane itself and the downstream section of two-lane highway where platooning is lower than it would have been without the passing lane. Table 4 presents the estimated reductions in percent time delay for three different effective lengths -- 3, 5, and 8 mi (5, 8, and 13 km) -- as well as for different lengths of passing lane.

The selection of the design length of a passing lane and the effective length used for analysis purposes are discussed in the following sections. Once these are determined, Table 4 can be used to predict the percent time delay and, hence, the level of service on a highway section which includes a passing lane.

It should be noted that the base values of percent time delay for a normal two-lane highway in Table 4 are significantly higher than those specified in the HCM (see Table 3) for ideal conditions. This is because the simulated results were derived for non-ideal conditions of terrain, no-passing zones, and traffic composition. Since these conditions can vary from one case to another, it is recommended that Table 4 be entered using a given base value of percent time delay, rather than the traffic flow. In other words, the estimated two-lane highway percent time delay should be used to select the appropriate row of Table 4, regardless of traffic flow. Linear interpolation in Table 4 is acceptable.

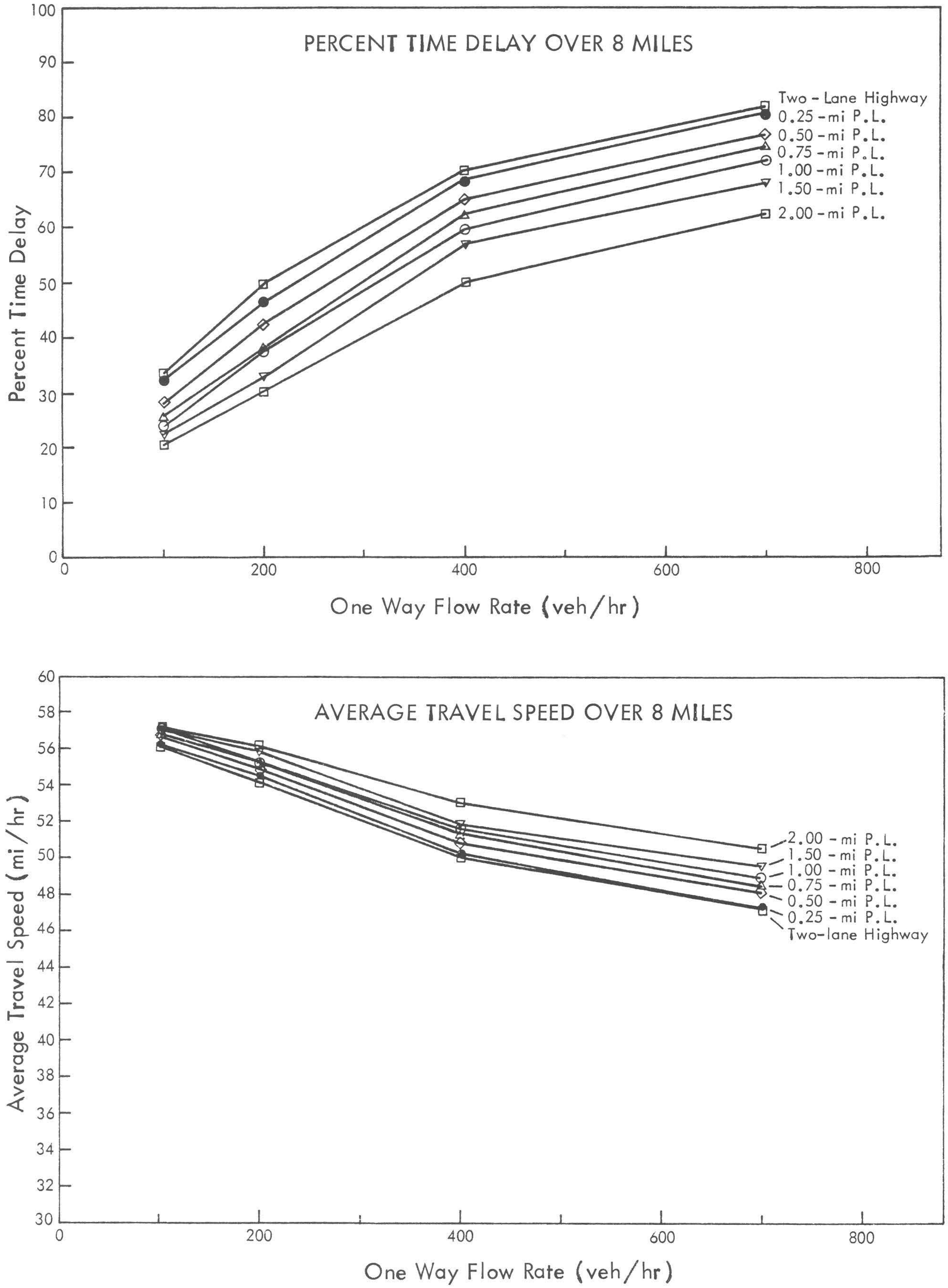


Figure 9 - Computer Simulation Results for Operational Effectiveness of Passing Lanes

TABLE 4

EFFECT OF PASSING LANES ON PERCENT TIME DELAY OVER AN EXTENDED ROAD LENGTH

	PERCENT TIME DELAY						
Effective Length (mi)	Passing Lane Length (mi)						
	0	0.25	0.50	0.75	1.00	1.50	2.00
One-way Flow Rate = 100 veh/hr							
3	33	30	20	17	17	17	17
5	33	31	25	22	19	17	17
8	33	32	28	26	24	22	20
One-way Flow Rate = 200 veh/hr							
3	50	39	29	25	25	25	25
5	50	44	37	31	29	25	25
8	50	46	42	38	37	33	30
One-wqy Flow Rate = 400 veh/hr							
3	70	67	57	49	43	35	35
5	70	68	62	57	54	49	38
8	70	69	65	62	60	57	50
One-way Flow Rate = 700 veh/hr							
3	82	79	69	63	55	45	41
5	82	80	74	71	66	60	52
8	82	81	77	75	72	68	63

Note: 1 mi = 1.609 km

Optimal Design Length for Passing Lanes

The optimal design length for a passing lane can be determined through a cost-effectiveness analysis. This can be illustrated by the cost-effectiveness data in Table 5. This table presents the percent time delay over an effective length of 8 mi (13 km) for passing lanes of various

TABLE 5

REDUCTION IN PERCENT TIME DELAY PER UNIT LENGTH OF PASSING LANE

One-Way Flow Rate (veh/hr)	Passing Lane Length (mi)[a]					
	0.25	0.50	0.75	1.00	1.50	2.00
100	2.8	8.2	8.1	8.1	6.8	6.2
200	11.1	13.1	14.0	11.7	10.6	9.5
400	2.8	8.2	13.1	9.0	8.1	9.5
700	2.8	8.2	8.1	9.0	8.7	9.0

a/ Unit length of passing lanes increased by 600 ft to account for cost of constructing lane addition and lane drop tapers.

Note: 1 mi = 1.609 km
1 ft = 0.305 m

design lengths, the difference between the percent time delay for each design length and a conventional two-lane highway and the ratio of this difference to the design length. This ratio represents the effectiveness of passing lanes in reducing vehicle platooning per unit length. The use of design length in the denominator of the cost-effectiveness ratio represents the cost of constructing passing lanes, which can vary widely depending on terrain. The passing lane lengths used to compute the cost-effectiveness ratios in Table 5 have been increased by 600 ft (180 m), half of the combined length of typical lane addition and lane drop tapers, to account for the cost of constructing these transition areas.

The optimal design lengths for passing lanes, based on the data in Table 5, are tabulated in Table 6. For flow rates of 200 veh/hr or less in one direction of travel, the highest cost-effectiveness per unit length is obtained for passing lanes with design lengths between 0.5 and 0.75 mi (0.8 and 1.2 km). Passing lanes shorter than 0.5 mi (0.8 km) or longer than 0.75 mi (1.2 km) are not as desirable because they provide less operational benefit per unit length. As flow rate increases above 200 veh/hr, the optimal design length for a passing lane also increases. At a flow rate of 400 veh/hr in one direction of travel, the optimal design length for a passing lane is 0.75 to 1.0 mi (1.2 to 1.6 km). At very high flow rates, such as 700 veh/hr in one direction of travel, the optimal design length of passing lanes ranges from 1.0 to 2.0 mi (1.6 to 3.2 km). However, the use of passing lanes longer than 1.0 mi (1.6 km) in length may not be desirable,

TABLE 6

OPTIMAL DESIGN LENGTHS FOR PASSING LANES

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

Note: 1 mi = 1.609 km

even for highways with peak flow rates of 700 veh/hr in one direction of travel, because longer passing lanes would be suboptimal throughout the remainder of the day when traffic volumes are lower.

Cost-effectiveness analysis indicates that short passing lanes are usually more effective per unit length, and therefore per dollar spent on construction, than long passing lanes. Thus, the overall level of service on a highway can often be improved more by constructing three 0.5-mi (0.8-km) passing lanes spaced at intervals than by constructing one 2-mi (3.2-km) passing lane. The optimal design length for passing lanes on a specific section of two-lane highway could be based on the highest hourly flow rate that occurs frequently (e.g., on a daily basis) on that specific highway section. The design hour volume, which occurs in only a few hours out of each year, may be too high to serve as the basis for the choice of a cost-effective passing lane length. It may be useful to evaluate traffic operations for several design hours, especially when the composition of traffic differs between weekdays and weekends.

Changes in Spot Platooning Over the Length of a Passing Lane

The effectiveness of a passing lane in improving traffic operations on a two-lane highway can be determined by using the TWOPAS model to compare the percentage of vehicles delayed in platoons immediately downstream of a passing lane to the percentage of vehicles that would have been delayed in platoons at the same location had the passing lane not been present. A statistical analysis of results from over 85 computer simulation runs produced the following predictive equation for the effectiveness of a passing lane:[13]

$$RPD = -\ 6.84 + 10.9\ \ell n(LEN) + 0.0823\ FLOW - \frac{471}{FLOW} + 9.59\ \ell n(UPD) - 0.0247\ FLOW * \ell n(UPD) \qquad (1)$$

where,

RPD = reduction in percentage of vehicles delayed in platoons due to presence of a passing lane;
LEN = length of passing lane not including tapers (mi);
FLOW = flow rate in one direction of travel (veh/hr);
UPD = percentage of vehicle delayed in platoons upstream of passing lane.

Equation (1) has been established with high statistical confidence and illustrates the complexity of the relationships and interactions that influence the effectiveness of passing lanes. Equation (1) is valid for the range of passing lane lengths from 0.25 to 2.00 mi (0.4 to 3.2 km), for the range of flow rates from 100 to 700 veh/hr, and for the range of upstream percentage of vehicles delayed in platoons from 20 to 70 percent. Equation (1) can be used for passing lanes on highways with up to 30 percent heavy vehicles in the traffic stream in level, moderately rolling, or severely rolling terrain. However, Equation (1) is not applicable to climbing lanes in mountainous terrain.

International Research and Operational Experience

Early Australian research related to passing lanes focused on traffic operational conditions at transitions between two-lane and four-lane roads. Field studies by Hoban[14] in 1980 found that when traffic from a two-lane road entered a four-lane road, speeds increased immediately, and continued to increase over a distance of 1.4 mi (2.2 km). Half of the increase in speed in the four-lane section occurred in the first 1,300 ft (400 m). This increase in speed results directly from breaking up vehicle platoons and enabling following drivers to travel closer to their desired speed. At the transition from the four-lane road back to the two-lane road, the traffic operational benefits from breaking up vehicle platoons were found to carry over for many miles downstream. The decrease in mean speed downstream of the lane drop was much more gradual than the increase in mean speed at the lane addition. Similar results were found for traffic platooning, which decreased rapidly on entry to the four-lane road and increased only gradually on return to the two-lane road.

In another 1980 study, Hoban[15] used a microscopic computer simulation model, a predecessor of the TRARR model currently used in Australia, to evaluate the operational effectiveness of both complete four-laning and construction of relatively short passing lanes over an extended section of highway. The results of this study, illustrated in Figure 10, show that a four-lane road can increase operating speeds by up to 15 mi/hr (24 km/hr) above those found on a two-lane highway. Lower speed increases were found for the addition of one, two or three passing lanes along the same highway section. However, the construction of short passing lanes costs substantially less than complete four-laning, and Hoban[16] found in 1982 that passing lanes had higher benefit-cost ratios than complete four-laning and could be justified at lower traffic volume levels.

Table 7 presents minimum volume guidelines for installation of passing lanes that have been developed on the basis of Australian research. For any particular highway section, the recommended minimum design hour volume for installation of passing lanes can be determined from Table 7 on the basis of the availability of passing opportunities in the preceding 2 to 6 mi (3 to 10 km) and the percentage of slow vehicles in the traffic stream.[17] Passing lanes are now becoming a common feature on busy two-lane highways in Australia, and reactions from both drivers and highway engineers have been very favorable.[10]

Research on passing lanes has been active in Canada since 1975, especially in the provinces of Ontario and Alberta. The Canadian criteria for the installation of passing lanes on two-lane highways are intended to achieve a balance between the demand for passing and the supply of passing opportunities.

In 1975, the Ontario Ministry of Transport and Communications[18] introduced criteria for the design of two-lane highways based on the need to provide a percentage of highway length with "assured" passing opportunities. Under these criteria, passing lanes are provided where the combination of restricted sight distance and high opposing traffic volumes do not provide an adequate level of passing opportunities.

The concept used by Ontario has been refined by Alberta Transportation to provide a method to determine the net passing opportunities available on a two-lane highways.[19] These are defined by:

$$NPO = GAO \times APSD \tag{2}$$

where,

NPO = percentage of net passing opportunities;
GAO = fraction of time with gaps adequate for overtaking (> 25 sec);
APSD = percentage of highway length with adequate passing sight distance.

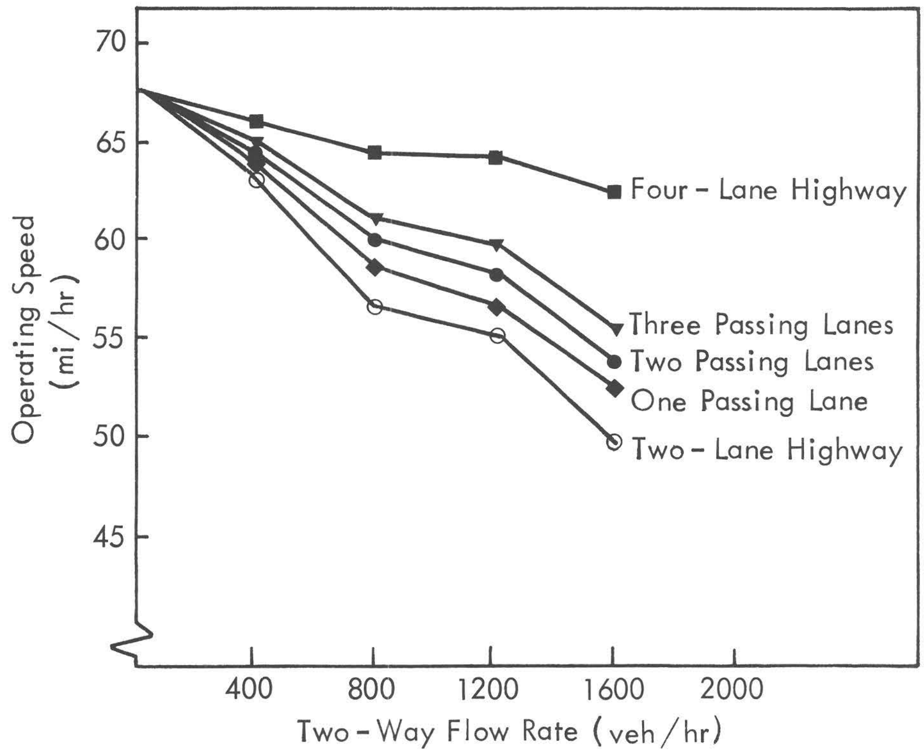


Figure 10 - Computer Simulation Results for Operational Effectiveness of Passing Lanes and Four-Lane Highways in Australia[15]

TABLE 7

RECOMMENDED MINIMUM VOLUME GUIDELINES FOR PASSING LANES IN AUSTRALIA[17]

Passing Opportunities Over Preceding 2 to 6 mi		Current-Year Design Volume (AADT)		
		Percentage of Slow Vehicles		
Description	Percent of Length Providing Passing[a]	5	10	20
Excellent	70-100	5,670	5,000	4,330
Good	30-70	4,530	4,000	3,470
Moderate	10-30	3,330	3,000	2,670
Occasional	5-10	2,270	2,000	1,730
Restricted	0-5	1,530	1,330	1,130
Very Restricted	0	930	800	670

[a] Note that the Australian definition of the percent of length providing passing produces much lower values than criteria used in North America.

Note: 1 mi = 1.609 km

From field studies on the Trans-Canada Highway in Banff National Park, the fraction of each hour with gaps for overtaking is defined as:

$$GAO = e^{-.0018626\ OFLOW} \tag{3}$$

where,

OFLOW = opposing flow rate (veh/hr).

Using these equations, it can be shown that approximately 40 percent of the hour is available at a flow rate of 500 veh/hr for drivers in the opposing traffic stream to pass. If such a road has 70 percent adequate passing sight distance, the net passing opportunities would be:

$$NPO = 0.40 \times 70 = 28\%$$

Rural highway design in Alberta is based on the objective of providing at least 50 percent net passing opportunities in both directions of travel in the peak period traffic during the design year. Where traffic volumes are low enough and available sight distance adequate to achieve this level, a two-lane highway will suffice. Where 50 percent net passing opportunities are not available in the design year, additional passing lanes or four-lane sections should be added. Passing lane and four-lane sections provide 100% net passing opportunities throughout their length and can, thus, raise the net passing opportunities over an extended section of highway to the level desired for design.

The 50 percent criterion for net passing opportunities is based on field observations of highways in Alberta that operated well at this level. It should be noted that most of Alberta is in the prairies, with relatively level terrain and good sight distance. Highway agencies in more restrictive terrain might find it economically infeasible to provide 50 percent net passing opportunities.

3.3.6 Safety Effectiveness

Safety evaluations have shown that passing lanes and short four-lane sections reduce accident rates below the levels found on conventional two-lane highways.

Table 8 compares the results of two before-after evaluations of passing lane installation. A California study by Rinde[20] at 23 sites in level, rolling, and mountainous terrain found accident rate reductions due to passing lane installation of 11 to 27 percent, depending on road width. The accident rate reduction effectiveness at the 13 sites in level or rolling terrain was 42 percent. In data from 22 sites in four States, Harwood and St. John[12] found the accident rate reduction effectiveness of passing lanes to be 9 percent for all accidents and 17 percent for fatal and injury accidents. The combined data from both studies indicates that passing lane installation reduces accident rate by 25 percent.

TABLE 8

ACCIDENT REDUCTION EFFECTIVENESS OF PASSING LANES

Source	Type of Terrain	Total Roadway Width (ft)[a]	No. of Passing Lane Sites	Percent Reduction: All Accidents	Percent Reduction: Fatal and Injury Accidents
Rinde[20]	Level, rolling, and mountainous	36	4	11	-
		40	14	25	-
		42-44	5	27	-
	Level and rolling sites only	36-44	13	42	-
Harwood and St. John[12]	Level and rolling	40-48	22	9	17
Combined Totals for Level and Rolling Terrain			35	25	-

Note: 1 ft = 0.305 m

[a] Total roadway width includes both traveled way and shoulders.

Harwood and St. John[12] found no indication in the accident data of any marked safety problem in either the lane addition or lane drop transition areas of passing lanes. In field studies of traffic conflicts and erratic maneuvers at the lane drop transition areas of 10 passing lanes, lane drop transition areas were found to operate smoothly. Overall, 1.3 percent of the vehicles passing through the lane drop transition area created a traffic conflict, while erratic maneuver rates of 0.4 and 0.3 percent were observed for centerline and shoulder encroachments, respectively. The traffic conflict and encroachment rates observed at lane drop transition areas in passing lanes were much smaller than the rates found in lane drop transition areas at other locations on the highway system, such as in work zones.

An evaluation of cross-centerline accidents involving vehicles traveling in opposite directions on the highway found no safety differences between passing lanes with passing prohibited in the opposing direction and passing lanes with passing permitted in the opposing direction where adequate sight distance was available.[12] The provision for passing by vehicles traveling in the opposing direction does not appear to lead to any safety problems at the types of sites and flow rate levels (up to 400 veh/hr in one direction of travel), where it has been permitted by the highway agencies that participated in the Harwood and St. John study. Both types of passing lanes had cross-centerline accident rates lower than those of comparable sections of conventional two-lane highway.

Reviewing a small number of climbing lane sites in the United States, Jorgensen[21] found no change in accident experience. In the United Kingdom, Voorhees[22] found a 13 percent reduction in accidents where a climbing lane was provided.

A safety evaluation of nine short four-lane sections in three states found a 34 percent lower total accident rate and a 43 percent lower fatal and injury accident rate on the short four-lane sections than rates on comparable sections of conventional two-lane highways.[12] These differences, although substantial, were not statistically significant because of the limited number of sites available. The cross-centerline accident rates for the short four-lane sections were generally less than half the rates for the comparable two-lane sections.

Table 9 summarizes the relative accident rates found in recent research for passing lane sections and short four-lane sections, expressed as ratios between the expected accident rate for each and the expected accident rate of a conventional two-lane highway.

TABLE 9

RELATIVE ACCIDENT RATES FOR IMPROVEMENT ALTERNATIVES

Alternative	All Accidents	Fatal and Injury Accidents
Conventional two-lane highway	1.00	1.00
Passing lane section	0.75	0.70
Four-lane section	0.65	0.60

3.3.7 Summary

Passing lanes have been found to be effective in improving overall traffic operations on two-lane highways, and they provide a lower cost alternative to four-laning extended sections of highway. Passing opportunities on two-lane highways can be increased by the installation of passing lanes in level and rolling terrain, of climbing lanes on sustained grades, and of short sections of four-lane highway. The traffic operational effectiveness of passing lanes can be predicted as a function of flow rate, passing lane length, and the percentage of traffic traveling in platoons using the procedure presented above. The installation of a passing lane on a two-lane highway reduces accident rate by approximately 25 percent. Recommended geometric design, signing, and marking practices for passing lanes are presented in this Guide.

3.4 ALTERNATIVE PASSING IMPROVEMENTS

This section of the Guide discusses several alternative improvements that can be used to provide additional passing opportunities, including turnouts, shoulder driving, and shoulder use sections.

3.4.1 Turnouts

Turnouts are an important minor passing improvement that has been used most widely in the western United States, but are applicable to any winding or mountainous two-lane highway with limited passing opportunities. Turnouts are not a substitute for passing lanes, but are an alternative that can provide some operational benefits on highways where passing lanes are not constructed.

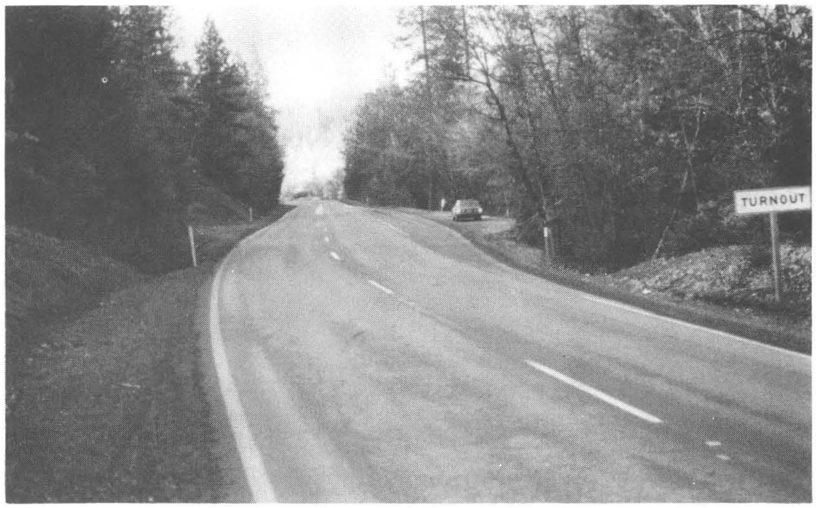
A turnout is a widened, unobstructed shoulder area on a two-lane highway that allows slow-moving vehicles to pull out of the through lane to permit following vehicles to pass. Turnouts are relatively short, generally less than 600 ft (190 m) in length, as compared with to passing lanes, which are generally at least 1,000 ft (190 m) in length. Figure 11 illustrates three typical turnouts in California, Oregon, and Washington.

The driver of a slow-moving vehicle that is delaying one or more following vehicles and is approaching a turnout is expected to pull out of the through lane into the turnout and allow the following vehicle(s) to pass. The driver of the slow-moving vehicle should remain in the turnout only long enough for the following vehicle(s) to pass and should then return to the through lane. When there are only one or two following vehicles, this maneuver can be completed smoothly and there is no need for the driver of the turnout vehicle to stop. When there are three or more following vehicles, however, the driver of the turnout vehicle should stop, if necessary, to allow all of the following vehicles to pass.

Location and Configuration

Turnouts may be used on nearly any type of two-lane highway that has limited passing opportunities, but they are most often used on lower volume highways, where long platoons are rare, and in difficult terrain with steep grades and/or isolated slow-moving vehicles where the construction of a passing or climbing lane may not be cost-effective.

Turnouts can be employed successfully both on sustained grades and in level or rolling terrain. Turnouts are especially well-suited to mountainous terrain where the cost of providing a passing 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. A 500-ft (150-m) turnout in relatively uncomplicated terrain can be constructed for as little as $18,000.



California

Oregon

Washington

Figure 11 - Typical Turnout Sites on Two-Lane Highways

Turnouts are often used in series along a highway section to provide passing opportunities. A typical spacing between turnouts in a series is 1 to 3 mi (2 to 5 km).

To maximize usage, turnouts should be located where the drivers of slow-moving vehicles do not consider use of the turnout to involve substantial delay to themselves. For example, it is better to locate a turnout in the middle of a sustained grade where the speeds of trucks and recreational vehicles (RVs) are depressed, than at or near the crest of the grade where truck and RV drivers are intent on recovering their lost speed.

Ideally, to avoid driver confusion, turnouts and passing lanes should not be interspersed. Either turnouts alone or passing lanes alone should be used to provide passing opportunities over extended sections of two-lane highway. The use of signing to introduce a series of turnouts is recommended so that drivers consider the section of highway with turnouts to be distinct from previous highway sections that may have used passing lanes or other treatments. On some low volume roads, however, it may be desirable to provide turnouts initially and to progressively replace these turnouts with passing lanes as traffic volumes increase; this could lead to some mixing of the two types of treatments.

Geometrics

Proper geometric design of turnouts should consider turnout length, turnout width, and the location of the turnout with respect to horizontal and vertical curves that limit sight distance.

The recommended lengths for turnouts are as follows:

Mean Approach Speed (mph)	Turnout Entry Speed (mph)	Recommended Turnout Length (ft)
20	15	200
30	25	200
40	35	280
45	40	350
50	45	440
55	50	530

1 mile = 1.609 km
1 ft = 0.305 m

The recommended lengths are based on the assumption that slow-moving vehicles enter the turnout at 5 mi/hr (8 km/hr) slower than the mean speed of the through traffic. These lengths are sufficient to allow a vehicle to enter the turnout at the assumed speed, coast to the midpoint of the turnout without braking, and then, if necessary, brake to a stop in the remaining turnout length using a deceleration rate not exceeding 10 ft/sec^2 (3 m/sec^2).

Field observations have confirmed that in turnouts of the recommended length one or two following vehicles can pass without requiring the turnout user to stop.[12]

The recommended lengths for turnouts include the entry and exit tapers which, due to the nature of turnouts, can be much shorter than the tapers recommended for passing lanes. Typical entry and exit taper lengths for turnouts are 50 to 100 ft (15 to 30 m). Turnouts shorter than 200 ft (60 m) are not recommended even for very low approach speeds. Turnouts longer than 600 ft (180 m) are not recommended on high-speed roads because some drivers may use them as passing lanes. For the same reason, shorter turnouts should be used on lower speed roads.

The minimum recommended turnout width is 12 ft (3.7 m), and widths up to 16 ft (4.9 m) are considered desirable. Turnouts wider than 16 ft (4.9 m) may confuse drivers about the correct path to follow in the turnout. An edge line marking on the right side of the turnout is desirable to guide drivers, especially in wider turnouts.

A turnout should not be located on or adjacent to a horizontal or vertical curve that limits sight distance in either direction. The available sight distance on the approach to the turnout should be at least 1,000 ft (305 m). A driver approaching a turnout should have a clear view of the entire turnout in order to determine whether the turnout is available for use and in order to anticipate whether a vehicle using the turnout is about to reenter the traffic stream. Operational experience suggests that turnouts that cannot be seen for some distance by approaching drivers are less likely to be used.

Signing and Marking

Proper signing and marking of turnouts are necessary both to assure safe operations and to maximize turnout usage. Signing is needed to convey information to drivers at three locations in advance of turnouts:

- In advance of series of turnouts;
- In advance of each turnout; and
- At the entrance to each turnout.

Signing in advance of a series of turnouts is desirable, while signing at the latter two locations is recommended for each and every turnout. The recommended signing practices for turnouts are illustrated in Figure 12.

The SLOW VEHICLES USE TURNOUTS sign to alerts drivers to an upcoming series of turnouts. The distance contained in the sign legend will vary, but should be appropriate to the length of the highway over which turnouts are provided. While use of this sign is not mandatory, it is desirable to alert drivers to a change in the character of the highway where a series of turnouts is introduced after a series of passing lanes or after

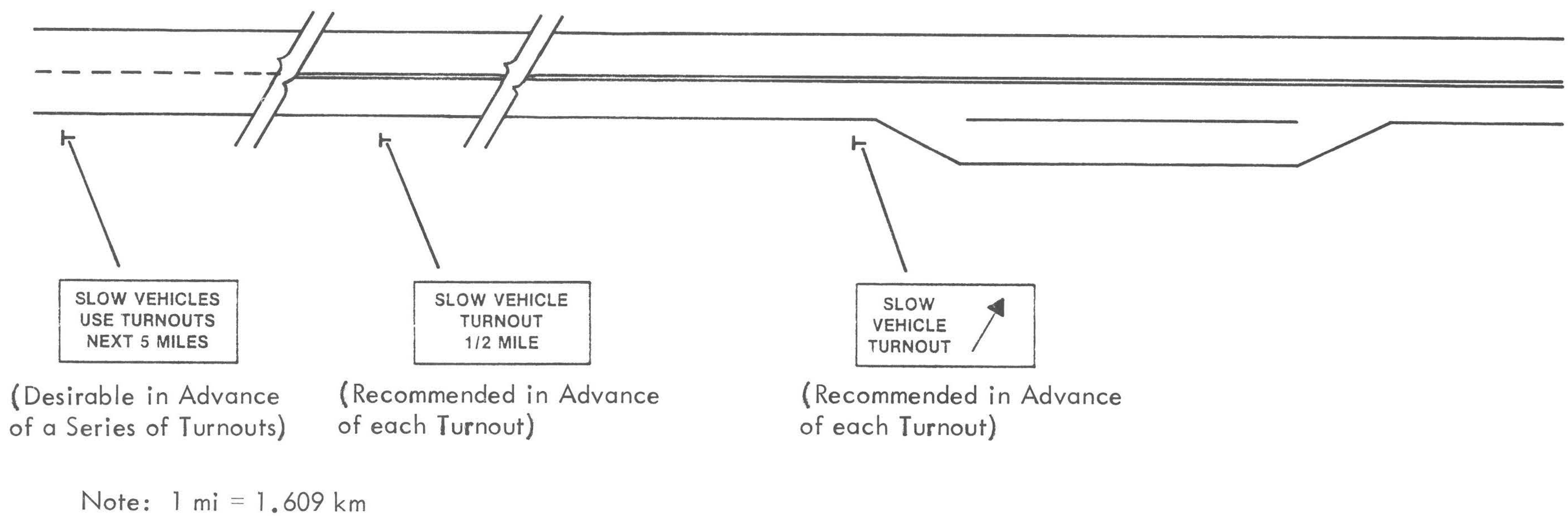


Figure 12 - Recommended Signing Practice for Turnouts

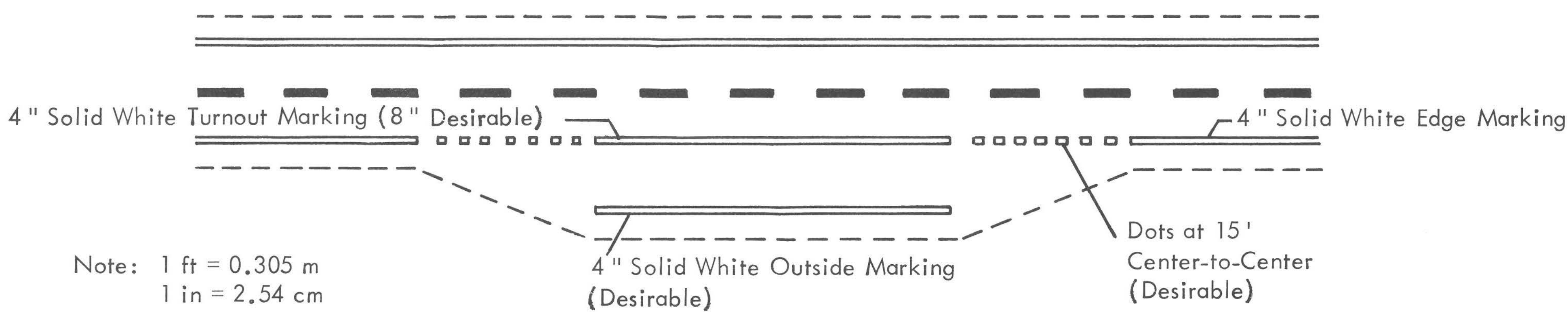


Figure 13 - Recommended Marking Practice for Turnouts

an extended section of two-lane highway with a high percentage of adequate passing sight distance where passing improvements were not needed.

The SLOW VEHICLE TURNOUT 1/2 MILE sign provides drivers with advance notification of a specific turnout. Alternatively, a 1/4-mi distance can be used, especially on lower speed roads. This advance sign is recommended prior to each turnout so that drivers of both slow-moving vehicles and following vehicles are alerted to the passing opportunity provided by the upcoming turnout. Care should be taken in the location of this advance sign; there should be no roadside areas between the advance sign and the turnout that might be mistaken for the turnout by drivers.

The SLOW VEHICLE TURNOUT sign with an upward-sloping arrow is used immediately in advance of each turnout to identify the turnout for drivers. The combination of this sign and the driver's view of the turnout alerts the driver of a slow-moving vehicle to enter the turnout.

On two-lane highways with narrow shoulders, parking areas or scenic overlooks may be needed at intervals to provide a roadside area for drivers to pull off the road and stop. The provision of parking areas is desirable in order to discourage drivers from using turnouts for stopping, but parking areas should be clearly distinguished from turnouts by different paving and appearance, if necessary, white-on-blue PARKING AREA signs, as specified in MUTCD Section 2D-42, may also be used.

The pavement markings recommended for turnouts are shown in Figure 13. A 4-in (10-cm) solid white line separating the turnout from the through travel lane should be provided at all turnouts. Breaks of approximately 50 to 100 ft (15 to 31 m), between the highway edge lines and the turnout marking identify the turnout entry and exit areas. This turnout marking serves two functions. The first function is to distinguish the turnout from the through travel lane for turnout users; the second function is to continue the highway edge line to guide drivers not using the turnout along the through travel lane. The latter function is especially important in fog or other limited visibility conditions.

Several desirable additions to turnout marking practice may be used by highway agencies to enhance turnout operations. These include (a) an outside edgeline to delineate the right-hand edge of the turnout; (b) a double width line (8 inches or 20 cm wide) to separate the turnout from the through lane; and (c) painted dots at 15-ft (4.5-m) center-to-center spacing to extend the highway edge line through the turnout entry and exit areas. The use of an outside edgeline is especially appropriate where the paved area of the turnout is larger than needed for the turnout maneuver.

Legal Considerations

Several States have adopted legislation requiring drivers of slow-moving vehicles to use turnouts. If legislation of this type is adopted, it is recommended that the law require any driver who is proceeding at less than the normal speed of traffic and who is delaying one or more following vehicles to use signed and marked turnouts so that the vehicle(s) being delayed may pass. States that require slow-moving vehicles to use turnouts may choose to reinforce this requirement with appropriate signing.

Some State laws require a slow-moving vehicle to use a turnout only if it is delaying five or more vehicles. The five-vehicle delay requirement is not recommended because many vehicles being delayed by slow-moving vehicles may be unnecessarily denied the opportunity to pass. Highways that regularly experience platoons of six or more vehicles should be candidates for the installation of passing or climbing lanes rather than turnouts, unless the cost of constructing an added lane is prohibitive.

Operational Effectiveness

Turnouts can be effective in providing passing opportunities on two-lane highways, although they are not as effective as passing or climbing lanes. Turnouts must be well-designed and well-located to be operationally effective. A single well-designed and well-located turnout can be expected to provide 20 to 50 percent of the number of passes that would occur in a 1-mi (1.6-km) passing lane in level terrain.[12] However, a poorly located or poorly designed turnout will receive little usage and will provide almost no operational benefits. A series of well-designed and well-located turnouts is needed to even approach the level of operational benefits provided by a single passing lane.

Drivers tend to avoid turnouts that appear to be too short, too narrow, or poorly paved, and such turnouts add almost nothing to the quality of traffic service on a highway. Turnouts also receive little use if they are shorter than the recommended minimum lengths tabulated above; if they are narrow, or have a curb along their right-hand edge that appears to unnecessarily constrain the driver; if they have rough pavement surface; or if they are unpaved. Turnouts should not be located on the inside of a horizontal curve where an approaching driver is unable to see from a distance whether or not the turnout is being used. To maximize turnout usage, drivers of slow-moving vehicles should feel that they are not making too great a sacrifice in speed when providing a passing opportunity for following vehicles. Turnout usage, especially by large combination trucks, is likely to be minimal when the drivers of the slow-moving vehicles perceive that they will be forced to stop when entering a turnout.

Field observations have shown that, in most cases, when a slow-moving vehicle enters a turnout, all of the vehicles in the following platoon are able to pass.[12] When platoons of more than two vehicles build up, drivers of slow-moving vehicles may be reluctant to use a turnout because they may have to stop. The solution to this dilemma is to provide turnouts at more frequent intervals, so that long platoons are less likely to build up behind slow-moving vehicles.

Field observations also suggest that some drivers are uncertain about whether or not to use turnouts, and whether or not to stop in the turnout if the following platoon has not cleared. Driver education may be needed to overcome these uncertainties.

The operational effectiveness of turnouts may also be reduced if turnouts are used by drivers for other purposes than those for which they are intended. On two-lane highways with narrow shoulders, drivers often have difficulty finding suitable locations to stop and park for rest, to read a map, to eat a meal, or to change drivers. Highway agencies should consider providing parking areas so that drivers are not tempted to use turnouts for this purpose. Parking areas should have distinct signing and a contrasting pavement surface type so that drivers can distinguish them from turnouts.

Safety Effectiveness

Turnouts have been evaluated in two research studies and have been found to operate safely. Rooney[23,24] found no evidence that a significant number of accidents occur at turnouts. Sixteen turnouts in California were found to experience only one accident per 80,000 turnout users.

Harwood and St. John[12] evaluated 42 turnouts in three states and found that a typical turnout experiences only one accident every 5 years. At seven turnouts where usage rates were observed in the field, the evaluation found only one accident per 400,000 turnout users, an even lower rate than that found by Rooney. A safety comparison between the turnout sites and adjacent sections of conventional two-lane highway from 0.20 to 0.50 mi (0.30 to 0.80 km) away found that the turnout sites had accident rates approximately 30 percent lower than the adjacent untreated sites.

Field observations by Harwood and St. John[12] found that 5 to 10 percent of turnout users caused a traffic conflict (such as braking by a following vehicle) when reentering the highway from a turnout, but the accident experience associated with this maneuver was minimal. This finding implies that following drivers anticipate the possible return of the turnout vehicle to the through lanes and that their braking is a controlled response which does not indicate the likelihood of a collision.

Summary

Turnouts provide a limited increase in passing opportunities and should be considered for use on two-lane highways where passing lanes are not provided. Safety evaluations have not found any accident problems caused by motorists using turnouts. Many turnouts receive little usage because they are poorly designed or poorly located. The geometric design, traffic control, and location criteria presented in this Guide should be followed to assure that turnouts are effective in improving two-lane highway operations.

3.4.2 Shoulder Driving

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[25] has catalogued 21 other uses of 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 United States 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 return to the travel lane after the following vehicle has passed. This custom is regarded as a courtesy to other motorists and requires little or no sacrifice in speed by either motorist. In effect, paved shoulders can function as continuous turnouts. This practice is most common in Texas, but it has also been observed in many western States and in Western Canada. Figure 14 illustrates a slow-moving recreational vehicle using the shoulder so that following vehicles can pass.

Figure 14 - Slow-Moving Recreational Vehicle Using Shoulder to Allow Following Vehicles to Pass

A paved shoulder can also function as a climbing lane on a sustained upgrade although, in this case, slow-moving vehicles often ascend the remainder of the grade on the shoulder rather than returning to the travel lane after being passed.

Shoulder driving provides a great deal of operational flexibility for two-lane highways. Highway agencies should consider encouraging the usage of paved shoulders by slow-moving vehicles as one means of improving passing opportunities in two-lane highways without a major capital investment.

Shoulder Design and Geometrics

For effective use as a passing aid, shoulders on two-lane highways must be paved and reasonably continuous. The shoulder must be in good condition so that drivers are not reluctant to use it, and it should have enough structural strength to withstand regular usage by trucks and recreational vehicles. Shoulder widths of at least 8 ft (2.4 m), and preferably 10 ft (3.1 m), are required.

Highway agencies should consider the mileage of two-lane highways with paved shoulders and their structural quality when deciding whether to encourage shoulder driving as a passing aid. It should be kept in mind that where shoulder driving becomes common, it is not limited to a few selected sites but will occur anywhere on the highway system where paved shoulders are provided. In Texas, where shoulder driving is most common, many two-lane highways have a 44-ft (13.4-m) total roadway width, including both the travel lanes and full-depth paved shoulders. This design is appropriate to accommodate passing; and, while it may cost more to construct than the two-lane cross-sections with narrower and weaker shoulders, it may be a very cost-effective design if it reduces the need for more expensive passing improvements. In addition to providing passing opportunities, paved shoulders have been found to reduce maintenance costs and to improve traffic safety.[26,27] The additional width provided by a shoulder can also be used to provide turning lanes with very little extra cost.

Legal Considerations

Driving on the shoulder is illegal in most States. Even in Texas, shoulder driving may technically be prohibited by law although this prohibition is not enforced. Thus, a highway agency considering the encouragement of shoulder driving as a passing aid might need to consider both legislation to authorize shoulder driving by slow-moving vehicles and a public education campaign to familiarize motorists with the new law.

Signing and Marking

Shoulder driving, as practiced in Texas and other western States, requires no special signing. The practice of shoulder driving has grown up through local custom rather than through the use of any particular traffic control device to encourage it.

Most two-lane highways where slow-moving vehicles use the shoulder have only conventional pavement markings: a yellow centerline indicating passing and no-passing zones and two white edge lines separating the through travel lanes from the shoulders. Once drivers become familiar with shoulder driving, no special markings are needed to encourage it. However, Texas has recently begun marking some two-lane highways with two 12 ft (3.6 m) travel lanes separated by two yellow centerlines spaced 2 to 4 ft (0.6 to 1.2 m) apart, referred to as a two-lane double-striped highway.[28] This marking, illustrated in Figure 15, can be accommodated within a 44-ft (13.4-m) cross-section. This marking allows two vehicles traveling in the same direction

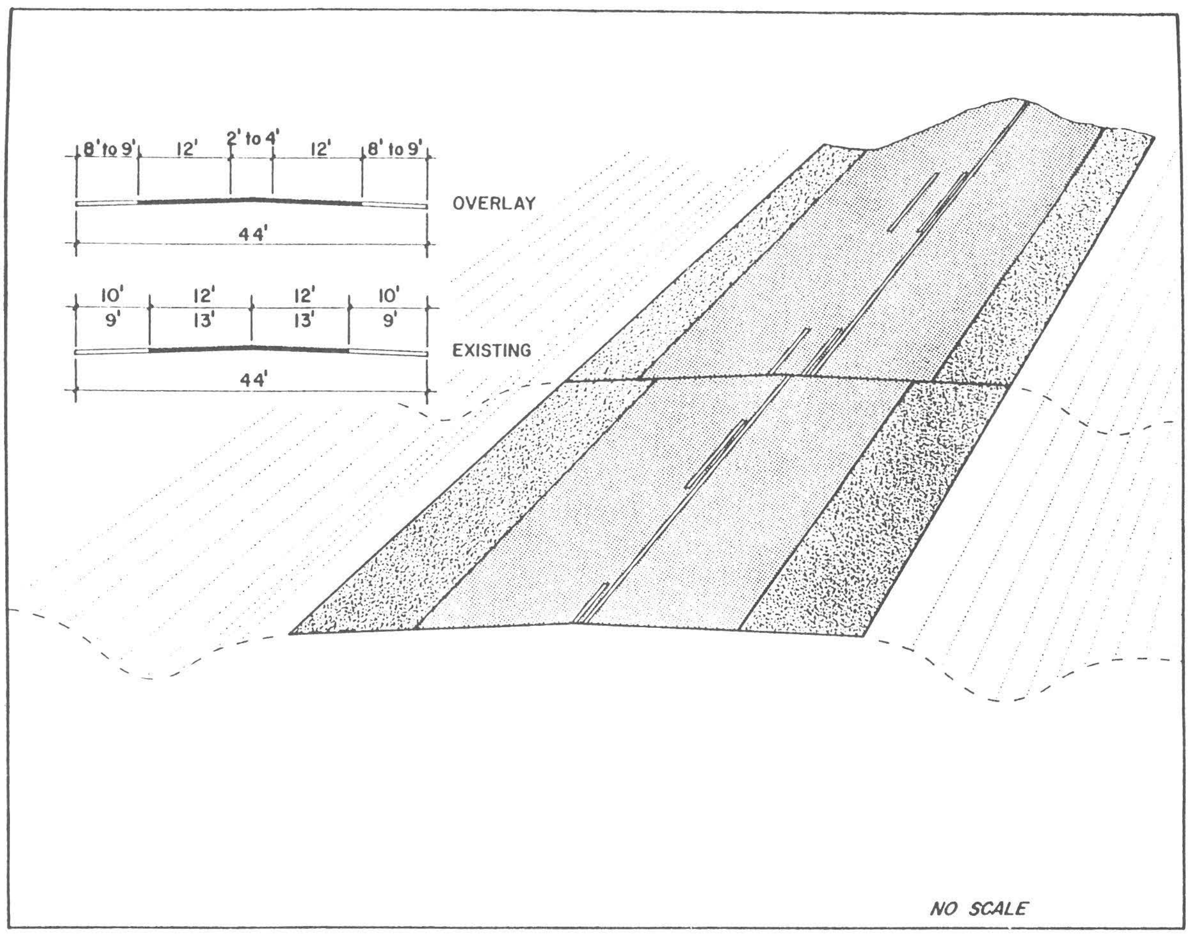


Figure 15 - Two-Lane Double Striped Highway Used in Texas to Accommodate Passing Without Encroachment on the Opposing Lane[28]

to pass abreast without encroaching on the travel lane for the opposing direction, and it reduces the potential for vehicles to use part of the opposing lane when passing a vehicle that has moved to the shoulder.

Operational Effectiveness

The traffic operational benefits of shoulder driving in Texas have been evaluated by Fambro et al.[29] This study evaluated the effectiveness of shoulder driving on two-lane highways by comparing operational conditions on highways without paved shoulders. In Texas, shoulder driving on two-lane highways with paved shoulders is widespread while shoulder driving on highways without paved shoulders is minimal.

The Fambro study found that the operational benefits of providing a full-width paved shoulder increase as the traffic volume increases. These benefits are minimal at flow rates below 200 veh/hr in one direction of travel; however, at flow rates above 200 veh/hr, paved shoulders appear to raise the average speed on the roadway by at least 10 percent. This increase in speed is undoubtedly due in part to shoulder driving, but it is also possible that the presence of the paved shoulder outside the travel lane encourages higher speeds, even when it is not used for shoulder driving.

The percentage of traffic in platoons was generally lower on highways with paved shoulders than on highways without paved shoulder. On highways with paved shoulders the percentage of vehicles following in platoons was typically less than 20 percent. At any given location, however, only about 5 percent of the traffic actually used the shoulders.

Based on the study findings, Fambro et al.,[29] recommended that it be legal for a motorist to pull onto a paved shoulder in order to let faster vehicles pass, but that it should not be a requirement for motorists to do so. To obtain the operational benefits from shoulder driving, it was concluded that paved shoulders should be added to all two-lane roads in Texas with traffic volumes in excess of 200 veh/hr in one direction of travel.

Similar results were obtained from field studies under higher volume conditions on the Trans-Canada highway in the province of Alberta.[30] A survey of passing maneuvers on a 6.2 mi (10 km) section of two-lane highway with a two-way volume of 1,000 veh/hr (75/25 percent directional split) found that 25 percent of all passing maneuvers involved shoulder driving, with almost all of these passing maneuvers in the heavy direction of flow. However, it was observed that as volume increases, drivers of slower vehicles become more reluctant to move to the shoulder for fear of not being able to reenter the traffic stream. Clearly, paved shoulders do aid in providing passing opportunities and dispersing platoons; but because some drivers are reluctant to use them, shoulders cannot be considered to be a substitute for passing lanes. Morrall[30] also found some uncertainty among the drivers of slower vehicles as to whether or not they should use the shoulder, as well as some frustration among following drivers when others failed to pull onto the shoulder. The provision of a continuous paved shoulder to encourage courtesy by drivers of slow-moving vehicles can also lead to undesirable traffic behavior. Cases of passing on the right and three vehicles traveling abreast have been observed. However, these cases are outweighed by the overall safety benefits of paved shoulders discussed below.

Safety Evaluation

The study of shoulder driving in Texas by Fambro et al.[29] compared the safety records of two-lane 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, over the ADT range from 1,000 to 7,000 veh/day. Below the ADT of 3,000 veh/day, almost all of the benefits of paved shoulders came from reduction in single vehicle run-off-road accidents. Above 3,000 veh/day, the provision of paved shoulders reduced both single-vehicle and multiple-vehicle non-intersection accidents. There was no indication of any pattern of accidents related to shoulder driving.

Summary

Shoulder driving can provide a limited increase in passing opportunities on two-lane highways without the expenditures required for passing lane construction and without the accident risks inherent in passing in the opposing traffic lane. Shoulder driving does require high-quality, full width paved shoulders and a driver population willing to show courtesy by moving to the shoulder to make way for faster vehicles. There is no indication of any safety problem associated with shoulder driving on two-lane highways.

3.4.3 Shoulder Use Sections

Another approach to the use of paved shoulders by slow-moving vehicles is to permit this practice at selected sites designated by specific signing. Signs are erected at both the beginning and end of the highway section where shoulder use is allowed. This approach is a much more limited application of shoulder use by slow-moving vehicles than the shoulder driving described in Section 3.4.2. In areas like Texas, drivers will use paved shoulders wherever they are provided; in this approach, paved shoulders can be used only where permitted by specific signing.

Typically, drivers of slow-moving vehicles in shoulder use sections move to the shoulder only long enough for following vehicles to pass and then return to the through travel lane. Thus, a shoulder use section functions as an extended turnout.

This approach enables a highway agency to encourage shoulder use by slow-moving vehicles only where it has been established that additional passing opportunities are needed and that the shoulder is structurally adequate to handle the anticipated traffic loads. Shoulder use sections can be established by installation of signs at locations with existing paved shoulders or by construction of new paved shoulders at locations where passing opportunities are needed. Shoulder use sections designated by signs have been used only in the State of Washington, but the concept has wide applicability to increase passing opportunities, and it may be the most effective approach to obtain operational benefits from paved shoulders in regions where shoulder driving is not customary.

Location and Configuration

Shoulder use sections are appropriate on steep upgrades and at locations where vehicle platooning is high due to lack of passing opportunities. Shoulder use sections can range in length from 1,000 ft (305 m) to 3 mi (5 km). Depending on the site geometrics, shoulder use sections can be provided in one or both directions of travel. For example, relatively short shoulder use sections might be provided in opposite directions of travel approaching a hill crest; a longer shoulder use section might be provided for several miles in one direction of travel on a gradual upgrade.

Shoulder Design and Geometrics

The shoulder design and geometrics required for shoulder use sections are similar to those required for general shoulder driving, discussed in Section 3.4.2. Shoulder use should be encouraged by signing only where shoulders are at least 8 ft (2.4 m), and preferably 10 ft (3.1 m), wide. Adequate structural strength and good surface conditions are required in designated shoulder use sections. Particular attention should be placed on shoulder maintainance; drivers of slow moving vehicles are unlikely to use the shoulder if it is rough, broken, or covered with debris.

Legal Considerations

There is no provision in the laws of most States for vehicles to use the shoulder at some locations but not at others. Shoulder driving is not generally permitted under Washington State law, so the State enacted legislation specifically permitting the DOT to determine which portions of the two-lane highway system could be safely used for shoulder driving to permit passing and to establish such sections by the erection of appropriate signs. Similar legislation would probably be required in most States for highway agencies to be permitted to designate shoulder use sections.

Signing and Marking

Signing of shoulder use sections is not addressed in the MUTCD. The following signs are recommended for use at the beginning and end of shoulder use sections:

SLOW VEHICLES
MAY USE
SHOULDER

At Beginning of
Shoulder Use Section

END
SHOULDER
DRIVING

At End of
Shoulder Use Section

Shoulder use sections should be introduced with a black-on-white regulatory sign with the legend SLOW VEHICLES MAY USE SHOULDER. The end of the shoulder use section should be designated with a black-on-white regulatory sign with the legend END SHOULDER DRIVING.

Figure 16 presents photographs of the signs for shoulder use sections in the State of Washington. The Washington signing practice is identical to the practice recommended above except that the sign at the beginning of a shoulder use section carries the supplementry restriction DAYLIGHT HOURS ONLY. The prohibition of shoulder driving to permit passing at night is not recommended as a requirement in this Guide; however, this restriction may have little negative impact on traffic operations since high levels of vehicle platooning are unlikely after dark on many two-lane highways.

The pavement markings at a designated shoulder use section should be identical to those on a conventional two-lane highway, including the indication of passing and no-passing zones by the yellow centerline marking. Washington State law carries a specific provision that signing for shoulder use sections supersedes the meaning of pavement edge lines separating the shoulder from the travel lanes.

Operational Effectiveness

A field evaluation of three shoulder use sections in Washington State by Harwood and St. John[12] found shoulder use sections to be effective in increasing passing opportunities and reducing vehicle platooning, but less effective than passing lanes. Between 2 and 9 percent of all vehicles traveling through the Washington sites used the shoulder to permit other vehicles to pass. However, between 21 and 43 percent of all platoon leaders used the shoulder at the Washington sites, creating passing opportunities for following vehicles that would not otherwise have been available. This level of shoulder use is comparable to that observed by Fambro et al.[29] for shoulder driving in Texas.

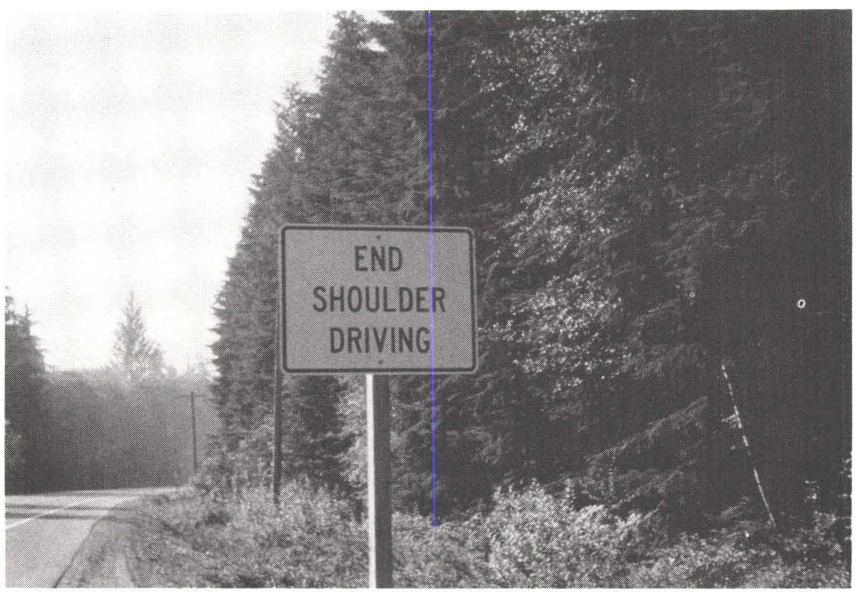


Figure 16 - Signing Used in Washington State to Introduce and Terminate a Shoulder Use Section

Shoulder use sections were found to be about 20 percent as effective in reducing traffic platooning as passing lanes of comparable length.[12] The rate of passing maneuvers in shoulder use sections was between 20 and 50 percent of the observed rates in passing lanes of comparable length.

Safety Effectiveness

An evaluation of safety conditions at two sites in Washington State before and after installation of designated shoulder use sections found no evidence of any influence of shoulder use on accidents.[12] The accident rate increased slightly at one site and decreased slightly at the other. However, at both sites, none of the accidents that occurred during the study period involved multiple-vehicle collisions. There was no evidence of any safety problems related to collisions between vehicles using the shoulder and vehicles using the travel lanes. Therefore, it is concluded that installation of a shoulder use section designated by signing is unlikely to increase accident experience.

Summary

Shoulder use sections provide the traffic operational benefits of shoulder driving at sites designated by signs. The use of signs to permit shoulder driving allows a highway agency to select sites where passing opportunities are needed and where the paved shoulder is structurally adequate to carry the anticipated traffic loads. Shoulder use sections designated by signs have been used by one highway agency and no safety problems associated with shoulder use by slow-moving vehicles have been encountered.

4. TURNING IMPROVEMENTS

This section of the Guide addresses improvements to reduce delays caused by turning movements on two-lane highways. It begins with a discussion of the need for turning improvements and the improvement alternatives that should be considered. Specific guidance is then provided for the use of intersection turn lanes, shoulder bypass lanes, and two-way left-turn lanes. The scope of this section is limited to improvements at driveways and unsignalized intersections.

4.1 NEED FOR TURNING IMPROVEMENTS

Turning movements at intersections and driveways may decrease the level of traffic service on two-lane highways by delaying following vehicles. Vehicles making a right turn off a highway must slow to a speed appropriate for the turning radius and width of the intersecting road or driveway and may cause following vehicles to slow as well. Left turns off a two-lane highway typically involve more delay than right turns, because following vehicles may be forced to slow, or even stop, depending upon whether or not a gap in the opposing traffic stream is available for the left-turning vehicle to complete its turn.

The justification for turning improvements on a two-lane highway is usually to reduce delay or accidents at a specific intersection or series of driveways. In limited situations, on highways with high traffic volumes, turning improvements are needed to prevent the formation and growth of vehicle platoons. The operational problems created by turning vehicles are illustrated in Figure 17. Note that the vehicle stopped to make a left turn

Figure 17 - Vehicle Waiting to Make a Left Turn Creates Delays to Following Vehicles

has delayed a queue of following vehicles; one of the following vehicles is using the shoulder to bypass the turning vehicles.

4.1.1 Delay Reduction

Evaluation of the need for turning improvements to reduce delay on two-lane highways requires engineering data including counts of hourly traffic volumes in each direction of travel, counts of turning volumes at intersections and/or driveways, and field observations or HCM estimates of the level of vehicle platooning in the approaching traffic streams.

The need for turning improvements to improve traffic operations has been assessed by Hoban[31] with a simple computer model of an intersection that simulates delays to through vehicles caused by turning vehicles for any specified combination of traffic volume, composition, directional split, and percentage of left turns. Some of the key results obtained by Hoban[31] are illustrated in Figures 18 and 19. Figure 18 shows the delay reductions which can be achieved by removing the effects of turns from the through traffic on a two-lane road, expressed as hours of delay reduced per hour of traffic. Figure 19 shows the effects of delays caused by turning vehicle on percent time delay over a road section. The top line in Figure 19 shows that turning vehicles can substantially increase percent time delay for through traffic over a short road length. However, when these effects are averaged over a longer road section, the increase in percent time delay is greatly reduced, as shown by the center line in Figure 19.

The major results of this investigation[31] are:

- At traffic volumes below 200 veh/hr in one direction of travel, there is virtually no delay to following vehicles. At such low flow rates, the probability that a left-turning vehicle will be closely followed by another vehicle is minimal.

- Turn delays have only small effects on overall traffic operations on a 0.5-mi (0.8-km) highway section at volumes below about 600 veh/hr with 5 percent turning vehicles or below about 400 veh/hr with 20 percent turning vehicles.

- At higher traffic volumes on a 0.5-mi (0.8-km) highway section, turn delays can cause substantial decrease in average speeds and increases in percent time delay, and individual vehicles can be delayed as much as three or four times their normal travel time.

- When turn delays are evaluated over a longer highway section (e.g., 10 mi or 16 km in length), their effect on overall traffic operations is quite small, even at high traffic flows. This is because turn delays are quite localized, while delays due to traffic platooning behind slow vehicles are typically experienced over many miles of travel.

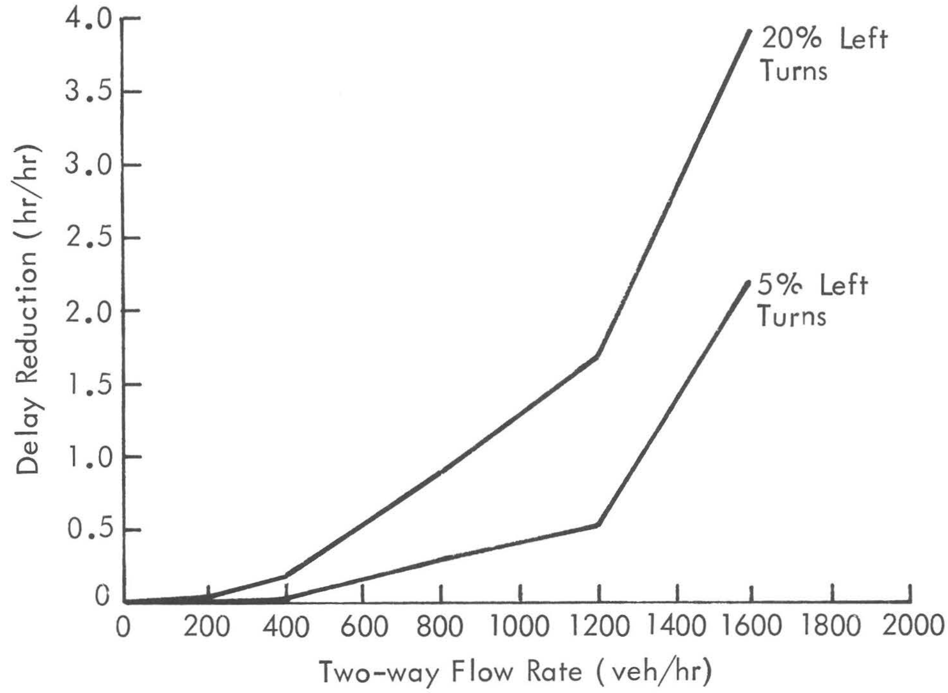


Figure 18 - Estimated Delay Savings from Turn Lanes[31]

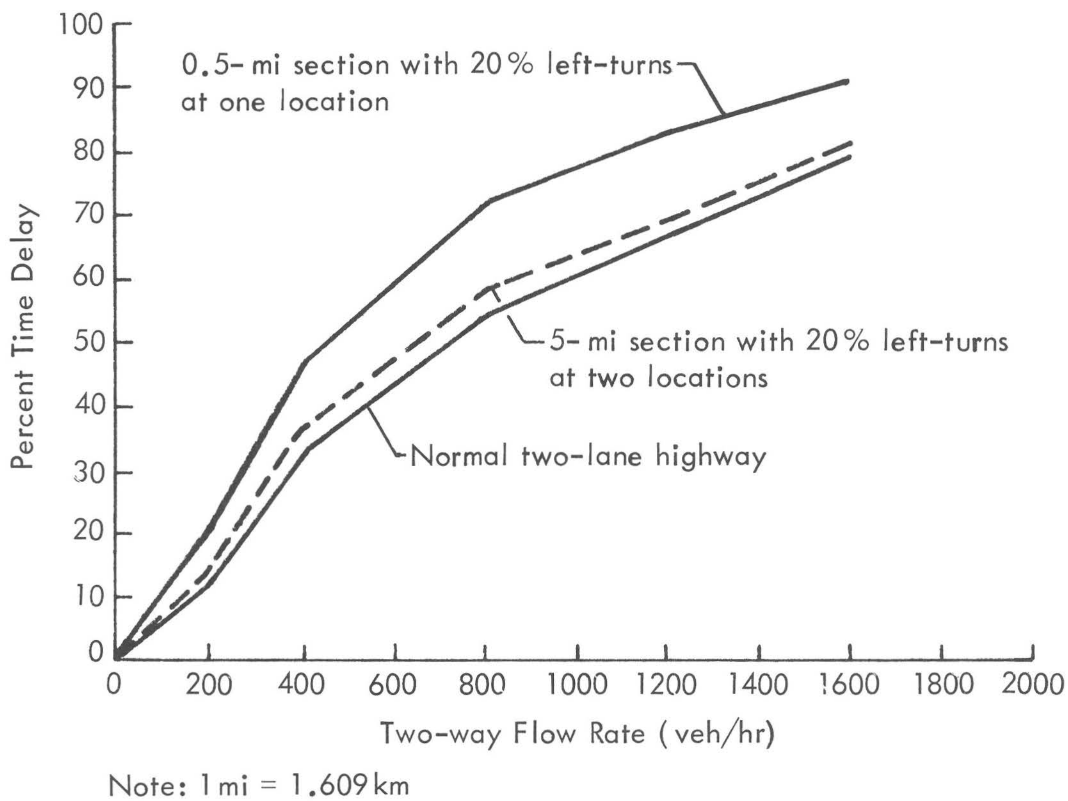


Figure 19 - Simulated Percent Time Delay for Three Cases[31]

- When turn delays are compared with an extreme case of a 10-mi (16-km) no-passing zone, the delays caused by the no-passing zone are considerably greater than those due to turning vehicles.

It is difficult to generalize about the traffic operational effects of delays caused by turning traffic, since there are many possible combinations of road and traffic conditions which may need to be considered. When traffic delays due to platooning are already severe, for example, the additional delays due to turning traffic may be unacceptable, and may be easier to remedy than the platooning delays.

One of the most important conclusions which may be drawn from this investigation is that both passing and turning problems must be considered in any evaluation of an extended road section. Improvements which reduce turn delays while creating no-passing zones, for example, may not provide a better overall quality of service on a road.

4.1.2 Accident Reduction

The need for turning improvements to reduce accidents should be based on consideration of approach speeds, geometric factors such as sight distance, and accident history. Two types of accident analysis are typically used to justify turning improvements. First, turning improvements may be warranted at sites where accident rates are high; e.g., substantially higher than the statewide or areawide average. Accident rates for individual intersections should be expressed as accidents per million vehicles entering the intersection, while accident rates for highway sections should be expressed as accidents per million vehicle-miles. Second, turning improvements may be warranted at sites where collision diagrams or tabulations of accident types indicate persistent patterns of left-turn, right-turn, angle, or rear-end accidents associated with turning maneuvers at intersections and driveways.

4.2 TURNING IMPROVEMENT ALTERNATIVES

Three alternative turning improvements are available to reduce delay and accidents on two-lane highways. These are:

- Intersection turn lanes. Separate right-turn and left-turn lanes can be provided at intersections and, in special cases, at high-volume driveways.

- Shoulder bypass lanes. Bypass lanes can be provided at T-intersections to enable through vehicles to pass to the right of vehicles that are stopped waiting to make a left turn.

- Two-way left-turn lanes can be installed in the center of a two-lane highway to allow vehicles traveling in either direction to make left-turns into intersections and driveways.

These alternatives are discussed in the following sections.

4.3 INTERSECTION TURN LANES

Intersection turn lanes are desirable at selected locations in order to reduce delays to through vehicles caused by turning vehicles and to reduce accidents related to turning maneuvers. Separate right-turn and left-turn lanes may be provided, as appropriate, to remove turning vehicles from the through travel lanes. Left-turn lanes, in particular, provide a protected location for turning vehicles to wait for a gap in opposing traffic to complete their turn. Such protection reduces the potential for rear-end accidents between left-turning and following vehicles, and may also encourage drivers of left-turning vehicles to wait for an adequate gap in opposing traffic before completing their turn. Figure 20 illustrates a typical two-lane highway intersection with left-turn lanes.

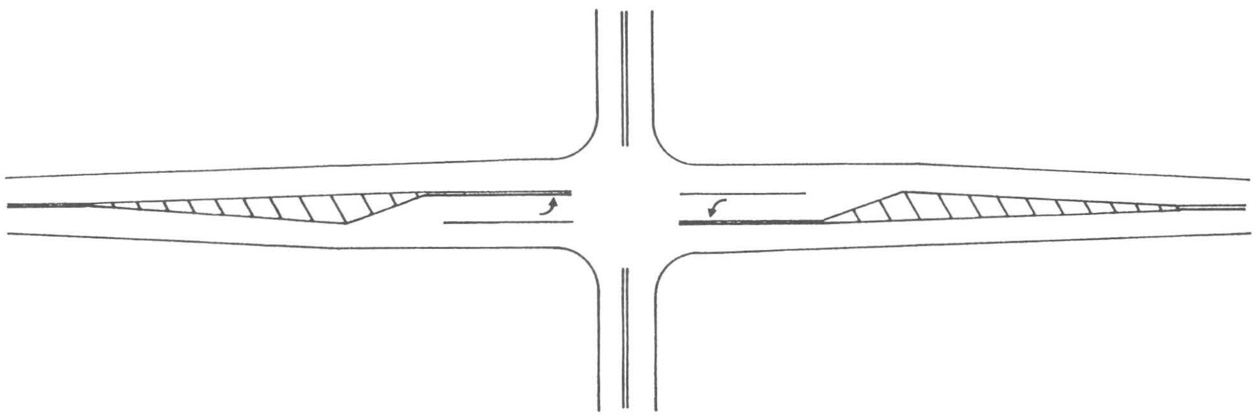


Figure 20 - Plan View of Typical Intersection with Left-Turn Lanes

Intersection turn lanes are not discussed extensively in this Guide because they are less novel to most highway agencies than some of the other passing and turning improvements. Virtually every highway agency has established geometric design and traffic control criteria for channelized intersections. Intersection turn lanes should be designed in accordance with the AASHTO Green Book[7] and highway agency policies. Excellent design guides for intersection turn lanes have recently been published by Neuman[32] in NCHRP Report 279 on Intersection Channelization and by the California Department of Transportation.[33] Similarly, signing and marking of channelized intersections is fully addressed in the MUTCD[6] and in highway agency traffic control policies and handbooks.

Table 10 indicates the traffic operational conditions under which the installation of a left-turn lane is considered warranted to reduce delay at an unsignalized intersection on a two-lane highway.[7,34] Table 10 is entered with the percentage of left turns at an unsignalized intersection and the opposing traffic volume; the entries in the table indicate the flow rate at which the installation of a left-turn lane is considered warranted. The table is based on the assumption that the volumes of left turns and right turns from the side road are less than or equal to the volume of left turns from the main highway.

TABLE 10

OPERATIONAL WARRANTS FOR LEFT-TURN LANES AT INTERSECTIONS ON TWO-LANE HIGHWAYS[7,34]

Opposing Volume (veh/hr)	Advancing Volume to Warrant a Left-Turn Lane (veh/hr): 5 Percent Left Turns	10 Percent Left Turns	20 Percent Left Turns	30 Percent Left Turns
	50-mph Operating Speed			
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
	60-mph Operating Speed			
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

Note: 1 mile = 1.609 km.

Varying estimates of the effectiveness of intersection turn lanes in reducing through vehicle delay have been developed by Ring and Carstens,[35] Lee,[36] and McCoy et al.[37] However, none of these sources considers the downstream effects of intersection delay on traffic operations, which are minimal at low flow rates but can be substantial at high flow rates. The TURNER model, developed by Hoban[31], is the only currently available method for assessing these downstream effects.

There are no widely accepted criteria that indicate when the installation of an intersection turn lane is warranted to improve safety. One agency considers installation of a left-turn lane to be warranted at an unsignalized intersection if four left-turn-related accidents occur in one year or if six occur in two years.[33] Installation of intersection turn lanes may also be justified to improve safety at locations where restricted sight distance on an intersection approach creates a potential for rear-end accidents.

Intersection turn lanes not only improve traffic operations and safety, but may also reduce fuel consumption, vehicle operating costs, and pollutant emissions by reducing vehicle stops and speed changes. These effects can be estimated using procedures presented by Dale.[38]

4.4 SHOULDER BYPASS LANES

Shoulder bypass lanes are a low-cost alternative to intersection turn lanes for reducing delays to through vehicles caused by left-turning vehicles. Where a side road intersects a two-lane highway at a three-leg or T-intersection, a portion of the paved shoulder opposite the intersection may be marked as a lane for through traffic to bypass vehicles making a left turn. While used most commonly at unsignalized intersections, shoulder bypass lanes may also be used at major driveways. Table 10 in Section 4.3 shows that relatively high flow rates are needed to warrant construction of a left-turn lane. Where an adequate paved shoulder is already available, however, installation of a shoulder bypass lane may be as simple as remarking the highway edge line. Thus, provision of a shoulder bypass lane is often much less expensive than construction of a left-turn lane. At other locations, construction of a paved shoulder for use as a bypass lane may be justified either to improve traffic operations or reduce accident experience.

Figure 21 illustrates a typical shoulder bypass lane at a T-intersection on a two-lane highway. If a vehicle is stopped in the through travel lane waiting to make a left turn, following vehicles can use the bypass lane to avoid having to stop themselves. If there are no turning vehicles present, drivers of through vehicles should continue in the through travel lane without entering the bypass lane. Many drivers already use paved shoulders to bypass turning vehicles, although use of shoulders for this maneuver is illegal in many States. The marking of a bypass lane encourages drivers to avoid unnecessary delay and assures that the maneuver is legal by designating a portion of the paved shoulder as part of the traveled way.

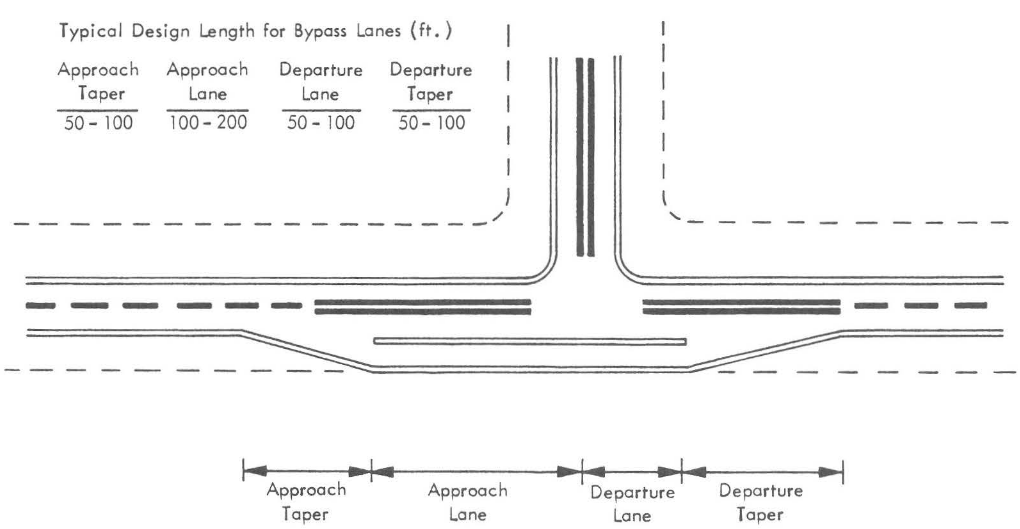


Figure 21 - Plan View of Typical Intersection with Shoulder Bypass Lane

4.4.1 Geometrics and Shoulder Design

The key geometric elements of shoulder bypass lanes at T-intersections are indicated in Figure 21. These elements are lane width, approach taper length, approach lane length, departure lane length, and departure taper length. For design purposes, the approach lane ends and the departure lane begins at the centerline of the intersecting road.

There are no established geometric design criteria for shoulder bypass lanes. A survey of State highway agencies by Buehler[39] found that shoulder bypass lanes have been used in 23 States and operate effectively with a wide variety of geometric designs.

The minimum recommended width for a shoulder bypass lane is 10 ft (3.1 m), with a width of 12 ft (3.7 m) considered desirable. Bypass lanes less than 10 ft (3.1 m) wide should be avoided because they encourage drivers to straddle the boundary between the bypass lane and the through travel lane. If a road carries substantial bicycle traffic, it may be desirable to provide a narrow shoulder outside the bypass lane.

Shoulder bypass lanes should be relatively short; if a bypass lane is too long, drivers may mistake it for a passing lane or feel that they are required to use the bypass lane even when no turning vehicle is present. The total length of a shoulder bypass lanes should typically be 250 to 500 ft (76 to 153 m) depending on traffic volumes and site conditions.

The approach and departure tapers should be relatively short because most vehicles use shoulder bypass lanes at reduced speeds. Typical taper lengths for shoulder bypass lanes are 50 to 100 ft (15 to 31 m). Where a shoulder bypass lane is used, it should be designed to encourage drivers to slow down before entering the bypass lane. It is probably preferable to provide a separate left-turn lane (see Section 4.3) rather than providing a shoulder bypass lane with long tapers. Tapers as long as those used in passing lanes might encourage drivers to enter the bypass lane unnecessarily when no turning vehicles are present.

The length of the approach lane should be the same as the length of the left-turn lane that would be used if that intersection were channelized; i.e., long enough to accommodate the maximum number of left-turning vehicles expected to be stopped at any one time. At the flow rates found on most two-lane highways, an approach lane length of 100 to 200 ft (31 to 61 m) can be used. The departure lane is typically 50 to 100 ft (15 to 31 m) long. Buehler[39] found that the most common operational problem at shoulder bypass lanes -- conflicts caused by a vehicle moving into the lane after its following vehicle has already entered -- could be minimized by making the approach lane <u>shorter</u> and the departure lane slightly <u>longer</u>. A shorter approach lane minimizes the potential for conflicts between vehicles entering the bypass lane.

The paved shoulder must have adequate structural strength for the anticipated traffic loads.

4.4.2 Signing and Marking

No specific signing is needed for shoulder bypass lanes. Pavement markings alone have been found adequate to encourage the intended usage.[39]

The recommended practice for marking shoulder bypass lanes is illustrated in Figure 21. The pavement edge line is tapered from the edge of the through travel lane to the outside of the shoulder, along the outside edge of the shoulder through the approach and departure lanes, and back to the edge of the through travel lane. A 4-in. (10-cm) solid white line should be provided at the edge of the through travel lane throughout the length of the approach lane and the departure lane. The use of a solid line here discourages late entry into the bypass lane and premature return to the through travel lane. The breaks between the pavement edge line and the lane line in the area of the approach and departure taper areas define the locations where drivers are expected to enter and leave the bypass lanes.

4.4.3 Legal Considerations

As noted earlier in this Guide, many States prohibit driving on shoulders. In most States, however, no special legislation would be required to implement shoulder bypass lanes because the pavement markings used designate the bypass lane as part of the traveled way. It should be noted that, even where shoulder bypass lanes are not provided, many drivers already use paved shoulders to bypass left-turning vehicles, although this may be illegal. The State of Delaware has adopted legislation to allow through vehicles to bypass left-turning vehicles on their right both on paved shoulders and at shoulder bypass lanes.[40]

4.4.4 Operational Effectiveness

Shoulder bypass lanes have been shown to be effective in reducing delay to through vehicles at T-intersections, as well as reducing fuel consumption, vehicle operating costs, and pollutant emissions.

No quantitative estimates are available for the delay reduction effectiveness of shoulder bypass lanes. However, a Delaware study found that, where shoulder bypass lanes are provided, 97 percent of the drivers who needed them to avoid delay did in fact use them.[40] Similarly, an Illinois study observed over 90 percent usage of shoulder bypass lanes by drivers who needed them.[39] Even bypass lanes as short as 150 ft (46 m) were used effectively by drivers.

Shoulder bypass lanes were found to be more effective than paved shoulders alone in improving traffic operations. In Delaware, where use of both paved shoulders and shoulder bypass lanes to bypass left-turning vehicles is legal, only 81 percent of drivers used paved shoulders to bypass left-turning vehicles, whereas 97 percent of drivers used shoulder bypass lanes where necessary.

4.4.5 Safety Effectiveness

The accident experience of shoulder bypass lanes compared with that of separate left-turn lanes or compared with that of paved shoulders alone has not been formally evaluated. However, Nebraska has reported a marked decrease in rear-end accidents at shoulder bypass lanes, and other States have reported relatively few accidents occurring at shoulder bypass lane installations.[40]

4.4.6 Summary

Shoulder bypass lanes are a low-cost alternative to separate left-turn lanes at T-intersections on two-lane highways. Drivers of through vehicles use the shoulder lane to bypass vehicles stopped in the through travel lane waiting to make a left turn. Field evaluations have shown that drivers understand the purpose of shoulder bypass lanes are more likely to use a bypass lane than an ordinary paved shoulder to go around a left-turning vehicle. Many highway agencies have used shoulder bypass lanes and none have reported any accident problems associated with the shoulder bypass maneuver.

4.5 TWO-WAY LEFT-TURN LANES

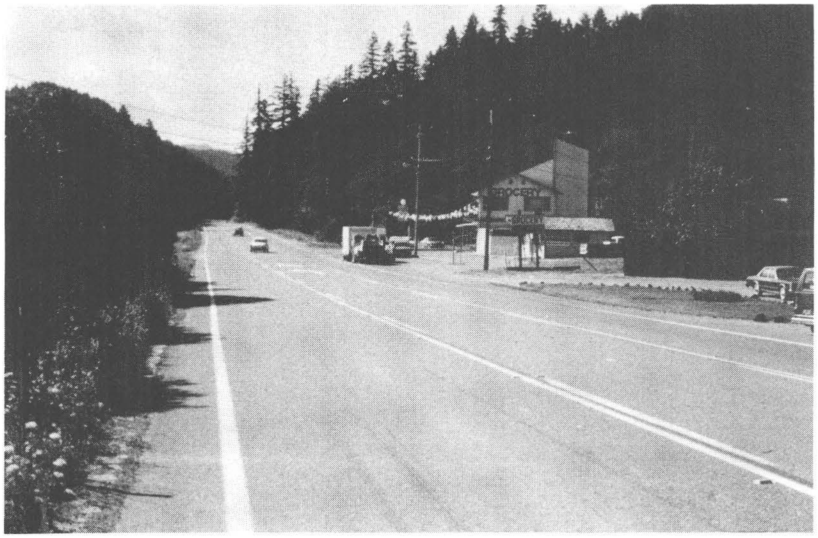
Another type of turning improvement, the two-way left-turn lane (TWLTL), is appropriate where turning demands are present at several adjacent intersections and driveways along a section of highway. A TWLTL is a paved area in the highway median that extends continuously along a highway section and is 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.

TWLTLs have been used for many years on urban and suburban arterial streets with strip commercial development to improve safety and to reduce delays to through vehicles caused by turning traffic. Highway agencies have recently begun to use TWLTLs in rural and urban fringe areas to obtain these same types of operational and safety benefits.

Figure 22 illustrates typical TWLTLs on rural two-lane highways in California and Kansas.

4.5.1 Location and Configuration

TWLTLs are typically employed in rural areas at isolated developments (often roadside businesses), where speeds are higher and traffic flow rates are lower than those of TWLTL sections used in urban and suburban areas. TWLTLs are also employed in small towns and in urban fringe areas, which often have the potentially hazardous combination of dense development, frequent turning maneuvers, and high approach speeds. At a location with a single high volume intersection or driveway, a conventional left-turn lane



California

Kansas

Figure 22 - Typical Two-Way Left-Turn Lane Sites on Two-Lane Highways

is more appropriate than a TWLTL. However, TWLTLs are particularly appropriate at locations where high left-turn volumes are spread over several adjacent driveways and unsignalized intersections and at locations where there is a documented pattern of left-turn accidents spread over several intersections or driveways.

TWLTLs are also well-suited for use in conjunction with passing lanes. A TWLTL can be provided for left-turning traffic at isolated developments while at the same time serving as a buffer between alternating passing lanes. TWLTLs can also be provided through small towns on highways with passing lanes. Thus, extended sections of highway can be constructed with a mixture of passing lanes provided in one direction or the other and TWLTLs or left-turn lanes provided, as appropriate to site-specific geometries and roadside development. In some cases, this could be constructed with a continuous three-lane cross-section.

Care should be taken not to overuse TWLTLs on two-lane highways because passing is prohibited in TWLTL sections. If used in areas with minimal development, TWLTLs can be operationally detrimental by denying drivers the opportunity to pass slow-moving vehicles, without any corresponding safety benefit. When evaluating whether to install a TWLTL, highway agencies should consider the availability of passing opportunities on the adjacent highway section. If the only good passing zone for miles in either direction is replaced by a TWLTL, illegal passing maneuvers are likely, and the potential for conflicts between passing and turning vehicles is increased.

4.5.2 Signing and Marking

Figure 23 illustrates the recommended signing and marking practices for TWLTLs.

TWLTL sections should be signed in accordance with MUTCD requirements using the black-on-white regulatory symbol sign (R3-9b), shown in Figure 23, or a nonsymbol sign with an equivalent message (e.g., CENTER LANE LEFT TURN ONLY). These signs should be post-mounted outside the right shoulder in each direction of travel at the beginning of the TWLTL and should be repeated at appropriate intervals within the TWLTL section. Some highway agencies install the TWLTL signing overhead on span wires for increased visibility at the beginning of the TWLTL section in each direction of travel.

The pavement markings used to designate TWLTLs should comply with the requirements of MUTCD Sections 3B-1 and 3B-2. As shown in MUTCD Figure 3-5a, a TWLTL should be marked on each side with a double yellow line consisting of a solid yellow line on the side adjacent to the travel lane and a broken yellow line on the side adjacent to the TWLTL.

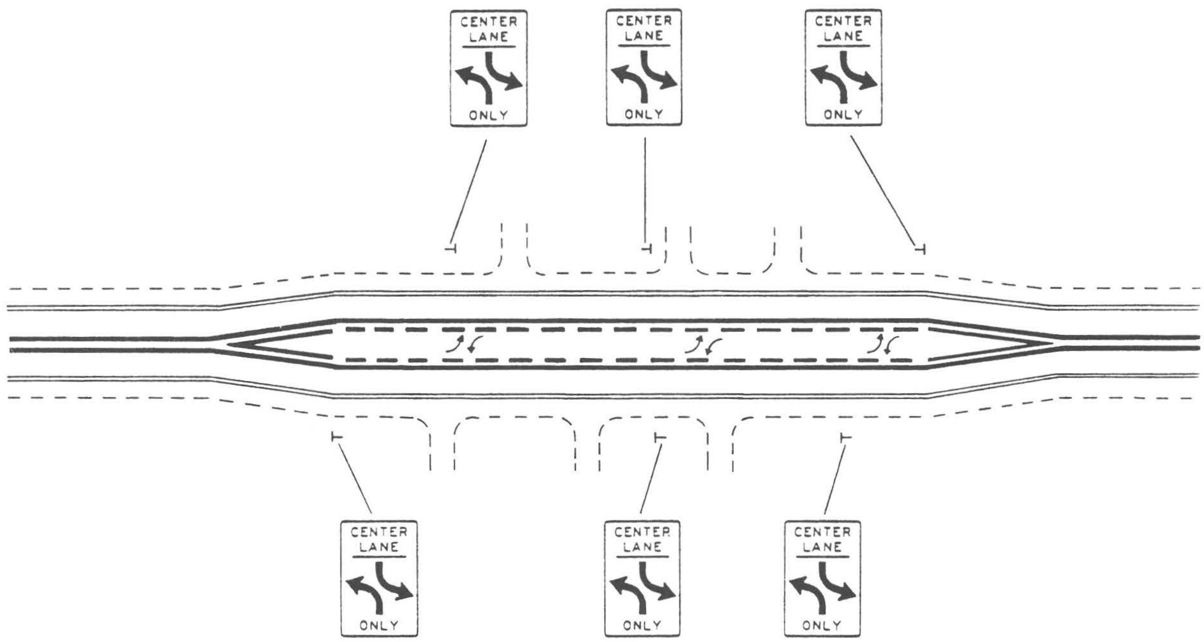


Figure 23 - Recommended Signing and Marking Practice for a Two-Way Left-Turn Lane on a Two-Lane Highway

4.5.3 Operational Effectiveness

Many studies have shown TWLTLs to be effective in reducing delay to through vehicles on multilane highways in urban and suburban areas.[41-43] The operational effectiveness of TWLTLs on two-lane highways in rural and urban fringe areas was evaluated by Harwood and St. John[12] at seven TWLTL sites located in six states. These TWLTL field studies estimated the potential delay to through vehicles that would have been caused by left-turning vehicles if the TWLTL had not been present. Through vehicles that could potentially be delayed by a left-turning vehicle included vehicles in the platoon immediately behind the left-turning vehicle and vehicles that passed the left-turning vehicle while it was stopped waiting for a gap in opposing traffic.

Very little potential delay to through vehicles was observed at rural TWLTL sites, especially those with flow rates below 300 veh/hr in one direction of travel. In fact, during 28 out of the 48 hr that data were collected at the rural TWLTL sites, there was no potential delay at all. The highest level of potential delay observed at a rural TWLTL site was 3.4 sec per left-turn vehicle. Thus, at rural sites with low flow rates the installation of TWLTLs is justified only on a safety basis because very few operational benefits can be expected.[12]

Substantially more potential delay to through vehicles was observed at the higher volume urban fringe sites. The highest level of potential delay observed was 121 sec per left-turn vehicle at a site with strip commercial development and a flow rate over 500 veh/hr in one direction of travel. Under such conditions, TWLTL installation can be readily justified on an operational basis.[12]

Figure 24 presents a model that can be used to predict the amount of delay reduced by a TWLTL on a two-lane highway in a rural or urban fringe area.[12] Figure 17 indicates that the operational benefits of a TWLTL are minimal at low flow rates. There is no delay reduction due to a TWLTL at flow rates below 120 veh/hr in one direction of travel and less than 10 sec of delay reduction per left-turn vehicle at flow rates below 300 veh/hr.

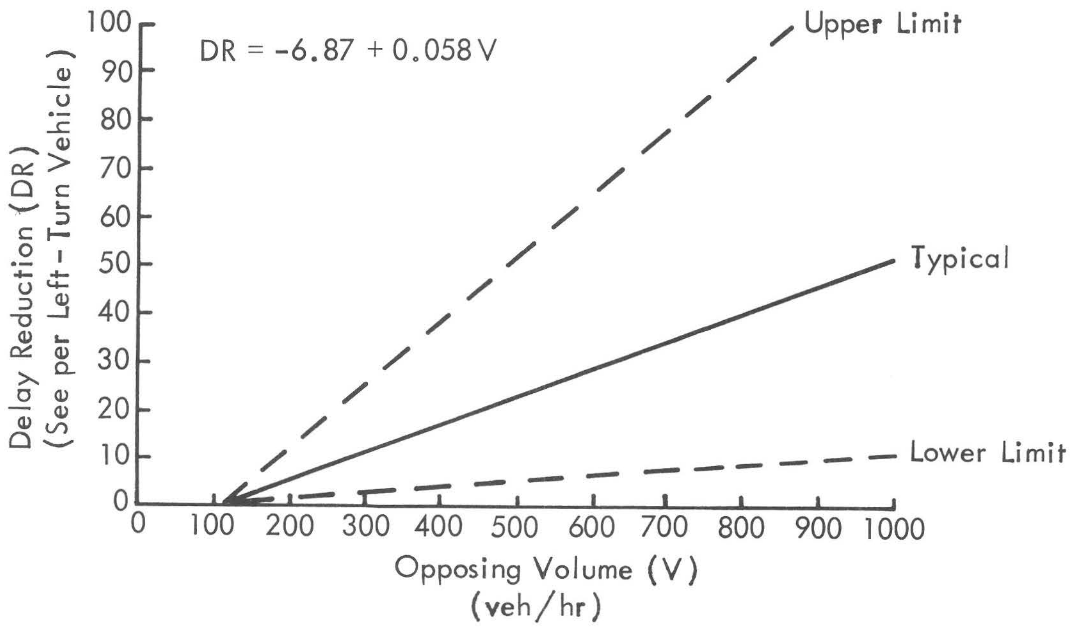


Figure 24 - Predictive Relationship for Delay Reduction Due to Two-Way Left-Turn Lane Installation on a Two-Lane Highway[12]

In the Harwood and St. John[12] study, the percent of vehicles platooned was essentially unchanged from upstream to downstream of TWLTL sites. This finding is another indication of the operational effectiveness of TWLTLs since, without the presence of the TWLTL, platooning would be increased by vehicles stopped in the travel lane waiting to make a left turn.

4.5.4 Safety Effectiveness

TWLTLs are effective in reducing left-turn accident rates. TWLTLs have been found to reduce accident rates by approximately 35 percent, when installed at urban and suburban sites, primarily on multilane highways.[41] Comparable accident reduction effectiveness was found by Harwood and St. John[12] for installation of TWLTLs on two-lane highways in urban fringe areas. In rural areas, the number of accidents at candidate TWLTLs on two-lane highways is small, but TWLTLs can reduce these accidents by up to 85 percent.[12]

When TWLTLs first came into common use, many engineers were concerned about the potential for head-on collisions between left-turn vehicles travelling in opposing directions. However, there is no indication in the literature or in the operating experience of many highway agencies of any problem related to head-on collisions in TWLTLs.[12,39,40]

A field study of traffic conflicts and erratic maneuvers at four rural TWLTL sites on two-lane highways found only one problem that was consistent: illegal passing in the TWLTL was observed by a relatively small fraction (0.4 percent) of vehicles.[12] Since it is evident that some drivers will pass illegally in TWLTLs, a careful evaluation is recommended of any proposed TWLTL installation that would eliminate an existing passing zone.

4.5.5 Summary

Two-way left-turn lanes (TWLTLs) provide a deceleration and storage area in the center of the highway to reduce delay to through vehicles and rear-end accident potential caused by left-turning vehicles at intersections and driveways over a length of highway. The operational benefits of TWLTLs in rural areas with flow rates below 300 veh/hr are minimal, but TWLTLs in rural areas can reduce accident rates up to 85 percent. At flow rates above 300 veh/hr, TWLTLs provide both operational and safety benefits. In urban fringe areas, TWLTLs typically reduce accident rates by 35 percent.

5. PLANNING AND DESIGN FOR ROUTE CONSISTENCY

To maximize the effectiveness of the operational improvements along a section of two-lane highway, the location and design of the improvements should be carefully integrated with the remainder of the highway. The formal planning process described in this section of the Guide provides a tool for highway agencies to manage long-term improvements on an extended section of two-lane highway so that drivers can get the most benefit from the low-cost operational improvements provided. With consistent geometrics and a consistent pattern of signing, drivers will come to expect operational improvements along the highway and to rely upon them to provide passing opportunities and reduce turning delays.

5.1 PLANNING REQUIREMENTS

5.1.1 Length of Highway for Planning Analysis

The provision of a passing or turning improvement on a two-lane highway affects traffic operations for some distance in both directions, and the effectiveness of the improvement depends upon upstream and downstream road characteristics. It is therefore important at the planning stage to consider a longer highway section beyond the length of the improvement itself.

The highway length to be used for analysis depends on the type of improvement being considered. For a single climbing lane or turning lane provided at a problem location, the analysis length could be as short as 2 to 5 mi (3 to 8 km). In many cases, however, the objective of the analysis is to improve overall traffic operations and quality of service over an extended route length. Planning sections of 15 to 50 mi (24 to 80 km) in length should then be used, to ensure that operational improvements are located to maximize effectiveness, minimize construction costs, and provide a consistent overall road standard. The boundaries of planning sections will usually be cities, towns, or major junctions where a substantial proportion of motorists enter or leave a particular route. Points of major change in the character of the highway, such as the beginning and end of freeway sections, are also appropriate as the boundaries of planning sections.

5.1.2 Developing a Long-Term Plan

Planning for operational improvements should begin with a realistic assessment of the long-term design standard expected for a particular highway. In particular, the planner should consider whether major reconstruction or four-laning is likely within 10 to 15 years. If this is the case, operational improvements should be designed either to fit in with long-term plans, or to provide temporary relief from unsatisfactory operating conditions. In either case, special attention is required to ensure that the initial construction fits in with the existing highway design, while retaining flexibility for future development.

A common problem arises where major four-lane construction may be desirable, but is very uncertain in the foreseeable future, possibly due to limited highway funds or uncertain traffic growth projections. In such cases, the use of passing lanes and turning treatments can provide a very effective interim improvement, but designers must decide whether to use a three-lane cross-section or a four-lane divided or undivided cross-section. Such decisions are an important element of the long-term planning analysis.

The planning analysis should take account of traffic volumes, composition, directional splits, projected growth, and platooning, as well as major turning volumes and safety problems. Improvements can then be developed to address long-term needs, and these may be constructed in stages, if desired, to make best use of available highway funds.

5.1.3 Balancing Conflicting Needs

The planning process may need to take account of conflicting improvement needs on a length of highway, especially in areas with substantial roadside development. The provision of turning lanes, for example, may restrict already limited passing opportunities, and the planner may have to decide which is the greater need, or whether both needs can be accommodated. In the same way, the need for roadside parking should be considered in any turnout or passing lane proposal which reduces roadside parking space to provide passing opportunities. In some areas, bicyclists are encouraged to use paved shoulders, and the use of these shoulders by slow vehicles, or the conversion of shoulders into passing and turning lanes, could conflict with the needs of cyclists.

5.2 DESIGN REQUIREMENTS

5.2.1 Design Consistency

The design of localized improvements on a two-lane highway should give the impression of a consistent road standard, rather than a series of disjointed elements. This impression must be maintained even if the improvements are constructed in stages over a number of years. To achieve this consistency, the following components should be considered:

- <u>Geometric standards</u> such as lane width, shoulder width, and design speed should not change abruptly from one road element to the next, if this can be avoided.

- <u>Signs and markings</u> should be provided to a uniform standard along the road. If the improved section is to be well signed, then this standard should be carried through to adjacent road sections. Signs and markings can also assist in keeping the driver informed of changes in the road ahead.

- Driver perceptions of route consistency should be considered as well as technical design criteria. If a passing lane is located on a long tangent following a series of curves, for example, it can give the appearance of a very different road standard, even though this was not the intention of the designer.

5.2.2 Selecting Improvement Locations

The first step in determining the locations for operational improvements should be to determine where there are specific bottleneck sites, such as steep grades or high volume intersections, that may require a site-specific operational improvement, either now or in the future. Site-specific improvements could include climbing lanes at steep grades, intersection turn lanes, or two-way left-turn lanes in areas of roadside development. Then, the need for passing opportunities on the remainder of the highway section should be assessed to determine whether additional passing opportunities are needed. If additional passing opportunities are needed, specific locations for passing lanes or turnouts should be selected and made part of the operational plan. The location and design of each improvement should be evaluated for consistency with the adjacent sections of conventional two-lane highway and the adjacent operational treatments. Even if these improvements are not constructed immediately, the existence of the plan will focus the attention of the highway agency on preserving access control and acquiring needed right-of-way in areas where future operational improvements are planned.

A useful technique in selecting improvement locations is to draw up a construction cost profile for a road, as illustrated in Figure 18. The cost of constructing a particular type of improvement, such as a passing lane or four-lane section, can usually be broken down into:

- Base costs for pavement, surfacing, and signs which do not vary with location; and

- Variable costs for earthwork, drainage, structures, right-of-way acquisition, and other factors which vary along the road length.

In the example of Figure 25, the higher cost "spikes" represent small bridges, while other sections require higher earthwork and drainage costs. A profile of this type is very useful for minimizing the costs for localized improvements, subject to the constraints and guidelines discussed in Section 3.3.1. Other types of location constraints can also be shown in the cost profile.

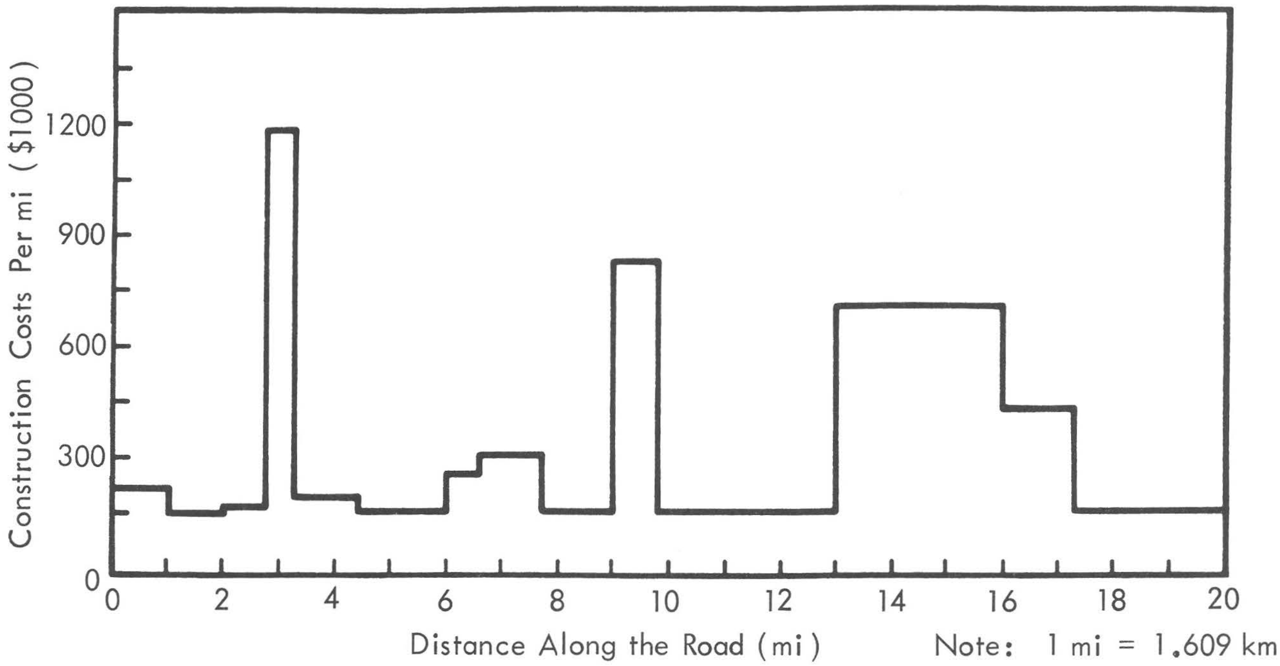


Figure 25 - Construction Cost Profile for Passing Lanes on a Section of Two-Lane Highway[15]

5.2.3 Transition Design

Transitions between improved and existing road elements require particular attention, and should be designed for smooth operation at high travel speeds. Special problems arise when passing and turning improvements are located adjacent to one another. In some cases, a turn lane can be designed as part of a transition at one end of a passing lane, or between two passing lanes. However, the presence of the turn lane can distract attention from the beginning or end of the passing lane, and conflicts may arise between passing and turning vehicles. Good signing and adequate transition tapers are especially important at these locations.

Transitions from two-lane to passing lane, four-lane, or four-lane divided sections should appear to be planned for the motorists' benefit. The addition of signs can improve driver expectancy of these transitions by providing information such as PASSING LANE 5 MILES or DIVIDED HIGHWAY NEXT 5 MILES. Signing of this type helps drivers to get the most benefit out of the operational improvements provided.

Contrasting types of passing improvements, such as passing lanes and turnouts, should not generally be mixed together on the same highway section. However, at some locations, such as at a major change in terrain or traffic volume, it may be desirable to switch from one type of improvement to another. Signing can be very helpful at such locations in changing driver expectations before the first encounter with the new type of operational improvement.

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APPENDIX A: TRAFFIC OPERATIONS AND LEVEL OF SERVICE ON TWO-LANE ROADS

Two-lane road traffic operations and service are discussed briefly in Sections 2 and 3 of the main text of this Guide. This Appendix presents a more detailed review of these topics as background to the discussion of operational improvement alternatives in the main text.

PASSING DEMAND AND SUPPLY

Vehicles traveling along a road usually have a wide range of desired speeds. When other traffic is present, each vehicle can only maintain its desired speed by a continuous process of passing. If passing is restricted for any reason, platoons develop and vehicles experience some degree of delay and constraint. The following sections consider the factors which affect passing demand and supply, and hence the extent of traffic platooning on a given road.

Demand for Passing

The demand for passing in a stream of traffic traveling in one direction is equivalent to the rate at which vehicles would catch up if passing was unrestricted. Wardrop[44] has shown that this can be given by:

$$D = \frac{1}{\sqrt{\pi}} \frac{Q^2 \sigma}{V^2} \tag{4}$$

where,

D = passing demand (passes/hr/mi);
Q = traffic flow in one direction (veh/hr);
V = mean desired speed (mi/hr); and,
σ = standard deviation of desired speeds (mi/hr).

Equation (4) shows that passing demand increases with the square of traffic flow, and with the spread of traffic speeds, as given by the ratio of the standard deviation to the mean (σ/V). Demand also increases as mean speed decreases.

While the strong effect of flow on passing demand is important, the effects of traffic speed deserve particular attention. Demand is lowest on roads where speeds are fairly uniform, e.g., level terrain with similar vehicles and journey types. Passing demand can be much greater when speeds are lower and more varied, due to either terrain or a mixture of traffic characteristics. For example:

- Trucks and recreational vehicles have similar speeds to cars on level road sections, but are generally slower on upgrades.

- Local traffic is usually slower than long-distance traffic.

- Farm and tourist vehicles tend to be slower than others.

- Turning vehicles and very slow agricultural or construction traffic can have a large effect on passing demand.

These effects are illustrated in Figure 26, which shows passing demand for three traffic situations. The middle line represents the traffic conditions used in the development of the 1985 Highway Capacity Manual (HCM) procedures,[2] using a mean desired speed of 60 mi/hr (96 km/hr) and a spread (σ/V) of 12 percent. The lower line is for faster and more uniform traffic (mean = 62 mi/hr (100 km/hr), spread = 10 percent), such as the morning journey to work. The top line has a greater mixture of speeds (mean = 54 mi/hr (87 km/hr), spread = 16 percent). This could occur when long-distance through traffic passes through a farm or recreational area, with many slower vehicles and turning movements. At a flow of 800 veh/hr, demand varies from 578 to 1,062 passes/hr/mi (359 to 600 passes/hr/km) depending on traffic characteristics.

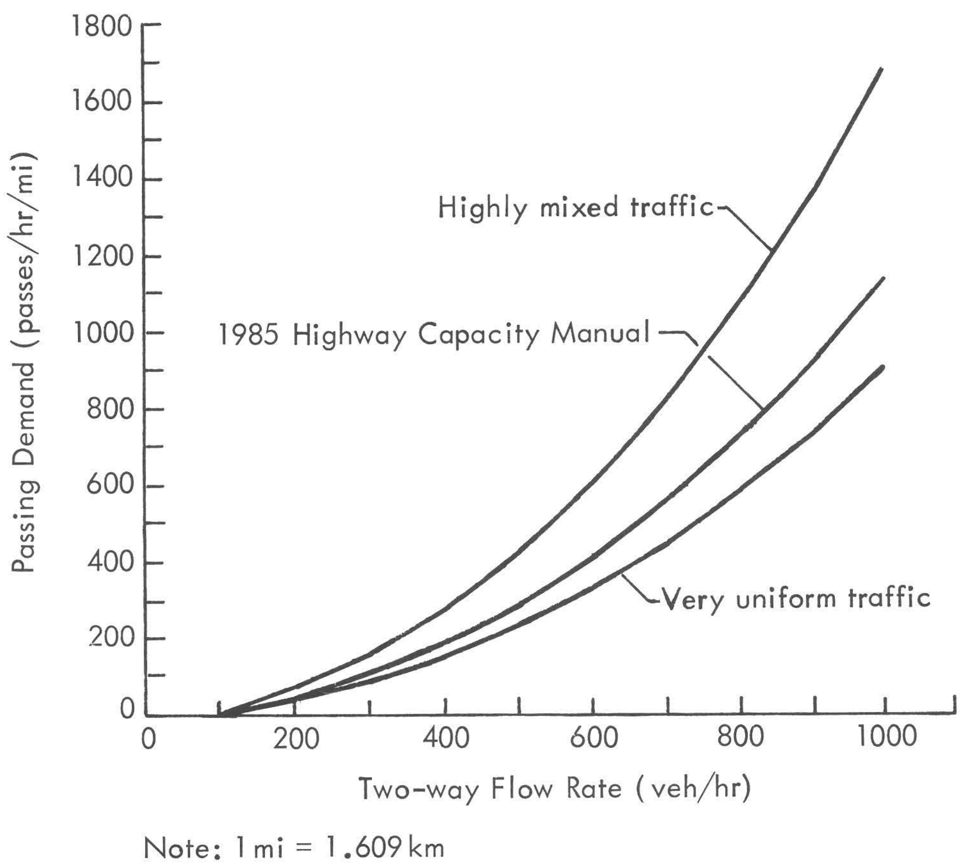


Figure 26 - Passing Demand for Three Traffic Cases

The main point to be noted here is that the degree of traffic problems and need for improvement vary substantially with road and traffic conditions, and not all of these factors are covered in most analysis procedures. In particular, the type of traffic on a road is an important determinant of improvement needs. Some engineering judgment is needed in applying "standard" analysis procedures in cases where traffic characteristics are not standard.

Supply of Passing Opportunities

On a multilane road, passing opportunities are available continuously at all points along the road, and require a gap of only a few seconds in the adjacent traffic stream. The situation is quite different on two-lane roads, where passing is carried out in the lane usually used by oncoming traffic. A passing opportunity in this case requires a gap in the oncoming traffic stream large enough for the passing maneuver itself, plus the distance traveled by the oncoming vehicle, plus a safety margin. In addition, this opportunity can only be used if the sight distance is adequate. Passing opportunities are thus not continuous, and two-lane roads can be considered as a series of passing and no-passing zones.

The supply of passing opportunities is often measured by the percentage of the road length with no-passing zones, defined as:

> ". . . any marked no-passing zone or, as a surrogate, any section of road wherein the passing sight distance is 1,500 ft or less."[2]

The 1,500-ft (460-m) criterion allows for a large proportion of passing demand by vehicles immediately behind a slow vehicle. However, it is not sufficient for some cases, such as:

- Long platoons;
- Timid drivers;
- Low-powered or unfamiliar vehicles (e.g., recreational vehicles);
- Drivers whose vision is obstructed by larger lead vehicles; and
- Small speed differences between lead and following vehicles.

When a passing opportunity is not adequate, the rejecting vehicle remains in position behind the platoon leader, obstructing passing maneuvers at the next passing zone. This can result in timid drivers congregating at the head of platoons, and greatly reducing the achieved passing rate. Because of these factors, the passing opportunity provided by several short 1,500-ft (460-m) sections is not as useful as one long tangent which can be utilized by more cautious drivers.

Measuring passing opportunities by percent no-passing also ignores the need for gaps in the oncoming traffic stream. This can be overcome by the Canadian concept of "assured" or net passing opportunities, which takes account of both passing sight distance and gap-acceptance requirements.[19] It is given by:

$$APO = GAO \times APSD \qquad (5)$$

where,

APO = percent of road providing assured passing opportunities;
GAO = fraction of time with gaps adequate for overtaking ($>$ 25 sec);
APSD = percentage of road length with adequate passing sight distance.

In this case, both APSD and GAO are based on fixed criteria which allow for most, but not all, passing demand.

The passing opportunities provided by a passing lane are similar to those of a multilane road, in that they are available to all vehicles wishing to pass, and are not constrained by oncoming traffic. The increase in passing supply is thus much greater than that indicated by assuming zero percent no-passing, or even 100 percent assured passing opportunity, since there is no problem of gap rejection by timid drivers. Turnouts and shoulder use by slow vehicles also provide some of the benefits of passing lanes.

The preceding discussion shows that the percentage of road length with no-passing zones cannot be used to measure the improvements in passing supply provided by passing lanes, turnouts, or shoulder use. Even on two-lane roads, some roads may appear to provide a high percentage of passing zones, yet have a low passing rate due to long platoons, cautious drivers, and obstructed sight distance on curves. As with speeds, some engineering judgment may be required to determine whether the calculated percent no-passing truly reflects observed passing behavior on a two-lane road.

Platooning

The extent of platooning on a two-lane road reflects the balance between passing demand and supply, and hence the degree of constraint or freedom experienced by drivers. Platooning is easy to measure on the road, and appears to reflect drivers' perceptions of level of service. Platooning also automatically allows for variations in desired speed. On a recreational route, for example, both speeds and platooning may be low, suggesting that drivers are satisfied with the lower speeds. In other cases, high platooning can occur even with high speeds, indicating that drivers are experiencing some constraint despite the high speeds.

Platooning may be measured at a point on the road by the percentage of vehicles following at headways (time gaps) of less than 5 sec. In fact, the 5-sec cutoff is arbitrary; some degree of interaction may be observed at headways of 2.5 to 9 sec, and there is no precise method for defining following vehicles from the point of view of a roadside observer. In practice, the use of a constant cutoff headway works well, as long as a fixed value is used in all studies.

Platoon size is usually defined to include platoon leaders. For convenience, a single vehicle is sometimes defined as a platoon of size 1. In this case, mean platoon size (MPS) is directly related to spot percent platooned (SPP) by:

$$MPS = \frac{100}{100 - SPP} \tag{6}$$

Platooning over a road section may be measured by the percentage of travel time spent delayed, or percent time delay (PTD). This is not easy to measure, but may be estimated from spot platooning data. If spot platooning is recorded at several locations along a road, the average value is an estimate of percent distance delayed (PDD). This tends to underestimate percent time delay, since average speed is slower while delayed. A vehicle which spends 50 percent of its journey distance in platoons, for example, will spend more than 50 percent of its journey time in platoons, since it will travel faster over the other 50 percent of distance. This bias is illustrated in Table 11, which is taken from an Australian computer simulation study. The table shows that spot platooning was 3 to 7 percent lower than percent time delay, and the difference was greatest at 900 veh/hr.

TABLE 11

DIFFERENCES BETWEEN SPOT AND JOURNEY PLATOONING

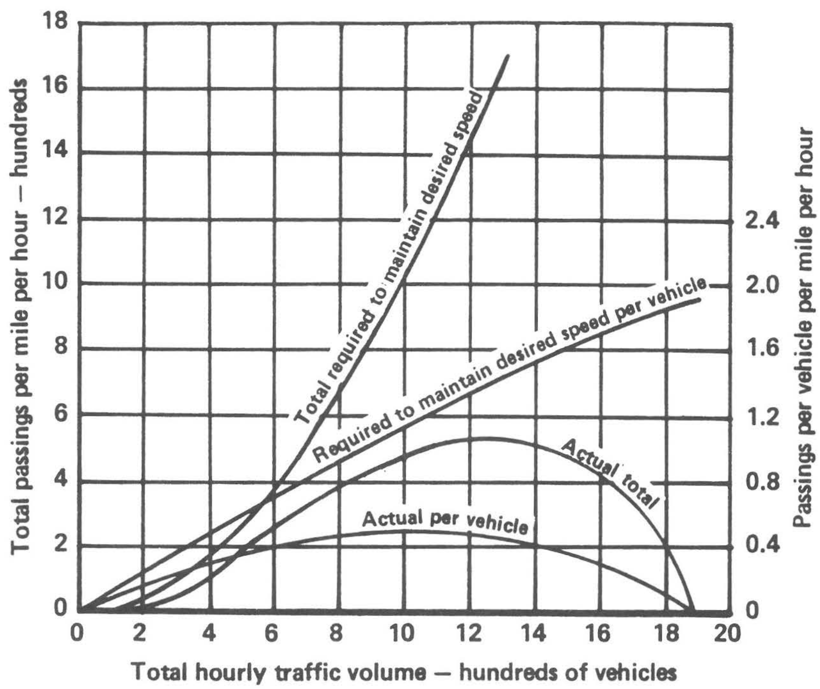
Two-Way Flow Rate (veh/hr)	Spot Platooning (%)	Percent Time Delay	Difference
200	14.5	14.3	---
400	31.7	34.7	3.0
900	63.3	70.3	7.0
1400	75.8	80.5	4.7
1700	80.9	85.2	4.3

Many researchers have used the balance of passing supply and demand, and the extent of traffic platooning, as measures of the quality of traffic operations on a two-lane road. As long ago as 1939, O. K. Normann[45] noted that:

> "It is apparent that a measure other than average vehicle speed along must be used to determine the capacities of highways."

Normann used relative speeds to determine the extent of platooning in the traffic stream. In determining the "practical" capacity of a road (the forerunner of service volume), Normann compared the demand for passing with observed actual passing rates at various flow rates, as shown in Figure 27.[46] He noted that:

> "The fact that the average driver on a two-lane tangent highway should increase the number of passings he makes as the traffic volume goes above 800 veh/hr, but can make no material increase due to traffic density, is a very important consideration in the determination of practical capacities for rural highways."



Note: 1 mi = 1.609 km

Figure 27 - Comparison of Actual Passings and Number Required to Maintain Free Speeds[46]

Harwood and St. John[12] compared Normann's observed passing rate on a two-lane road with recent field observations of passing rates on passing lanes, as shown in Figure 28. The figure shows that the passing rates achieved on a passing lane are far above those on a normal two-lane road. Figure 28 also shows that a passing lane can greatly increase passing opportunities for traffic in the opposing direction of travel, where this is permitted.

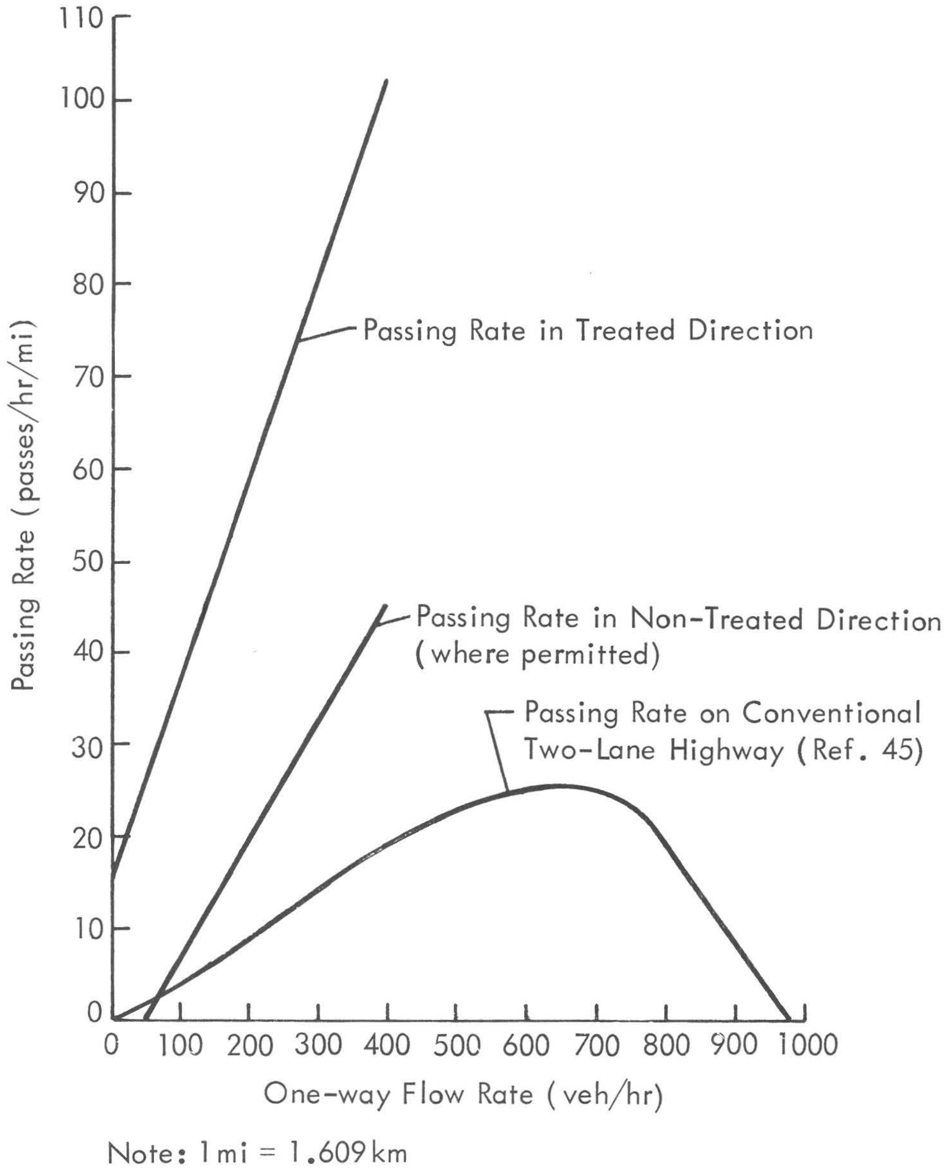


Figure 28 - Passing Rates on Passing Lanes and Two-Lane Roads[12]

Messer[3] used a similar approach to Normann[46] in the selection of level of service criteria for the 1985 Highway Capacity Manual. This is discussed further in the following section. At this stage, there is still no simple procedure for predicting traffic platooning for given conditions of passing demand and supply. Thus while platooning gives a good indication of the balance between demand and supply, the three factors cannot be directly related.

LEVEL OF SERVICE ANALYSIS

This section provides a brief summary of the 1985 Highway Capacity Manual[2] analysis procedures for two-lane highways. Rather than simply repeating the HCM text and tables, the procedures are presented in a different form to give some insight into their effects.

Definition of Levels of Service

The major change in the 1985 two-lane highway chapter is the introduction of entirely new criteria for determining level of service. In the 1965 manual, levels of service were defined in terms of a minimum operating speed and a maximum volume/capacity ratio. The new manual has redefined the six levels using maximum percent time delay over a length of road and minimum average upgrade speed on steep grades. These are illustrated in Table 12.

TABLE 12

LEVEL-OF-SERVICE CRITERIA FOR TWO-LANE HIGHWAYS[2]

Level of Service	Percent Time Delay on General Segments	Average Upgrade Speed (mi/hr) on Specific Grades
A	≤ 30	≥ 55
B	≤ 45	≥ 50
C	≤ 60	≥ 45
D	≤ 75	≥ 40
E	> 75	$\geq$ 25-40
F	100	$<$ 25-40

Note: 1 mi = 1.609 km

The percent time delay criterion is a considerable improvement over the earlier definitions of level of service. Speed criteria present some problems because it is difficult to set a fixed speed standard which is applicable to all roads, and observed speeds do not change greatly as traffic flow and congestion increase. Volume/capacity ratios are also inappropriate for two-lane highways, since these roads are not designed for capacity, and the capacity of a given road is difficult to measure or estimate. Percent time delay, on the other hand, has several advantages:

- It is more sensitive to variations in traffic flow rate.
- It reflects drivers' perceptions of constraint or freedom to maneuver.
- It is not dependent on speed, and so is equally applicable on high-speed and low-speed roads.
- It can be easily measured in the field (using spot platooning).

The cutoff points in percent time delay for each level of service in Table 12 do not produce the same service flow rates as the 1965 HCM. For levels D and E, service flows have increased because the new manual uses a higher capacity for two-lane roads. Maximum service flows for levels A to C, on the other hand, are generally lower than those given by the 1965 manual. This is because the cutoff values of percent time delay have been selected from a consideration of relationships between passing demand and supply and platooning,[3] and do not necessarily match the old level of service definitions. Level-of-service A, for example, was chosen to match the flow at which unrestricted passing demand approximately equals supply on an ideal road. This is about the flow at which platoon building starts to occur.

Table 13 shows the estimated average speed at maximum percent time delay for each level of service, for various road conditions. It should be noted that these values are for conditions of 100 percent passenger cars, 50:50 directional split, and standard speed characteristics. The manual gives no procedures for estimating average speeds for other cases.

Analysis Procedures

Once the level of service criteria of Table 12 are adopted, engineers and planners can use information from a variety of sources to analyze particular cases. Field data, simulation, and other research studies can provide estimates of service volume or percent time delay, and indeed may be necessary to analyze cases not well covered by the HCM. The manual, however, provides a set of standard analysis procedures, which should be satisfactory for a large number of situations.

For general terrain segments, the basic service flow for a given case can be found from Table 14. Table 14 presents maximum service flows for each level of service, for various terrain and sight distance conditions.

TABLE 13

ESTIMATED AVERAGE SPEED AT MAXIMUM SERVICE VOLUME

Level of Service	Level Terrain			Rolling Terrain			Mountainous Terrain		
	Design Speed (mi/hr)			Design Speed (mi/hr)			Design Speed (mi/hr)		
	>60	50	40	>60	50	40	>60	50	40
A	58	54	50	57	53	49	56	52	48
B	55	51	47	54	50	46	54	50	46
C	52	48	44	51	47	43	49	45	41
D	50	46	42	49	45	41	45	41	37
E	45	41	37	40	36	32	35	31	27

Note: 1 mi = 1.609 km

TABLE 14

MAXIMUM SERVICE FLOWS (pc/hr) FOR GENERAL SEGMENTS

Level of Service	Percent No Passing					
	0	20	40	60	80	100
Level Terrain						
A	420	336	252	196	140	112
B	756	672	588	532	476	448
C	1204	1092	1008	952	924	896
D	1792	1736	1680	1652	1624	1596
E	2800	2800	2800	2800	2800	2800
Rolling Terrain						
A	420	280	196	140	112	84
B	728	644	532	476	420	364
C	1176	1092	980	896	840	784
D	1736	1596	1456	1344	1288	1204
E	2716	2632	2576	2548	2520	2520
Mountainous Terrain						
A	392	252	196	112	56	28
B	700	560	448	364	336	280
C	1092	924	784	644	560	448
D	1624	1400	1260	1120	1036	924
E	2548	2436	2352	2296	2240	2184

These were derived for the specified levels of percent time delay using traffic simulation, with some limited support from field data. The HCM presents these as volume/capacity ratios, to provide some link with previous analysis procedures. However, they were originally derived as actual flow rates, and simply converted to v/c ratios using the maximum capacity of 2,800 veh/hr. This is not a true measure of capacity utilization, because many sections have a capacity of less than 2,800 veh/hr. Since v/c must be multiplied by this value anyway, it would be more useful to work directly with service flows, as given in Table 14.

It is sometimes convenient to consider limiting service flows in terms of average daily traffic (ADT), rather than design hour flows. If we assume that the design hour flow is 15 percent of ADT, the comparable ADT flows for ideal conditions are given in Table 15. These would be reduced for narrow roads, uneven directional split and non-car traffic, as discussed in the following paragraphs.

TABLE 15

SELECTED SERVICE FLOWS EXPRESSED AS ADTs (pc/day)

Level of Service	Percent No Passing					
	0	20	40	60	80	100
Level Terrain						
B	5040	4480	3920	3545	3175	2985
C	8025	7280	6720	6345	6160	5975
D	11945	11575	11200	11015	10825	10640
Rolling Terrain						
B	4855	4295	3545	3175	2800	2425
C	7840	7280	6535	5975	5600	5225
D	11575	10640	9705	8960	8585	8025
Mountainous Terrain						
B	4665	3735	2985	2425	2240	1865
C	7280	6160	5225	4295	3735	2985
D	10825	9335	8400	7465	6905	6160

The manual then provides adjustment factors which reduce this flow to account for:

- Directional split of traffic;
- Lane and shoulder width; and
- Effects of heavy vehicles, including trucks, buses, and recreational vehicles.

Each of these adjustment factors includes some changes from the 1965 HCM procedures. The directional split factor recognizes that capacity of a two-lane road is highest when directional split is even, i.e., 50 percent in each direction. A 90:10 split, for example, reduces service flow by 25 percent. Service flows are also reduced on narrow roads; however, the reductions for most widths are not as severe as in the 1965 HCM. The 1985 HCM introduces heavy vehicle factors for recreational vehicles as well as trucks and buses. Many of the values of the heavy vehicle factors have been reduced, e.g., for trucks on level terrain and especially on very steep grades.

The 1985 HCM also introduces new procedures for steep grades. In addition to meeting the service flows based on percent time delay over a road section, the average upgrade speed must not fall below the values set in Table 12. The analysis steps are described in the HCM and need not be repeated here. One new feature is the introduction of an adjustment factor which allows for the speed reductions of cars on steep upgrades.

Factors Not Covered by the HCM Analysis Procedures

The HCM analysis procedures cannot be used to assess roads with improved passing opportunities, such as passing lanes, turnouts, or shoulder driving. There are two reasons for this. First, the improved passing opportunities are much greater than would be indicated by having zero no-passing zones in the HCM analysis, as discussed earlier in this Appendix. Second, the HCM procedures are all based on equilibrium traffic operations, and cannot take account of the downstream benefits of short improvements such as passing lanes. The HCM also does not consider the effects of turning vehicles on the through traffic operations on a two-lane road. These limitations are not surprising, because the percent time delay concept is new, and there have not been many research studies of passing and turning improvements until recent years. Nevertheless, some special procedures are required to take account of these treatments in this Guide.

Procedures for predicting percent time delay for passing and turning improvements are provided in Sections 3 and 4 of the Guide. For passing lanes, these are based on the concept of an "effective length" over which some benefit is experienced. An improved value of percent time delay may be estimated over this effective length, and averaged with estimated values for nearby untreated road sections. Climbing lanes and short four-lane sections also provide a similar degree of improvement, in addition to the removal of delays on steep grades in the case of climbing lanes.

Turnouts and shoulder driving provide some proportion of the benefits of passing lanes. While no detailed measures of improvement effectiveness are available, a good estimate could be obtained by selecting a value of percent time delay between those for passing lanes and untreated two-lane roads. Depending on utilization, the benefit of well-designed turnouts may be 20 to 50 percent of those of passing lanes. The effective length of turnouts is probably less than for passing lanes, although this will depend on downstream passing opportunities and oncoming traffic flow. For continuous shoulder driving, downstream benefits need not be considered, and the improvement should be estimated as a proportion of the reduction of percent time delay obtained within a passing lane.

In some cases it will be desirable to estimate the effects of turn delays on the level of service experienced by through traffic. This can be done by predicting the effects of turns on the percent time delay for non-turning vehicles. The TURNER program developed by Hoban[31] provides a method for doing this. While turn effects can be quite severe to those affected at a specific location, their overall effects for all traffic over a longer road section are often relatively small. It should also be noted that many turning improvements create no-passing zones, which can have a negative effect on level of service. The evaluation of turning improvements should therefore compare the existing road including turn delays with the improved road which has no turn delays but possibly less passing opportunities.

Apart from these analytical techniques, it is always possible to measure traffic platooning directly to assess the quality of traffic operations on a road. Spot platooning should be recorded at a number of locations, since it often fluctuates along a road. The average over several locations gives an estimate of percent time delay, which should be adjusted upwards by a few percent to allow for the bias discussed earlier in this appendix. Spot data are also useful for identifying problem locations and choosing sites for possible improvements. Note that it is quite acceptable for spot platooning to rise for short periods in no-passing zones if the overall percent time delay is satisfactory. In fact, the use of low-cost passing improvements creates a cyclic pattern of platoon building and break-up, which makes best use of short passing opportunities.

ECONOMIC ANALYSIS

Level of service procedures do not take any account of the cost of providing an improvement, or the relative costs and benefits of alternatives. It is often necessary to conduct some form of economic analysis to provide this information, especially when establishing planning and design standards. A simple approach is to calculate cost-effectiveness ratios, typically expressed as a percentage improvement per thousand dollars construction cost. The improvement in this case may be a reduction in percent time delay, travel time, accident rate, or some other measure.

Benefit-cost analysis provides a more rigorous consideration of the economics of a road project. For each alternative being considered, it requires estimates of the costs of construction, maintenance, vehicle operation, travel time, accidents, and other factors. These may be expressed as total costs, or savings provided by a given improvement, and some items may be ignored if changes are expected to be small. A recommended procedure for benefit-cost evaluation is provided by the AASHTO "Manual on User Benefit Analysis of Highway and Bus-Transit Improvements."[5]

Travel times and vehicle operating costs should be evaluated for a range of different hourly traffic volumes, and then summed according to the frequency of each hourly volume range over a year. The hourly flow rates can be typically related to average daily traffic (ADT) by a distribution of the following form:

- 250 hr at 16 percent of ADT;
- 250 hr at 10 percent of ADT;
- 1,000 hr at 8 percent of ADT;
- 2,500 hr at 5 percent of ADT; and
- 4,760 hr at 2 percent of ADT.

Note that the hours add to 8,760 (365 x 24), while the area under this distribution adds to 365 days.

Benefit-cost analysis then requires estimates of the dollar values of savings in travel time and accidents; the analysis cannot consider intangible benefits, such as reductions in traffic platooning. The evaluation is usually carried out over a specified project life, using estimated traffic growth and economic discount rates to adjust future costs and benefits to current values. Many of these parameters are controversial or uncertain, and widely varying results can be obtained by using different values of time or discount rates.

A particular problem is the valuation of small travel time savings, especially for weekend and recreational traffic. These are often considered to have little economic value, since drivers do not usually have a productive use for the small amounts of time saved. However, these time savings reflect large differences in perceived quality of traffic operations, and much of the public pressure for two-lane road improvements comes from weekend drivers. This suggests a public willingness to pay for these improvements.

Economic analysis is nevertheless valuable for comparing different road improvement strategies and making the most of limited road funds.

APPENDIX B: EXAMPLE OF PASSING LANE EFFECTIVENESS ANALYSIS

This Appendix presents an example of an analysis of the operational effectiveness of passing lanes based on the information presented in Section 3.3 of the Guide. The example illustrates the approach that should be used to the selection of appropriate passing lane locations, configurations, lengths, and spacings and to the assessment of the operational effectiveness of particular passing lane alternatives.

STUDY SITE

The study site considered in the example is a two-lane highway in the western United States. This highway is a major recreational and trucking route along a river valley connecting a coastal recreational and industrial area with inland population centers. The location that is the focus of this example is a section of the highway through sparsely developed agricultural and timberland between two small towns, known as Town A and Town B, located 10 mi (10 km) apart. This site provides a good illustration of the effective use of passing lanes because passing lanes have actually been provided in the field at this site.

GEOMETRICS

The existing highway through the study site is a conventional two-lane undivided highway with two 12-ft (3.6-m) lanes and 10-ft (3.01-m) paved shoulders. The road passes through relatively level terrain, with no grades over 3 percent. Parts of the road are winding, with numerous horizontal curves, but none of the curves is sharper than 3 degrees. Passing is prohibited on approximately 50 percent of the highway length due to horizontal and vertical sight distance restrictions.

TRAFFIC VOLUMES

The annual average daily traffic volume (AADT) at this site is approximately 3,000 veh/day. During the peak summer season, traffic volumes are typically 30 percent higher than average and traffic volumes are typically 20 percent lower than average during the winter months. The traffic volume on this highway has less seasonal variation than many recreational routes that can experience traffic volumes in peak months that are twice the average. Only moderate traffic volume growth (2 percent per year) is predicted in the foreseeable future (10 to 20 years).

The design hour flow rate for this highway is 360 veh/hr in both directions of travel, which corresponds to a K-factor (percentage of AADT in the design hour) of 12 percent. The peak hour factor (PHF) for the site is 0.9 indicating that the traffic volume during the peak 15 min of the design hour is about 111 percent of the traffic volume for the design hour as a whole. The typical directional split of traffic volumes at this site during peak periods is 50/50. Based on these data, the service flow rate for this site is 400 veh/hr in both directions, or 200 veh/hr in each direction.

The highway typically carries 35 percent heavy vehicles, consisting of 20 percent trucks and 15 percent recreational vehicles. These heavy vehicles frequently travel at slower speeds than the rest of the traffic stream and this, together with relatively high hourly volumes and the winding alignment, results in long platoons of vehicles during peak travel periods. Field observations found approximately 50 percent of the traffic following in platoons at several spot locations along the section. Thus, according to the evaluation procedures in Chapter 8 of the 1985 Highway Capacity Manual (HCM), this site operates at level-of-service C during peak periods.

Evaluation of these traffic flow data using the procedures of HCM Chapter 8 confirms the field observation that this site operates at level-of-service C during peak periods.

NEED FOR PASSING LANES

There are no formal warrants for passing lanes used in the United States. Thus, the need for passing lanes is assessed through comparison of the actual level of service to the desired level of service. If, in this case, traffic operations at level-of-service B are desired for this highway, passing lanes are needed to achieve this goal. Furthermore, the warrants used in Australia and Canada, presented in Section 3.3.5 of this Guide, both indicate that passing lanes are needed at this site.

The Australian minimum volume guidelines for passing lanes, presented in Table 16, are based on the current AADT, the percentage of slow vehicles, and the level of passing opportunities available over the preceding 2 to 6 mi (3 to 10 km). For this site, the current AADT is 3,000 veh/day and the percentage of slow vehicles is over 20 percent (given that there are 35 percent heavy vehicles in the traffic stream). The Australian guidelines are based on a more stringent definition of the percent of highway length providing passing opportunities than is used in the United States. Although the site in question has passing zones for 50 percent of its length, much of this length is in short passing zones that cannot be used effectively to pass vehicles at highway speeds. It is estimated that approximately 25 percent of the highway length at this site provides good passing opportunities, so the passing opportunities at this site are rated as "Moderate" according to the Australian guidelines. Table 16 indicates that passing lanes are warranted at this site, because the current AADT exceeds 2,670 veh/day.

The installation of passing lanes is also warranted according to the Canadian guidelines presented in Section 3.3.5. The fraction of each hour with gaps available for overtaking (GAO) is estimated from Equation (3) as:

$$GAO = e^{-0.0018626(200)} = 0.69$$

TABLE 16

RECOMMENDED MINIMUM VOLUME GUIDELINES FOR PASSING LANES IN AUSTRALIA[17]

Passing Opportunities Over Preceding 2 to 6 mi		Current-Year Design Volume (AADT)		
		Percentage of Slow Vehicles		
Description	Percent of Length Providing Passing[a/]	5	10	20
Excellent	70-100	5,670	5,000	4,330
Good	30-70	4,530	4,000	3,470
Moderate	10-30	3,330	3,000	2,670
Occasional	5-10	2,270	2,000	1,730
Restricted	0-5	1,530	1,330	1,130
Very Restricted	0	930	800	670

a/ Note that the Australian definition of the percent of length providing passing produces much lower values than criteria used in North America.

Note: 1 mi = 1.609 km

Since 50 percent of the road length has adequate passing sight distance, the net passing opportunities (NPO) on the road are estimated from Equation (2) as:

$$NPO = 0.69 \times 50 = 34.5\%$$

Thus, the estimate net passing opportunities available at this site are 34.5 percent. According to the criteria used in Alberta, passing lanes are needed at this site since the net passing opportunities available on the road falls below 50 percent.

OPTIMUM LENGTH OF PASSING LANES

The optimum lengths of passing lanes for specific levels of traffic flow rate are presented in Table 6 in Section 3.3.5 of this Guide, which is reproduced in Table 17. Table 17 shows that the optimum length of passing lane for a site with a flow rate of 200 veh/hr in one direction of travel is between 0.5 and 0.75 mi (0.8 and 1.2 km). At this site, since the flow rates are below 200 veh/hr during most hours of the year, the best choice for passing lane length is 0.5 mi (0.8 km).

TABLE 17

OPTIMAL DESIGN LENGTHS FOR PASSING LANES

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

Note: 1 mi = 1.609 km

ALTERNATIVES CONSIDERED

Five possible passing lane alternatives for this study site are evaluated in this example. These alternatives are illustrated in Figure 29. Alternatives 1 through 3 each involve one passing lane in each direction of travel on the study site, although the location of the passing lanes are varied between the alternatives. Alternative 4 has two passing lanes in each direction of travel, while Alternative 5 has three passing lanes in each direction of travel. Each of the passing lanes for all five alternatives is 0.5 mi (0.8 km) long, since that length was found to be optimal for the flow rate on the study site.

OPERATIONAL EFFECTIVENESS OF PASSING LANE ALTERNATIVES

The operational effectiveness of each of the passing lane alternatives is evaluated below. Three separate sources -- field observations, the procedures in HCM Chapter 8, and Table 4 in Section 3.3.5 in this Guide -- are in agreement that the existing two-lane highway without passing lanes will operate at level-of-service C with percent time delay of approximately 50 percent (roughly corresponding to the percentage of vehicles following in platoons). The operational effectiveness assessment is based on the reduction in percent time delay that is likely to result from each improvement alternative. An operational effectiveness assessment would usually be performed separately for each direction of travel, but, in this case, the passing lane alternatives considered are symmetrical and identical results are obtained from analysis of each direction of travel.

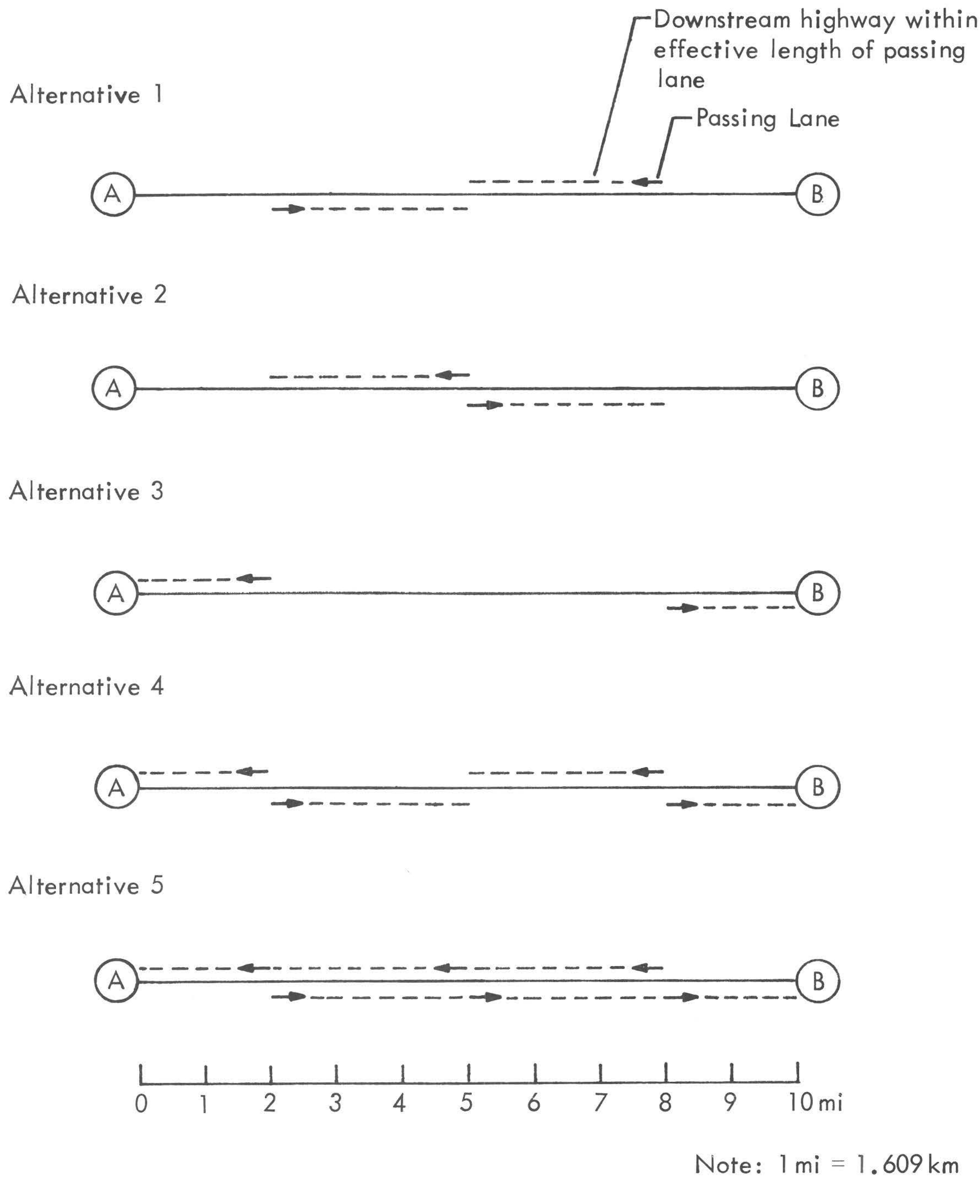


Figure 29 - Passing Lane Alternatives Considered in Operational Effectiveness Example

Alternative 1

Alternative 1 has a single passing lane in each direction of travel, located 2 mi (3 km) from the upstream end of the study site.

The first step in the operational effectiveness assessment of this alternative is to determine the effective length of the passing lane, as discussed in Section 3.3.5 of the Guide. The effective length of a passing lane generally ranges from 3 to 8 mi (5 to 13 km) depending on passing lane length, traffic flow rate and composition, and downstream passing opportunities. Figure 8 in Section 3.3.5 of the Guide presents some examples of the gradual increase in platooning downstream of a passing lane which can be used to estimate effective length. At this study site -- with 0.5-mi (0.8-km) passing lanes, a flow rate of 200 veh/hr in each direction, a relatively high percentage of heavy vehicles in the traffic stream, and fairly restricted downstream passing opportunities -- a relatively short 3-mi (5-km) effective length is expected.

It should be noted that for Alternative 1 the end of the passing lane in each direction of travel is located 7.5 mi (12 km) upstream of the small town at the end of the site. Thus, there are no downstream interferences that would prevent each passing lane from achieving its full effective length.

Table 4 in Section 3.3.5 of the Guide indicates that on a highway with a one-way flow rate of 200 veh/hr, a 0.5-mi (0.8-km) passing lane will reduce the percent time delay from 50 percent to 29 percent over a 3-mi (5-km) effective length. For flow rates, passing lane lengths or effective lengths not shown directly in Table 4, the reduction in percent time delay may be determined by interpolation. The effective length of the passing lane includes both the passing lane itself and a 2.5-mi (4-km) section of normal two-lane highway downstream of the passing lane. Traffic operations on the remaining 7 mi (11 km) of the study site are not improved by the passing lane and, thus, would be expected to experience the same percent time delay as the unimproved highway section, 50 percent. The overall level of service for the study site as a whole can be determined by computation of a weighted average percent time delay. The weighted average of 7 mi (11 km) with 50 percent time delay and 3 mi (5 km) with 29 percent time delay can be estimated as:

$$\frac{7(50) + 3(29)}{10} = 43.7\%$$

Thus, a single passing lane in each direction of travel would be expected to reduce percent time delay on the study site from 50 percent to 43.7 percent. The overall quality of service for the site is improved to level-of-service B.

Alternative 2

Alternative 2 is similar to Alternative 1, with a single passing lane in each direction of travel. For this alternative, the passing lanes in each direction of travel are located 5 mi (8 km) from the upstream end of the site.

The operational effectiveness of Alternative 2 is identical to Alternative 1. The entire 3-mi (5-km) effective length of each passing lane falls within the study site so, as in the case of Alternative 1, the study site also consists of 3 mi (5 km) of highway with improved traffic operations and 7 mi (11 km) of two-lane highway with normal traffic operations. As with Alternative 1, the overall percent time delay for the study site as a whole would be 43.7 percent and the site would operate at level-of-service B.

It should be noted that, while both Alternatives 1 and 2 have the same overall average percent time delay, the zones of platoon buildup and breakdown on the site would follow a different, though equivalent, pattern. Under Alternative 2, traffic platooning would build up to a higher level upstream of the passing lane because the passing lane is located further downstream, but there would be less platoon buildup downstream of the passing lane before the end of the site is reached.

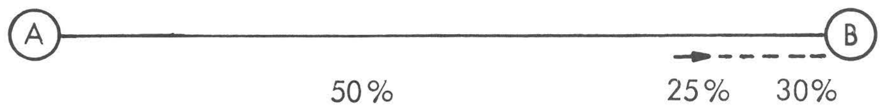
Alternative 3

Alternative 3 is similar to Alternatives 1 and 2 with a single passing lane in each direction of travel. However, for this alternative, the passing lane is located 8 mi (13 km) from the upstream end of the site.

In the configuration of Alternative 3, the end of the site where the highway enters one of the small towns is reached after only 2 mi (3 km) of the 3-mi (5-km) effective length of the passing lanes. The operational effectiveness of the passing lanes will be reduced in this case because the effective length of the passing lane is limited by the presence of the small town.

The reduced effectiveness of the passing lane in this case can be estimated in the following manner. Table 4 in Section 3.3.5 of the Guide shows that the passing lane can be expected to reduce percent time delay to 29 percent over a 3-mi (5-km) effective length. This reduced percent time delay is composed of 25 percent time delay (approximately half of the upstream level) within the 0.5-mi (0.8-km) passing lane and a higher level on the 2.5-mi (4-km) downstream section. A simple distance weighted average shows that the percent time delay in this 2.5-mi (4-km) downstream section must be 30 percent in order to have an overall percent time delay of 29 percent for the 3-mi effective length. The downstream portion of the effective length is reduced from 2.5 to 1.5 mi (4 to 2.4 km) due to the presence of the small town. The expected percent time delay in each portion of the study site for Alternative 3 is illustrated below:

Alternative 3



Percent Time Delay for One Direction of Travel

The overall percent time delay on the section can be estimated as the weighted average of 50 percent time delay for 8 mi (13 km) upstream of the passing lane; 25 percent time delay for the 0.5-mi (0.8-km) passing lane; and 41.8 percent time delay for the 1.5-mi (2.4-km) downstream section. Thus, the overall percent time delay is estimated as:

$$\frac{8(50) + 0.5(25) + 1.5\ (30)}{10} = 45.8\%$$

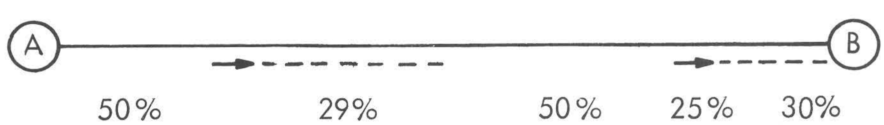
Under Alternative 3, the study site operates at level-of-service C, although the overall percent time delay is only slightly higher than for Alternatives 1 and 2.

Alternative 4

Alternative 4 consists of a combination of Alternatives 1 and 3 with two passing lanes, located 2 mi (3 km) and 8 km (11 km) from the upstream end of the study site.

The operational effectiveness of this configuration can be estimated from data already presented above. The following diagram illustrates the expected percent time delay in each portion of the study site for Alternative 4:

Alternative 4



Percent Time Delay for One Direction of Travel

Under Alternative 4, the study site consists of: 2 mi (3 km) with 50 percent time delay upstream of the first passing lane; 3 mi (5 km) with 29 percent delay for the first passing lane and its effective length downstream; 3 mi (5 km) with 50 percent time delay upstream of the second passing lane; 0.5 mi (0.8 km) with 25 percent time delay within the second passing lane; and 1.5 mi (2.4 km) with 30 percent time delay located downstream of the second passing lane. Thus, the overall percent time delay is estimated as:

$$\frac{2(50) + 3(29) + 3(50) + 0.5(25) + 1.5\ (30)}{10} = 39.5\%$$

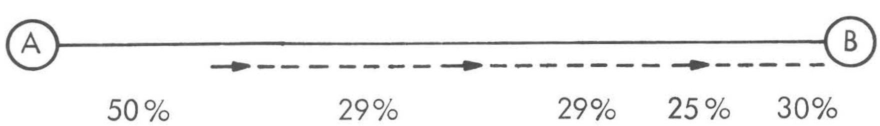
With two passing lanes, traffic platooning is reduced to a point well within level-of-service B in the peak hour.

Alternative 5

The final alternative considered is Alternative 5, which combines Alternatives 1, 2, and 3 and consists of three passing lanes located 2, 5, and 8 mi (3, 8, and 11 km) from the upstream end of the site.

The expected percent time delay in each portion of the study site for Alternative 5 is illustrated below:

Alternative 5



Percent Time Delay for One Direction of Travel

Thus, this alternative results in a total of 2 mi (3 km) with 50 percent time delay; 6 mi (10 km) with 29 percent time delay; 0.5 mi (0.8 km) with 25 percent time delay; and 1.5 mi (2.4 km) with 30 percent time delay. For Alternative 5, the overall percent time delay is estimated as:

$$\frac{2(50) + 6(29) + 0.5(25) + 1.5(30)}{10} = 33.2\%$$

Peak hour traffic operations under this alternative are at level-of-service B, and are approaching the upper boundary of level-of-service A (at 30 percent time delay).

Summary

Table 18 summarizes the expected percent time delay and level of service for the existing two-lane highway and for each passing lane alternative.

TABLE 18

SUMMARY OF OPERATIONAL EFFECTIVENESS ESTIMATES FOR PASSING LANE ALTERNATIVES

	Expected Percent Time Delay	Expected Level of Service
Existing Highway	50	C
Alternative 1	43.7	B
Alternative 2	43.7	B
Alternative 3	45.8	C
Alternative 4	39.5	B
Alternative 5	33.2	B

INTERPRETATION OF RESULTS

The results of the operational analysis indicate that construction of a single passing lane in each direction of travel within the study site provides only a limited improvement in traffic operations. The existing peak hour level-of-service C is either not changed or is reduced to barely within level-of-service B. Alternatives incorporating two or three passing lanes in each direction of travel within the study site are needed to reach traffic operations well within level-of-service B. These alternatives could, thus, improve traffic operations by one level of service; with the anticipated traffic growth of 2 percent per year, installation of passing lanes could prevent any deterioration of traffic operations from the current level of service over a 20-year period. Further improvements in traffice operations could be obtained by installation of more frequent passing lanes or passing lanes with lengths longer than 0.5 mi (0.8 km).

The operational analysis presented above provides a measure of effectiveness for each alternative, but does not indicate which alternative should be chosen. The choice of the most appropriate alternative depends on the design level of service being sought, which may vary from site to site.

If the design level of service for the study site in this example were level-of-service B for current conditions or level-of-service C for a 20-year design period, then Alternatives 4 and 5 evaluated above would be appropriate choices.

The measures of effectiveness for these alternatives are expressed in terms of percent time dealy, which does not lend itself directly to economic analysis since there are no established monetary equivalents to improvements in percent time delay. An example of an economic evauation of passing lane alternatives based on speed measures from a computer simulation model is presented by Harwood and St. John.[13]