NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT **282**

MULTILANE DESIGN ALTERNATIVES FOR IMPROVING SUBURBAN HIGHWAYS

SPORTATION RESEARCH BOARD
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MULTILANE DESIGN ALTERNATIVES FOR IMPROVING SUBURBAN HIGHWAYS

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AREAS OF INTEREST:

FACILITIES DESIGN
TRANSPORTATION SAFETY
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TRANSPORTATION RESEARCH BOARD
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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Staff
Transportation
Research Board

Highway designers, traffic planners, and traffic engineers involved in the reconstruction of suburban highways will be interested in the research findings of this report. The safety records of alternatives multilane design types were investigated through an analysis of accident data from California and Michigan, and operational characteristics were compared using computer simulation. A systematic process is described for designers and planners to follow in the selection of the most appropriate design for a given situation.

Because of the limited funds available for highway improvements, transportation agencies must search for the most cost-effective means to provide the additional highway capacity needed to accommodate the increasing traffic demand within urban fringe areas. In the selection of a capacity improvement, the designer must evaluate safety, operational characteristics, and access to adjacent properties while taking right-of-way and other costs into consideration.

NCHRP Project 2-13 was initiated to investigate and compare the safety, operational, and cost characteristics of selected multilane design alternatives for use in suburban areas. Information was developed on the advantages and disadvantages of each alternative to assist in the selection of the most appropriate design for a given condition. This information will assist transportation agencies in saving time and costs in the decision-making process while assuring maximum benefits to the public. The four primary design types investigated included:

- Three-lane divided including a two-way left-turn lane in the median.
- Four-lane undivided.
- Four-lane divided with one-way left-turn lanes in the median.
- Five-lane divided including a two-way left-turn lane in the median.

This research was directed to two of the most difficult areas typically considered in the design process—the prediction of accidents and the estimation of motorist delay. In both cases, the problem rests with attempting to transfer data based on "average" conditions to a specific location that may have atypical features. The report includes guidance and cautions in the application of the research findings, and the reader should become familiar with this information before attempting to use the summary tables and figures directly. With an understanding of the nature of the data, the findings should provide valuable insights into the design process.

The collection of actual operational data for the various design alternatives was planned initially, even though it was recognized that the available funding would permit only a small data collection effort. As the research progressed, it became clear that the collection of any new field data was not practical. At that point, existing data and a recently developed simulation model were employed to develop the operational data. Although the model had not been extensively validated, it did provide a useful method of comparing alternatives and produced generally logical results.

At the same time that Project 2-13 was being conducted, the Federal Highway Administration was sponsoring a directly related study, entitled "Alleviation of Operational Problems on Two-Lane Highways." This FHWA research focused on relatively low-cost operational improvements, e.g., passing lanes; whereas, the NCHRP study addressed new multilane design alternatives. A preliminary report, "Passing Lanes and Other Operational Improvements on Two-Lane Highways," will be available from the FHWA in the spring of 1986 and the final report will be available in mid-1986, The reports can be obtained from the FHWA Office of Safety and Traffic Operations, Research and Development, Safety Design Division, 6300 Georgetown Pike, McLean, Virginia 22101. This combination of FHWA and NCHRP research represents a comprehensive treatment of improvements to two-lane highways.

CONTENTS

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Mr. Douglas W. Harwood, Principal Traffic Engineer, was the principal investigator for Project 2-13 and the author of this report. Mr. Jerry L. Graham of Graham-Migletz Enterprises, Inc. and Dr. John C. Glennon of John C. Glennon, Chartered, served as consultants to the project. Other project staff members at Midwest Research Institute who contributed to the project include Dr. Jairus D. Flora, Ms. Karin M. Bauer, Ms. Rosemary Moran, Mr. Patrick J. Heenan, and Ms. Debra Hodge. The computer simulation of traffic operations on arterial streets was performed by Dr. John L. Ballard of the University of Nebraska-Lincoln.

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MULTILANE DESIGN ALTERNATIVES FOR IMPROVING SUBURBAN HIGHWAYS

SUMMARY

The objective of this research was to investigate and compare the safety, operational, and cost characteristics of selected multilane design alternatives for suburban highways. Operational characteristics of interest to the study included capacity, level of service, and accessibility. Safety characteristics included the frequency, severity, and type of accidents.

The multilane design alternatives that were the major focus of the research included: three-lane divided including a two-way left-turn lane in the median; four-lane undivided; four-lane divided with a raised-median; and five-lane divided including a two-way left-turn in the median. Other multilane design alternatives that were considered in the study included: five-lane divided with a continuous alternating left-turn lane in the median; six-lane divided with a raised median; and seven-lane divided with a two-way left-turn in the median. A two-lane undivided suburban highway served as the base condition for the study.

A safety data base was assembled for suburban highways on the state highway systems of California and Michigan to quantify the safety performance of multilane design alternatives. Accident rate estimates for multilane design alternatives were obtained as a function of type of development (commercial/residential), driveways per mile, intersections per mile, truck percentage, and presence or absence of a full shoulder. The percentage of accidents involving a fatality or injury and the percentage of accidents susceptible to correction by median treatments (including head-on, rear-end and angle accidents) were also quantified by design alternative and type of development.

Traffic operational comparisons of suburban highway sections with and without two-way left-turn lanes were made using a computer traffic simulation model developed at the University of Nebraska-Lincoln. The results of these comparisons provide quantitative estimates of the delay reduction effectiveness of installing two-way left-turn lanes on two-lane and four-lane arterials. These traffic operational results were extended analytically to obtain estimates of the operational effects of installing a raised median on a four-lane arterial.

The research provides a comparison of the advantages, disadvantages, and relative merits of the various design alternatives for suburban highways, including both their traffic operational and safety performance, as well as the less quantitative aspects such as the impacts on land use and development, abutting businesses, and pedestrians and bicycles.

A stepwise process for selecting an appropriate design alternative for use on a suburban highway is suggested. The process emphasizes the consideration of the traffic operational and safety performance of design alternatives and less quantitative factors such as community and highway agency priorities and constraints. The process considers current and projected future conditions on the facility and emphasizes both the selection of an ultimate design alternative for each facility and possible staged construction options to reach that ultimate design alternative.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

An ever-important challenge facing highway agencies in the United States is the need to alleviate operational problems on suburban arterial highways. The increased accessibility resulting from expansion of the freeway system, the development of regional shopping centers and industrial plants, and the spread of strip commercial development have increased the operational problems on suburban highways, which often were to designed for their current functional uses or traffic volumes. Furthermore, the operational problems common to suburban highways are often accompanied by substantial safety problems, particularly angle and rear-end collisions associated with turning maneuvers.

Congestion and accidents on suburban highways usually result from two major causes. The first is an insufficient number of lanes for through traffic. Two-lane highways, in particular, have the most limited level of service for any given traffic volume and can be major "bottlenecks" in the arterial system. The second cause of congestion and accidents is the interference to through traffic caused by turning vehicles (particularly left-turns). Turning traffic demands both at intersections and at driveways can be major causes of delay and accidents.

The geometric and traffic operational improvements implemented by transportation agencies to alleviate these problems have two basic functional objectives that address the two major causes of operational problems discussed previously. Improvement projects are generally intended (1) to provide additional through capacity and/or (2) to reduce or eliminate the conflicts between through and turning traffic. Projects that involve pavement widening without a median treatment address only the first objective, while projects that involve both pavement widening and median treatments (such as raised medians, left-turn bays, and two-way left-turn lanes), address both objectives.

Because of the limited funds available for highway improvements, transportation agencies must search for the most cost-effective means to provide the additional highway capacity needed to accommodate the increasing traffic demand within urban fringe areas. In the selection of a capacity improvement, the designer must evaluate safety, operational characteristics, and access to adjacent properties while taking right-of-way and other costs into consideration. The existence of developed properties adjacent to the in-place roadway is a major problem in suburban areas because substantial cost increases are incurred if additional right-of-way is needed.

Previous research has not addressed a full range of multilane design alternatives appropriate for a suburban setting. More information is needed on the advantages and disadvantages of each alternative to assist in the selection of the most appropriate design for a given condition. This information will assist transportation agencies in saving time and costs in the decision-making process while assuring maximum benefits to the public.

RESEARCH OBJECTIVES AND SCOPE

The objective of NCHRP Project 2–13 was to investigate and compare the safety, operational, and cost characteristics of selected multilane design alternatives for suburban highways. Operational characteristics of interest to the study included capacity, level of service, and accessibility. Safety characteristics included the frequency, severity, and type of accidents.

Existing suburban two-lane highways were investigated to serve as the base condition for the study. Alternatives to the two-lane base condition that were investigated extensively included:

- Three-lane divided including a two-way left-turn in the median.
- Four-lane undivided.
- Four-lane divided with one-way left-turn lanes in the median.
- Five-lane divided including a two-way left-turn lane in the median.

Three other design alternatives for suburban highways were also investigated, but in less detail.

Each design alternative was investigated under both no shoulder and full shoulder conditions. Of particular concern in the research were highways with traffic volumes over 7,000 vpd and speeds between 35 and 50 mph. These conditions usually indicate that a two-lane highway can no longer handle the demand.

RESEARCH APPROACH

The general approach to the research was to combine findings from the literature with findings of data analyses performed in the study to obtain a comprehensive description of the advantages and disadvantages and potential applicability of particular design alternatives.

A critical review was conducted of the literature related to the design, traffic operations, and safety characteristics of each type of suburban multilane highway. The following factors were considered in the review: median width and type; shoulder presence; access to roadside development; right-of-way requirements; capacity; operational characteristics; and accident experience. Relevant information was obtained from published papers, research reports, and design guides to minimize the data collection effort required in the research.

A set of critical factors that should be considered in making meaningful comparisons of design alternatives was identified. These factors include existing conditions, projected future conditions, constraints on the choice of design alternatives, priorities that favor one particular design alternative over others, and potential benefits and disbenefits of design alternatives.

Some estimates of the safety performance of multilane design alternatives were found in the literature, particularly for two-way left-turn lanes. To provide a complete evaluation of the safety performance of multi-lane design alternatives, accident and operational data on suburban highways were obtained from the records of two state highway agencies. These data were carefully assessed to avoid mistaking an effect of traffic volume or density of development on safety for an effect of the design alternatives themselves.

The available data in the literature on the operational performance of arterial highways do not deal specifically with the effects of median dividers, roadside development, or two-way left-turn lanes. Therefore, a combination of simulation and analytical modeling was employed in the research to assess these effects.

The information from the literature and from the data collection and analysis was combined to assess the relative merits of the design alternatives in terms of operations, safety, and costs. The primary advantages, disadvantages, and limitations of each alternative are presented in this report, together with the best available quantitative estimates of their operational and safety performance. The primary emphasis in the research was on the assessment of the safety performance of design alternatives. Traffic operational performance is also assessed in the report, while construction costs are addressed only indirectly. Construction costs for design alternatives can, in general, be envisioned as proportional to section length and roadway width. Site-specific cost determinations are essential to evaluation of trade-offs between design alternatives, since site-specific cost factors such as utility relocation and right-of-way acquisition may render otherwise desirable design alternatives infeasible.

A selection process for design alternatives on suburban highways is presented. This process is intended to illustrate a general approach to the selection of multilane design alternatives rather than a rigid methodology. Three design examples were developed to illustrate how all of the critical factors would typically be considered by state or local authorities in the selection of a particular design.

CHAPTER TWO

FINDINGS

The research examined a broad range of multilane design alternatives suitable for use on suburban arterial highways. This chapter presents a description of each of these design alternatives together with the research findings that influence the selection of one design alternative or another for a particular traffic situation. The discussion of the selection of design alternatives addresses the general advantages and disadvantages of the alternatives and key considerations in the selection process including operational and safety effectiveness and other, less quantitative, selection criteria.

SUBURBAN ARTERIAL HIGHWAYS

The research scope was limited to geometric design alternatives appropriate for use on suburban arterial highways. In this report, a suburban arterial highway is defined on the basis of a particular set of operational conditions, rather than on the basis of geographic location in a "suburban" community. Any highway that meets the following criteria is considered to be a suburban arterial highway:

- Traffic volume over 7,000 vpd.
- Speeds between 35 and 50 mph.
- Spacing of at least one-quarter mile between signalized intersections.
- Direct driveway access from abutting properties.
- No curb parking.
- Location in or near a populated area.

The first three criteria define a set of suburban operational conditions that are generally less congested than urban conditions, but more congested than rural conditions. The fourth criterion distinguishes suburban arterial highways from freeways, or expressways based on the presence of direct driveway access from abutting properties. The fifth criterion, exclusion of curb parking, recognizes that arterials with curb parking are more typical of urban than suburban conditions; most suburban arterials tend to be developed with residential properties with individual driveway access or with commercial properties that provide off-street parking for their customers. Finally, the sixth criterion recognizes that suburban conditions of the type intended for this study only occur in or near a populated area, although this need not necessarily be a large metropolitan area. A further discussion of the rationale for the criteria that define a suburban arterial, particularly as they relate to the selection of data collection sites for this study, is found in Appendix A of this report.

While the research was intended to address the improvement of suburban arterial highways, most of the general findings, if not the specific quantitative results, are also useful in applying the same design alternatives to other types of urban arterial highways.

DESIGN ALTERNATIVES

The research presented here was intended to evaluate geometric design alternatives for use on suburban highways. A design alternative is defined here by the cross section of the roadway between major intersections. Design alternatives are distinguished from one another primarily by the basic number of through lanes and by the presence or absence of a median treatment to control left turns at driveways and minor intersections. The research evaluated both two-lane undivided highways, as a base condition, and multilane design alternatives that could be used to upgrade an existing two-lane highway.

The research considered eight design alternatives that are widely used on suburban arterial highways. These are:

- Two-lane undivided.
- Three-lane divided including a two-way left-turn in the median.
- Four-lane undivided.
- Four-lane divided with one-way left-turn lanes in the median.
- Five-lane divided including a two-way left-turn lane in the median.
- Five-lane divided including continuous alternating one-way left-turn lanes in the median.
- Six-lane divided with one-way left-turn lanes in the median.
- Seven-lane divided including a two-way left-turn lane in the median.

The general geometric design characteristics of these design alternatives are shown in Figure 1.

The quantitative aspects of operational and safety performance in the research focused on the first five design alternatives listed above. The latter three design alternatives were considered qualitatively on the basis of their similarities to the first five alternatives. It is recognized that other design alternatives that are not considered here, such as six-lane undivided and eight-lane divided arterials, can be used effectively on suburban arterial highways in particular situations. Furthermore, it is also recognized that many geometric variations of the basic design

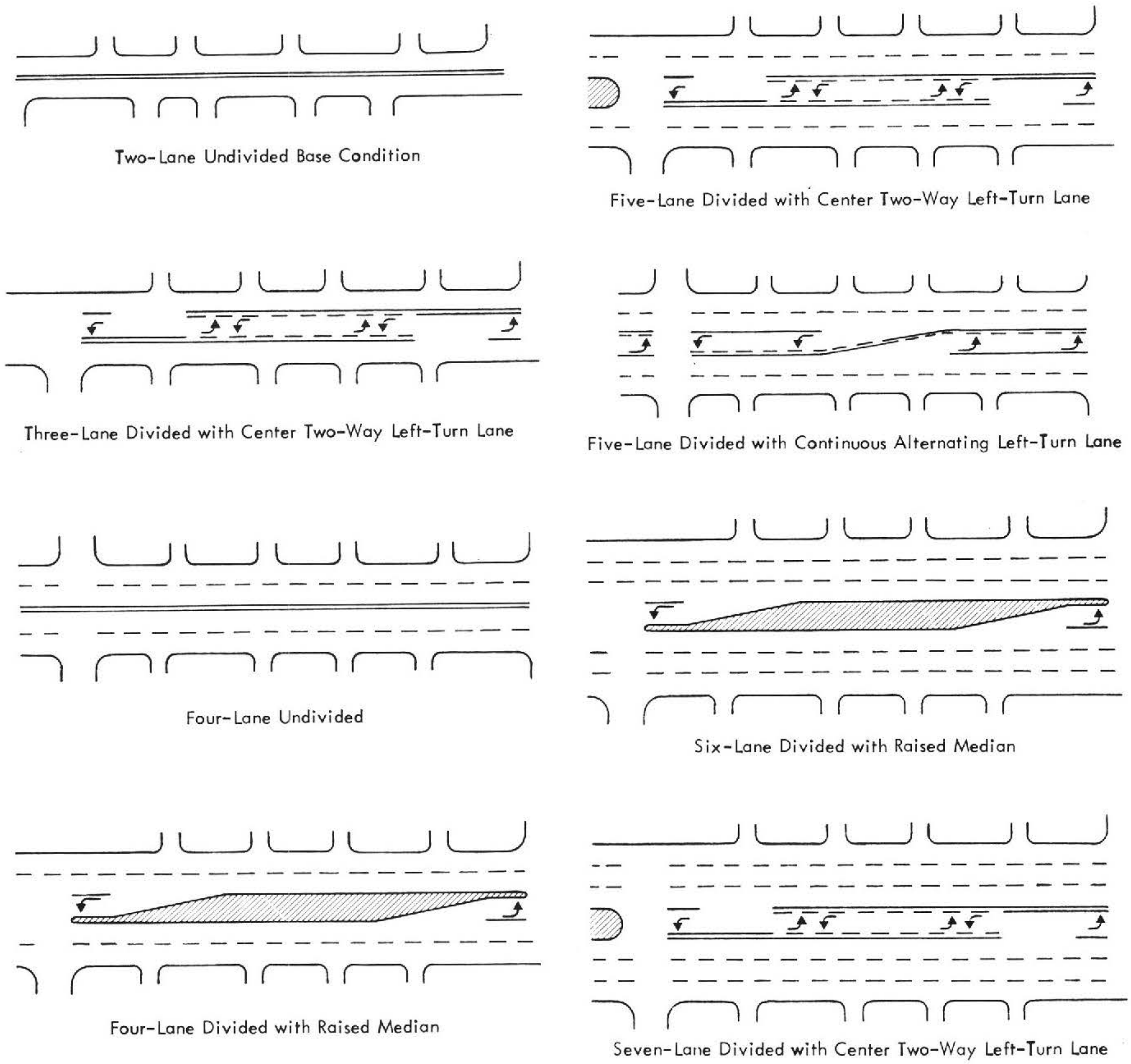

Figure 1. Design alternatives for improving suburban arterial highways.

alternatives considered here are possible. For example, each design alternative can be constructed with a range of lane, median, and shoulder widths. An issue of particular interest in the research was to compare the effectivness of design alternatives with full shoulders (8-ft wide and over) and with no shoulders (e.g., curb-and-gutter sections).

Each basic design alternative is briefly discussed below and illustrated with one or more photographs. The advantages and disadvantages of these alternatives are more fully discussed later in this chapter.

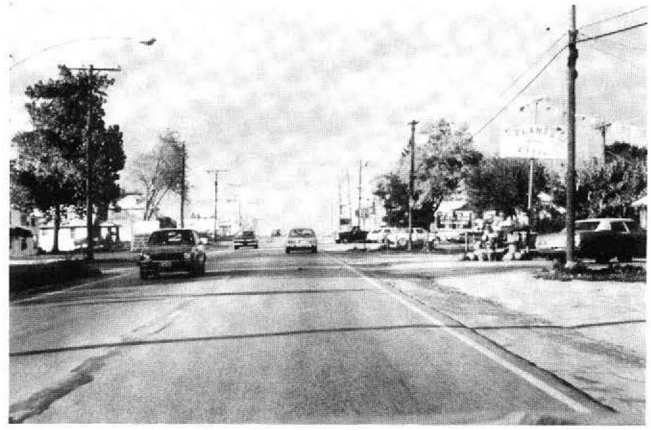

Figure 2. Two-lane undivided highway.

Two-Lane Undivided

A two-lane arterial served as the base condition for the study. This design alternative, shown in Figure 2, consists of one lane of travel in each direction separated by a painted centerline. Two-lane undivided roadways range in width from a minimum of 20 ft (with 10-ft lanes and no shoulder) to 40 ft (with 12-ft lanes and full shoulders). (The lane widths presented in this section are based on the range of lane widths actually found in the field. While there are many existing facilities with 10-ft lanes, the use of 11-ft lanes for upgrading projects on suburban arterial highways is recommended and the use of 12-ft lanes is highly desirable.) While Figure 2 illustrates a two-lane undivided highway with a full shoulder, two-lane undivided highways with no shoulder are also common on suburban highways. Throughout this report, the two-lane undivided design alternative has been abbreviated as the 2U alternative.

Figure 3. Three-lane divided highway with center two-way left-turn lane.

Three-Lane with Two-Way Left-Turn Lane

A three-lane design including a two-way left-turn lane (TWLTL) in the median is a simple improvement from the two-lane undivided alternative, requiring 10 to 16 ft of additional roadway width depending on the width of the center turn-lane. The TWLTL in the median provides a deceleration and storage area for vehicles that desire to turn left at a driveway or an unsignalized intersection so that the turning vehicles do not delay through vehicles as they wait for a gap in opposing traffic to complete their turn. As illustrated in Figure 1, the TWLTL is delineated by a broken and a solid yellow centerline adjacent to the through travel lane on each side of the TWLTL.

Five-lane TWLTL designs (see below) have been used effectively on suburban arterials for many years, but the use of the three-lane TWLTL alternative has become widespread only recently. It serves as a low-cost alternative to designs with multiple through lanes in each direction and is appropriate for highways with relatively low through traffic volumes, with frequent left-turn demands between intersections and where available funds and/or right-of-way are limited. A typical suburban highway with a three-lane TWLTL design is shown in Figure 3. The three-lane TWLTL design alternative has been abbreviated throughout this report as the 3T alternative.

Four-Lane Undivided

The most simple design alternative with multiple lanes for through traffic in each direction of travel is the four-lane undivided highway. This alternative has two through lanes in each direction of travel separated by a double yellow centerline and requires a total roadway width of 40 to 64 ft, depending on lane and shoulder widths. Typical suburban four-lane undivided highways with and without full shoulders are shown in Figures 4 and 5, respectively. The four-lane undivided design alternative has been abbreviated as 4U in this report.

Four-Lane Divided

Another four-lane alternative is the four-lane divided highway with a raised median and one-way left-turn lanes at intersections and/or major driveways. Suburban four-lane divided highways typically have raised medians from 10 to 30 ft in width, with total roadway widths ranging from 48 to 94 ft. Median openings, either with or without one-way left-turn lanes, are provided at signalized intersections and at selected unsignalized intersections and major driveways to facilitate crossing movements and left-turn movements onto and off of the arterial. A typical four-lane divided suburban arterial is shown in Figure 6. The four-lane divided alternative is abbreviated as 4D in this report.

Figure 4. Four-lane undivided highway with full shoulders.

Figure 5. Four-lane undivided highway with no shoulders.

Figure 6. Four-lane divided highway with raised median.

Five-Lane with Two-Way Left-Turn Lane

The five-lane design alternative including a two-way left-turn lane in the median has, in the past 15 years, become the single most common multilane design alternative for upgrading suburban arterials. This design alternative has two through lanes of travel in each direction and a center TWLTL to provide for left-turn maneuvers at driveways and minor intersections. The total roadway width for a five-lane TWLTL section on a suburban highway ranges from 50 to 80 ft, depending on the lane widths and shoulder widths employed. Figures 7 and 8 illustrate a typical suburban highway with a five-lane TWLTL design. The five-lane TWLTL design alternative is referred to as the 5T alternative throughout this report.

Five-Lane with Continuous Alternating Left-Turn Lanes

A final multilane design alternative with two through lanes in each direction is the five-lane with continuous alternating left-turn lanes. This alternative is intended to incorporate the best features of both the four-lane divided and five-lane TWLTL alternatives. This design incorporates one-way left-turn lanes in

Figure 7. Five-lane divided highway with center two-way left-turn lane.

Figure 8. Five-lane divided highway with center two-way left-turn lane.

the median that are continuous or nearly continuous along a section of highway, but alternate from one direction of travel to another. Figure 9 shows a five-lane alternating left-turn lane section incorporating a raised median that limits left turns to specific median openings, while Figure 10 shows a similar design with a flush median where the left-turn channelization is indicated by pavement markings.

The raised median design shown in Figure 9 differs from the four-lane divided alternative in that there is a left-turn lane in one direction of travel or the other nearly continuously along the length of a highway section, and there is little or no length of highway with a full width median. This design has been referred to as a "Z-pattern" because of the shape of the raised median sections between median openings. The flush median design in Figure 10 differs from a five-lane TWLTL section in that the median turn lane, although continuous, is marked for use by only one direction of travel at any given location. The flush median design is less restrictive than the raised median design in that left turns are permitted not just at designated median openings but also at midblock driveway locations where a left-turn lane is provided for one particular direction of travel. The five-lane design with continuous alternating left-turn lanes has been designated the 5C alternative in this report.

Six-Lane Divided

Six-lane divided highways with a raised median and one-way left-turn lanes at intersections and/or major driveways are appropriate for use on higher volume suburban highways. This alternative functions in a manner similar to the four-lane divided design alternative except that it provides three through lanes for travel in each direction. A typical six-lane divided suburban arterial is shown in Figure 11. The six-lane divided design alternative is abbreviated in this report as 6D.

Figure 9. Five-lane divided highway with continuous alternating left-turn lane and raised median.

Seven-Lane with Two-Way Left-Turn Lane

The seven-lane TWLTL design alternative operates in a manner similar to the five-lane TWLTL alternative, except that three through lanes are provided in each direction of travel. Figure 12 shows a typical seven-lane TWLTL design on a suburban highway. The seven-lane TWLTL design alternative is abbreviated as 7T in this report.

Figure 10. Five-lane divided highway with continuous alternating left-turn lane and flush median.

Figure 11. Six-lane divided highway.

Figure 12. Seven-lane divided highway with center two-way left-turn lane.

SELECTION CONSIDERATIONS

The remainder of Chapter Two focuses on the key issue of selecting an appropriate multilane alternative for a particular section of suburban highway. This discussion provides the decision-maker with the best available information on the advantages and disadvantages of the various design alternatives and their relative effectiveness and presents a recommended approach to the selection of multilane design alternatives.

The next section addresses two key cost-effectiveness considerations in the selection of multilane design alternatives: safety performance and traffic operational performance. The subsequent section presents the general advantages and disadvantages of the eight design alternatives. In that section, the safety and operational analysis results developed in this study are compared and contrasted with other results reported in the literature. The final section presents a recommended approach to considering both the general advantages and disadvantages and the operational and safety effectiveness in the selection of a design alternative.

Cost-Effectiveness Considerations

The primary cost-effectiveness considerations in the selection of multilane design alternatives for suburban highways are operational effectiveness, safety effectivensss, and construction cost. This section presents quantitative estimates of operational and safety effectiveness that are appropriate for use in cost-effectivness evaluations. No formal cost-effectiveness procedure for considering trade-offs between these effectiveness measures and construction cost is provided here, although the procedures of the AASHTO *A Manual on User Benefit Analysis of Highway and Bus Transit Improvements—1977* (*1*) could be used for this purpose.

The recommended approach to the selection of design alternatives has intentionally been kept informal and flexible. A rigid, formal cost-effectiveness procedure for the selection of design alternatives has not been provided for three reasons. First, it is our assessment that the formalized evaluation and cost-effectiveness procedures often provided in research reports are generally not used by highway agencies, at least in the form presented. Therefore, it is the fundamental principles behind the procedure that are most important to convey. Second, the formal procedures usually presume a much greater certainty about the safety impact of a particular alternative than is usually warranted. An informed judgment about the relative safety effectiveness of particular design alternatives may often provide the most reliable estimate. Third, the nonquantifiable factors that influence the selection of a design alternative, such as impacts on land use and development, impacts on abutting businesses, and impacts on pedestrians and bicycles, are often just as important as the quantifiable factors. For these reasons, a general approach has been presented to selection of design alternatives rather than a stepwise procedure.

Safety Effectiveness

There are two methods that can be used to assess the safety effectiveness of design alternatives for suburban highways: before-after studies and comparative evaluations.

Before-after studies are used to compare the accident rates of selected highway sections during selected time periods before and after construction of a particular design alternative. A strength of the before-after design is that each site is matched to itself in time, so that traffic volumes, traffic characteristics, and land use are unlikely to change radically between the before and after periods. However, a common weakness of before-after studies is the lack of a control group, consisting of highway sections that were not improved, to assure that a general time trend in accident rates is not mistaken for an effect of the geometric improvements. Despite this weakness, the results of uncontrolled before-after studies must often be relied on because of the lack of other results in the literature.

A comparative study, on the other hand, is intended to compare the accident rates of similar sites with different design alternatives. A strength of this approach is that the accident rate comparison can be made for a common time period. A potential weakness of this approach is that highway sections with different design alternatives may also differ in other factors such as geometrics, traffic volume, traffic characteristics, and/or land use. Because of this potential weakness, statistical methods must be used to account for such differences.

Several before-after studies (without control groups) evaluating multilane design alternatives, particularly three- and five-lane TWLTLs, were found in the literature. While some of the highway sections used in these studies may be more urban than suburban in character (e.g., speed limits of 30 mph or below), it is probable that many of the sites meet the criteria for suburban highways established in this study. Because of the availability of before-after studies in the literature, it was decided to conduct a comparative evaluation in this project and to use the results of the comparative study together with the results from the literature to assess the safety effectiveness of multilane design alternatives.

The safety evaluation performed in this study used safety data obtained for suburban highways in two states. The development of this safety data base, which contains a five-year accident history for 469 miles of suburban highways on the state highway systems of California and Michigan, is documented in Appendix A of this report. The results obtained from the analysis of this data base are summarized below and are documented in more detail in Appendix B. These results are compared and contrasted with other results from the literature in the discussion of the advantages and disadvantages of design alternatives that follows in this chapter.

Accident Rates

A key measure of effectiveness for the design alternatives in the study was the accident rate per million vehicle-miles. An important element of the analysis of accident rates on suburban highways was statistical control for the differences between the design alternatives in geometrics, traffic volume, traffic characteristics, and land use. The effects of nine independent variables, in addition to the design alternative, were considered in the analysis. These independent variables were:

- ADT.
- Truck percentage.
- Type of development.
- Estimated level of left-turn demand.

- Lane width.
- Shoulder width.
- Speed.
- Driveways per mile.
- Unsignalized intersections per mile.

The importance of controlling for the effects of these independent variables can be illustrated by an example. The raw accident data for Michigan show that five-lane TWLTL (5T) sections have higher nonintersection accident rates than four-lane undivided (4U) sections, while the reverse was found to be true when the effects of the other independent variables were controlled for.

The effects on suburban highway accident rates of truck percentage, type of development, shoulder width, driveways per mile and unsignalized intersections per mile were found to be statistically significant, while the effects of ADT, lane width, estimated left-turn demand, and speed were found to be not statistically significant.

The results of the accident rate analysis are summarized in Tables 1, 2, and 3. Table 1 presents the average nonintersection accident rates for suburban highways. The expected accident rate for any particular highway section is determined as the sum of a basic accident rate for each design alternative and type of development (commercial/residential), and adjustment factors for driveway density and truck percentage. Similar data for the unsignalized intersection accident rates of highway sections are given in Table 2 as a function of design alternative, type of development, unsignalized intersections per mile, and truck percentage. Table 3 presents the expected accident rates for nonintersection accidents and unsignalized intersection accidents combined. Signalized intersection accident experience should be considered separately because the geometrics of signalized intersections may vary widely and are not necessarily determined by the design alternative used between signalized intersections.

Table 1. Average accident rates for nonintersection accidents on suburban arterial highways.

BASIC ACCIDENT RATES
(accidents per million vehicle-miles)

Type of Development	Design Alternative 2U	3T	4U	4D	5T
Commercial	2.39	1.56	2.85	2.90	2.69
Residential	1.88	1.64	0.97	1.39	1.39

ADJUSTMENT FACTORS

	Under 30	30-60	Over 60
Driveways per mile	-0.41	-0.03	+0.35
	Under 5%	5-10%	Over 10%
Truck percentage	+0.18	-0.07	-0.33

Note: Accident rates should be decreased by 5% for highway sections with full shoulders and increased by 5% for highway sections with no shoulders.

Table 2. Average accident rates for unsignalized intersection accidents on suburban arterial highways.

BASIC ACCIDENT RATES
(accidents per million vehicle-miles)

Type of Development	Design Alternative 2U	3T	4U	4D	5T
Commercial	2.11	2.43	4.77	4.71	3.11
Residential	2.88	1.91	3.03	2.71	1.85

ADJUSTMENT FACTORS

	Under 5	5-10	Over 10
Intersections per mile	-0.99	+0.28	+1.55
	Under 5%	5-10%	Over 10%
Truck percentage	+0.22	-0.08	-0.38

Table 3. Total accident rates for suburban arterial highways (including nonintersection and unsignalized intersection accidents).

BASIC ACCIDENT RATES
(accidents per million vehicle-miles)

Type of Development	Design Alternative 2U	3T	4U	4D	5T
Commercial	4.50	3.99	7.62	7.61	5.80
Residential	4.76	3.55	4.00	4.10	3.24

ADJUSTMENT FACTORS

	Under 30	30-60	Over 60
Driveways per mile	-0.41	-0.03	+0.35
	Under 5%	5-10%	Over 10%
Intersections per mile	-0.99	+0.28	+1.55
	Under 5%	5-10%	Over 10%
Truck percentage	+0.40	-0.15	-0.71

The accident rates in Tables 1, 2, and 3 should be interpreted as average or expected values. Substantial site-to-site and state-to-state variations in accident rate are not unusual. Decisions based on accident data for the particular site in question will always be preferable to decisions based solely on the averages in Tables 1, 2, and 3. These tables provide a valid method to predict the expected accident rates of suburban highway sections, but users should be cautious in interpreting the adjustment factors as precise estimates of the incremental effects of those variables. For example, the inverse relationship between accident rate and truck percentage could represent, in part, the effect of other factors correlated with truck percentage and cannot necessarily be interpreted as a cause and effect relationship.

Tables 1 and 2 can be used to determine the expected accident rate for a section of suburban highway between signalized intersections. Consider, for example, a suburban two-lane undivided arterial with commercial development, an ADT of 12,500 vpd, 45 driveways per mile, 7.5 intersections per mile, and 7.5

percent trucks. According to Table 1, such a highway section would be expected to experience 2.39 − 0.03 − 0.07 = 2.29 accidents per million vehicle-miles, or 10.4 accidents per mile per year. According to Table 2, the same highway section would experience 2.11 + 0.28 − 0.08 = 2.31 unsignalized intersection accidents per million vehicle-miles, or 10.5 accidents per mile per year. Thus, the highway section would be expected to experience a total accident rate of 4.60 accidents per million vehicle-miles, or 20.9 accidents per mile per year. For convenience, accident frequencies per mile per year based on Tables 1, 2, and 3 have been tabulated in Appendix C.

The tables illustrate that, with minor exceptions, suburban highways with residential development tend to have lower rates than highways with commercial development. Three-lane TWLTL sections have lower accident rates than two-lane undivided sections, while five-lane TWLTL sections have lower accident rates than either four-lane undivided or four-lane divided sections. The average accident rates of four-lane undivided and four-lane divided sections appear to be roughly comparable.

The differences in average accident rate between the design alternatives, as shown in Tables 1, 2, and 3, provides one measure of safety effectiveness that can be used to evaluate a proposed improvement project. For example, since the average total accident rate for a commercially developed 3T section is 11 percent lower than the average accident rate for commercially developed 2U section, 11 percent is a reasonable estimate for the accident reduction effectiveness of a project to improve an existing 2U section to a 3T design. However, both engineering judgment and design examples developed from the safety data base suggest that highly congested sites have higher accident rates than the average and improvement projects at such sites are more effective than average in improving safety. Although this conclusion cannot be quantified or proved statistically from the safety data base, it appears reasonable and it can form the basis for judgments about increased safety effectiveness estimates for some projects on congested highways. A design example presented later in this report illustrates the exercise of engineering judgment in such a case.

Shoulder Width

Each of the design alternatives for suburban highways addressed in this report can be constructed either with full shoulders or with no shoulders (e.g., with a curb-and-gutter section). The safety effectiveness of full shoulders plays an important role in the consideration of design alternatives because, at some sites with right-of-way restrictions, operational benefits can be obtained only by eliminating the shoulder so that a median or a TWLTL can be installed. Elimination of the shoulder could increase accident rate by narrowing the roadside clear area and increasing the likelihood that a vehicle running off the road will strike an object. The key issue is whether or not this potential increase in accident rate is offset by the decrease in accident rate due to the median treatment.

There are no studies in the literature that address the safety effectiveness of shoulders on urban or suburban highways. There has been a great deal of research over the years on the effects of shoulders on rural highways, but the results are inconclusive. A recent state-of-the-art review by Zegeer and Perkins (*2*) evaluated three studies that reported increases in accident rate with wider shoulders, two studies that reported mixed effects or no effect of wider shoulders on accident rate, and six studies that reported decreases in accident rate with wider shoulders. One problem with virtually all of the research to date on the safety effects of shoulders is the lack of experimental control for roadside features, which can produce large disparities in reported accident rates for otherwise similar highways. Most rural highways with wide shoulders tend also to have better roadside designs. One's best judgment is that shoulders do have a positive effect on safety, but this effect may be much smaller than reported in many studies.

The safety data base developed in this study was used to investigate the safety effectiveness of shoulders on suburban highways. It was found that the accident rates in Table 1 should be decreased by 5 percent for sites with full shoulders and increased by 5 percent for sites with no shoulder. This positive relationship between accident rate and the presence of a shoulder is small, but statistically significant. Although it is reasonable to expect that the safety effectiveness of a full shoulder is different for different design alternatives, there was no discernable interaction effect of this type in the data base. It should be noted that like the shoulder studies reported in the literature, this shoulder analysis did not consider the effect of roadside design; this lack of data on roadside design may be less critical because highway sections without roadside obstacles are much less frequent on suburban highways than on rural highways.

The findings of the shoulder width analysis suggest that the full shoulder condition is more desirable than the no shoulder condition for any given alternative. However, where right-of-way restrictions dictate, the elimination of the shoulder to improve traffic operations by upgrading from one design alternative to another appears justified whenever the anticipated accident reduction effectiveness of the project is at least 10 percent.

Accident Severity

The safety analysis also quantified the differences in the severity distribution between design alternatives. Table 4 presents the percentage of accidents involving a fatality or injury by design alternative, type of development (commercial/residential), and accident location (nonintersection/unsignalized intersection). For each column in the table, the severity data have been combined for pairs of design alternatives that do not differ significantly in the proportion of fatal and injury accidents; for example, there is no statistically significant difference between the proportion of fatal and injury accidents for nonintersection accidents on commercial 2U and 4U sections, so a combined

Table 4. Accident severity distribution for suburban arterial highways.

Design Alternative	Percent of Accidents Involving a Fatality or Injury: Nonintersection Accidents, Commercial	Nonintersection Accidents, Residential	Unsignalized Intersection Accidents, Commercial	Unsignalized Intersection Accidents, Residential
2U	38.4	43.6	39.0	32.9
3T	29.9	43.6	32.1	32.9
4U	38.4	38.8	32.1	32.9
4D	33.7	43.6	26.9	45.1
5T	33.7	38.8	32.1	26.6

proportion of fatal and injury accidents (38.4 percent) is used for both.

The accident severity results given in Table 4 should also be considered in the selection of multilane design alternatives for suburban highways. For example, upgrading from a 2U to a 3T design on a commercially developed section not only reduces accident rate (see Tables 1, 2, and 3), but also reduces the percentage of fatal and injury accidents from 38.4 percent to 29.9 percent for nonintersection locations and from 39.0 percent to 32.1 percent at unsignalized intersections.

Table 5. Distribution of accident types susceptible to correction by multilane design alternatives.

Design Alternative	Percent of Accidents Susceptible to Correction[a]: Nonintersection Accidents		Unsignalized Intersection Accidents	
	Commercial	Residential	Commercial	Residential
2U	50.5	44.3	55.9	50.5
3T	45.0	49.4	65.2	56.7
4U	45.8	51.6	65.0	63.5
4D	58.6	43.2	55.3	42.4
5T	50.5	60.0	44.6	55.0

[a] Head-on, rear-end, and angle accidents.

Accident Types

There are three types of accidents that are generally susceptible to correction by installation of multilane design alternatives on suburban highways. These are: head-on accidents, rear-end accidents, and angle accidents. Each of these three types of accidents involves multiple-vehicle collisions that could be ameliorated by installation of a raised median or a TWLTL. To minimize differences in accident classification systems, opposing direction sideswipe accidents have been classified as head-on accidents and same direction sideswipe accidents have been classified as rear-end accidents.

Table 5 presents the proportion of all accidents represented by these accident types that are susceptible to correction for each design alternative and type of development. The recommended use of the data in Table 5 is to judge whether particular sites have a higher than average proportion of correctable accident types. The installation of an improved design alternative at such sites is likely to be more effective than suggested by the differences in average accident rates derived from Tables 1, 2, and 3. However, the percentages of correctable accidents in Table 5 should be used only in a general sense to judge the magnitude of a problem at a particular site. Direct comparisons between design alternatives may be misleading because alternatives with higher volumes of turning maneuvers are more likely to have a higher percentage of correctable accident types, and no data are available to control for the volume of turning maneuvers.

Operational Effectiveness

The operational effectiveness of multilane design alternatives was evaluated in this study for four pairs of alternatives. These are:

- Improving a two-lane undivided (2U) design to a three-lane TWLTL (3T) design.
- Improving a four-lane undivided (4U) design to a five-lane TWLTL (5T) design.
- Improving a four-lane undivided (4U) design to a four-lane divided (4D) design.
- Improving a four-lane divided (4D) design to a five-lane TWLTL (5T) design.

The operational comparison of the 2U and 3T design alternatives and of the 4U and 5T design alternatives was performed using a computer traffic simulation model, known as TWLTL-SIM, developed at the University of Nebraska. The development of these operational estimates is presented in detail in Appendix D. The operational comparison of the 4U and 4D design alternatives and the 4D and 5T design alternatives combined the results of the simulation analysis with analytical estimates of the impact of a median divider on adjacent signalized intersection(s). This analysis is presented in detail in Appendix E.

The operational effectiveness of TWLTLs and raised medians on arterial streets is not addressed directly by either the 1965 or 1985 *Highway Capacity Manual* (HCM) procedures. The first attempts to quantify the delay reduction effectiveness were made recently in papers published by McCoy, Ballard and Wijaya (*3*) and Ballard and McCoy (*4*) of the University of Nebraska. Their work using an earlier version of the TWLTL-SIM computer simulation model has been updated in this report. The TWLTL-SIM model has been validated for a limited set of field data collected in Omaha and Lincoln, Nebraska. The traffic operational predictions obtained from this model are more highly variable than was desired, and inconsistencies in the model predictions were found in a few cases. Nevertheless, the model results presented in this report, while not as quantitatively precise as desired, demonstrate some fundamental findings concerning the operational effectiveness of TWLTLs. Further development of the TWLTL-SIM model to produce a more consistent tool for operational analysis is recommended.

Table 6 presents estimates of the reduction in delay to through vehicles caused by left-turn vehicles for TWLTLs on suburban highways developed using the TWLTL-SIM model. The table illustrates that the delay reduction due to a TWLTL is a function of flow rate and driveway density. The delay reduction effectiveness estimates in the table are in units of veh-sec of delay reduced per left-turn vehicle. For example, if a TWLTL were installed on a 0.5-mile section of a four-lane undivided highway with a flow rate of 650 vph in each direction, a driveway density of 60 driveways per mile and 20 percent of the through volume turning left per mile, the estimated delay reduction in each direction of travel would be:

$$8.7 \text{ veh-sec} \times 0.2(650) \text{ veh/hr/mi} \times 0.5 \text{ mi} = 565.5 \frac{\text{veh-sec}}{\text{hr}}$$

Interpolation in Table 6 to obtain delay estimates for other flow rates or driveway densities is acceptable.

Table 6 shows that the installation of a TWLTL on either a two-lane or a four-lane highway reduces delay for each combination of flow rates and driveway densities evaluated. At the flow rate evaluated for both design alternatives, installation of a TWLTL results in greater delay reduction on a two-lane

Table 6. Delay reduction estimates for installation of TWLTLs on suburban highways.

Flow Rate (vph)[a]	Driveways per Mile	Delay Reduction (veh-sec per left-turn vehicle)[a] 2U vs. 3T	4U vs. 5T
400	30	+19.7	+6.3
	60	+13.1	+5.4
	90	+13.1	+4.8
650	30	-	+10.2
	60	-	+8.7
	90	-	+7.8
900	30	-	+65.4
	60	-	+56.3
	90	-	+47.8
1,100	30	-	+764.2
	60	-	+673.5
	90	-	+531.1

[a] In one direction of travel.

highway than on a four-lane highway. This finding is not unexpected since every following vehicle is delayed by a vehicle waiting to turn left on a two-lane highway, while vehicles may change lanes to avoid a vehicle waiting to turn left on a four-lane highway. The delay reduction estimates for TWLTLs in two-lane highways are based on the assumption that there is no shoulder available for through vehicles to bypass vehicles waiting to turn left.

No delay reduction estimates are presented in Table 6 for the installation of a TWLTL on a two-lane highway at flow rates of 650 vph in each direction and above. The simulation results indicate that above the level of 500 to 600 vph in each direction, even moderate left-turn volumes on a two-lane undivided roadway will result in overcapacitated conditions with unacceptable operational conditions and rapidly increasing delay. Thus, the delay reduction effectiveness for these conditions is large but unquantifiable. On four-lane highways, the simulation model results in Table 6 indicate a very rapid increase in left-turn delay between 900 and 1,100 vph, similar to the results observed for two-lane highways but at a higher volume level. These results suggest that at flow rates of approximately 1,000 vph or higher with even moderate midblock left-turn volumes, four-lane undivided highways become very congested and some type of operational improvement—TWLTL or raised median—is needed.

The results reported in Table 6 indicate that from an operational standpoint, the use of a TWLTL is a highly desirable alternative in a wide variety of design situations. The delay reduction estimates in Table 6 are suitable for use in operational evaluations and cost-effectiveness evaluations to justify the installation of a TWLTL. One drawback to the use of such estimates is the need for left-turn volume data not only at intersections, but also at midblock locations (driveways), to quantify the operational benefits of a TWLTL. Midblock turning volumes are not usually obtained in the design of suburban highway improvements, but may be a desirable addition to the design process because they can be used together with Table 6 to determine delay reduction estimates.

The operational comparison of the 4U and 4D design alternatives and the 4D and 5T design alternatives combined the results obtained with the TWLTL-SIM model with analytical estimates of the other operational effects of a median divider. It was assumed that drivers denied the opportunity to turn left by the presence of a median divider would proceed to the next signalized intersection, make a U-turn during a separate left-turn phase, and return to their desired destination in the opposing direction of travel. The results of this analysis (see Table E-7 in Appendix E) show that the installation of a median divider on a four-lane undivided highway generally increases delay up to a flow rate of approximately 1,000 vph in each direction of travel. Above that flow rate, drivers making midblock left turns are better served by the indirect U-turn routing than by waiting for a gap in opposing traffic to complete a left turn. Because of the variability inherent in the simulation model results, the 1,000 vph flow rate should not be regarded as a precise boundary between conditions appropriate for a four-lane undivided highway and for installation of a raised median. However, the results strongly suggest that as flow rates approach or exceed 1,000 vph, the installation of a raised median becomes more desirable. Furthermore, this finding does not mean that raised medians should not be used at flow rates lower than 1,000 vph, but it does imply that they should be used only when there are other benefits that offset the operational disadvantages of a raised median.

The operational comparison of the four-lane divided and five-lane TWLTL alternatives shows that, similar to the installation of a TWLTL on an undivided highway, the replacement of a median divider with a TWLTL reduces delay over the entire range of flow rates, left-turn demands, and driveway densities studied.

The operational effectiveness of highway improvements involving a change in the basic number of through lanes on a facility, such as upgrading from a 2U to a 5T design, can be estimated as the sum of the TWLTL effectivness (2U to 3T from Table 5) plus the delay reduction due to the addition of a second through lane in each direction. The latter quantity can be best estimated using the procedures of Chapter 11 of the 1985 HCM on Urban and Suburban Arterials to assess the difference in midblock running speeds between a two-lane and a four-lane facility (*31*).

Advantages and Disadvantages of Design Alternatives

This section presents the general advantages and disadvantages of the eight design alternatives identified earlier in this chapter as appropriate for use on suburban highways. The advantages and disadvantages identified here are based on the findings of the research performed in this study, the research reported in the literature, the experience and design practices of highway agencies contacted during the study, and judgments and assessments made by the author. The primary intent of this section is to present the nonquantitative advantages and disadvantages of the design alternatives. However, because many of these advantages and disadvantages are closely related to traffic operations and safety issues, a discussion of traffic operational and safety evaluations in the literature is also included. The traffic operational and safety findings reported in the literature are compared and contrasted to the findings of the analyses performed in this study, which were reported above in the discussion of Cost-Effectiveness Considerations. Thus, the discussion of the advantages and disadvantages presented below constitutes a guideline for the appropriate uses of each multilane design alternative.

Table 7 presents an overview of the general advantages and disadvantages of the eight basic design alternatives. These advantages and disadvantages are addressed in the following in individual discussions of each design alternative.

Table 7. Advantages and disadvantages of design alternatives for suburban highways.

DESIGN ALTERNATIVE	ADVANTAGES	DISADVANTAGES
Two-lane undivided (2U)	1. Least expensive alternative 2. Minimal right-of-way required	1. Minimal capacity for through traffic movement 2. Delay to through vehicles by left-turning vehicles
Three-lane with TWLTL (3T)	1. Reduces delay to through vehicles by left-turning vehicles 2. Reduces frequency of rear-end and angle accidents associated with left-turn maneuvers 3. Provides spatial separation between opposing lanes to reduce head-on accidents 4. Increases operational flexibility	1. May eliminate shoulders 2. No refuge area in median for pedestrians 3. May encourage strip commercial development
Four-lane undivided (4U)	1. Provides additional lanes to increase capacity for through traffic movement 2. Requires less width than 4D and 5T alternatives	1. Delay to through vehicles by left-turning vehicles 2. May generate safety problems associated with rear-end and lane-changing conflicts
Four-lane divided (4D)	1. Provides additional lanes to increase capacity for through traffic movement 2. Reduces rear-end and angle accidents associated with left-turn maneuvers 3. Provides physical separation to reduce head-on accidents 4. Discourages strip commercial development 5. Provides a median refuge area for pedestrians	1. Required pavement and right-of-way width may not be available 2. Increased delay to left-turning vehicles 3. Indirect routing required for large trucks 4. Lack of operational flexibility due to fixed median
Five-lane with TWLTL (5T)	1. Provides additional lanes to increase capacity for through traffic movement 2. Reduces delay to through vehicles by left-turning vehicles 3. Reduces frequency of rear-end and angle accidents associated with left-turn maneuvers 4. Provides spatial separation between opposing lanes to reduce head-on accidents 5. Increases operational flexibility	1. Required pavement and right-of-way width may not be available 2. No refuge area in median for for pedestrians 3. May generate safety problems at closely spaced driveways and intersections 4. May encourage strip commercial development
Five-lane with alternating left-turn lanes (5C) (raised median)	1. Same as 4D alternative 2. More frequent median openings for left turns	1. Same as 4D alternative
Five-lane with alternating left-turn lanes (5C) (flush median)	1. Same as 5T alternative	1. Same as 5T alternative 2. Provides left-turn lane for only one direction at a time
Six-lane divided (6D)	1. Same as 4D alternative 2. Increased turning radius for U-turns	2. Same as 4D alternative
Seven-lane with TWLTL (7T)	1. Same as 5T alternative	2. Same as 5T alternative

Two-Lane Undivided

The two-lane undivided (2U) base condition is the simplest design alternative for a suburban highway, and also the least evaluated, because most studies focus on upgrading two-lane undivided highways to an improved design rather than on the two-lane undivided condition itself. Most surburban highways, except in very rapidly developing areas, were originally constructed as two-lane undivided highways, often in a rural or semirural environment, but many of these two-lane undivided highways require upgrading as suburban development continues, driveway densities rise, and traffic volumes increase.

The major advantages of the 2U design alternative are relatively low construction cost and minimum right-of-way requirements. The disadvantages of the 2U alternative are minimal through traffic capacity, because there is only one through lane in each direction of travel; and delays to through vehicles by vehicles making left turns, because there are no physical restrictions and no deceleration and storage areas for left turns.

Two-lane undivided facilities generally provide acceptable service levels on suburban highways with traffic volumes less than 5,000 to 7,000 vpd. Some two-lane undivided facilities without closely spaced signals or commercial development or both may provide adequate service on highways with traffic volumes up to 15,000 vpd. However, more typically, two-lane undivided facilities above the 5,000 to 7,000 ADT level experience peak-hour congestion and/or increased accidents that suggest the need to upgrade the facility with one of the multilane design alternatives presented in this report. The peak-hour traffic volumes, especially on signalized arterials, may require more than one lane to serve the through traffic volume, while the left-turn traffic generated by commercial development may create unacceptable delays to through motorists. Such congestion can lead to rear end and angle accidents associated with turning maneuvers. For example, one two-lane undivided surburban highway section used as a design example later in this report, with an ADT of 11,700 vpd, experienced an accident rate four times the expected rate for 2U facilities.

The level of traffic service for two-lane undivided highways under suburban conditions cannot be evaluated adequately with Chapter 8 on Two-Lane Highways in the 1985 *Highway Capacity Manual* (HCM). This chapter is intended for application to two-lane highways with uninterrupted flow, and such conditions do not usually exist on suburban arterials. The procedures of HCM Chapter 11 on Arterial Streets are most applicable to suburban 2U facilities. These procedures include consideration of the combined effect of traffic conditions on signalized intersection approaches and in midblock sections between signalized intersections.

There has not been a complete safety evaluation of accident rates and patterns on surburban two-lane highways in previous literature, but accident rate estimates for two-lane highways are presented above and in Appendix B of this report.

Three-Lane with TWLTL

The three-lane TWLTL (3T) design alternative has several important advantages over the two-lane undivided base condition, which can be gained for only a minimal increase in pavement width. In fact, some two-lane undivided facilities with wide lanes and/or full shoulders can be converted to three-lane with TWLTL simply by restriping.

The primary advantages of a three-lane facility is that the TWLTL provides a storage area in the median for left-turning vehicles. The removal of these vehicles from the through traffic lanes minimizes the delay to through vehicles caused by left-turning vehicles and reduces the risk of rear-end and angle accidents associated with left-turn maneuvers. The provision of a TWLTL in the median may encourage drivers to wait for an adequate gap in opposing traffic when waiting to turn left; without the TWLTL, left-turning drivers may become anxious or impatient and select an inadequate gap when they are delaying a queue of following vehicles. The TWLTL also introduces a spatial separation between the lanes of traffic moving in opposite directions which may reduce the risk of head-on accidents. Finally, the presence of a TWLTL provides operational flexibility on a suburban arterial that can increase the freedom of movement for emergency vehicles and simplify the traffic control arranagements when maintenance or construction activity requires a lane to be closed.

The 3T design alternative has some disadvantages. First, the installation of a TWLTL provides a wider pavement for pedestrians to cross without providing a refuge area in the median; however, this disadvantage is of much less concern for a three-lane TWLTL design than for a five-lane TWLTL design (see below). A second disadvantage is that increased pavement and/or right-of-way width may be needed and, in some cases, this width may be obtained by eliminating a full shoulder on a two-lane undivided facility. The sacrifice of a full shoulder may partially offset the accident rate reduction gained from the installation of a TWLTL and eliminate the operational flexibility provided by the use of the shoulder to store disabled vehicles out of the through lanes. On the other hand, where congested conditions on a 2U facility encourage frequent use of the shoulder to bypass vehicles stopped in the through lanes to make a left turn, the 3T design is probably a safer alternative for use of the existing pavement width. Finally, the installation of a TWLTL may encourage strip commercial development. If the established land use plan for a particular facility or corridor is to discourage strip commercial development, the use of a wider design alternative with a raised median should be considered. However, if strip commercial development is not considered undesirable or if it has already occurred, the TWLTL may be the best way to provide access to that development.

Three-lane TWLTL sections have not been evaluated as extensively as five-lane TWLTL sections. The following discussion focuses on findings that are specifically applicable to the three-lane TWLTL. A more general discussion of TWLTL effectiveness will be found in the section of the five-lane TWLTL design later in this chapter.

A recent study of median treatments by Walton et al. (*5*) concluded that the use of the three-lane TWLTL design alternative is most appropriate on highways with traffic volumes in the range from 5,000 to 12,000 vpd. Effective applications of the three-lane TWLTL alternative have been noted in the field at even higher traffic volume levels.

It has long been recognized that TWLTLs are effective in reducing congestion on suburban highways with heavy left-turn demands, but efforts to quantify that effectiveness have been made only within the last three years. Harwood and St. John (*6*) performed a field study of three, three-lane TWLTL sites in urban fringe areas. It was found that the delay reduction effectiveness of a three-lane TWLTL design, in comparison to a two-lane undivided design, was correlated with the left-turn volume, the through traffic volume, the opposing traffic volume, and the percent of traffic platooned in the opposing direction. However, the latter variables were so strongly correlated with each other that a regression relationship using any one of these variables to predict delay was as good as a relationship using several of them. The opposing traffic volume was found to have the strongest relationship and the following regression equation was developed to predict delay reduction:

$$DPLTV = -6.87 + 0.058 \text{ OFLOW}$$

where DPLTV = delay reduction per left-turn vehicle (sec); and OFLOW = opposing flow rate (vph). This regression model explains 32 percent of the variation in the dependent variable (i.e., $R^2 = 0.32$).

McCoy, Ballard and Wijaya (*3*) performed a simulation study in 1982 to predict the reduction in delay and stops by through vehicles due to installation of a TWLTL on a two-lane undivided street. An updated version of the model used in that study, known as TWLTL-SIM, was used to obtain the operational estimates for converting from a 2U to a 3T design that were presented in Table 6 of this report. This table shows that the operational benefits of installing a TWLTL on a 2U facility are substantial and should be considered on many densely developed facilities.

There are no procedures in the 1985 *Highway Capacity Manual* that directly address the effectiveness of a three-lane TWLTL section. However, on a two-lane undivided arterial without signals or with widely spaced signals, it is suggested that the installation of a TWLTL can restore traffic operations approaching the level of service for uninterrupted flow conditions determined from the procedures of Chapter 8.

The safety effectiveness of the three-lane TWLTL design alternative has been evaluated more extensively than the operational effectiveness. The safety analysis presented earlier in this report found that accident rates were 11 percent lower for 3T sections than for 2U sections on suburban arterial highways with commercial development and 25 percent lower for highways with residential development. Thakkar (*7*) reports a reduction in accident rate of 32 percent for all accidents and 31 percent for fatal and injury accidents with installation of a three-lane TWLTL section. One site evaluated by Harwood and St. John (*6*), where a 2U facility was converted to a 3T design, resulted in a 35 percent reduction in accident rate. Thus, the safety effectiveness of converting from the 2U to the 3T design alternative is expected to be in the range from an 11 to 35 percent accident rate reduction.

A case study of a two-lane undivided highway restriped as a three-lane TWLTL section was performed by Nemeth (*8*). A 0.8-mile section of two-lane highway with an ADT of 13,000 to 14,000 vpd was restriped to include a 13-ft wide TWLTL. The restriping reduced the width of the through lanes from 15 to 11.5 ft, and the shoulder width on part of the section was reduced to less than 3 ft. The evaluation of this site found a statistically significant increase in running speed of nearly 3 mph and a 40 to 60 percent reduction in traffic conflicts due to braking and weaving after installation of the TWLTL. It was concluded that the introduction of the TWLTL resulted in a measurable improvement in traffic flow and safety, despite the narrowing of the through lanes and shoulder. The results of a traffic conflict study by McCormick and Wilson (*9*), presented in Table 8, found that the 3T design alternative had a lower conflict rate than the 4D alternative, but a higher conflict rate than the 5T alternative.

Table 8. Comparison of traffic conflict rates for four design alternatives (*9*).

Design Alternative	Adjusted Mean Conflict Rate[a]
Four-lane undivided (4U)	22.1
Three-lane with TWLTL (3T)	17.6
Five-lane with alternating left-turn (5C with flush median)	9.1
Five-lane with TWLTL (5T)	4.8

[a] Conflicts per hour per 300 ft.

Two studies have examined the conversion of an existing 4U section to a 3T design. Nemeth (*8*) found that the installation of a 3T design on a highway with an existing 4U design and an ADT of 16,000 vpd resulted in an increase in delay because of the reduction in the number of through lanes. He concluded that the access function of the roadway was improved at the price of a measurable delay in the traffic movement function. On the other hand, on a facility with a lower traffic volume, Jomini (*10*) found no significant increase in delay, as well as a substantial reduction in accidents, resulting from a 4U to 3T conversion.

The three-lane TWLTL design appears to be an effective alternative to a two-lane undivided highway for locations with substantial midblock left-turn demands. The three-lane TWLTL may also be a useful alternative to an existing four-lane divided highway for sites with low volumes of through traffic and high left-turn volumes.

Four-Lane Undivided

The four-lane undivided (4U) design alternative has the advantage over the 2U and 3T design alternatives of increased capacity for through traffic because two through lanes are provided for travel in each direction. The major disadvantage of the 4U design alternative is that there is no special provision for left turns, so that through vehicles are frequently delayed by left-turn vehicles. Traffic turning both left and right at intersections and driveways can create rear-end conflicts and lane changes to avoid delay that are often symptomatic of safety problems.

Guidelines developed by Klatt (*11*) for the city of Omaha, Nebraska, concluded that the 4U design alternative is best suited for use on streets functionally classified as collectors or minor arterials. The 4U design alternative is most suitable for residential and light commercial areas, without high left-turn demands. The use of the 4U design alternative is not recommended on a highway that is, or could become, a major arterial; either the 4D design alternative or the 5T design alternative or both would be more appropriate for a major arterial. However, the 4U design alternative could be appropriate as a stage to an ultimate 4D or 5T design.

Figure 13. Four-lane undivided highway where right-of-way width restricts widening.

Although it would be desirable to upgrade many 4U arterials to a 4D or 5T design, right-of-way restrictions make this infeasible at many locations. For example, Figure 13 shows commercial development on a 4U arterial with building setbacks of less than 20 ft where the widening of the roadway would eliminate off-street parking and reduce the viability of retail operations at this location. On 4U facilities that cannot be widened, the use of the variety of access control techniques catalogued by Glennon et al. (*12, 13*) to improve traffic operations and safety at individual driveways is recommended. Table 9 presents a summary of these techniques.

The capacity of suburban arterial highways with a four-lane undivided cross section is addressed in Chapter 11 on Urban and Suburban Arterials in the 1985 *Highway Capacity Manual* (HCM). Four-lane undivided arterials with signal spacings greater than 2 miles can also be addressed with the procedures of Chapter 7 on Multilane Highways in the 1985 HCM. However, neither procedure adequately addresses the effects of suburban development and associated midblock turning maneuvers on level of service and capacity. The operational analysis of multilane highway sections performed in this study found the 4U design alternative to be less desirable than the 5T design alternative under virtually all operating conditions and less desirable than the 4D design alternative under high-volume conditions (over 1,000 vph in one direction of travel).

Four-lane undivided highways generally have higher accident rates than other multilane design alternatives. The safety effectiveness estimates for improving a 4U design to a 4D or 5T alternative are addressed below in the discussion of those two design alternatives.

In summary, nearly any highway, where the 4U design alternative is in use, could be improved in traffic operations and/or safety by installation of a TWLTL or a raised median. The use of the 4U design alternative is recommended only (1) for facilities with residential or light commercial development without heavy left-turn demands that are not expected major arterials; (2) for facilities with right-of-way restrictions that make wider design alternatives infeasible; or (3) as a stage toward the construction of a facility with an ultimate 4D or 5T design.

Table 9. Driveway location, design and control techniques to improve driveway operations.

- Regulate minimum spacing of driveways.
- Regulate minimum corner clearance.
- Regulate minimum property line clearance.
- Regulate maximum number of driveways per property frontage.
- Regulate maximum width of driveways.
- Consolidate access for adjacent properties.
- Encourage connections between adjacent properties.
- Deny access for small frontage.
- Require access on collector street (where available) in lieu of additional driveway on highway.
- Channelize driveway to eliminate conflicts between entering and exiting vehicles.
- Use one-way driveways in lieu of two-way driveways.
- Restrict turning maneuvers by signing or channelization.
- Improve corner radii to increase turning speeds.
- Improve vertical geometrics of driveways to increase turning speeds.
- Require driveway paving to increase turning speeds.
- Install right-turn acceleration and deceleration lanes.
- Move sidewalk-driveway crossing further from highway.

Source: Glennon, et al. (Refs. 12 and 13).

Four-Lane Divided

The primary advantages of the four-lane divided (4D) design alternative are increased capacity for through traffic by the provision of two through lanes in each direction of travel and the protection of that through traffic capacity by the elimination of left turns except at selected intersections and major driveways. The installation of a median divider also reduces the likelihood of head-on accidents between vehicles traveling in opposite directions and rear-end and angle accidents associated with left-turn maneuvers. Finally, on suburban highways with adjacent land that is not fully developed, the installation of a median can be used to discourage new strip commercial development and preserve the traffic movement function of the roadway.

A major disadvantage of the 4D design alternative is the increased travel time for vehicles that desire to turn left at locations where median openings are not provided. These vehicles must either make a U-turn at a location where a median opening is provided or use some other indirect route to reach their destination. While residents or retail customers driving passenger cars may be able to make U-turns at signalized intersections, the geometrics are usually not adequate for large trucks to make U-turns, so delivery vehicles must often use indirect routes. For some kinds of retail businesses, installation of a median may discourage customers who desire to turn left to reach the establishment and make midblock locations less desirable (*14*). The installation of a median also reduces the operational flexibility of the roadway to serve special conditions including emergency vehicle movements and work zones with lane closures.

The 4D design alternative is best suited for use on major arterials with high volumes of through traffic and limited access points. The use of the 4D design alternative is recommended only for highways with less than 45 driveways per mile; on highways with more than 45 driveways per mile, the 5T design alternative is probably better suited to serve the existing development. The 4D design alternative is better suited than the 5T design alternative to serve suburban highways with isolated major traffic generators (e.g., shopping centers or office complexes), which have widely spaced, high-volume driveways. Suburban highways with existing strip commercial development are probably better served with a 5T design.

The installation of a raised median is the best available technique to preserve the through traffic movement function on a suburban highway, although this is accomplished at the expense of the land access function. Thus, the 4D design alternative is appropriate when a highway agency makes a conscious choice to favor the traffic movement function. In rapidly developing suburban areas, the choice of the 4D design alternative may be used to influence the course of future development so that the traffic movement function is preserved. Figure 14 shows a suburban highway in a rapidly developing area where the 4D design alternative was selected in conjunction with zoning policies to discourage strip commercial development and encourage isolated major traffic generators whose access to the facility could be carefully controlled.

Where the 4D design alternative is selected for a suburban highway with existing development, careful consideration needs to be given by the design agency to the adequacy of alternative routes to complete left turns that are prevented by the median. This consideration may include the geometric design, signal timing and signal phasing at adjacent signalized intersections, the length of separate left-turn lanes at median openings and signalized intersections, the turning radius required to complete U-turns, and the availability and adequacy of alternate routes including parallel streets, alleys, and service roads.

The operational evaluation performed in this study found that relative to the 4U design alternative, the combined delay to through and left-turn vehicles was reduced by the 4D design alternative only for flow rates above 1,000 vph in one direction of travel. The use of the 4D design alternative for highways with peak flow rates less than 1,000 vph is recommended only where other offsetting benefits such as improved safety, land use control, or preservation of through traffic capacity are expected.

Table 3, presented earlier in this chapter, indicates that the average accident rate for the 4U and 4D alternatives are nearly the same. However, despite this finding, there are two important reasons why the installation of a raised median on some 4U facilities will provide safety benefits. First, Table 4 shows that, on suburban highways with commercial development, the percentage of fatal and injury accidents is lower by 5 percent on 4D facilities than on 4U facilities. The opposite appears to be the case on residential facilities. Second, many existing facilities with a 4U design alternative have accident rates much higher than the average for all 4U facilities. If a suburban 4U facility has an above-average accident rate, and if the proportion of accidents susceptible to correction by the installation of a median (head-on accidents and rear-end and angle accidents associated with left turns) is large enough to account for the increase above the average rate, upgrading to the 4D design alternative can be expected to reduce the accident rate to the average for 4D sections. Design examples illustrating this principle are presented later in this report.

Figure 14. Four-lane divided highway with raised median used to limit strip commercial development.

Five-Lane with TWLTL

The five-lane TWLTL (5T) design alternative has several important advantages. The 5T design alternative reduces delay to through vehicles by providing two lanes for through traffic in each direction of travel and a continuous TWLTL in the highway median to minimize delay to through vehicles by vehicles turning left. The 5T design alternative is effective in reducing the frequency of rear-end and angle accidents associated with left-turn maneuvers and may also reduce head-on accidents through spatial separation of the lanes of traffic moving in opposite directions. Thus, the 5T alternative reduces the same type of accidents as the 4D alternative without the increased delays often resulting from installation of a raised median. Finally, the installation of a TWLTL enhances the operational flexibility of the facility to meet special situations such as movement of emergency vehicles and lane closures due to traffic accidents or work zones. Another aspect of the operational flexibility of the 5T design alternative is that the center TWLTL lends itself well to reversible flow operation; some agencies have operated the center lane as a travel lane in one direction of travel during the morning peak period, in the opposite direction during the evening peak period, and as a TWLTL during off-peak periods. Such operation takes advantage of the temporal distribution of traffic, since the peak periods for through movements do not necessarily occur simultaneously with the peak period for left-turn movements. The safety and operational benefits of TWLTLs are substantial and have made the 5T design the single most widely used multilane design alternative for suburban highways in many jurisdictions.

Despite their many advantages, the five-lane TWLTL design has several disadvantages that should be considered at sites where its use is contemplated. First, the increased pavement and right-of-way width required for a TWLTL may not be available at all locations; the installation of a TWLTL may not be feasible at all at some locations because of the right-of-way restrictions (see Figure 13) and, at other locations, may require elimination of shoulders that may partially offset the accident reduction resulting from the TWLTL.

Second, unlike the 4D design alternative, the 5T alternative provides no refuge area in the highway median for pedestrians. Although pedestrian movements are usually infrequent on sub-

Figure 15. Problems encountered by pedestrian crossing a five-lane divided highway with TWLTL.

Figure 16. Inappropriate wrong-way use of intersection left-turn lane on five-lane divided highway with TWLTL.

urban highways compared to urban and central business district locations, pedestrians do cross the highway, both at intersections and at midblock locations. Figure 15 shows the difficulty that a pedestrian can encounter crossing a 5T facility; having reached the median, the pedestrian is forced to wait in a highly exposed position for an opportunity to cross safely to the far side of the highway.

Third, inappropriate use of the TWLTL by drivers and potential conflicts between turning vehicles may occur at driveways located close to a major intersection (e.g., within 100 ft). Although this problem arises not directly from the TWLTL but from lack of adequate access control policies concerning driveway locations, it nevertheless becomes a consideration in selecting and in marking a TWLTL. The usual method of marking a TWLTL section is to provide one-way left-turn lanes at major intersections and permit the TWLTL to be carried up to or across minor intersections. While this policy appears appropriate, the literature provides no formal evidence either for or against this practice. Figure 16 shows that where a one-way left-turn lane is provided at an intersection on a 5T section, vehicles in the opposing direction may continue to use it as a TWLTL to turn left into driveways near the intersection. Some agencies have reported accident problems related to such movements that could be alleviated by installation of a raised median on the intersection approach.

A final disadvantage of the 5T alternative is that on suburban highways that are not fully developed, the installation of a TWLTL may encourage strip commercial development rather than other types of development that land-use planners may consider more desirable. On existing facilities that already have strip commercial development, however, the 5T alternative may be the design alternative best suited to serve the existing development. However, on an arterial street that is not fully developed, future commercial development and higher turning volumes resulting from installation of a TWLTL could partially or totally offset the operational and safety benefits initially gained from the TWLTL. The 4D design alternative should also be considered in such cases.

The 5T design alternative is most appropriate for surburban highways with commercial development, driveway densities greater than 45 driveways per mile, low-to-moderate volumes of through traffic, high left-turn volumes, and for high rates of rear-end and angle accidents associated with left-turn maneuvers. There has been little effort in the past to measure left-turn demand or to establish traffic volume ranges that would warrant installation of a TWLTL. The operational evaluation performed in this study indicates that the installation of a TWLTL on existing 4U facilities provides operational benefits at all volume levels. These benefits are relatively modest (7.8 to 10.2 seconds of delay reduced per left-turn vehicle) at a flow rate of 650 vph in each direction of travel, but are substantial at a flow rate of 900 vph (as much as one minute of delay reduced per left-turn vehicle) and even greater at higher flow rates.

Many safety evaluations of the 5T design alternative have been conducted. An extensive literature review by Glennon et al. (*12, 13*) estimated the accident reduction effectiveness of TWLTLs at 35 percent of the total accident experience prior to installation. This estimate was based primarily on a series of before-after evaluations in Michigan (*15, 16, 17*) as well as studies in Sacramento, California (*18*), and Seattle, Washington (*19*). The Michigan studies evaluated approximately 6.58 miles of TWLTL in the 15,000 to 30,000 ADT range and found an average 33 percent reduction in total accident frequency. The general accuracy of this estimate is reinforced by several more recent studies. In 1975, Busbee (*20*) reported a 38 percent reduction in accident frequency for one TWLTL project, and, in 1979, the Arizona Department of Transportation (*21*) reported a 35.9 percent reduction in accident frequency for 12 TWLTL projects totaling 12.2 miles in length. Thakkar (*7*) found a 27.7 percent reduction in total accident rate for the 5T design alternative, while the safety comparison performed in this study (see Table 3) found the total accident rate of the 5T alternative to be 24 percent lower than 4U sections for commercial sections and 19 percent lower for residential sections. As with the 4D design alternative, it is probably true that the installation of the 5T design alternative will have greater than average effectiveness at sites with a high proportion of rear-end and angle accidents associated with left-turn maneuvers. Furthermore, in all cases, the average accident severity for 5T sections was found to be the same or lower than that for 4U sections (see Table 4).

These findings concerning the safety effectiveness of the 5T alternative are reinforced further by the traffic conflict evaluation by McCormick and Wilson (*9*) (see Table 8), which found the 5T alternative to have the lowest traffic conflict rate for all of the design alternatives considered.

The published literature on the safety effectiveness of TWLTLs universally discounts the possibility of substantial increases in head-on accidents between vehicles in opposing directions trying to use the TWLTL to turn left at the same location. Although the potential for such accidents exists, drivers appear to understand the operation of a TWLTL clearly and avoid such situations. Those before-after studies that have looked at TWLTL effectiveness by accident type have found that head-on accidents usually decrease with TWLTL installation, although not by as much as other accident types such as rear-end accidents.

Five-Lane with Continuous Alternating Left-Turn Lanes

Another five-lane alternative (5C) uses continuous alternating one-way left-turn lanes in the median to control left-turn movements on surburban arterials. When implemented with a raised median the 5C alternative operates in a manner similar to the 4D alternative, whereas when implemented with a flush median, it operates in a manner similar to the 5T alternative except that a median left-turn is provided for only one direction of travel at a time.

The advantages and disadvantages of the 5C design alternative implemented with a raised median are essentially the same as for the 4D alternative. A major advantage of the 5C alternative over the 4D alternative is that median openings are generally provided more frequently.

The advantages and disadvantages of the 5C alternative implemented with a flush median are similar to the advantages and disadvantages of the 5T alternative. The operational effectiveness of the 5C flush median alternative is lower than the 5T alternative if development is uniform along both sides of the road, because a left-turn lane is provided for either one direction or the other but not for both at any given location. Limited studies of the safety effectiveness of the 5C flush median alternative suggest that it is less effective in reducing accidents than the 5T alternative. Thomas (*22*) found that this design alternative reduced accidents by 28 percent, which is slightly less than the generally accepted safety effectiveness estimate of 35 percent for TWLTLs. McCormick and Wilson (*9*) found the 5C flush median alternative to have nearly twice the traffic conflict rate of the 5T alternative. The only possible advantage of the 5C flush median alternative is the elimination of the potential for head-on collisions in the TWLTL and this potential problem has not, in fact, been found to occur. Thus, the 5T design alternative is considered to be preferable to the 5C design alternative with a flush median at any site where the latter might be considered.

Six-Lane Divided

The advantages and disadvantages of the six-lane divided (6D) design alternative are similar to the advantages and disadvantages of the 4D design alternative discussed earlier. One advantage of the 6D alternative over the 4D alternative is that the additional roadway width provides a more generous turning radius for vehicles to make U-turns at signalized intersections to complete midblock left-turn maneuvers that are prevented by the median.

Table 10. Critical factors in selection of design alternatives for suburban highways.

EXISTING CONDITIONS	• Existing geometrics and traffic control • Existing operational demands • Existing operational conditions (level of service, speed, delay) • Existing safety conditions • Existing land use • Other existing conditions
PROJECTED FUTURE CONDITIONS	• Projected future operational demands • Projected safety conditions • Land use planning/anticipated land use changes
CONSTRAINTS	• Physical constraints (available right-of-way width, intersection spacing) • Economic constraints (available funds) • Access control laws and ordinances • Zoning policies • Public opinion
PRIORITIES	• Functional classification • Priority for serving through traffic vs. land access traffic • Priority for control of future development
POTENTIAL BENEFITS AND DISBENEFITS	• Operational effectiveness • Safety effectiveness • Impact on through traffic vs. land access traffic • Impact on land use and development • Impact on abutting businesses • Impact on future traffic volumes • Impact on pedestrians • Impact on bicycles • Impact on transit

Seven-Lane with TWLTL

The advantages and disadvantages of the seven-lane TWLTL (7T) design alternative are similar to the advantages and disadvantages of the 5T design alternative. While the 7T alternative could be used to provide additional through traffic capacity at any location where the 5T alternative was under consideration, in actual practice highway agencies appear to limit the use of the 7T design alternative to residential and light commercial areas with relatively low left-turn volumes. In more heavily commercialized areas, the higher left-turn demands generated by the commercial development may not be adequately served by a TWLTL because of the high volume of opposing traffic. Thus, on facilities with heavy commercial development, the 6D design alternative may be preferable to the 7T alternative.

Selection Process

This section outlines the recommended process for selecting an appropriate design alternative for a suburban highway. The purpose of this discussion is to show how the various effectiveness measures and advantages and disadvantages of design alternatives discussed above can be considered together in the decision-making process.

The critical factors that influence the selection of a multilane design alternative are presented in Table 10. These critical factors are classified into five major categories: existing conditions, projected future conditions, constraints, priorities, and potential benefits and disbenefits. The critical factors set the framework for the design alternative selection process.

Table 11. Steps in recommended process for selecting design alternatives.

Step 1 - Determine existing conditions.

Step 2 - Determine projected future conditions.

Step 3 - Identify constraints.

Step 4 - Identify priorities.

Step 5 - Determine basic number of through lanes to serve current and projected future traffic volumes.

Step 6 - Identify feasible design alternatives with required number of through lanes.

Step 7 - Examine possible geometric variations in design alternatives (e.g., shoulders).

Step 8 - Determine benefits and disbenefits of feasible alternatives.

Step 9 - Select the ultimate design alternative for the site.

Step 10 - Examine possible staged construction options to reach the ultimate design.

Table 11 presents 10 steps in the recommended process for selecting a design alternative. Each step is discussed in the following.

Step 1—Determine Existing Conditions. The first step in the process of selecting an appropriate design alternative for a particular site is to document the existing conditions at the site. Table 12 presents a list of existing conditions relevant to the selection of a design alternative. These include existing geometrics and traffic control; existing operational demands; existing operational conditions such as capacity, level of service and delay (which are the combined results of geometrics, traffic control, and operational demands); existing safety conditions; existing land use; and other relevant site specific conditions.

The documentation of existing conditions for a major design project may require extensive field work, including surveys and traffic counts, and assembly of data from existing records, such as construction plans and previous traffic studies. For planning studies, a reduced set of data related to traffic operational demand and operational conditions should be collected or estimated, in addition to existing geometrics, to allow preliminary consideration of an appropriate design alternative. Table 13 presents a stratification system or framework representing the minimum data required for planning purposes. This stratification system includes the key variables needed to assess traffic operation conditions and estimate expected traffic accident rates for a suburban highway. At the very least, the traffic engineer or the designer selecting a preliminary design alternative should determine where the site in question falls within the levels for each factor in the stratification system.

A key operational variable included in both Tables 12 and 13 is the left-turn volume for minor intersections and driveways along a section of highway, which is necessary for any quantitative assessment of the operational effects of installing a raised median or a TWLTL. Greater emphasis needs to be placed in the future on collecting data on midblock left-turn volumes for use in the assessment of design alternatives, because without such data the traffic engineer or designer must rely on surrogates, such as driveway density or type of development, and engineering judgment to determine operational effectiveness.

Safety conditions at the site are also a key consideration including the accident rate per million vehicle-miles on the highway section, the proportion of fatal and injury accidents, and the proportion of accidents susceptible to correction by installation of a median or a TWLTL (head-on, rear-end, and angle accidents).

Table 12. Existing conditions relevant to selection of design alternatives for suburban highways.

Existing Geometrics and Traffic Control

- Current design alternative
- Pavement and lane widths
- Presence or absence of shoulder
- Shoulder width
- Presence or absence of curb parking
- Presence or absence of median
- Type of median
- Median width
- Right-of-way width
- Speed limit
- Spacing between major intersections (and/or major driveways)
- Intersection geometrics
- Intersection traffic controls

Existing Operational Demand

- Average daily traffic (vpd)
- Hourly traffic volumes and peaking characteristics
- Percent trucks
- Directional split
- Turning volumes at intersections and driveways (especially left turns)
- Bicycle volumes
- Pedestrian volumes and desired movements
- Type and frequency of transit service

Existing Operational Conditions

- Capacity (vph)
- Level of Service
- Volume/capacity ratio
- Mean and 85%ile speed (mph)
- Travel-time or delay (veh-sec)

Existing Safety Conditions

- Accident rate (accidents per million vehicle-miles)
- Accident frequency per mile per year
- Accident severity distribution (fatal/injury/PDO)
- Accident type distribution (by relationship to intersection, number of vehicles involved and type of collision)
- Existing accident problems (specific locations and/or specific accident types)

Existing Land Use

- Type of development (commercial/residential)
- Continuity of development (strip development/isolated major traffic generators)
- Driveway density (driveways per mile)
- Intersection density (minor intersections per mile)

Other Existing Conditions

- Site-specific conditions relevant to design alternatives

Step 2—Determine Projected Future Conditions. Projected future conditions at the site over the design life of the proposed improvement should also be determined. The projections should include, at the minimum, the stratification factors given in Table 13. The design life of the project should normally be 20 years.

Step 3—Identify Constraints. Constraints that limit the feasibility of particular design alternatives or make particular alternatives more or less desireable should be identified. Such constraints may include physical constraints, economic constraints set by availability of funds, access control laws and ordinances, zoning policies, and public opinion. The physical constraints are design controls which, for all practical purposes, cannot be changed, such as intersection spacing and the maximum right-of-way width that can be obtained without interfering with existing development.

Step 4—Identify Priorities. Highway agency, land use, and community priorities that affect the choice of a design alternative should be identified at an early stage. One important consideration is the priority assigned to through traffic movement as

Table 13. Stratification system for characterizing traffic operations on suburban highways.

Traffic Volume

Average Daily Traffic

7,000 - 10,000 vpd
10,000 - 15,000 vpd
15,000 - 20,000 vpd
Over 20,000 vpd

Peak Hour Flow Rate (one-way)

Under 300 vph
300 - 600 vph
600 - 1,000 vph
Over 1,000 vph`

Left-Turn Volume

Under 100 left turns/hr/mile
100 - 200 left turns/hr/mile
200 - 400 left turns/hr/mile
Over 400 left turns/hr/mile

Truck Volumes (Percent Trucks)

Under 5% trucks
5 - 10% trucks
Over 10% trucks

Type of Development

Commercial
Residential

Driveway Density

Under 30 driveways per mile
30 - 60 driveways per mile
Over 60 driveways per mile

Intersection Density

Under 5 unsignalized intersections per mile
5 - 10 unsignalized intersections per mile
Over 10 unsignalized intersections per mile

opposed to land access traffic. The functional classification of the roadway is an indicator of this priority. Design alternatives with raised medians that limit access to abutting property, such as the 4D and 6D alternatives, should generally be assigned higher priorities on facilities classified as major arterials than on facilities classified as minor arterials or collectors.

Another consideration is the priority assigned to control of future development. Alternatives incorporating a raised median may be preferred on relatively undeveloped facilities to prevent future strip commercial development, while alternatives incorporating a TWLTL may be preferred on facilities where strip commercial development has already occurred.

Step 5—Determine the Basic Number of Through Lanes. The first analytical step in the selection process is to determine the basic number of through lanes needed to maintain an adequate level of service, both for existing traffic volumes and for projected future traffic volumes. The basic number of through lanes is determined through a capacity analysis. Chapter 11 on Urban and Suburban Arterials in the 1985 *Highway Capacity Manual* (HCM) provides a procedure to consider the lane requirements both for midblock sections and for signalized intersection approaches. On suburban highways without signals or with widely spaced signals, the capacity analysis could be performed with the procedures of 1985 HCM Chapter 7 on Multilane Highways or Chapter 8 on Two-lane Highways. These procedures do not generally address the interrupted flow conditions produced by suburban development and midblock turning movements, but these issues can be addressed with the data in this report on the effectiveness of raised median and/or TWLTL design alternatives.

Step 6—Identify Feasible Design Alternatives. The next step in the selection process is to identify all feasible design alternatives with the required number of through lanes to serve the projected future traffic. Feasible alternatives should include all design alternatives that could be constructed within the physical constraints of the site. Right-of-way restrictions have been emphasized in the previous discussion of design alternatives because, in most cases, alternatives that involve demolition of existing structures or eliminating off-street parking for commercial establishments will be considered infeasible. Design alternatives that require utility relocation (e.g., utility poles or street lights) should be included as well as the cost for utility relocation included in the project cost.

Step 7—Examine Possible Geometric Variations. Possible geometric variations of the feasible design alternatives should be considered including the widths of lanes, medians, and shoulders. The choice between full shoulders and no shoulders for each design alternative should be considered at this stage both because of the potential impact on the project cost and, especially, because the reduced roadway width from elimination of the shoulder may make an infeasible design alternative physically feasible at some sites. The estimated 10 percent increase in accident rate that results from elimination of a shoulder may be more than offset by the decrease in accident rate that results from installation of a median and/or TWLTL that would not otherwise be feasible. The design speed of the facility and the actual operating speeds used by drivers should be considered in design of the detailed geometrics of the facility.

Step 8—Determine Benefits and Disbenefits. Each feasible design alternative and possible geometric variations of each alternative should be evaluated to determine the quantitative and nonquantitative benefits and disbenefits.

The traffic operational and safety effects of each alternative can be quantified using the effectiveness estimates presented earlier in this report. The operational effectiveness of TWLTLs for sites that require one or two through lanes in each direction can be determined from interpolation in Table 6. The operational effectiveness for installation of a raised median on a highway with two through lanes in each direction of travel can be estimated from Table E-7. The estimates of delay reduction per left-turn vehicle from the tables should be multiplied by the left-turn volume to obtain a delay estimate in units of vehicle-seconds.

The safety effectiveness of each design alternative, relative to the design alternative currently in use, can be determined from the accident rate estimates given in Tables 1 and 2 and the accident severity distribution data given in Table 4. For example, improvement of a commercially developed section with an ADT of 15,000 vpd, 6 percent trucks, 70 driveways per mile, and 7.5 unsignalized intersections per mile from the 4U to the 5T design alternative would be expected to decrease the accident rate by 22 percent from 8.10 to 6.28 accidents per million vehicle-miles and to decrease the percentage of fatal and injury accidents at nonintersection locations from 38.4 percent to 33.7 percent.

The effect of geometric variations on the safety effectiveness of design alternatives should also be considered. In particular, the elimination of a full shoulder for a particular design alternative would be expected to increase the accident rate for that alternative by 10 percent.

At sites where the actual accident rate for existing conditions is substantially greater than the rate for the existing design alternative predicted from Tables 1 and 2 and/or the percentage of head-on, rear-end, and angle accidents at the site is greater than the percentage found in Table 5, there may be a correctable accident problem at the site. In such cases, the safety effectiveness of design alternatives that involve installation of a raised median or a TWLTL is likely to be greater than average. The magnitude of the accident reduction for sites with a correctable safety problem must be based on engineering judgment considering the magnitude of the existing problem, the impact of particular design alternative(s) on that type of problem, and each agency's experience with similar types of improvements.

Table 14 and Figure 17 have been developed as a summary of the traffic operational and safety impacts of design alternatives and form a basis for making judgments of the type discussed above. Table 14 lists 11 operational factors and 13 safety factors whose relative merits have been rated for a range of geometric variations for five major design alternatives. Figure 17 presents the ratings that were developed by the project staff. Each design alternative has been rated for a range of roadway widths (traveled way plus shoulder) that correspond to narrow lane, wide lane, narrow shoulder, full shoulder, and wide median design. A five-unit ordinal scale was used to rate each operational and safety factor; from least desirable to most desirable, the ratings used were − −, −, 0, +, + +. The more operational safety factors are improved by a particular design alternative and the greater the improvement in the rating for those factors, the greater the safety effectiveness that would be expected from the improvement.

Other, less quantitative benefits and disbenefits of design alternatives should also be identified, because these nonquantitative factors may often be as important an influence as traffic operations and safety on the choice of a design alternative. The nonquantitative impacts to be considered include the two issues for which priorities were established in an earlier stage of the selection process: the impact of the design alternative on through traffic vs. land access traffic and the impact of the design alternative on land use and development. Other benefits and disbenefits that should be considered are the impact on abutting businesses, the impact on growth of future traffic volumes, the impact on pedestrians, the impact on bicycles (particularly important if no shoulder is provided), and the impact on bus transit operations.

Step 9—Select the Ultimate Design Alternative. The next step in the process is to consider the trade-offs among the benefits, and costs of the feasible design alternatives and select the most appropriate design alternative for the site in question. The design alternative that best serves the projected future traffic at the site is referred to as the ultimate design alternative. The trade-offs among design alternatives are usually considered through engineering judgment, although a formal cost effectiveness procedure, such as the procedure of the AASHTO User Benefit Analysis Manual (*1*), could be used to examine the quantitative aspects of traffic operations, traffic safety, and construction cost.

Step 10—Examine Staged Construction Options. The final step in the selection process is to consider whether to construct the ultimate design alternative immediately or whether staged construction could be employed to construct a less costly design alternative now and construct the ultimate design alternative later.

Table 14. Operational and safety factors rated for design alternatives on suburban highways.

Operational Factors

1. Minimize or eliminate delay to through vehicles by left-turning vehicles
2. Minimize delay to through vehicles by right-turning vehicles
3. Allow provision of turning lanes at intersections and high volume driveways
4. Ease the movement of emergency vehicles
5. Provide for storage of disabled vehicles
6. Compatible with use of frontage roads
7. Facilitate U-turns
8. Shadow vehicles making crossing maneuvers at unsignalized intersections (eliminate blocking of one direction while waiting for gap in the other direction)
9. Facilitate pedestrian crossings
10. Encourage access development on side streets off of the arterial
11. Minimize high-volume of left-turn and U-turn movements at intersections

Safety Factors

1. Minimize rear-end conflicts between left-turning and through vehicles and allow left-turn drivers time to evaluate opposing gaps
2. Minimize high concentration of driveways and overlapping conflict patterns
3. Control conflicts between left turns into and out of driveways
4. Minimize or eliminate conflicts between opposing left-turns off of the arterial
5. Minimize or eliminate conflicts between left turns and right turns from/to the same lane
6. Minimize or eliminate conflicts caused by encroachment on opposing lanes of vehicles turning right into and out of driveways
7. Minimize or eliminate conflicts caused by encroachment on adjacent lanes of vehicles turning right into and out of driveways
8. Minimize or eliminate conflicts in opposing lanes of vehicles turning left off of the arterial
9. Minimize time during which left-turn conflicts with opposing traffic can occur
10. Provide protected position in median for crossing vehicles
11. Provide protected position in median for crossing pedestrians
12. Minimize conflicts between bicycles and motor vehicles.
13. Increase width of roadside clear recovery area

DESIGN ALTERNATIVE	DESCRIPTION OF GEOMETRICS	TOTAL AVAILABLE WIDTH (FT)	OPERATIONAL FACTORS											SAFETY FACTORS												
			1	2	3	4	5	6	7	8	9	10	11	1	2	3	4	5	6	7	8	9	10	11	12	13
Two-Lane Undivided (2U)	Narrow Lanes	20 - 22	--	--	--	--	--	--	--	+	+	--	+	--	--	--	--	--	--	0	--	+	--	--	--	--
	Wide Lanes	24 - 26	--	--	--	--	--	--	--	+	+	--	+	--	--	--	--	--	-	0	+	+	--	--	-	--
	Narrow Shoulder	28 - 36	-	-	--	+	+	--	-	+	+	--	+	--	--	--	--	-	+	0	+	+	--	--	+	-
	Full Shoulder	38 - 40	+	++	+	++	++	--	-	+	+	--	+	--	--	--	--	-	++	0	++	+	--	--	++	+
Three-Lane with TWLTL (3T)	Narrow Lanes	30 - 32	+	--	--	+	--	--	--	--	+	--	+	+	--	-	-	-	++	--	--	+	--	--	--	--
	Wide Lanes	34 - 40	++	--	--	+	--	--	-	-	-	--	+	++	--	-	-	+	++	+	+	+	-	--	-	--
	Narrow Shoulder	42 - 48	++	+	--	++	+	--	-	-	-	--	+	++	--	-	-	+	++	++	+	+	-	--	+	-
	Full Shoulder	50 - 56	++	++	+	++	++	-	-	-	-	--	+	++	--	-	-	++	++	++	++	+	-	--	++	+
Four-Lane Undivided (4U)	Narrow Lanes	40 - 42	-	-	--	-	--	--	-	--	--	--	+	--	--	--	--	++	++	--	-	-	--	--	--	--
	Wide Lanes	44 - 52	-	-	--	-	--	--	-	--	--	--	+	--	--	--	--	++	++	+	+	-	--	--	-	--
	Narrow Shoulder	54 - 58	-	+	--	+	+	--	-	--	--	--	+	--	--	--	--	++	++	++	++	-	--	--	+	-
	Full Shoulder	60 - 64	-	++	+	++	++	--	-	--	--	--	+	--	--	--	--	++	++	++	++	-	--	--	++	+
Four-Lane Divided with Raised Median (4D)	Narrow Lanes	48 - 54	+	-	--	-	--	++	-	--	+	++	-	+	++	++	++	++	++	--	-	-	--	++	--	--
	Wide Lanes	56 - 64	++	-	--	-	--	++	+	-	++	++	-	++	++	++	++	++	++	+	++	-	-	++	-	--
	Narrow Shoulder	66 - 70	++	+	--	+	+	+	++	-	++	++	-	++	++	++	++	++	++	++	++	-	-	++	+	-
	Full Shoulder	72 - 80	++	++	+	++	++	+	++	-	++	++	-	++	++	++	++	++	++	++	++	-	-	++	++	+
	Wide Median	72 - 94	++	-	--	++	++	+	++	++	++	+	-	++	++	++	++	++	++	+	++	-	++	++	-	--
Five-Lane with TWLTL (5T)	Narrow Lanes	50 - 54	+	-	--	+	--	--	-	--	--	--	+	+	--	-	-	++	++	--	--	-	--	--	--	--
	Wide Lanes	56 - 64	++	-	--	+	--	--	+	-	--	--	+	++	--	-	-	++	++	+	++	-	-	--	-	--
	Narrow Shoulder	66 - 68	++	+	--	+	+	--	++	-	--	--	+	++	--	-	-	++	++	++	++	-	-	--	+	-
	Full Shoulder	70 - 80	++	++	+	++	++	--	++	-	--	--	+	++	--	-	-	++	++	++	++	-	-	--	++	+

Scale of Operational and Safety Ratings:

++ Most desirable
+
0
−
−− Least desirable

Figure 17. Relative ratings of operational and safety factors for design alternatives.

A comparison of the basic number of through lanes required for the existing traffic volume and for the projected future traffic volume should indicate whether alternatives with fewer through lanes than the ultimate design alternative should be considered. If the ultimate design alternative includes a raised median or a TWLTL, the current need for the median treatment should be assessed. Any design alternative considered as the first stage should be compatible with the ultimate design alternative; for example, it would not make sense to build a first-stage alternative with a raised median if the ultimate design alternative involved a TWLTL. The 3T and 4U design alternatives may be particularly appropriate as the first stage to an ultimate 4D or 5T design. If a design alternative less costly than the ultimate design alternative is capable of serving the current traffic demand, the choice between immediate construction of the ultimate design alternative and the staged construction approach should be based on available funds and on the length of time that the first stage improvement could continue in service.

CHAPTER THREE

INTERPRETATION, APPRAISAL, APPLICATION

The findings of the study reported in Chapter Two illustrate the traffic operational and safety characteristics of multilane design alternatives for improving suburban highways. These findings form the basis for the selection of appropriate design alternatives for particular suburban highway facilities.

The findings concerning the relative safety of multilane design alternatives have been presented in Tables 1, 2, 3, 4, and 5 in Chapter Two. The typical accident rates for suburban arterials given in Tables 1, 2, and 3 represent average safety conditions for individual design alternatives and types of development. Although some site-to-site variation from these averages is inevitable, major departures from the typical rates may be interpreted as the presence of a safety problem that is potentially correctable through installation of an improved design alternative. A predominance of head-on, rear-end, and angle accidents above the levels suggested in Table 5 may also indicate the presence of a correctable safety problem.

The evaluation of safety problems in this manner requires judgment on the part of the designer or traffic engineer to determine whether the accident experience at a particular site is susceptible to correction by a design alternative improvement. The exercise of this type of judgment is essential because average accident rates (for example, those presented in Tables 1, 2, and 3) suggest that the 4U, 4D, and 5T design alternatives have higher accident rates than the 2U and 3T design alternatives. In fact, an uncongested four- or five-lane facility is likely to have a lower accident rate than a highly congested two- or three-lane facility.

The operational findings obtained in the study indicate the clear operational advantages of design alternatives involving TWLTLs over undivided and/or raised-median alternatives over a wide range of traffic volume levels and driveway densities. Installation of a raised median provides an operational advantage on a four-lane undivided facility only for flow rates over 1,000 vph in one direction.

The study results suggest that design alternatives involving two-way left-turn lanes are very appropriate as the ultimate design alternative for a wide variety of suburban highway conditions, since the 3T and 5T alternatives have both traffic operational and safety advantages over comparable undivided and raised median alternatives. Raised medians should be used only where other potential benefits outweigh their operational disadvantages. The use of raised medians may be appropriate on suburban highways with high through traffic volumes and relatively low turning volumes, highways in undeveloped or lightly developed areas where strip commercial development is considered undesirable, highways with high pedestrian crossing volumes, and highways where a physical separation or median barrier is needed between the lanes of traffic moving in opposite directions.

A nine-step process has been suggested for selecting multilane design alternatives for suburban highways. Three design examples have been developed to illustrate the selection process and the use of the traffic operational and safety findings presented in Chapter Two. These examples address the following design situations:

- Improvement of a two-lane undivided (2U) design to the three-lane TWLTL (3T) design alternative.
- Improvement of a two-lane undivided (2U) design alternative to the five-lane TWLTL (5T) design alternative.
- Improvement of a four-lane undivided (4U) design alternative to the four-lane divided (4D) design alternative.

The design alternatives are summarized here and presented in detail in Appendix F.

Design Example 1 illustrates a suburban two-lane highway with moderate peak-hour flow rates (450 vph in each direction), but with strip commercial development and relatively high turning volumes (90 left-turns per hour per mile). These conditions have resulted in peak-hour congestion and accident rates that are nearly four times the accident rate for a typical two-lane undivided highway. It was found that substantial safety benefits (60 to 80 percent accident rate reduction) would result from each of three design alternatives—3T, 4D, and 5T—that would reduce the peak-hour congestion. The 5T design alternative was selected as the ultimate design alternative for this site. However, because of relatively slow current traffic volume growth, immediate construction of the 3T design alternative was recommended as a first stage that could serve the traffic demand for at least 5 years. The ultimate 5T design alternative would be constructed if and when traffic volumes warrant.

Design Example 2 illustrates a commercially developed suburban two-lane undivided highway with greater operational demands but less serious safety problems than Design Example 1. This site has a current peak-hour flow rate of 950 vph in each direction with 190 left-turns per hour per mile with rapid growth of traffic volume expected. The accident rate at the site is 1.5 times the expected accident rate for a two-lane undivided highway. Despite the contrasting traffic operational and safety conditions to Design Example 1, a two-way left-turn lane is still the appropriate median treatment for this site. The current traffic operational demands warrant the construction of the 5T design alternative with possible later conversion to a six- or seven-lane facility.

Design Example 3 illustrates an existing suburban four-lane undivided highway. This example presents a contrasting case to the previous examples, with rapid growth of traffic volumes expected, but with no correctable safety problems. This site is in a developing area, and the responsible highway agency assigns a high priority to the preservation of through traffic capacity and the control of commercial development through installation of a raised median. The four-lane divided (4D) alternative is recommended for construction at this site.

CHAPTER FOUR

CONCLUSIONS AND RECOMMENDATIONS

The major conclusions of the research address the appropriate uses of multilane design alternatives for improving suburban highways. A brief summary of the appropriate uses of each design alternative is given below.

The three-lane TWLTL (3T) design alternative has substantial traffic operational and safety advantages over a two-lane undivided highway and requires only a minimal increase in roadway width. The 3T design alternative can be expected to reduce accident rates, on the average, by 11 to 35 percent below the accident rate for a two-lane undivided facility, with even greater reductions possible for highly congested two-lane undivided facilities. The 3T design alternative will provide a substantial reduction in delay to through vehicles caused by left-turning vehicles, especially for flow rates above 500 to 600 vph in one direction. The three-lane TWLTL design alternative has been underutilized on suburban highways until recent years, but may be appropriate as the ultimate design alternative for some sites or as the first stage of a more extensive improvement, depending on current and projected future traffic volume levels. In some situations with high left-turn volumes and relatively low through volumes, restriping of a four-lane undivided (4U) facility as a 3T facility may promote safety without sacrificing operational efficiency.

The 4U design alternative is most appropriate for residential and light commercial areas on suburban highways classified as collectors and minor arterials. The 4D and 5T design alternatives, if physically feasible, would be more desirable than the 4U design alternative on highways that have dense commercial development, have heavy left-turn volumes, or are classified as, or could become, major arterials. The 4U design alternative may also be appropriate as the first stage toward construction of a wider roadway with a median treatment.

The four-lane divided (4D) design alternative is best suited for use on major arterials with high volumes of through traffic and less than 45 driveways per mile. The 4D design alternative is operationally preferable to the 4U design alternative only for sites with peak-hour flow rates over approximately 1,000 vph in one direction, although this alternative could be used at lower flow rates where offsetting benefits, such as improved safety, land use control, or preservation of through traffic capacity, are expected. The average accident rates for the 4U and 4D design alternatives are approximately the same, although a reduction in accident rate would be expected from improved traffic flow with installation of the 4D design alternative on a congested 4U facility. The 4D design alternative is not well suited to highways with strip commercial development and may, in fact, be used to discourage such development from occurring. However, the 4D design alternative is better suited than the 5T design alternative to serve suburban highways with isolated major traffic generators that have widely spaced, high-volume driveways.

The five-lane TWLTL (5T) design alternative is most appropriate for suburban highways with commercial development, driveway densities greater than 45 driveways per mile, low-to-moderate volumes of through traffic, high left-turn volumes, and/or high rates of rear-end and angle accidents associated with left-turn maneuvers. The 5T design alternative was found to provide traffic operational benefits, relative to the 4U and 4D design alternatives, for all levels of through traffic volume, left-turn volume, and driveway density evaluated. The installation of the 5T design alternative on an undivided facility is expected to reduce accident rate by 19 to 35 percent, on the average, with even greater reductions possible for highly congested facilities. The 5T design alternative has been used extensively over the last 20 years and is likely to continue as the most common multilane design alternative improvement for suburban highways.

The five-lane continuous alternating left-turn lane (5C) design with a raised median is similar in traffic operations and safety to the 4D design alternative, although more frequent median openings are provided. The use of the 5C design alternative with a flush median is not recommended, because the 5T design alternative would be superior in traffic operations and safety in any situation where the 5C design alternative with a flush median might be considered.

The traffic operational and safety performance of the six-lane divided (6D) and seven-lane TWLTL (7T) design alternatives has not been quantified, but is expected to be similar to their four- and five-lane counterparts.

The provision of a full shoulder on a suburban highway is expected to reduce the accident rate by 10 percent from the accident rate for a similar highway with no shoulder. No differences between design alternatives in the safety effectiveness of shoulders were found; however, such differences would be very difficult to detect in the available data base.

The use of a stepwise selection process for multilane design alternatives is recommended to assure that both present and future requirements for the facility are considered before a particular design alternative is selected. A general approach to this selection process is presented in this report.

It should be recognized that the quantitative operational results presented in this report are based on a traffic simulation model that is in need of further development and validation. While the model results do suggest some fundamental findings concerning the operational effectiveness of TWLTLs and raised medians, the results are not as precise as desired and should be interpreted as approximate rather than exact.

The safety effects of multilane design alternatives have been quantified in this report, but it should be recognized that engineering judgment is required in the application of these estimates to particular sites. The estimates in this report are based on data from two states—California and Michigan. However, both accident rates and the quality of accident reporting systems vary from state-to-state and from jurisdiction-to-jurisdiction. The safety measures presented in this report can be used most effectively in conjunction with the actual experience of particular highway agencies.

Further research is needed on the traffic operational effects of raised medians, two-way left-turn lanes, and suburban development. In existing capacity procedures, suburban highways of the type addressed in this report tend to slip through the cracks between procedures for highways with uninterrupted flow and procedures for signalized intersections. Publication of the 1985 *Highway Capacity Manual* should partially remedy this deficiency, although the effects assigned to raised medians, two-way left-turn lanes, and suburban development in the new Chapters 7 and 11 of the HCM are not very precise. It is recommended that both future research on multilane highway operations and design of future improvement projects should be based on an explicit measure of left-turn demand between major intersections expressed, for example, as left-turn volume per hour per mile.

Finally, it should be recognized that, while traffic operations and safety are the key factors in most decisions concerning multilane design alternatives for improving suburban highways, other less quantitative factors, priorities, and constraints should receive due consideration. Such factors may include available funding levels, impacts on land use and development, impacts on abutting businesses, impact on pedestrians and bicycles, access control laws and ordinances, zoning policies, and public opinion.

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APPENDIX A

DEVELOPMENT OF SAFETY DATA BASE

The safety analyses performed in this study required the assembly of a data base containing geometric, traffic, and accident data for typical suburban arterial highways with a range of multilane design alternatives. This data base was assembled from the records of the California Department of Transportation and the Michigan Department of Transportation and contains data on 377.6 miles of suburban arterials with the following design alternatives:

- Two-lane undivided
- Three-lane with TWLTL
- Four-lane undivided
- Four-lane divided
- Five-lane with TWLTL

The safety data for these five alternatives were extensively analyzed in the study to determine the safety differences between the design alternatives and the safety differences between a no-shoulder and a full-shoulder condition for each alternative.

A supplementary data base was also assembled containing data on 91.1 miles of suburban arterial for three additional design alternatives:

- Five-lane divided with continuous alternating left-turn lane
- Six-lane divided
- Seven-lane with TWLTL

This supplementary data base has not been extensively analyzed.

DEFINITION OF A SUBURBAN ARTERIAL HIGHWAY

The project scope was intended specifically to address the operational and safety problems of arterial highways in a suburban setting. The reason for this particular focus was that suburban arterials often present a unique combination of large and growing traffic volumes, high speeds, and rapid development of adjoining land that confront highway agencies with traffic operational and safety problems that are particularly acute.

The focus on suburban conditions required the development of a working definition of a suburban arterial highway as distinct from other functional classes and from urban and rural conditions. This definition was based strictly on traffic operational conditions rather than on whether a highway section was located within a central city or an outlying suburban community. By this definition, a higher speed arterial street could be considered a suburban highway even though located within a major city, while a low-speed arterial could be considered urban if located within the congested portion of a suburban community.

The following criteria were used to define a suburban arterial highway in this study:

- Traffic volume over 7,000 vpd.
- Speeds between 35 and 50 mph.
- Spacing of at least one-quarter mile between signalized intersections.
- Direct driveway access from abutting properties.
- No curb parking.
- Located in or near a populated area.

The purpose of the minimum traffic volume criterion (7,000 vpd) was to focus the study on highways with enough traffic that a two-lane undivided cross section is, or soon would become, inadequate to handle the demand. This criterion was adhered to rigorously in the study; each highway section in the data base had a traffic volume over 7,000 vpd in at least one year of the study period.

The speed criterion (35 to 50 mph) was based on the posted speed limit, rather than actual operating speeds, because of the lack of speed data for most of the highway sections studied. This criterion was intended to limit the study to the suburban environment and eliminate highways that were too urban or too rural. Arterial highways with speed limits of 30 mph and below are generally more urban than suburban in character and were excluded from the study. Highways with 55-mph speed limits generally represent either rural or controlled access conditions that were also not considered suburban in character. Speed limits from 35 to 50 mph represent the range typically found on suburban highways, with the higher speeds usually associated with less developed highways.

The signalized intersection spacing criterion was intended to exclude urban arterials with congested signalized conditions, such as a central business district with a signal at every intersection. In most cases, the average spacing between signalized intersections on the study sections is ½-mile or more.

Highways with controlled access where direct driveway access to the arterial were from abutting properties were excluded from the study.

The curb parking criterion was included because arterials with curb parking permitted were thought to be more urban than suburban in character. Suburban arterials typically have commercial establishments that provide off-street parking for their customers. However, because it was found in California that curb parking was permitted on more than half of the four-lane divided sections that would otherwise be classified as suburban arterials, it was decided to collect data on highway sections with curb parking, but to record for each section whether or not curb parking was permitted. Most of the safety analyses performed in the study excluded all sections where curb parking was permitted.

The final criterion, location in or near a populated area, was introduced to eliminate the possibility of a short section of rural highway with a depressed speed limit due to special conditions being classified as a suburban arterial. Both state highway agencies that participated in the study have classified their highway system into urban, suburban, and rural areas, and those sections classified as rural were excluded from the study. The distinction between urban and suburban conditions was based on the operational criteria presented here and not on the basis of the state criteria. (For example, California classifies a highway section as "suburban" if it is located within an urban area, but outside city limits, or within a rural area, but inside city limits. This very distinction implies that the location with respect to city limits is not a good indicator of the true character of the highway.) It should be noted that the study sections were not necessarily located in large metropolitan areas, but could be located in any community with a population greater than approximately 20,000 where the conditions defined as suburban are found.

SELECTION OF STUDY SECTIONS

Study sections located on suburban arterial highways in California and Michigan were selected in a two-step process. First, a geometric inventory tape was obtained from each state and reviewed to identify candidate sites for each design alternative of interest to the study. Second, the candidate sites were reviewed on the state's photolog to confirm or update the data included on the inventory, to eliminate sections that were not appropriate for the study, and to subdivide sections where necessary.

The geometric inventory tapes obtained from each state were developed by the state primarily from photolog data and were in current use as part of the state's accident surveillance system. The geometric inventory file used in California was the Caltrans Highway Data Base (HDB) from the Traffic Accident Surveillance and Analysis System (TASAS); the Michigan data were from the Michigan Dimensional Accident Surveillance System (MIDAS). A review of these files identified approximately 175 route sections in California and 165 route sections in Michigan that were under state jurisdiction and were classified as suburban arterials.

Each of these route sections was reviewed on the photolog. Based on the computerized data and the photolog review, the route sections were subdivided into shorter lengths that were used as study sections. Each study section was required to be homogeneous in five parameters: design alternative, lane width, shoulder width, speed limit, and ADT. The maximum variation in ADT permitted within a study section was 20 percent of ADT or 3,000 vpd, whichever is less. A minimum length of 0.25 miles over which these five parameters must remain constant was established to define a study section. No maximum length was established, and the sections were allowed to be as long as possible given the homogeneity requirements. In no case were homogeneous sections subdivided merely to increase the sample size.

The process of subdividing the route sections into homogeneous study sections and eliminating inappropriate sections resulted in 147 study sections in California and 273 study sections in Michigan. Table A-1 shows the number of study sections and the total length for each design alternative in both California and Michigan. Table A-2 gives the total length of study sections for each design alternative in the combined California and Michigan data, broken down separately by nine key study variables: ADT, truck percentage, number of driveways per mile, number of unsignalized intersections per mile, traffic speed, lane width, shoulder width, adjacent land use, and estimated level of left-turn demand.

DATA COLLECTION

Data on the geometric and traffic control characteristics, traffic volumes, and accident history of each study section were obtained from the cooperating states. Table A-3 gives the data elements that were obtained. The sources of these data included computerized files, photologs, and manual files.

Geometric and Traffic Control Data

Most of the geometric and traffic control data needed for the study sections were available in the computerized data bases, including: number of lanes, median treatment (if any), lane width, presence of shoulder, shoulder width, speed limit, and number of signalized and unsignalized intersections. These data were updated or confirmed in a review of the most recent, available photolog film for each site, and additional data were obtained from the photolog including presence or absence of curb parking, type of development on adjacent land (residential/commercial), and number of residential, commercial, and industrial driveways.

Traffic Volume Data

The traffic volume data for the study sections were drawn from both computerized and manual files. Separate estimates of average daily traffic volume and peak-hour traffic volume for each year of the 5-year study period (1978–1982) were used,

Table A-1. Number and total length of study sections in California and Michigan.

	California		Michigan		Combined	
Design Alternative	Number of Sections	Total Length (miles)	Number of Sections	Total Length (miles)	Number of Sections	Total Length (miles)
Two-lane undivided (2U)	55	28.2	38	27.9	93	56.1
Three-lane with TWLTL (3T)	8	3.7	11	8.7	19	12.4
Four-lane undivided (4U)	30	16.9	99	56.4	129	73.3
Four-lane divided (4D)	28	14.4	16	7.4	44	21.8
Five-lane with TWLTL (5T)	26	16.1	109	75.1	135	91.2
	147	79.3	273	175.5	420	254.8

Table A-2. Total length (miles) of study sections classified by design alternative and other variables.

Design Alternative	Average Daily Traffic (veh/day) 7,000-10,000	10,000-15,000	15,000-20,000	Over 20,000	Truck Percentage < 5	5 - 10	> 10	Driveway Density (Driveways Per Mile) < 30	30-60	> 60	Number of Intersections Per Mile < 5	5 - 10	> 10
2U	15.1	25.8	6.9	8.3	23.9	28.7	3.5	21.8	28.4	5.9	28.8	20.6	6.8
3T	3.4	2.6	4.6	1.8	4.3	7.1	1.0	1.6	7.3	3.5	4.1	2.5	5.8
4U	4.7	22.8	21.5	24.3	29.5	33.5	10.3	13.7	34.4	25.2	25.2	23.5	24.6
4D	1.0	4.2	6.7	9.9	8.8	10.9	2.1	8.5	8.0	5.3	15.4	5.6	0.8
5T	1.3	8.8	9.5	71.6	21.7	62.3	7.2	18.2	45.0	28.0	43.7	36.9	10.6
	25.5	64.2	49.2	115.9	88.2	142.5	24.1	63.8	123.1	67.9	117.2	89.1	48.6

Design Alternative	Traffic Speed Low (35-40 mph)	High (45-50 mph)	Lane Width 10 ft	11 ft	12 ft	Shoulder Width 0-4 ft	8 ft & Over	Adjacent Land Use Residential	Commercial	Level of Left Turn Demand Light	Moderate	Heavy
2U	23.4	32.7	11.7	11.6	32.8	25.5	30.6	35.5	20.6	49.4	6.7	0.0
3T	3.7	8.7	1.5	2.8	8.1	4.8	7.6	5.7	6.7	7.2	5.2	0.0
4U	47.2	26.1	12.8	18.1	42.4	46.6	26.7	25.1	48.2	54.1	17.3	1.9
4D	10.9	10.9	0.0	0.3	21.5	7.1	14.7	5.7	16.1	13.6	8.2	0.0
5T	36.2	55.0	0.4	10.0	80.8	78.4	12.8	10.9	86.3	28.6	36.8	25.8
	121.4	133.4	26.4	42.8	185.6	162.4	92.4	82.9	171.9	152.9	74.2	27.7

whenever possible. A single estimate of truck traffic as a percentage of the average daily traffic volume was used for all 5 years.

Many of the design alternatives evaluated in this study were intended either to improve the safety of left-turn maneuvers or to restrict left-turn maneuvers at driveways and at minor unsignalized intersections. It would have been highly desirable in the safety analysis to have data on the volume of such left-turn maneuvers for either the peak hour or for a full 24-hour period. However, such data were not available for the study sections because highway agencies do not typically maintain inventory files of left-turn volumes at midblock locations or at minor unsignalized intersections. Two data elements were considered as potential surrogates for midblock left-turn volumes in the safety analysis:

- Driveway density (driveways per mile).
- Level of left-turn demand (light/moderate/heavy) estimated from number of driveways and character of development observed on the photolog.

It should be noted that left-turn volumes were considered explicitly in the traffic operational analysis presented in Appendix D. Results indicating the impact of left-turn volumes on delay to through vehicles are presented in that appendix.

Table A-3. Data elements obtained from cooperating states.

	Computerized Data Files	Photolog	Manual Files
GEOMETRIC AND TRAFFIC CONTROL DATA			
Number of lanes	X	X	
Median treatment (if any)	X	X	
Lane width	X		
Presence of shoulder	X	X	
Shoulder width	X	X	
Presence of curb parking		X	
Speed limit	X	X	
Type of development		X	
Number of driveways		X	
Number of signalized intersections	X	X	
Number of unsignalized intersections	x	x	
TRAFFIC VOLUME DATA			
Average daily traffic (ADT)	X		X
Peak hour volume			X
Truck percentage			X
Estimated level of left-turn demand (light/moderate/heavy)		X	
ACCIDENT DATA (1978-1982)			
Location (milepost)	X		
Date	X		
Time of day	X		
Severity	X		
Number of vehicles involved	X		
Accident type	X		
Manner of collision	X		
Intersection involvement	X		
Turning maneuver involvement	X		
Light condition	X		
Pavement surface condition	X		
Weather	X		

Accident Data

Data on the accident history of each study section were obtained from the computerized accident records system of each cooperating state. The specific accident data elements and the categories used for each are presented in Table A-4.

The accident data include the details of each individual accident occurring on each study section for a 5-year period from 1978 through 1982, inclusive. The history of construction activity on each section during the 5-year study period was reviewed. Where major construction work, such as widening or median contruction, was found during a particular year, the entire calendar year was excluded from the study. There were 730 section-years of accident data available for the 147 study sections in California, indicating that the average length of the study period for each section was 4.97 years. Similarly, there were 1,327 section-years of accident data available for the 273 study sections in Michigan, representing an average study period of 4.86 years. Each section-year of accident data was used as a separate observation in the safety analysis.

Table A-4. Accident data categories.

Accident Severity

- Fatal
- Injury
- Property-Damage-Only

Accident Type

- Collision with another motor vehicle
- Collision with parked vehicle
- Collision with pedestrian
- Collision with bicycle
- Collision with animal
- Collison with fixed object
- Other collision
- Noncollision

Manner of Collision
(multiple vehicle accidents only)

- Head-on
- Rear-end
- Sideswipe-same direction
- Sideswipe-opposite directions
- Angle
- Other

Turn Involvement

- Right-turn
- Left-turn
- U-turn
- No turn involved

Light Condition

- Daylight
- Dusk, dawn
- Dark-street lighted
- Dark-not lighted

Pavement Surface Condition

- Dry
- Wet
- Ice and snow
- Other

Weather

- Clear
- Cloudy
- Raining
- Snowing
- Other

SUMMARY OF SAFETY DATA BASE

Tables A-5 and A-6 summarize the study sections in the safety data base and the traffic exposure and accident history on those sections during the study period.

Table A-5 presents the number of study sections, number of section-years of data, total length of study sections, average ADT, and million vehicle-miles of exposure broken down by design alternative, by type of development (commercial/residential), and by state. There were over 9 billion vehicle-miles of exposure on the study sections during the study period. Nearly one-half of the total exposure occurred on five-lane TWLTL sections. The ADT data show that the average ADT for each design alternative increases with the number of lanes. The ADT for five-lane TWLTL sections is about twice the ADT for two-lane undivided sections.

Table A-6 summarizes the accident experience on the study sections. The accident data are broken down by the same key variables used in Table A-5 and also by severity and intersection involvement. Accidents at signalized intersections, unsignalized intersections, and at nonintersection locations are tabulated separately. In total, the study sections experienced 60,791 accidents. Of these accidents, 16,608 (or 27.3 percent) occurred at or on the approaches to signalized intersections, 22,325 (or 36.7 percent) occurred at or on the approaches to nonsignalized intersections, and 21,858 (or 36.0 percent) occurred at nonintersection locations. The analysis of these accident data is presented in Appendix B.

Table A-5. Summary of study sections and exposure in safety data base.

Design Alternative	Type of Development	State	No. of Study Sections	No. of Section-Years	Average ADT (veh/day)	Total Length (miles)	Total Travel in Study Period (million veh-mile)
Two-lane undivided (2U)	Commercial	California	17	83	10,600	7.32	138.21
		Michigan	18	90	18,032	13.25	436.04
			35	173	15,474	20.57	574.25
	Residential	California	38	190	11,857	20.91	452.47
		Michigan	20	94	12,585	14.66	316.51
			58	284	12,096	35.57	768.98
	2U TOTALS		93	457	13,340	56.14	1,343.23
Three-lane with TWLTL (3T)	Commercial	California	4	19	13,245	1.39	31.92
		Michigan	7	35	15,270	5.32	148.26
			11	54	14,986	6.71	180.18
	Residential	California	4	19	15,226	2.34	61.77
		Michigan	4	20	13,617	3.37	83.75
			8	39	14,323	5.71	145.52
	3T TOTALS		19	93	14,678	12.42	325.70
Four-lane undivided (4U)	Commercial	California	17	85	20,184	9.13	336.31
		Michigan	64	313	18,931	39.04	1,319.35
			81	398	19,165	48.17	1,655,66
	Residential	California	13	65	22,156	7.76	313.77
		Michigan	35	171	15,069	17.33	465.70
			48	236	17,312	25.09	779.47
	4U TOTALS		129	734	18,529	73.26	2,435.13
Four-lane divided (4D)	Commercial	California	21	105	23,217	9.72	411.85
		Michigan	14	70	18,922	6.43	222.04
			35	175	21,507	16.15	633.89
	Residential	California	7	35	22,025	4.66	187.31
		Michigan	2	10	12,789	0.97	22.64
			9	45	20,434	5.63	209.95
	4D TOTALS		44	220	21,229	21.78	843.84
Five-lane with TWLTL (5T)	Commercial	California	24	119	26,445	15.34	734.13
		Michigan	94	455	27,374	65.02	3,144.56
			118	574	27,185	80.36	3,878.69
	Residential	California	2	10	11,938	0.75	16.34
		Michigan	15	69	24,293	10.05	409.92
			17	79	23,269	10.80	426.26
	5T TOTALS		135	153	26,749	91.16	4,304.95
	ENTIRE DATA BASE		420	2,057	20,317	254.76	9,252.85

Table A-6. Summary of accident experience in project data base.

Design Alternative	Type of Development	State	All Accidents F&I	All Accidents PDO	All Accidents Total	Nonintersection Accidents F&I	Nonintersection Accidents PDO	Nonintersection Accidents Total	Unsignalized Intersection Accidents F&I	Unsignalized Intersection Accidents PDO	Unsignalized Intersection Accidents Total	Signalized Intersection Accidents F&I	Signalized Intersection Accidents PDO	Signalized Intersection Accidents Total
Two-lane undivided (2U)	Commercial	California	190	210	400	137	148	285	45	51	96	8	11	19
		Michigan	581	1,029	1,610	280	501	781	262	410	672	39	118	157
			771	1,239	2,010	417	649	1,066	307	461	768	47	129	176
	Residential	California	554	722	1,276	370	412	782	111	181	292	73	129	202
		Michigan	548	1,011	1,539	218	393	611	268	508	776	62	110	172
			1,102	1,733	2,835	588	805	1,393	379	689	1,068	135	239	374
	2U TOTALS		1,873	2,972	4,845	1,005	1,454	2,459	686	1,150	1,836	182	368	550
Three-lane with TWLTL (3T)	Commercial	California	26	50	76	13	28	41	3	4	7	10	18	28
		Michigan	255	555	810	83	198	281	136	282	418	36	75	111
			281	605	886	96	226	322	139	286	425	46	93	139
	Residential	California	77	78	155	56	49	105	15	16	31	6	13	19
		Michigan	124	266	390	28	59	87	86	181	267	10	26	36
			201	344	545	84	108	192	101	197	298	16	39	55
	3T TOTALS		482	949	1,431	180	334	514	240	483	723	62	132	194
Four-lane undivided (4U)	Commercial	California	689	824	1,513	372	383	760	128	202	330	184	239	423
		Michigan	3,622	7,868	11,490	1,215	2,246	3,461	1,866	4,091	5,957	541	1,531	2,072
			4,311	8,692	13,003	1,592	2,629	4,221	1,994	4,293	6,287	725	1,770	2,495
	Residential	California	688	940	1,628	174	239	413	68	91	159	446	610	1,056
		Michigan	964	2,026	2,990	197	367	564	590	1,254	1,844	177	405	582
			1,652	2,966	4,618	371	606	977	658	1,345	2,003	623	1,015	1,638
	4U TOTALS		5,963	11,658	17,621	1,963	3,235	5,198	2,652	5,638	8,290	1,348	2,785	4,133
Four-lane divided (4D)	Commercial	California	839	1,626	2,465	381	637	1,018	105	199	304	353	790	1,143
		Michigan	509	1,379	1,888	166	423	589	228	621	849	115	335	450
			1,348	3,005	4,353	547	1,060	1,607	333	820	1,153	468	1,125	1,593
	Residential	California	177	250	427	107	124	231	24	55	79	46	71	117
		Michigan	46	64	110	5	14	19	41	50	91	0	0	0
			223	314	537	112	138	250	65	105	170	46	71	117
	4D TOTALS		1,571	3,319	4,890	659	1,198	1,857	398	925	1,323	514	1,196	1,710
Five-lane with TWLTL (5T)	Commercial	California	1,127	1,356	2,483	569	678	1,247	221	233	454	337	445	782
		Michigan	8,926	18,800	27,726	3,195	6,850	10,045	2,807	5,877	8,684	2,924	6,073	8,997
			10,053	20,156	30,209	3,764	7,528	11,292	3,028	6,110	9,138	3,261	6,518	9,779
	Residential	California	36	22	58	13	17	30	5	4	9	18	1	19
		Michigan	649	1,088	1,737	200	308	508	268	738	1,006	181	42	273
			685	1,110	1,795	213	325	538	273	742	1,015	199	43	242
	5T TOTALS		10,738	21,266	32,004	3,977	7,853	11,830	3,301	6,852	10,153	3,460	6,561	10,021
	ENTIRE DATA BASE		20,627	40,164	60,791	7,784	14,074	21,858	7,277	15,048	22,325	5,566	11,042	16,608

APPENDIX B

SAFETY ANALYSIS

This appendix describes the analyses that were performed in the project to compare the safety characteristics of multilane design alternatives on suburban highways. The findings presented here are the combined result of formal statistical analyses of the data base documented in Appendix A and less formal interpretations of the data and comparisons between design alternatives. Included in the following discussion are the objectives of the analysis, the variables used in the analysis, the statistical approach adopted, the results obtained from the statistical analysis, and the interpretation of these results.

ANALYSIS OBJECTIVES

The safety analyses performed in the project had three objectives:

1. To quantify the differences in accident rate between multilane design alternatives.
2. To compare the accident rates between similar highway sections with full shoulder conditions and no shoulder conditions.
3. To characterize and compare the distributions of accident severity, accident type, etc., for each design alternative.

ANALYSIS APPROACH

The statistical approach to the safety analysis consists of three elements: (1) measures of effectiveness, or dependent variables; (2) study section characteristics, or independent variables; and (3) statistical techniques to compare the measures of effectiveness between design alternatives while accounting for the effects of the other independent variables. Each of these elements is discussed independently, as follows.

Measures of Effectiveness

The primary measure of effectiveness used in the study is the accident rate per million vehicle-miles, defined as:

$$AR = \frac{(N)(10^6)}{(ADT)(D)(L)}$$

where: AR = accident rate, accidents per million vehicle-miles;
N = number of accidents during study period;
ADT = average daily traffic volume, veh/day;
D = duration of study period, days (based on 365 days per year); and
L = length of study section, miles.

The accident rate in this form is an appropriate measure of effectiveness for the study sections in the safety data base which vary in both length (ranging from 0.25 to 4.50 miles) and traffic volume (ranging from 7,000 to 25,000 vpd). These variations in length and traffic volume represent variations between the study sections in the exposure to or risk of an accident.

The total accident rate for a section of suburban arterial highway was subdivided in two ways in the study: by accident severity level and by relationship to intersection.

Accidents are classified by severity level as fatal, injury, or property-damage-only (PDO) accidents based on the most severe injury to any individual involved in the accident. The accident rate per million vehicle-miles for fatal and injury accidents and for property-damage-only accidents were analyzed in the study. Fatal and injury accidents were considered together because the relative frequency of fatal accidents is usually too small to obtain statistically valid results. The accident experience of the study sections is classified by severity level in Table A-6.

Another key variable in the safety assessment of multilane design alternatives is the relationship of accidents to intersections. Three types of accidents that occur on suburban arterial streets merit separate consideration: nonintersection accidents, unsignalized intersection accidents, and signalized intersection accidents. The accident experience of the study sections is classified by relationship to intersection in Table A-6.

Nonintersection accidents constitute approximately 36 percent of the accidents on the suburban arterial highways in the safety data base. Nonintersection accidents may include collisions involving vehicles entering or leaving driveways; rear-end sideswipe and head-on accidents between vehicles traveling along the arterial street; single-vehicle accidents involving collisions with pedestrians, bicycles, animals, and fixed objects; and single-vehicle noncollision accidents. For the design alternatives studied, driveway accidents constitute 30 to 50 percent of all nonintersection accidents.

Accidents related to unsignalized intersections constitute approximately 37 percent of the accidents on the study sections. These accidents were identified in the data base using the criteria of the cooperating states, which include all accidents occurring within the intersection limits and accidents on the approaches that are classified related to the intersection. Intersection-related accidents on the approaches are generally within 150 to 250 ft of the intersection, but could be located farther away if the intersection was clearly the cause of the accident.

Accidents related to signalized intersections constitute approximately 27 percent of the accidents on the study sections. For purposes of this study, all accidents within the limits of a signalized intersection or within 250 ft of a signalized intersection were treated as related to the intersection.

The multilane design alternatives evaluated in this study, which include the addition of raised medians or TWLTLs to undivided arterials, have a direct impact on the accident experience between intersections and at unsignalized intersections. In the case of nonintersection accidents, median dividers and two-way left-turn lanes can reduce driveway-related accidents by eliminating left-turns at driveways or separating vehicles waiting to turn left from through traffic; they also increase the separation between vehicles traveling in opposite directions and, thus, reduce the potential for head-on and sideswipe accidents. Median dividers and two-way left-turn lanes also provide similar benefits at unsignalized intersections, which typically do not have separate left-turn lanes on undivided arterials. However, median dividers and two-way left-turn lanes often have no direct impact on the operation of signalized intersections, which often have intersection channelization, separate left-turn lanes, or separate left-turn signal phases even on undivided arterials.

A decision to exclude signalized intersection accidents from the safety analysis was reached because multilane design alternatives have their primary impact on accidents that occur between intersections and at unsignalized intersections. Installation of a multilane design alternative on a suburban arterial highway may have no impact on the operation of a signalized intersection, especially if the basic number of through lanes is unchanged, because signalized intersections often have intersection channelization, separate left-turn lane, and/or separate left-turn phases even on undivided arterials. Furthermore, signalized intersections vary much more in geometrics and cross-traffic volumes than unsignalized intersections so that evaluation of their safety performance would require an intersection-oriented, rather than a section-oriented, safety analysis that was regarded as beyond the scope of the study. Thus, the study was limited to consideration of safety at nonintersection and unsignalized intersection locations. In the event that construction of a multilane design alternative does improve the geometrics of a signalized intersection, the safety benefits of that improvement should be considered in addition to those predicted in this appendix.

Independent Variables

Eight study section characteristics were used as independent variables in the safety analysis. Each of these variables has from two to four levels that were investigated in the safety analysis. These variables, together with the levels used for each, are:

INDEPENDENT VARIABLE	LEVELS
Average daily traffic volume (ADT)	7,000 to 10,000 vpd 10,000 to 15,000 vpd 15,000 to 20,000 vpd Over 20,000 vpd
Truck percentage	Under 5 percent 5 to 10 percent Over 10 percent
Type of development	Commercial Residential
Estimated level of left-turn demand	Light Moderate/heavy
Shoulder width	No shoulder (0 to 4 ft) Full shoulder (8 ft and over)
Speed	Low (35 to 40 mph) High (45 to 50 mph)
Driveways per mile	Under 30 per mile 30 to 60 per mile Over 60 per mile
Unsignalized intersections per mile	Under 5 per mile 5 to 10 per mile Over 10 per mile

It should be noted that four of the eight variables—average daily traffic volume, truck percentage, driveways per mile, and unsignalized intersections per mile—are by nature continuous rather than categorized variables. These variables were used in the statistical analyses both as continuous and as categorical variables.

Preliminary Review of Accident Rates

A preliminary review of the accident rate data illustrates a first cut at the comparison of accident rates between design alternatives required by the analysis objectives. Table B-1 illustrates the rates for nonintersection accidents and unsignalized intersection accidents. This table is based directly on the accident and exposure data in Tables A-5 and A-6 and is broken down by design alternative, type of development, and state.

The raw accident rates in Table B-1 should be interpreted cautiously because any apparent differences between design alternatives, development types, or states could be caused by variables whose effects are not included in the table such as traffic volume, driveway density, etc. For example, it appears from the data in Table B-1 that five-lane TWLTL sections with commercial development in Michigan have higher nonintersection accident rates than either four-lane undivided or four-lane divided sections. In fact, just the opposite was found to be the case, after accounting for the greater driveway densities found on five-lane TWLTL sections.

Despite the need for cautious interpretation, several interesting observations can be drawn from Table B-1. First, the nonintersection accident rates for residential street sections appear to be generally lower, but more variable, than for commercial sections. This variability may be attributable to the fact that some residential sections are completely residential, while others include commercial development at scattered locations.

Second, a review of the nonintersection accident rates shows relatively good agreement between the California and Michigan data, which provides some assurance that the data from the two states represent comparable conditions and can be combined.

In contrast, the accident rates for unsignalized intersections in California are much lower than the rates in Michigan. Possible differences between the California and Michigan sections that could explain this difference in accident rate were examined, including the possibility that there were more unsignalized intersections per unit length on the Michigan sections than on the California sections or that the Michigan intersections carried higher traffic volumes. However, it was found that neither of these factors could explain the observed difference between the states. Possible differences in the accident records systems of the two states that could explain why fewer accidents were included at California intersections than at Michigan intersections were examined but no valid explanation was found. Both states have similar accident reporting criteria and both states classify intersection accidents in a similar manner (both accidents involving vehicles on the arterial and on the side street are included in the data base, even if the side street is not under state jurisdiction). However, in the opinion of the author, it is possible that intersection-related accidents on side streets not under state jurisdiction in some municipalities in California are not reported or included in the state accident records system as consistently as they are in Michigan.

Because the California data for unsignalized intersection ac-

Table B-1. Unadjusted accident rates classified by design alternative, type of development, and state.

Design Alternative	Type of Development	State	Accident Rate (accidents per million vehicle-miles) Nonintersection Accidents	Unsignalized Intersection Accidents
Two-lane undivided (2U)	Commercial	California	2.06	0.69
		Michigan	1.79	1.54
	Residential	California	1.73	0.64
		Michigan	1.93	2.45
Three-lane with TWLTL (3T)	Commercial	California	1.28	0.22
		Michigan	1.89	2.82
	Residential	California	1.70	0.50
		Michigan	1.04	3.19
Four-lane undivided (4U)	Commercial	California	2.26	0.98
		Michigan	2.62	4.52
	Residential	California	1.32	0.51
		Michigan	1.21	3.96
Four-lane divided (4D)	Commercial	California	2.47	0.74
		Michigan	2.65	3.82
	Residential	California	1.23	0.42
		Michigan	0.84	4.02
Five-lane with TWLTL (5T)	Commercial	California	1.70	0.61
		Michigan	3.19	2.79
	Residential	California	1.84	0.55
		Michigan	1.24	2.45

cident rates appeared extremely low, they were checked against some statewide summary data for urban unsignalized intersections on highways under state jurisdiction published by the California Department of Transportation (*23*). Although the urban classification used in the statewide summary data represents a more inclusive category than the suburban conditions defined in this study, it was expected that the accident experience should be similar. The average urban unsignalized intersection with STOP or YIELD sign control on a route under state jurisdiction in California experienced an accident rate of 0.46 accidents per million entering vehicles in 1980 to 1982 (*23*). Using an estimated ADT of 18,000 vpd for the arterial street and a conservative estimate of 1,000 vpd for the side street to estimate the total entering volume, this statewide rate corresponds to an expected accident frequency of 3.2 accidents per unsignalized intersection per year. The project data base includes 0.75 accidents per unsignalized intersection per year in California and 3.6 accidents per unsignalized intersection per year in Michigan. Thus, the complete statewide data for California is in far better agreement with the accident rates for the Michigan sections in the data base than the accident rates for California sections. For this reason, it was decided to base the estimates of unsignalized intersection accident rates solely on the Michigan data.

Statistical Approach

A statistical analysis of the difference in accident rate between multilane design alternatives was conducted. The effect on accident rate of the following independent variables was considered in the analysis, where appropriate:

- State
- Design alternative
- Average daily traffic volume
- Truck percentage
- Type of development
- Estimated level of left-turn demand
- Lane width
- Shoulder width
- Speed
- Driveways per mile
- Unsignalized intersections per mile

Separate analyses were conducted for nonintersection accident rates and unsignalized intersection accident rates. The state in which each study section was located was considered only in the analysis of the nonintersection accident rates, because the analysis of unsignalized intersection accident rates was based on the Michigan data alone. Shoulder width was also considered only in the analysis of nonintersection accidents, because the available shoulder width data were for locations between intersections and because shoulder widths are likely to vary at intersections based on the geometrics of individual intersections. The effect of driveways per mile was considered in the analysis of nonintersection accidents only, and the effect of intersections per mile was considered in the analysis of intersection accidents only.

The statistical analysis used a hierarchical analysis of covariance approach. Analysis of covariance is a statistical technique used to assess the effects of both independent variables with two or more discrete levels (known as factors) and independent variables with values on a continuous scale (known as covariates). Many analyses of covariance models were tried during the analysis, but those that proved most useful included state, design alternative, type of development, estimated level of left-turn demand, shoulder width, and speed as factors and average daily traffic volume, truck percentage, driveways per mile, and unsignalized intersections per mile as covariates.

The specific form of analysis of covariance that was used was a hierarchical analysis of covariance, in which the effects of the independent variables are accounted for in sequence, so that a factor or covariate is important only if it explains a significant proportion of the variance remaining after the variables considered previously have been accounted for.

In an analysis of variance or covariance with a balanced design, the best measure of effectiveness for each design alternative is simply the average (or arithmetic mean) accident rate for that alternative. The experiment designs used for this study were not balanced, however, because the sample sizes in the cells defined by the experimental factors were not equal and the covariates did not have the same mean in every cell. In such an unbalanced design, the best measure of effectiveness for each design alternative is the least square mean for that alternative. The least square mean compensates for the differences between the cells in sample sizes and covariate means. The least square mean is, in effect, the mean accident rate that would result if every cell had the same sample size and the same mean for every covariate.

All of the statistical analyses in this project were performed using the Statistical Analysis System (SAS) (*24*). The analyses of covariance, in particular, were performed using the SAS General Linear Model (GLM) procedure.

ANALYSIS RESULTS

The results of the statistical analysis indicated that many of the independent variables considered had statistically significant relationships to accident rate in some circumstances, but not in others, depending on which other variables are included in the model and the order in which those variables were combined. This situation arises because most of the independent variables are strongly correlated with one another and, in some cases, one variable may serve partially as a surrogate for the effect of another. Many different models were tried and those presented here, in the judgment of the investigators, best represent the actual effects of each factor and best serve the project objectives. However, given the complexities of the data base and the interrelationships between the geometrics and traffic variables in the study, other investigators could reach different conclusions about which variables to include in the final models.

Nonintersection Accident Rates

Table B-2 presents the results of an analysis of covariance illustrating the effects of seven independent variables or interactions found to have a statistically significant relationship to nonintersection accident rate. The factors found to be statistically significant in this model are state (California/Michigan), design alternative, type of development (commercial/residential), and shoulder width. The interaction between design alternative and type of development was also found to be significant, meaning that each combination of design alternative and development has a unique effect on nonintersection accident rate. The covariates found to be statistically significant in this model are driveway density and truck percentage.

The model represented by Table B-2 contains data for only four design alternatives (2U, 4U, 4D, and 5T) because not enough data were available for three-lane TWLTL (3T) sections to include in this particular analysis. A separate analysis conducted to compare 2U and 3T sections is illustrated by Table B-3. The shoulder width factor and the design alternative-development interaction have been omitted because they were not statistically significant in this analysis.

Before documenting the influence on accident rates of each of the statistically significant variables included in the foregoing models, a few words are appropriate about variables that were considered in the variables considered but found to be not significant. Average daily traffic volume, lane width, and speed were omitted from the models shown in Tables B-2 and B-3 because they were not statistically significant. Lane width and speed were not significant in any of the models tried. This result is consistent with the results of a similar study recently conducted by Walton et al. (*5*), who were unable to find a relationship between accident rate and lane width or speed limit on highways with 5T and 5C design alternatives. ADT would be significant if included in the model prior to design alternative and type of development, because the latter factors have a strong correlation to ADT. However, to attain the project objectives, it was considered to be more important to include design alternative, rather than ADT, in the model. The lack of a significant relationship between accident rate and ADT should not be interpreted to mean that accidents do not increase with increasing traffic volume; rather, because the measure of effectiveness is an accident *rate*, this indicates that accident frequency increases proportionally with ADT. Estimated level of left-turn demand (light/moderate/heavy) does have a significant relationship to nonintersection accident rate and could be used in the model as an alternative to driveways per mile. It was decided to retain driveway density, rather than estimated left-turn demand, in the model because it can be determined more easily and is less subject to variations in judgment between observers.

Table B-4 presents the least square mean nonintersection accident rates taken from the models in Tables B-2 and B-3. The accident rates in Table B-4 are averages that give equal weight to the data from California and Michigan. A basic nonintersection accident rate is indicated for each combination of design alternative and type of development. Adjustment factors that are added to the basic rates to account for different levels of driveway density and truck percentage are also indicated in the table. It is likely that these adjustment factors vary for different design alternatives and development types, but there is no statistical evidence that this is the case. Truck percentage was found to be inversely related to accident rate (i.e., accident rate decreases with increasing truck percentage, and vice versa). This result was unexpected and is possibly the result of correlations between truck volumes and other geometric or development variables. The results presented in Table B-4 provide a valid method of predicting the accident rates of suburban highway sections, but users should be cautious about interpreting the individual adjustment factors as precise measures of the incremental effects of those factors. The individual adjustment factors may arise partly from correlations with other variables and cannot necessarily be interpreted as representing cause and effect relationships.

The data in Table B-4 can be used to estimate the nonintersection accident rate for any arterial highway segment whose design alternative, type of development, driveway density, and truck percentage are specified. For example, a two-lane undivided arterial with commercial development, 45 driveways per mile and 6 percent trucks would be expected to have a nonintersection accident rate of $2.39-0.03-0.07=2.29$ accidents per

Table B-2. Analysis of covariance of nonintersection accident rate for 2U, 4U, 4D, and 5T sections.

Dependent Variable: Nonintersection accident rate
Data Base: California and Michigan data for 2U, 4U, 4D and 5T sections
Sample Size: N=1964

Source of Variation		Sum of Squares	Degrees of Freedom	Mean Square	F	Significance (at 95% confidence level)
STATE	(Factor)	11,071.91	2	5,535.96	1,112.66	SIG
ALT	(Factor)	114.80	3	38.27	7.69	SIG
DEV	(Factor)	586.99	1	586.99	117.98	SIG
ALT*DEV	(Interaction)	138.13	3	46.04	9.25	SIG
SW	(Factor)	22.77	1	22.77	4.58	SIG
DDEN	(Covariate)	173.43	1	173.43	34.86	SIG
TP	(Covariate)	54.72	1	54.72	11.00	SIG
Explained		12,162.75	12	1,013.56	203.71	SIG
Error		9,712.05	1952	4.98		(R^2=0.56)
TOTAL		21,874.80	1964			

ALT = Design alternative
DEV = Type of development
SW = Shoulder width
DDEN = Driveways per mile
TP = Truck percentage

Table B-3. Analysis of covariance of nonintersection accident rate for 2U and 3T sections.

Dependent Variable: Nonintersection accident rate
Data Base: California and Michigan data for 2U, and 3T sections
Sample Size: N=550

Source of Variation		Sum of Squares	Degrees of Freedom	Mean Square	F	Significance (at 95% confidence level)
STATE	(Factor)	1,930.63	2	965.32	432.86	SIG
ALT	(Factor)	10.60	1	10.60	4.75	SIG
DEV	(Factor)	12.63	1	12.63	5.66	SIG
DDEN	(Covariate)	29.47	1	29.47	13.22	SIG
TP	(Covariate)	33.67	1	33.67	15.10	SIG
Explained		2,017.00	6	336.17	150.74	SIG
Error		1,213.17	544	2.23		(R^2=0.56)
TOTAL		3,230.17	550			

ALT = Design alternative
DEV = Type of development
DDEN = Driveways per mile
TP = Truck percentage

million vehicle-miles. On a highway with an average daily traffic volume of 12,000 vpd, this corresponds to 10.0 accidents per mile per year.

Table B-4 shows that suburban arterial highway sections with residential development generally have lower nonintersection accident rates than sections with commercial development. On commercial facilities, 3T sections have lower nonintersection accident rates than 2U sections, and 5T sections have lower rates than either 4U or 4D sections. The pattern of variations in nonintersection accident rates for residential sections is less regular for 4U, 4D, and 5T sections. Nonintersection accident rates generally were found to increase with the density of driveways per mile but to decrease with increasing truck percentage.

The effect of the shoulder width factor, which indicates a difference between arterials with no shoulder and with a full shoulder, was found to be statistically significant in Table B-2. The results of the analysis indicate that the nonintersection accident rates for arterials with a full shoulder are about 5 percent lower than the rates shown in Table B-4 and the rates for arterials with no shoulder are about 5 percent higher than the table values. However, this result may not apply to 3T sections, as the shoulder width factor was not statistically significant when included in the model presented in Table B-3.

Table B-4. Least square mean rates and adjustment factors for nonintersection accidents.

BASIC ACCIDENT RATES
(accidents per million vehicle miles)

Type of Development	Design Alternative 2U	3T	4U	4D	5T
Commercial	2.39	1.56	2.85	2.90	2.69
Residential	1.88	1.64	0.97	1.39	1.39

ADJUSTMENT FACTORS

	Under 30	30-60	Over 60
Driveways per mile	-0.41	-0.03	+0.35
	Under 5%	5-10%	Over 10%
Truck percentage	+0.18	-0.07	-0.33

Tests of statistical contrasts were made to compare the full-shoulder and no-shoulder conditions for individual highway types (2U, 4U, 4D, and 5T). The purpose of the statistical contrast texts was to determine if the provision of a full shoulder was more effective for some highway types than for others. However, none of these contrasts for individual highway types was found to be statistically significant. Thus, it was not possible to conclude that the difference between the no-shoulder condition and the full-shoulder condition was higher or lower than 5 percent for any particular design alternative.

Tables B-5 through B-7 present results for unsignalized intersection accident rates for the study sections analogous to those presented above for nonintersection accident rates.

Table B-5 presents the results of an analysis of covariance of the accident rates for 2U, 4U, 4D, and 5T sections analogous to Table B-2. The analysis of unsignalized intersection accident rates incorporates unsignalized intersections per mile rather than driveways per mile, and does not include shoulder width as an independent variable. The analysis of covariance results presented in Table B-5 is similar to the model in Table B-2 except that the effect of truck percentage is statistically significant only at the 90 percent confidence level.

Table B-5. Analysis of covariance of unsignalized intersection accident rate for 2U, 4U, 4D, and 5T sections.

Dependent Variable: Unsignalized intersection accident rate
Data Base: Michigan data for 2U, 4U, 4D and 5T sections
Sample Size: N=1272

Source of Variation		Sum of Squares	Degrees of Freedom	Mean Square	F	Significance (at 95% confidence level)
ALT	(Factor)	16,528.96	4	4,132.24	500.28	SIG
DEV	(Factor)	32.95	1	32.95	3.99	SIG
ALT*DEV	(Interaction)	132.50	3	44.17	5.35	SIG
UINTPM	(Covariate)	1,789.90	1	1,789.90	216.70	SIG
TP	(Covariate)	28.40	1	28.40	3.44	NS[a]
Explained		18,512.71	10	1,851.27	224.13	SIG
Error		10,423.93	1262	8.26		(R^2=0.64)
TOTAL		28,936.64	1272			

[a] Statistically significant at 90% confidence level.

ALT = Design alternative
DEV = Type of development
UINTPM = Unsignalized intersections per mile
TP = Truck percentage

Table B-6. Analysis of covariance of unsignalized intersection accident rate for 2U and 3T sections.

Dependent Variable: Unsignalized intersection accident rate
Data Base: Michigan data for 2U and 3T sections
Sample Size: N=239

Source of Variation		Sum of Squares	Degrees of Freedom	Mean Square	F	Significance (at 95% confidence level)
ALT	(Factor)	1,680.59	2	840.30	250.61	SIG
DEV	(Factor)	81.10	1	81.10	24.19	SIG
UINTPM	(Covariate)	390.40	1	390.40	116.43	SIG
TP	(Covariate)	10.89	1	10.89	3.25	NS[a]
Explained		2,162.99	5	432.60	129.02	SIG
Error		784.61	234	3.35		(R^2=0.73)
TOTAL		2,947.60	239			

[a] Statistically significant at 90% confidence level.

ALT = Design alternative
DEV = Type of development
UINTPM = Unsignalized intersections per mile
TP = Truck percentage

Table B-7. Least square mean rates and adjustment factors for unsignalized intersection accidents.

BASIC ACCIDENT RATES
(accidents per million vehicle miles)

Type of Development	Design Alternative 2U	3T	4U	4D	5T
Commercial	2.11	2.43	4.77	4.71	3.11
Residential	2.88	1.91	3.03	2.71	1.85

ADJUSTMENT FACTORS

	Under 5	5-10	Over 10
Intersections per mile	-0.99	-0.28	+1.55

	Under 5%	5-10%	Over 10%
Truck percentage	+0.22	-0.08	-0.38

Table B-8. Least square mean rates and adjustment factors for suburban arterial highways (including nonintersection and unsignalized intersection accidents).

BASIC ACCIDENT RATES
(accidents per million vehicle-miles)

Type of Development	Design Alternative 2U	3T	4U	4D	5T
Commercial	4.50	3.99	7.62	7.61	5.80
Residential	4.76	3.55	4.00	4.10	3.24

ADJUSTMENT FACTORS

	Under 30	30-60	Over 60
Driveways per mile	-0.41	-0.03	+0.35

	Under 5	5-10	Over 10
Intersections per mile	-0.99	+0.28	+1.55

	Under 5%	5-10%	Over 10%
Truck percentage	+0.40	-0.15	-0.71

In a similar approach to the analysis of nonintersection accident rates, Table B-6 presents analysis of covariance of highway sections with the 2U and 3T design alternatives. These results are also analogous to the results presented in Table B-3, except that the truck percentage factor is statistically significant only at the 90 percent confidence level.

The results of the analysis of unsignalized intersection accident rates are summarized in Table B-7. The table presents basic accident rates for each design alternative and type of development and adjustment factors to account for the effects of intersections per mile and truck percentage.

Table B-8 summarizes the combined accident rates for both nonintersection and unsignalized intersection accidents. In a manner analogous to Tables B-4 and B-7, the table presents basic accident rates for individual design alternatives and types of development and adjustment factors for driveways per mile, unsignalized intersections per mile, and truck percentage. The results presented in Table B-8 indicate that the 3T alternative has accident rates that are 11 to 25 percent lower than the 2U design alternative. Highway sections with the 4U and 4D design alternative have very similar accident rates, while the 5T design alternative has accident rates that are 21 to 24 percent lower.

Accident Severities

The raw accident severity distributions from the project data base are given in Table B-9. The total accident experience for the study sections is broken down in the table by accident severity (fatal and injury vs. property-damage-only), by accident location (nonintersection vs. unsignalized intersection), by design alternative (2U/3T/4U/4D/5T), and by type of development (commercial/residential). For each combination of accident location and type of development, difference of proportions tests were performed to compare the differences between the design alternatives in the relative number of accidents involving a fatality or an injury. The results of these tests are indicated in the table.

The accident severity data were pooled to provide a single severity estimate for design alternatives found to be not significantly different in the percentage of fatal and injury accidents. These pooled estimates, given in Table B-10, can be used in conjunction with the accident rate estimates to determine the expected frequency of fatal and injury accidents and property-damage-only accidents on a suburban highway section.

Accident Types

There are three general types of accidents susceptible to correction through the installation of multilane design alternatives. These are multiple-vehicle head-on, rear-end, and angle collisions. Table B-11 indicates the relative frequency of these accident types on the suburban arterial highways in the safety data base, classified by design alternative and type of development.

Direct comparisons between the design alternatives in Table B-11 may be misleading because there is no formal statistical control for the fact that design alternatives with higher turning volumes are more likely to have higher rates for these accident types. Nevertheless, it is informative to note that the percentage of accidents susceptible to correction by installation of a multilane design alternative typically ranges from 40 to 65 percent of total accidents. Where these types of accidents are relatively more frequent, the percentage reduction in accident rate may be relatively higher. The table also demonstrates the predominance of rear-end accidents at nonintersection locations and angle accidents at unsignalized intersections.

No attempt has been made to break down the accident experience by the type of turning maneuver involved (left-turn, right-turn, U-turn, or crossing maneuver) or by driveway involvement, since examination of the raw data suggests that these classifications are not applied consistently in the accident reporting and coding process.

Table B-9. Comparison of accident severity distributions.

Design Alternative	Total Accidents	Fatal Injury Accidents (number (%))	Property-Damage -Only Accidents (number (%))	Difference of Proportions Test: Significantly Different From	Difference of Proportions Test: Not Significantly Different From
NONINTERSECTION ACCIDENTS--COMMERCIAL DEVELOPMENT (California and Michigan sections)					
2U	1,066	417 (39.1)	649 (60.9)	3T, 4D, 5T	4U
3T	322	96 (29.9)	226 (70.1)	2U, 4U, 4D, 5T	-
4U	4,221	1,592 (37.7)	2,629 (62.3)	3T, 4D, 5T	2U
4D	1,607	547 (34.0)	1,060 (66.0)	2U, 3T, 4U	5T
5T	11,292	3,764 (33.3)	7,528 (66.7)	2U, 3T, 4U	4D
NONINTERSECTION ACCIDENTS--RESIDENTIAL DEVELOPMENT (California and Michigan sections)					
2U	1,393	588 (42.2)	805 (57.8)	4U, 5T	3T, 4D
3T	192	84 (43.7)	108 (56.3)	4U, 5T	2U, 4D
4U	977	371 (38.0)	606 (62.0)	2U, 3T, 4D	5T
4D	250	112 (44.8)	138 (55.2)	4U, 5T	2U, 3T
5T	538	213 (39.6)	325 (60.4)	2U, 3T, 4D	4U
UNSIGNALIZED INTERSECTION ACCIDENTS--COMMERCIAL DEVELOPMENT (Michigan sections only)					
2U	672	262 (39.0)	410 (61.0)	3T, 4U, 4D, 5T	-
3T	418	136 (32.5)	282 (67.5)	2U, 4D	4U, 5T
4U	1,844	590 (31.4)	1,254 (68.6)	2U, 4D	3T, 5T
4D	849	228 (26.9)	621 (73.1)	2U, 3T, 4U, 5T	-
5T	8,684	2,807 (32.3)	5,877 (67.7)	2U, 4D	3T, 4U
UNSIGNALIZED INTERSECTION ACCIDENTS--RESIDENTIAL DEVELOPMENT (Michigan sections only)					
2U	776	268 (34.6)	508 (65.4)	4D, 5T	3T, 4U
3T	267	86 (32.2)	181 (67.8)	4D, 5T	2U, 4U
4U	1,844	590 (32.0)	1,254 (68.0)	4D, 5T	2U, 3T
4D	91	41 (45.1)	50 (54.9)	2U, 3T, 4U, 5T	-
5T	1,006	268 (26.6)	738 (73.4)	2U, 3T, 4U, 4D	-

Table B-10. Accident severity distribution for suburban arterial highways.

Design Alternative	Percentage of Accidents Involving a Fatality or Injury: Nonintersection Accidents: Commercial	Nonintersection Accidents: Residential	Unsignalized Intersection Accidents: Commercial	Unsignalized Intersection Accidents: Residential
2U	38.4	43.6	39.0	32.9
3T	29.9	43.6	32.1	32.9
4U	38.4	38.8	32.1	32.9
4D	33.7	43.6	26.9	45.1
5T	33.7	38.8	32.1	26.6

Table B-11. Relative frequency of accident types susceptible to correction on suburban arterial highways.

Design Alternative	Type of Development	Number of Accidents (percent of total accidents): Head-on[a]	Rear-End[b]	Angle	Total Susceptible to Correction	Total
		NONINTERSECTION ACCIDENTS				
2U	Commercial	96 (9.0)	386 (36.2)	57 (5.3)	539 (50.5)	1,066
	Residential	94 (6.7)	479 (34.4)	44 (3.2)	617 (44.3)	1,393
3T	Commercial	23 (7.1)	95 (29.5)	27 (8.4)	145 (45.0)	322
	Residential	12 (6.2)	79 (41.1)	4 (2.1)	95 (49.4)	192
4U	Commercial	216 (5.1)	1,575 (37.3)	142 (3.4)	1,933 (45.8)	4,221
	Residential	57 (5.8)	415 (42.5)	32 (3.3)	504 (51.6)	977
4D	Commercial	23 (1.4)	829 (51.6)	90 (5.6)	942 (58.6)	1,607
	Residential	5 (2.0)	99 (39.6)	4 (1.6)	108 (43.2)	250
5T	Commercial	436 (3.9)	4,668 (41.3)	596 (5.2)	5,700 (50.5)	11,292
	Residential	86 (16.0)	216 (40.1)	21 (3.9)	323 (60.0)	538
		UNSIGNALIZED INTERSECTION ACCIDENTS				
2U	Commercial	28 (3.6)	187 (24.3)	214 (27.8)	429 (55.9)	768
	Residential	34 (3.2)	229 (21.4)	276 (25.8)	539 (50.5)	1,068
3T	Commercial	13 (3.1)	132 (31.1)	132 (31.1)	277 (65.2)	425
	Residential	4 (1.3)	74 (24.8)	91 (30.5)	169 (56.7)	298
4U	Commercial	163 (2.3)	1,950 (31.0)	1,975 (31.4)	4,088 (65.0)	6,287
	Residential	70 (3.5)	640 (32.0)	562 (28.1)	1,272 (63.5)	2,003
4D	Commercial	12 (1.0)	279 (24.2)	347 (30.0)	638 (55.3)	1,153
	Residential	1 (0.6)	13 (7.6)	58 (34.1)	72 (42.4)	170
5T	Commercial	307 (3.4)	912 (10.0)	2,854 (31.2)	4,073 (44.6)	9,138
	Residential	42 (4.1)	293 (28.9)	223 (22.0)	558 (55.0)	1,015

[a] Including opposite direction sideswipe accidents.
[b] Including same direction sideswipe accidents.

APPENDIX C

TYPICAL ACCIDENT FREQUENCIES FOR SUBURBAN ARTERIAL HIGHWAYS

Table C-1 presents typical accident frequencies per mile per year for suburban arterial highways under various geometric and traffic conditions and types of development. Six parameters are needed to look up an estimated accident frequency for any particular highway segment:

- Design alternative
- Type of development (commercial/residential)
- Average daily traffic volume (ADT)
- Driveways per mile
- Unsignalized intersections per mile
- Truck percentage

Table C-1 provides estimates of the frequency of nonintersection accidents (labeled NINT in the table) and unsignalized intersection accidents (UINT) per mile per year and the frequency for both of these accident categories combined (TOT). The accident frequencies in Table C-1 are based directly on the accident rates developed in this study and presented in Tables B-4, B-7, and B-8.

The safety picture of suburban arterials is incomplete without consideration of signalized intersections. Table C-2 gives typical annual accident frequencies for signalized intersections on suburban arterials based on a predictive equation developed by Webb (*25*). This equation was based on approach speeds of 25 to 45 mph, which Webb called the semiurban group. The equation he developed to express the relationship between traffic volume and accidents was:

$$N = 0.17\, X^{0.45}\, Y^{0.38}$$

where: N = annual number of accidents;
X = ADT of major highway (hundreds of vehicles per day); and
Y = ADT of crossroad (hundreds of vehicles per day).

It should be noted that the tables in this appendix present average values of accident frequency and that the actual accident experience for particular locations can vary widely from these averages.

Table C-1. Estimated accident frequency per mile per year on suburban arterial highways.

Design Alternative	Type of Development	Driveways Per Mile	Average Daily Traffic Volume (veh/day) 7,000-10,000 NINT	UINT	TOT	10,000-15,000 NINT	UINT	TOT	15,000-20,000 NINT	UINT	TOT	Over 20,000 NINT	UINT	TOT
			TRUCK PERCENTAGE: UNDER 5% INTERSECTIONS PER MILE: UNDER 5											
2U	C	Under 30	5.9	3.7	9.6	9.9	6.1	16.0	13.8	8.6	22.4	17.7	11.0	28.7
		30-60	7.0	3.7	10.7	11.6	6.1	17.7	16.2	8.6	24.8	20.9	11.0	31.9
		Over 60	8.0	3.7	11.7	13.3	6.1	19.4	18.7	8.6	27.3	24.0	11.0	35.0
2U	R	Under 30	4.5	5.8	10.3	7.5	9.6	17.1	10.5	13.5	24.0	13.6	17.3	30.9
		30-60	5.6	5.8	11.4	9.3	9.6	18.9	13.0	13.5	26.5	16.7	17.3	34.0
		Over 60	6.6	5.8	12.4	11.0	9.6	20.6	15.4	13.5	28.9	19.8	17.3	37.1
3T	C	Under 30	3.6	4.5	8.1	6.1	7.6	13.7	8.5	10.6	19.1	10.9	13.6	24.5
		30-60	4.7	4.5	9.2	7.8	7.6	15.4	10.9	10.6	21.5	14.0	13.6	27.6
		Over 60	5.7	4.5	10.2	9.5	7.6	17.1	13.3	10.6	23.9	17.2	13.6	30.8
3T	R	Under 30	3.9	3.1	7.0	6.4	5.2	11.6	9.0	7.3	16.3	11.6	9.4	21.0
		30-60	4.9	3.1	8.0	8.2	5.2	13.4	11.4	7.3	18.7	14.7	9.4	24.1
		Over 60	5.9	3.1	9.0	9.9	5.2	15.1	13.9	7.3	21.2	17.8	9.4	27.2
4U	C	Under 30	7.2	11.0	18.2	12.0	18.3	30.3	16.7	25.6	42.3	21.5	32.9	54.4
		30-60	8.2	11.0	19.2	13.7	18.3	32.0	19.2	25.6	44.8	24.6	32.9	57.5
		Over 60	9.3	11.0	20.3	15.4	18.3	33.7	21.6	25.6	47.2	27.8	32.9	60.7
4U	R	Under 30	2.0	6.2	8.2	3.4	10.3	13.7	4.7	14.4	19.1	6.1	18.6	24.7
		30-60	3.1	6.2	9.3	5.1	10.3	15.4	7.2	14.4	21.6	9.2	18.6	27.8
		Over 60	4.1	6.2	10.3	6.8	10.3	17.1	9.6	14.4	24.0	12.3	18.6	30.9
4D	C	Under 30	7.3	10.8	18.1	12.2	18.0	30.2	17.1	25.2	42.3	21.9	32.4	54.3
		30-60	8.3	10.8	19.1	13.9	18.0	31.9	19.5	25.2	44.7	25.0	32.4	57.4
		Over 60	9.4	10.8	20.2	15.6	18.0	33.6	21.9	25.2	47.1	28.2	32.4	60.6
4D	R	Under 30	3.2	5.3	8.5	5.3	8.9	14.2	7.4	12.4	19.8	9.5	15.9	25.4
		30-60	4.2	5.3	9.5	7.0	8.9	15.9	9.8	12.4	22.2	12.6	15.9	28.5
		Over 60	5.3	5.3	10.6	8.8	8.9	17.7	12.3	12.4	24.7	15.8	15.9	31.7
5T	C	Under 30	6.7	6.4	13.1	11.2	10.7	21.9	15.7	14.9	30.6	20.2	19.2	39.4
		30-60	7.8	6.4	14.2	13.0	10.7	23.7	18.1	14.9	33.0	23.3	19.2	42.5
		Over 60	8.8	6.4	15.2	14.7	10.7	25.4	20.6	14.9	35.5	26.4	19.2	45.6
5T	R	Under 30	3.2	3.0	6.2	5.3	4.9	10.2	7.4	6.9	14.3	9.5	8.9	18.4
		30-60	4.2	3.0	7.2	7.0	4.9	11.9	9.8	6.9	16.7	12.6	8.9	21.5
		Over 60	5.3	3.0	8.3	8.8	4.9	13.7	12.3	6.9	19.2	15.8	8.9	24.7

Table C-1. Continued

Design Alternative	Type of Development	Driveways Per Mile	7,000-10,000 NINT	7,000-10,000 UINT	7,000-10,000 TOT	10,000-15,000 NINT	10,000-15,000 UINT	10,000-15,000 TOT	15,000-20,000 NINT	15,000-20,000 UINT	15,000-20,000 TOT	Over 20,000 NINT	Over 20,000 UINT	Over 20,000 TOT
			Average Daily Traffic Volume (veh/day)											
			TRUCK PERCENTAGE: UNDER 5% INTERSECTIONS PER MILE: 5-10											
2U	C	Under 30	5.9	7.1	13.0	9.9	11.9	21.8	13.8	16.7	30.5	17.7	21.4	39.1
		30-60	7.0	7.1	14.1	11.6	11.9	23.5	16.2	16.7	32.9	20.9	21.4	42.3
		Over 60	8.0	7.1	15.1	13.3	11.9	25.2	18.7	16.7	35.4	24.0	21.4	45.4
2U	R	Under 30	4.5	9.3	13.8	7.5	15.4	22.9	10.5	21.6	32.1	13.6	27.8	41.4
		30-60	5.6	9.3	14.9	9.3	15.4	24.7	13.0	21.6	34.6	16.7	27.8	44.5
		Over 60	6.6	9.3	15.9	11.0	15.4	26.4	15.4	21.6	37.0	19.8	27.8	47.6
3T	C	Under 30	3.6	8.0	11.6	6.1	13.4	19.5	8.5	18.7	27.2	10.9	24.1	35.0
		30-60	4.7	8.0	12.7	7.8	13.4	21.2	10.9	18.7	29.6	14.0	24.1	38.1
		Over 60	5.7	8.0	13.7	9.5	13.4	22.9	13.3	18.7	32.0	17.2	24.1	41.3
3T	R	Under 30	3.9	6.6	10.5	6.4	11.0	17.4	9.0	15.4	24.4	11.6	19.8	31.4
		30-60	4.9	6.6	11.5	8.2	11.0	19.2	11.4	15.4	26.8	14.7	19.8	34.5
		Over 60	5.9	6.6	12.5	9.9	11.0	20.9	13.9	15.4	29.3	17.8	19.8	37.6
4U	C	Under 30	7.2	14.4	21.6	12.0	24.0	36.0	16.7	33.7	50.4	21.5	43.3	64.8
		30-60	8.2	14.4	22.6	13.7	24.0	37.7	19.2	33.7	52.9	24.6	43.3	67.9
		Over60	9.3	14.4	23.7	15.4	24.0	39.4	21.6	33.7	55.3	27.8	43.3	71.1
4U	R	Under 30	2.0	9.7	11.7	3.4	16.1	19.5	4.7	22.5	27.2	6.1	29.0	35.1
		30-60	3.1	9.7	12.8	5.1	16.1	21.2	7.2	22.5	29.7	9.2	29.0	38.2
		Over 60	4.1	9.7	13.8	6.8	16.1	22.9	9.6	22.5	32.1	12.3	29.0	41.3
4D	C	Under 30	7.3	14.3	21.6	12.2	23.8	36.0	17.1	33.3	50.4	21.9	42.8	64.7
		30-60	8.3	14.3	22.6	13.9	23.8	37.7	19.5	33.3	52.8	25.0	42.8	67.8
		Over 60	9.4	14.3	23.7	15.6	23.8	39.4	21.9	33.3	55.2	28.2	42.8	71.0
4D	R	Under 30	3.2	8.8	12.0	5.3	14.6	19.9	7.4	20.5	27.9	9.5	26.4	35.9
		30-60	4.2	8.8	13.0	7.0	14.6	21.6	9.8	20.5	30.3	12.6	26.4	39.0
		Over 60	5.3	8.8	14.1	8.8	14.6	23.4	12.3	20.5	32.8	15.8	26.4	42.2
5T	C	Under 30	6.7	9.9	16.6	11.2	16.5	27.7	15.7	23.1	38.8	20.2	29.6	49.8
		30-60	7.8	9.9	17.7	13.0	16.5	29.5	18.1	23.1	41.2	23.3	29.6	52.9
		Over 60	8.8	9.9	18.7	14.7	16.5	31.2	20.6	23.1	43.7	26.4	29.6	56.0
5T	R	Under 30	3.2	6.4	9.6	5.3	10.7	16.0	7.4	15.0	22.4	9.5	19.3	28.8
		30-60	4.2	6.4	10.6	7.0	10.7	17.7	9.8	15.0	24.8	12.6	19.3	31.9
		Over 60	5.3	6.4	11.7	8.8	10.7	19.5	12.3	15.0	27.3	15.8	19.3	35.1

Table C-1. Continued

Design Alternative	Type of Development	Driveways Per Mile	Average Daily Traffic Volume (veh/day) 7,000-10,000 NINT	UINT	TOT	10,000-15,000 NINT	UINT	TOT	15,000-20,000 NINT	UINT	TOT	Over 20,000 NINT	UINT	TOT
TRUCK PERCENTAGE: UNDER 5% INTERSECTIONS PER MILE: OVER 10														
2U	C	Under 30	5.9	10.6	16.5	9.9	17.7	27.6	13.8	24.8	38.6	17.7	31.9	49.6
		30-60	7.0	10.6	17.6	11.6	17.7	29.3	16.2	24.8	41.0	20.9	31.9	52.8
		Over60	8.0	10.6	18.6	13.3	17.7	31.0	18.7	24.8	43.5	24.0	31.9	55.9
2U	R	Under 30	4.5	12.7	17.2	7.5	21.2	28.7	10.5	29.7	40.2	13.6	38.2	51.8
		30-60	5.6	12.7	18.3	9.3	21.2	30.5	13.0	29.7	42.7	16.7	38.2	54.9
		Over 60	6.6	12.7	19.3	11.0	21.2	32.2	15.4	29.7	45.1	19.8	38.2	58.0
3T	C	Under 30	3.6	11.5	15.1	6.1	19.2	25.3	8.5	26.8	35.3	10.9	34.5	45.4
		30-60	4.7	11.5	16.2	7.8	19.2	27.0	10.9	26.8	37.7	14.0	34.5	48.5
		Over 60	5.7	11.5	17.2	9.5	19.2	28.7	13.3	26.8	40.1	17.2	34.5	51.7
3T	R	Under 30	3.9	10.1	14.0	6.4	16.8	23.2	9.0	23.5	32.5	11.6	30.2	41.8
		30-60	4.9	10.1	15.0	8.2	16.8	25.0	11.4	23.5	34.9	14.7	30.2	44.9
		Over 60	5.9	10.1	16.0	9.9	16.8	26.7	13.9	23.5	37.4	17.8	30.2	48.0
4U	C	Under 30	7.2	17.9	25.1	12.0	29.8	41.8	16.7	41.8	58.5	21.5	53.7	75.2
		30-60	8.2	17.9	26.1	13.7	29.8	43.5	19.2	41.8	61.0	24.6	53.7	78.3
		Over 60	9.3	17.9	27.2	15.4	29.8	45.2	21.6	41.8	63.4	27.8	53.7	81.5
4U	R	Under 30	2.0	13.1	15.1	3.4	21.9	25.3	4.7	30.7	35.4	6.1	39.4	45.5
		30-60	3.1	13.1	16.2	5.1	21.9	27.0	7.2	30.7	37.9	9.2	39.4	48.6
		Over 60	4.1	13.1	17.2	6.8	21.9	28.7	9.6	30.7	40.3	12.3	39.4	51.7
4D	C	Under 30	7.3	17.7	25.0	12.2	29.6	41.8	17.1	41.4	58.5	21.9	53.2	75.1
		30-60	8.3	17.7	26.0	13.9	29.6	43.5	19.5	41.4	60.9	25.0	53.2	78.2
		Over 60	9.4	17.7	27.1	15.6	29.6	45.2	21.9	41.4	63.3	28.2	53.2	81.4
4D	R	Under 30	3.2	12.3	15.5	5.3	20.4	25.7	7.4	28.6	36.0	9.5	36.8	46.3
		30-60	4.2	12.3	16.5	7.0	20.4	27.4	9.8	28.6	38.4	12.6	36.8	49.4
		Over 60	5.3	12.3	17.6	8.8	20.4	29.2	12.3	28.6	40.9	15.8	36.8	52.6
5T	C	Under 30	6.7	13.4	20.1	11.2	22.3	33.5	15.7	31.2	46.9	20.2	40.1	60.3
		30-60	7.8	13.4	21.2	13.0	22.3	35.3	18.1	31.2	49.3	23.3	40.1	63.4
		Over 60	8.8	13.4	22.2	14.7	22.3	37.0	20.6	31.2	51.8	26.4	40.1	66.5
5T	R	Under 30	3.2	9.9	13.1	5.3	16.5	21.8	7.4	23.1	30.5	9.5	2.7	39.2
		30-60	4.2	9.9	14.1	7.0	16.5	23.5	9.8	23.1	32.9	12.6	29.7	42.3
		Over 60	5.3	9.9	15.2	8.8	16.5	25.3	12.3	23.1	35.4	15.8	29.7	45.5

Table C-1. Continued

Design Alternative	Type of Development	Driveways Per Mile	Average Daily Traffic Volume (veh/day) 7,000-10,000 NINT	UINT	TOT	10,000-15,000 NINT	UINT	TOT	15,000-20,000 NINT	UINT	TOT	Over 20,000 NINT	UINT	TOT
			TRUCK PERCENTAGE: 5-10% INTERSECTIONS PER MILE: UNDER 5											
2U	C	Under 30	5.2	2.8	8.0	8.7	4.7	13.4	12.2	6.6	18.8	15.7	8.5	24.2
		30-60	6.3	2.8	9.1	10.4	4.7	15.1	14.6	6.6	21.2	18.8	8.5	27.3
		Over 60	7.3	2.8	10.1	12.2	4.7	16.9	17.1	6.6	23.7	21.9	8.5	30.4
2U	R	Under 30	3.8	5.0	8.8	6.4	8.3	14.7	8.9	11.6	20.5	11.5	14.9	26.4
		30-60	4.9	5.0	9.9	8.1	8.3	16.4	11.4	11.6	23.0	14.6	14.9	29.5
		Over 60	5.9	5.0	10.9	9.9	8.3	18.2	13.8	11.6	25.4	17.7	14.9	32.6
3T	C	Under 30	3.0	3.7	6.7	4.9	6.2	11.1	6.9	8.7	15.6	8.9	11.2	20.1
		30-60	4.0	3.7	7.7	6.7	6.2	12.9	9.3	8.7	18.0	12.0	11.2	23.2
		Over 60	5.0	3.7	8.7	8.4	6.2	14.6	11.8	8.7	20.5	15.1	11.2	26.3
3T	R	Under 30	3.2	2.3	5.5	5.3	3.8	9.1	7.4	5.4	12.8	9.5	6.9	16.4
		30-60	4.2	2.3	6.5	7.0	3.	10.8	9.8	5.4	15.2	12.6	6.9	19.5
		Over 60	5.3	2.3	7.6	8.8	3.8	12.6	12.3	5.4	17.7	15.8	6.9	22.7
4U	C	Under 30	6.5	10.1	16.6	10.8	16.9	27.7	15.1	23.6	38.7	19.5	30.4	49.9
		30-60	7.5	10.1	17.6	12.5	16.9	29.4	17.6	23.6	41.2	22.6	30.4	53.0
		Over 60	8.6	10.1	18.7	14.3	16.9	31.2	20.0	23.6	43.6	25.7	30.4	56.1
4U	R	Under 30	1.3	5.4	6.7	2.2	8.9	11.1	3.1	12.5	15.6	4.0	16.1	20.1
		30-60	2.4	5.4	7.8	4.0	8.9	12.9	5.6	12.5	18.1	7.1	16.1	23.2
		Over 60	3.4	5.4	8.8	5.7	8.9	14.6	8.0	12.5	20.5	10.3	16.1	26.4
4D	C	Under 30	6.6	10.0	16.6	11.0	16.6	27.6	15.5	23.3	38.8	19.9	29.9	49.8
		30-60	7.0	10.0	17.7	12.8	16.6	29.4	17.9	23.3	41.2	23.0	29.9	52.9
		Over 60	8.7	10.0	18.7	14.5	16.6	31.1	20.3	23.3	43.6	26.1	29.9	56.0
4D	4	Under 30	2.5	4.5	7.0	4.2	7.5	11.7	5.8	10.5	16.3	7.5	13.5	21.0
		30-60	3.5	4.5	8.0	5.9	7.5	13.4	8.2	10.5	18.7	10.6	13.5	24.1
		Over 60	4.6	4.5	9.1	7.6	7.5	15.1	10.7	10.5	21.2	13.7	13.5	27.2
5T	C	Under 30	6.0	5.6	11.6	10.1	9.3	19.4	14.1	13.0	27.1	18.1	16.8	34.9
		30-60	7.1	5.6	12.7	11.8	9.3	21.1	16.5	13.0	29.5	21.3	16.8	38.1
		Over 60	8.1	5.6	13.7	13.6	9.3	22.9	19.0	13.0	32.0	24.4	16.8	41.2
5T	R	Under 30	2.5	2.1	4.6	4.2	3.6	7.8	5.8	5.0	10.8	7.5	6.4	13.9
		30-60	3.5	2.1	5.6	5.9	3.6	9.5	8.2	5.0	13.2	10.6	6.4	17.0
		Over 60	4.6	2.1	6.7	7.6	3.6	11.2	10.7	5.0	15.7	13.7	6.4	20.1

Table C-1. Continued

Design Alternative	Type of Development	Driveways Per Mile	Average Daily Traffic Volume (veh/day) 7,000-10,000 NINT	UINT	TOT	10,000-15,000 NINT	UINT	TOT	15,000-20,000 NINT	UINT	TOT	Over 20,000 NINT	UINT	TOT
			TRUCK PERCENTAGE: 5-10% INTERSECTIONS PER MILE: 5-10											
2U	C	Under 30	5.2	6.3	11.5	8.7	10.5	19.2	12.2	14.8	27.0	15.7	19.0	34.7
		30-60	6.3	6.3	12.6	10.4	10.5	20.9	14.6	14.8	29.4	18.8	19.0	37.8
		Over 60	7.3	6.3	13.6	12.2	10.5	22.7	17.1	14.8	31.9	21.9	19.0	40.9
2U	R	Under 30	3.8	8.4	12.2	6.4	14.1	20.5	8.9	19.7	28.6	11.5	25.3	36.8
		30-60	4.9	8.4	13.3	8.1	14.1	22.2	11.4	19.7	31.1	14.6	25.3	39.9
		Over 60	5.9	8.4	14.3	9.9	14.1	24.0	13.8	19.7	33.5	17.7	25.3	43.0
3T	C	Under 30	3.0	7.2	10.2	4.9	12.0	16.9	6.9	16.8	23.7	8.9	21.6	30.5
		30-60	4.0	7.2	11.2	6.7	12.0	18.7	9.3	16.8	26.1	12.0	21.6	33.6
		Over 60	5.0	7.2	12.2	8.4	12.0	20.4	11.8	16.8	28.6	15.1	21.6	36.7
3T	4	Under 30	3.2	5.8	9.0	5.3	9.6	14.9	7.4	13.5	20.9	9.5	17.3	26.8
		30-60	4.2	5.8	10.0	7.0	9.6	16.6	9.8	13.5	23.3	12.6	17.3	29.9
		Over 60	5.3	5.8	11.1	8.8	9.6	18.4	12.3	13.5	25.8	15.8	17.3	33.1
4U	C	Under 30	6.5	13.6	20.1	10.8	22.7	33.5	15.1	31.7	46.8	19.5	40.8	60.3
		30-60	7.5	13.6	21.1	12.5	22.7	35.2	17.6	31.7	49.3	22.6	40.8	63.4
		Over 60	8.6	13.6	22.2	14.3	22.7	37.0	20.0	31.7	51.7	25.7	40.8	66.5
4U	R	Under 30	1.3	8.8	10.1	2.2	14.7	16.9	3.1	20.6	23.7	4.0	26.5	30.5
		30-60	2.4	8.8	11.2	4.0	14.7	18.7	5.6	20.6	26.2	7.1	26.5	33.6
		Over 60	3.4	8.8	12.2	5.7	14.7	20.4	8.0	20.6	28.6	10.3	26.5	36.8
4D	C	Under 30	6.6	13.4	20.0	11.0	22.4	33.4	15.5	31.4	46.9	19.9	40.3	60.2
		30-60	7.7	13.4	21.1	12.8	22.4	35.2	17.9	31.4	49.3	23.0	40.3	63.3
		Over 60	8.7	13.4	22.1	14.5	22.4	36.9	20.3	31.4	51.7	26.1	40.3	66.4
4D	R	Under 30	2.5	8.0	10.5	4.2	13.3	17.5	5.8	18.6	24.4	7.5	23.9	31.4
		30-60	3.5	8.0	11.5	5.9	13.3	19.2	8.2	18.6	26.8	10.6	23.9	34.5
		Over 60	4.6	8.0	12.6	7.6	13.3	20.9	10.7	18.6	29.3	13.7	23.9	37.6
5T	C	Under 30	6.9	9.1	15.1	10.1	15.1	25.2	14.1	21.1	35.2	18.1	27.2	45.3
		30-60	7.1	9.1	16.2	11.8	15.1	26.9	16.5	21.1	37.6	21.3	27.2	48.5
		Over 60	8.1	9.1	17.2	13.6	15.1	28.7	19.0	21.1	40.1	24.4	27.2	51.6
5T	R	Under 30	2.5	5.6	8.1	4.2	9.4	13.6	5.8	13.1	18.9	7.5	16.8	24.3
		30-60	3.5	5.6	9.1	5.9	9.4	15.3	8.2	13.1	21.3	10.6	16.8	27.4
		Over 60	4.6	5.6	10.2	7.6	9.4	17.0	10.7	13.1	23.8	13.7	16.8	30.5

Table C-1. Continued

Design Alternative	Type of Development	Driveways Per Mile	Average Daily Traffic Volume (veh/day) 7,000-10,000			10,000-15,000			15,000-20,000			Over 20,000		
			NINT	UINT	TOT	NINT	UINT	TOT	NINT	UINT	TOT	NINT	UINT	TOT
			TRUCK PERCENTAGE: 5-10% INTERSECTIONS PER MILE: OVER 10											
2U	C	Under 30	5.2	9.8	15.0	8.7	16.3	25.0	12.2	22.9	35.1	15.7	29.4	45.1
		30-60	6.3	9.8	16.1	10.4	16.3	26.7	14.6	22.9	37.5	18.8	29.4	48.2
		Over 60	7.3	9.8	17.1	12.2	16.3	28.5	17.1	22.9	40.0	21.9	29.4	51.3
2U	R	Under 30	3.8	11.9	15.7	6.4	19.8	26.2	8.9	27.8	36.7	11.5	35.7	47.2
		30-60	4.9	11.9	16.8	8.1	19.8	27.9	11.4	27.8	39.2	14.6	35.7	50.3
		Over 60	5.9	11.9	17.8	9.9	19.8	29.7	13.8	27.8	41.6	17.7	35.7	53.4
3T	C	Under 30	3.0	10.7	13.7	4.9	17.8	22.7	6.9	24.9	31.8	8.9	32.0	40.9
		30-60	4.0	10.7	14.7	6.7	17.8	24.5	9.3	24.9	34.2	12.0	32.0	44.0
		Over 60	5.0	10.7	15.7	8.4	17.8	26.2	11.8	24.9	36.7	15.1	32.0	47.1
3T	R	Under 30	3.2	9.3	12.5	5.3	15.4	20.7	7.4	21.6	29.0	9.5	27.8	37.3
		30-60	4.2	9.3	13.5	7.0	15.4	22.4	9.8	21.6	31.4	12.6	27.8	40.4
		Over 60	5.3	9.3	14.6	8.8	15.4	24.2	12.3	21.6	33.9	15.8	27.8	43.6
4U	C	Under 30	6.5	17.1	23.6	10.8	28.5	39.3	15.1	39.9	55.0	19.5	51.2	70.7
		30-60	7.5	17.1	24.6	12.5	28.5	41.0	17.6	39.9	57.5	22.6	51.2	73.8
		Over 60	8.6	17.1	25.7	14.3	28.5	42.8	20.0	39.9	59.9	25.7	51.2	76.9
4U	R	Under 30	1.3	12.3	13.6	2.2	20.5	22.7	3.1	28.7	31.8	4.0	37.0	41.0
		30-60	2.4	12.3	14.7	4.0	20.5	24.5	5.6	28.7	34.3	7.1	37.0	44.1
		Over 60	3.4	12.3	15.7	5.7	20.5	26.2	8.0	28.7	36.7	10.3	37.0	47.3
4D	C	Under 30	6.6	16.9	23.5	11.0	28.2	39.2	15.5	39.5	55.0	19.9	50.8	70.7
		30-60	7.7	16.9	24.6	12.8	28.2	41.0	17.9	39.5	57.4	23.0	50.8	73.8
		Over 60	8.7	16.9	25.6	14.5	28.2	42.7	20.3	39.5	59.8	26.1	50.8	76.9
4D	R	Under 30	2.5	11.4	13.9	4.2	19.1	23.3	5.8	26.7	32.5	7.5	34.3	41.8
		30-60	3.5	11.4	14.9	5.9	19.1	25.0	8.2	26.7	34.9	10.6	34.3	44.9
		Over 60	4.6	11.4	16.0	7.6	19.1	26.7	10.7	26.7	37.4	13.7	34.3	48.0
5T	C	Under 30	6.0	12.5	18.5	10.1	20.9	31.0	14.1	29.3	43.4	18.1	37.6	55.7
		30-60	7.1	12.5	19.6	11.8	20.9	32.7	16.5	29.3	45.8	21.3	37.6	58.9
		Over 60	8.1	12.5	20.6	13.6	20.9	34.5	19.0	29.3	48.3	24.4	37.6	62.0
5T	R	Under 30	2.5	9.1	11.6	4.2	15.1	19.3	5.8	21.2	27.0	7.5	27.3	34.8
		30-60	3.5	9.1	12.6	5.9	15.1	21.0	8.2	21.2	29.4	10.6	27.3	37.9
		Over 60	4.6	9.1	13.7	7.6	15.1	22.7	10.7	21.2	31.9	13.7	27.3	41.0

Table C-1. Continued

			Average Daily Traffic Volume (veh/day)											
			7,000-10,000			10,000-15,000			15,000-20,000			Over 20,000		
Design Alternative	Type of Development	Driveways Per Mile	NINT	UINT	TOT	NINT	UINT	TOT	NINT	UINT	TOT	NINT	UINT	TOT
			TRUCK PERCENTAGE: OVER 10% INTERSECTIONS PER MILE: UNDER 5											
2U	C	Under 30	4.5	2.0	6.5	7.5	3.4	10.9	10.5	4.7	15.2	13.6	6.1	19.7
		30-60	5.6	2.0	7.6	9.3	3.4	12.7	13.0	4.7	17.7	16.7	6.1	22.8
		Over 60	6.6	2.0	8.6	11.0	3.4	14.4	15.4	4.7	20.1	19.8	6.1	25.9
2U	R	Under 30	3.1	4.1	7.2	5.2	6.9	12.1	7.3	9.6	16.9	9.4	12.4	21.8
		30-60	4.2	4.1	8.3	6.9	6.9	13.8	9.7	9.6	19.3	12.5	12.4	24.9
		Over 60	5.2	4.1	9.3	8.7	6.9	15.6	12.1	9.6	21.7	15.6	12.4	28.0
3T	C	Under 30	2.2	2.9	5.1	3.7	4.8	8.5	5.2	6.8	12.0	6.7	8.7	15.4
		30-60	3.3	2.9	6.2	5.5	4.8	10.3	7.7	6.8	14.5	9.9	8.7	18.6
		Over 60	4.3	2.9	7.2	7.2	4.8	12.0	10.1	6.8	16.9	13.0	8.7	21.7
3T	R	Under 30	2.5	1.5	4.0	4.1	2.5	6.6	5.7	3.4	9.1	7.4	4.4	11.8
		30-60	3.5	1.5	5.0	5.8	2.5	8.3	8.2	3.4	11.6	10.5	4.4	14.9
		Over 60	4.5	1.5	6.0	7.6	2.5	10.1	10.6	3.4	14.0	136	4.4	18.0
4U	C	Under 30	5.8	9.3	15.1	9.6	15.5	25.1	13.5	21.7	35.2	17.3	27.9	45.2
		30-60	6.8	9.3	16.1	11.4	15.5	26.9	15.9	21.7	37.6	20.4	27.9	48.3
		Over 60	7.9	9.3	17.2	13.1	15.5	28.6	18.3	21.7	40.0	23.6	27.9	51.5
4U	R	Under 30	0.6	4.5	5.1	1.0	7.6	8.6	1.5	10.6	12.1	1.9	13.6	15.5
		30-60	1.7	4.5	6.2	2.8	7.6	10.4	3.9	10.6	14.5	5.0	13.6	18.6
		Over 60	2.7	4.5	7.2	4.5	7.6	12.1	6.3	10.6	16.9	8.1	13.6	21.7
4D	C	Under 30	5.9	9.1	15.0	9.9	15.2	25.1	13.8	21.3	35.1	17.7	27.4	45.1
		30-60	7.0	9.1	16.1	11.6	15.2	26.8	16.2	21.3	37.5	20.9	27.4	48.3
		Over 60	8.0	9.1	17.1	13.3	15.2	28.5	18.7	21.3	40.0	24.0	27.4	51.4
4D	R	Under 30	1.8	3.7	5.5	3.0	6.1	9.1	4.2	8.6	12.8	5.3	11.0	16.3
		30-60	2.8	3.7	6.5	4.7	6.1	10.8	6.6	8.6	15.2	8.5	11.0	19.5
		Over 60	3.9	3.7	7.6	6.4	6.1	12.5	9.0	8.6	17.6	11.6	11.0	22.6
5T	C	Under 30	5.3	4.8	10.1	8.9	7.9	16.8	12.5	11.1	23.6	16.0	14.3	30.3
		30-60	6.4	4.8	11.2	10.6	7.9	18.5	14.9	11.1	26.0	19.1	14.3	33.4
		Over 60	7.4	4.8	12.2	12.4	7.9	20.3	17.3	11.1	28.4	22.3	14.3	36.6
5T	R	Under 30	1.8	1.3	3.1	3.0	2.2	5.2	4.2	3.1	7.3	5.3	3.9	9.2
		30-60	2.8	1.3	4.1	4.7	2.2	6.9	6.6	3.1	9.7	8.5	3.9	12.4
		Over 60	3.9	1.3	5.2	6.4	2.2	8.6	9.0	3.1	12.1	11.6	3.9	15.5

Table C-1. Continued

Design Alternative	Type of Development	Driveways Per Mile	Average Daily Traffic Volume (veh/day) 7,000-10,000 NINT	UINT	TOT	10,000-15,000 NINT	UINT	TOT	15,000-20,000 NINT	UINT	TOT	Over 20,000 NINT	UINT	TOT
		TRUCK PERCENTAGE: OVER 10% INTERSECTIONS PER MILE: 5-10												
2U	C	Under 30	4.5	5.5	10.0	7.5	9.2	16.7	10.5	12.8	23.3	13.6	16.5	30.1
		30-60	5.6	5.5	11.1	9.3	9.2	18.5	13.0	12.8	25.8	16.7	16.5	33.2
		Over 60	6.6	5.5	12.1	11.0	9.2	20.2	15.4	12.8	28.2	19.8	16.5	36.3
2U	R	Under 30	3.1	7.6	10.7	5.2	12.7	17.9	7.3	17.8	25.1	9.4	22.8	32.2
		30-60	4.2	7.6	11.8	6.9	12.7	19.6	9.7	17.8	27.5	12.5	22.8	35.3
		Over 60	5.2	7.6	12.8	8.7	12.7	21.4	12.1	17.8	29.9	15.6	22.8	38.4
3T	C	Under 30	2.2	6.4	8.6	3.7	10.6	14.3	5.2	14.9	20.1	6.7	19.1	25.8
		30-60	3.3	6.4	9.7	5.5	10.6	16.1	7.7	14.9	22.6	9.9	19.1	29.0
		Over 60	4.3	6.4	10.7	7.2	10.6	17.8	10.1	14.9	25.0	13.0	19.1	32.1
3T	R	Under 30	2.5	5.0	7.5	4.1	8.3	12.4	5.7	11.6	17.3	7.4	14.9	22.3
		30-60	3.5	5.0	8.5	5.8	8.3	14.1	8.2	11.6	19.8	10.5	14.9	25.4
		Over 60	4.5	5.0	9.5	7.6	8.3	15.9	10.6	11.6	22.2	13.6	14.9	28.5
4U	C	Under 30	5.8	12.8	18.6	9.6	21.3	30.9	13.5	29.8	43.3	17.3	38.4	55.7
		30-60	6.8	12.8	19.6	11.4	21.3	32.7	15.9	29.8	45.7	20.4	38.4	58.8
		Over 60	7.9	12.8	20.7	13.1	21.3	34.4	18.3	29.8	48.1	23.6	38.4	62.0
4U	R	Under 30	0.6	8.0	8.6	1.0	13.4	14.4	1.5	18.7	20.2	1.9	24.1	26.0
		30-60	1.7	8.0	9.7	2.8	13.4	16.2	3.9	18.7	22.6	5.0	24.1	29.1
		Over 60	2.7	8.0	10.7	4.5	13.4	17.9	6.3	18.7	25.0	8.1	24.1	32.2
4D	C	Under 30	5.9	12.6	18.5	9.9	21.0	30.0	13.8	29.4	43.2	17.7	37.9	55.6
		30-60	7.0	12.6	19.6	11.6	21.0	32.6	16.2	29.4	45.6	20.9	37.9	58.8
		Over 60	8.0	12.6	20.6	13.3	21.0	34.3	18.7	29.4	48.1	24.0	37.9	61.9
4D	R	Under 30	1.8	7.1	8.9	3.0	11.9	14.9	4.2	16.7	20.9	5.3	21.4	26.7
		30-60	2.8	7.1	9.9	4.7	11.9	16.6	6.6	16.7	23.3	8.5	21.4	29.9
		Over 60	3.9	7.1	11.0	6.4	11.9	18.3	9.0	16.7	25.7	11.6	21.4	33.0
5T	C	Under 30	5.3	8.2	13.5	8.9	13.7	22.6	12.5	19.2	31.7	16.0	24.7	40.7
		30-60	6.4	8.2	14.6	10.6	13.7	24.3	14.9	19.2	34.1	19.1	24.7	43.8
		Over 60	7.4	8.2	15.6	12.4	13.7	26.1	17.3	19.2	36.5	22.3	24.7	47.0
5T	R	Under 30	1.8	4.8	6.6	3.0	8.0	11.0	4.2	11.2	15.4	5.3	14.4	19.7
		30-60	2.8	4.8	7.6	4.7	8.0	12.7	6.6	11.2	17.8	8.5	14.4	22.9
		Over 60	3.9	4.8	8.7	6.4	8.0	14.4	9.0	11.2	20.2	11.6	14.4	26.0

Table C-1. Continued

Design Alternative	Type of Development	Driveways Per Mile	Average Daily Traffic Volume (veh/day) 7,000-10,000			10,000-15,000			15,000-20,000			Over 20,000		
			NINT	UINT	TOT	NINT	UINT	TOT	NINT	UINT	TOT	NINT	UINT	TOT
TRUCK PERCENTAGE: OVER 10% INTERSECTIONS PER MILE: OVER 10														
2U	C	Under 30	4.5	9.0	13.5	7.5	15.0	22.5	10.5	21.0	31.5	13.6	26.9	40.5
		30-60	5.6	9.0	14.6	9.3	15.0	24.3	13.0	21.0	34.0	16.7	26.9	43.6
		Over 60	6.6	9.0	15.6	11.0	15.0	26.0	15.4	21.0	36.4	19.8	26.9	46.7
2U	R	Under 30	3.1	11.1	14.2	5.2	18.5	23.7	7.3	25.9	33.2	9.4	33.3	42.7
		30-60	4.2	11.1	15.3	6.9	18.5	25.4	9.7	25.9	35.6	12.5	33.3	45.8
		Over 60	5.2	11.1	16.3	8.7	18.5	27.2	12.1	25.9	38.0	15.6	33.3	48.9
3T	C	Under 30	2.2	9.9	12.1	3.7	16.4	20.1	5.2	23.0	28.2	6.7	29.6	36.3
		30-60	3.3	9.9	13.2	5.5	16.4	21.9	7.7	23.0	30.7	9.9	29.6	39.5
		Over 60	4.3	9.9	14.2	7.2	16.4	23.6	10.1	23.0	33.1	13.0	29.6	42.6
3T	R	Under 30	2.5	8.4	10.9	4.1	14.1	18.2	5.7	19.7	25.4	7.4	25.3	32.7
		30-60	3.5	8.4	11.9	5.8	14.1	19.9	8.2	19.7	27.9	10.5	25.3	35.8
		Over 60	4.5	8.4	12.9	7.6	14.1	21.7	10.6	19.7	30.3	13.6	25.3	38.9
4U	C	Under 30	5.8	16.3	22.1	9.6	27.1	36.7	13.5	37.9	51.4	17.3	48.8	66.1
		30-60	6.8	16.3	23.1	11.4	27.1	38.5	15.9	37.9	53.8	20.4	48.8	69.2
		Over 60	7.9	16.3	24.2	13.1	27.1	40.2	18.3	37.9	56.2	23.6	48.8	72.4
4U	R	Under 30	0.6	11.5	12.1	1.0	19.2	20.2	1.5	26.8	28.3	1.9	34.5	36.4
		30-60	1.7	11.5	13.2	2.8	19.2	22.0	3.9	26.8	30.7	5.0	34.5	39.5
		Over 60	2.7	11.5	14.2	4.5	19.2	23.7	6.3	26.8	33.1	8.1	34.5	42.6
4D	C	Under 30	5.9	16.1	22.0	9.9	26.8	36.7	13.8	37.6	51.4	17.7	48.3	66.0
		30-60	7.0	16.1	23.1	11.6	26.8	38.4	16.2	37.6	53.8	20.9	48.3	69.2
		Over 60	8.0	16.1	24.1	13.3	26.8	40.1	18.7	37.6	56.3	24.0	48.3	72.3
4D	R	Under 30	1.8	10.6	12.4	3.0	17.7	20.7	4.2	24.8	29.0	5.3	31.9	37.2
		30-60	2.8	10.6	13.4	4.7	17.7	22.4	6.6	24.8	31.4	8.5	31.9	40.4
		Over 60	3.9	10.6	14.5	6.4	17.7	24.1	9.0	24.8	33.8	11.6	31.9	43.5
5T	C	Under 30	5.3	11.7	17.0	8.9	19.5	28.4	12.5	27.3	39.8	16.0	35.1	51.1
		30-60	6.4	11.7	18.1	10.6	19.5	30.1	14.9	27.3	42.2	19.1	35.1	54.2
		Over 60	7.4	11.7	19.1	12.4	19.5	31.9	17.3	27.3	44.6	22.3	35.1	57.4
5T	R	Under 30	1.8	8.3	10.1	3.0	13.8	16.8	4.2	19.3	23.5	5.3	24.8	30.1
		30-60	2.8	8.3	11.1	4.7	13.8	18.5	6.6	19.3	25.9	8.5	24.8	33.3
		Over 60	3.9	8.3	12.2	6.4	13.8	20.2	9.0	19.3	28.3	11.6	24.8	36.4

Table C-2. Typical annual accident frequencies for a signalized intersection on a suburban arterial highway (*25*).

Crossroad ADT (veh/day)	Arterial Highway ADT (veh/day) 7,000-10,000	10,000-15,000	15,000-20,000	Over 20,000
Low (< 1,000)	2.2	2.8	3.2	3.6
Medium (1,000-5,000)	4.3	5.4	6.3	7.1
High (> 5,000)	6.0	7.5	8.7	9.8

APPENDIX D

SIMULATION OF TRAFFIC OPERATIONS ON SUBURBAN ARTERIALS

This appendix documents the operation and application of a computer simulation model used to assess the traffic operational effectiveness of multilane design alternatives. The first portion of the appendix describes the simulation model used in this assessment. The latter portion of the appendix presents the results obtained from a traffic operational comparison of two-lane and four-lane suburban arterials both with and without two-way left-turn lanes.

OVERVIEW OF SIMULATION MODEL

The operational assessment of multilane design alternatives in this study was based on a traffic operational computer simulation model. This model, known as the Two-Way Left-Turn Lane Computer Simulation Model (TWLTL-SIM) was developed by Dr. John L. Ballard and Dr. Patrick T. McCoy at the University of Nebraska-Lincoln to provide a tool for traffic engineers to predict the operational effectiveness of a TWLTL. An earlier version of the TWLTL-SIM model was used in several published assessments of TWLTLs (*3, 4, 26*). The TWLTL-SIM model was written in the General Purpose Simulation System Version H (GPSSH) (*27, 28*), a special purpose computer language especially suited for modeling discrete systems. The TWLTL-SIM model was applied to the assessment of multilane design alternatives in this project by Dr. Ballard under a subcontract agreement with MRI.

The TWLTL-SIM model can be used to evaluate four basic geometric situations: (1) a two-way two-lane undivided street without a TWLTL; (2) a two-way two-lane street with a TWLTL; (3) a two-way four-lane undivided street without a TWLTL; and (4) a two-way four-lane street with a TWLTL. Thus, the model is capable of simulating four of the five design alternatives given primary attention in this study (all except four-lane divided). The following discussion presents the input, processing logic, output, and validation of the model.

Input

The input data to the model consist of street geometrics, traffic volume, and traffic characteristics data. The input parameters to the model include:

- Number of through lanes.
- Presence or absence of TWLTL.
- Length of simulated section.
- Locations of individual driveways.
- Entering traffic volume by lane in each direction (vph).
- Arrival distribution of entering traffic.
- Percentage of vehicles turning left at individual driveways.
- Percentage of vehicles turning right at individual driveways.
- Travel speed in each direction (mph).
- Random number seeds that serve as the basic for probabilistic generation of entering traffic headways, turning locations, and gap-acceptance criteria.

The traffic characteristics input to the models are the volume and average speed of traffic in each direction and the percentage of traffic volume turning left and right into each driveway on the street. Also, the arrival distribution of the traffic entering at each end of the simulated street section is specified. The model can generate both random and nonrandom arrival patterns, so the effects of upstream traffic signals can be simulated.

Because of the nature of the GPSS language, the street geometry is defined in terms of sections. Each lane on the street is divided into 20-ft sections, and driveway locations on the street are defined by the numbers of the sections in which they are located. Also specified for each driveway is the section number of the farthest point upstream at which a vehicle turning left into the driveway can enter the TWLTL. Typically, left-turn vehicles enter the TWLTL at a distance of 200 ft upstream from the driveway at which they desire to turn left.

As an example, the geometry of a 1,000-ft street segment for a three-lane highway with a TWLTL is shown in Figure D-1. Each lane is divided into 50, 20-ft sections, which are numbered as follows:

Lane 1: Sections 1–50,
Lane 2: Sections 51,100, and
TWLTL: Sections 101–150.

Processing Logic

In the TWLTL-SIM model, traffic enters the simulated street segment at either end in accordance with the traffic volumes and arrival patterns specified in input. Three vehicle paths are possible for any vehicle entering the segment; the vehicle may (1) traverse the entire length of the segment without turning and exit at the far end; (2) traverse a portion of the segment and exit by turning left at one of the driveways; or (3) traverse a portion of the segment and exit by turning right at one of the driveways. On entering the segment, the path to be taken by each vehicle is determined probabilistically in accordance with the turning percentages specified in input.

In addition to the vehicles entering the street segment at either end, some vehicles enter the segment by turning right onto the roadway from a driveway. All of these vehicles traverse the remainder of the segment and exit at the far end. The model does not include the capability to simulate left turns onto the roadway from driveways.

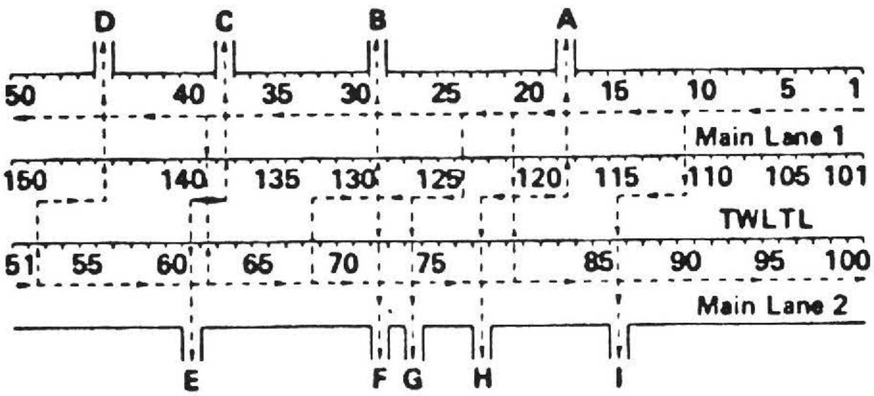

Figure D-1. Geometry of 1,000-ft street segment with TWLTL.

The section numbers of the driveway locations and their corresponding TWLTL entry points that would be input to the model for the TWLTL section shown in Figure D-1 are shown as follows:

DRIVEWAY	LANE NO. ENTERED FROM	DRIVEWAY LOCATION SECTION	TWLTL ENTRY POINT SECTION
A	2	18	121
B	2	29	133
C	2	38	139
D	2	45	149
E	1	61	139
F	1	72	124
G	1	74	124
H	1	78	121
I	1	86	111

In the case of a 1,000-ft street section without a TWLTL, sections 101–150 would not exist. Therefore, only the driveway location section numbers, and not the entry points, would be input to the model.

The left-turns off the roadway may delay following vehicles or force them to stop if no TWLTL is present. Such delay provides a measure of TWLTL effectiveness. The right-turns onto and off of the simulated roadway have no direct impact on TWLTL effectiveness. However, a right-turn off of the roadway can create a gap through which an opposing vehicle could subsequently turn left, while a right-turn onto the roadway could fill such a gap so that it was not available to opposing vehicles that desire to turn left.

The impact on delay measures of not simulating left-turn maneuvers out of driveways is uncertain, but it may not be large. While vehicles waiting to turn left from driveways experience substantial delay, may interrupt traffic flow in both directions of travel and may cause accidents, it is not clear that provision of a TWLTL is effective in reducing such delays and accidents. Some drivers use TWLTLs (especially those that are wider than average) as a median storage area to complete left-turns out of driveways. There are no hard data on the proportion of drivers who choose to complete left turns in this manner. However, the majority of drivers do not use TWLTLs in this manner and both the literature and practicing engineers are divided on the desirability of this practice.

Vehicles move through each 20-ft section in the main lanes at a constant speed specified in input and maintain at least a 2-sec headway behind their immediate leader. When a 2-sec headway cannot be maintained (because of a vehicle slowing or stopping to make a turn, for example), following vehicles will use a uniform deceleration rate of 5 ft/sec^2. Then, when system conditions warrant, vehicles will accelerate at a uniform rate of 5 ft/sec^2 to regain their desired speed. The constant speed assumption means that the entering headway distribution is preserved until modified by the responses to turning vehicles. The assumption of a constant desired speed and a constant minimum headway is an oversimplification of driver-speed selection and car-following behavior on actual highways. However, these assumptions are considered justified in this case because the objective of the model is not to estimate the actual travel speed on arterial streets, but is rather to simulate the left-turn gap-acceptance process and estimate its impact on vehicle delay.

In a model run with two through lanes, one in each direction, vehicles enter in one of the through lanes and continue in that lane until they turn or exit the roadway segment. The model has the capability to allow delayed vehicles to bypass on the right vehicles that are stopped to make a left turn, but this capability was not used in the study. Thus, the model implicitly assumes that there is not a usable shoulder for bypassing stopped vehicles.

In a model run with four through lanes, through vehicles are assigned probabilistically to either the right or left lane at the entrance point of the simulated roadway. Through vehicles that are delayed by turning vehicles on a four-lane section may change lanes to avoid delay. Vehicles that intend to turn right from the roadway are assigned to enter in the right lane and continue in that lane until they reach their desired turning point. A vehicle that intends to turn left enters the segment in the left lane and remains in the left lane until it reaches the entry point to the TWLTL, 200 ft upstream from the left-turn driveway. At this point, the left-turning vehicle begins decelerating to a velocity of 10 mph. Once it attains the desired speed of 10 mph, the vehicle enters the TWLTL and moves ahead in the TWLTL until it reaches the driveway at which it desires to turn or until it is stopped by vehicles already in the TWLTL waiting to turn left. The model continuously monitors the TWLTL and adjusts the entry point for left-turning vehicles as queues develop in the TWLTL.

If a turning vehicle reaches its entry point to the TWLTL and finds that the TWLTL section is already occupied by a left-turning vehicle from the other direction, it remains in the through lane and moves ahead until it either finds an unoccupied section in the TWLTL upstream from the driveway into which it wants to turn, reaches the driveway and stops in the through lane, or aborts the turn. In model runs at high flow rates both with and without a TWLTL, a vehicle will abort its turn and proceed ahead when stopping would precipitate a locked or jammed flow situation. This capability to abort a turn was added to the model to prevent it from ceasing operation due to jamming under very high flow conditions.

In all situations, a left-turning vehicle must have a minimum acceptable gap in the opposing traffic stream before it can turn left. The required length of gap is determined probabilistically in accordance with the left-turn gap-acceptance function derived by Gerlough and Wagner (*29*). The cumulative distribution function is as follows:

G GAP (SEC)	$P(G \leq G)$ PROBABILITY OF ACCEPTING A SHORTER GAP
3	0
3.5	0.15
4.0	0.32
4.5	0.52
5.0	0.69
5.5	0.82
6.0	0.90
6.5	0.95
7.0	0.97
7.5	0.986
8.0	0.993
8.5	0.997
9.0	0.998
9.5	0.999
10.0	1.0

If the left-turn vehicle is at the head of the queue and the required gap is available, the vehicle turns left. Otherwise, it waits for an acceptable gap. However, if a vehicle is not at the head of the queue, it will follow the leader across the opposing roadway as long as the available gap is longer than a minimum clearance time (1.5 sec to cross one lane, 2.86 sec to cross two lanes). The gap acceptance distribution based on Gerlough and Wagner was one of several candidates considered for the model and was found to produce the closest agreement with the field data collected for model validation.

Output Data

The output from the model includes the following data:

- Number of vehicles entering and exiting the segment.
- Number of left turns attempted and completed.
- Number of stops.
- Travel time in the segment.
- Stopped-time delay.

The travel time, stops and delay totals are output separately for through vehicles, left-turning vehicles, and all vehicles.

Model Validation

Time-lapse film of traffic flow on three arterial highway segments was obtained in order to validate the model. Two sites were located in Omaha, Nebraska (one 4U and one 5T site), and the third site (also 5T) was located in Lincoln, Nebraska. Altogether, 6 hours of time lapse film were taken.

The films were analyzed to determine the volumes, left-turn percentages, travel times, delays, and percentage of vehicles stopping on the three arterial street segments. The model was then run using the actual traffic volumes, left-turn percentages, and street geometrics as input data.

Table D-1 illustrates a comparison between the model results

Table D-1. Comparison of simulation model and field results.

Design Alternative	Measure of Effectiveness	Mean Difference (Model-Actual)	Std. Dev. of Difference	t	Significance[a]
Four-lane undivided (4U)	Percentage of vehicles stopping	30.33	24.58	2.14	NS
	Average stopped delay (veh-min/hr)	1.13	2.17	0.65	NS
Five-lane with TWLTL (5T)	Percentage of vehicles stopping	28.00	7.0	6.93	SIG[b]
	Average stopped delay (veh-min/hr)	0.17	4.05	0.07	NS

[a] Statistical significance at 95% confidence level (α=0.05) unless otherwise specified.
[b] Statistically significant at the 95% confidence level (α=0.05), but not at the 99% confidence level (α=0.01).

and the results obtained from the time lapse film. Paired t-tests were performed to compare the simulation model and actual mean delay times and percentage of vehicles stopping.

Table D-1 shows that for the 4U site there was no statistically significant difference at the 95 percent confidence level (α = 0.05) between the simulation and field results for the percentage of vehicles stopping and the average stopped delay. At the 5T sites, there was no significant difference in average stopped delay between the model output and the field data. However, there was a difference between the model and field results for the percentage of vehicles stopping at the 5T sites; this difference is statistically significant at the 95 percent confidence level (α 0.05), but not at the 99 percent confidence level. The observed difference is attributed to the difficulty of the judgments required by the time-lapse film analysts to distinguish vehicle stops.

Our assessment of the model after applying it in this research is that it can provide reasonable results, but that the simulation results were more consistent and repeatable for the reduction in delay due to a TWLTL than for the reduction in the number of stops. However, the use of several replicates for each situation of interest was found to be necessary, because the results of occasional runs can differ from the results of nominally identical runs by as much as an order of magnitude; the results of such deviant runs were discarded as outliers.

Table D-2. Conditions specified for simulation analysis of TWLTL effectiveness.

Flow Rate[a] (vph)	Driveway Density[b] (driveways/mile)	Percent of Traffic Turning Left Within a 1,000-ft Section: 5%	10%	15%	20%	25%
Two-Lane Undivided and Three-Lane With TWLTL						
400	30			X	X	X
	60			X	X	X
	90			X	X	X
650	30		X	X	X	
	60		X	X	X	
	90		X	X	X	
900	30	X				

Flow Rate[a] (vph)	Driveway Density[b] (driveways/mile)	Percent of Traffic Turning Left Within a 1,000-ft Section: 2.5%	5%	7.5%	10%	12.5%
650	30			X	X	X
	60			X	X	X
	90			X	X	X
900	30		X	X	X	
	60		X	X	X	
	90		X	X	X	
1,100	30	X	X	X		
	60	X	X	X		
	90	X	X	X		

[a] Flow rate in each direction of travel.
[b] Driveways per mile including driveways on both sides of highway.

SIMULATION ANALYSES OF TWLTL OPERATIONAL EFFECTIVENESS

The TWLTL-SIM model was used to investigate the operational effectiveness of 3T and 5T sections over comparable 2U and 4U alternatives, respectively.

Simulation Inputs

A series of runs was made to simulate traffic operations on a 1,000-ft section of suburban arterial highway while varying the design alternative, through traffic volume, left-turn percentage, and driveway density. Table D-2 indicates the specific combinations of these variables that were simulated for the 2U/3T and 4U/5T comparisons. Each X in Table D-2 represents a combination of flow rate, left-turn percentage, and driveway density for which paired runs were made with and without a TWLTL. Each pair of runs with and without a TWLTL were "clones" of one another, using the same random number seeds, so that the identical traffic stream entered highway sections that differed only in the presence or absence of a TWLTL. Three to five replicates were run for each paired comparison indicated in the table.

The combinations that were run were chosen for use in this study in light of the results of the previous TWLTL studies by McCoy, Ballard, and Wijaya (*3*) and Ballard and McCoy (*4*) using an earlier version of the TWLTL-SIM model. These combinations were intended to focus on the operational range of greatest interest to highway engineers; lower volume levels produce very little reduction in delay and higher volume levels produce jammed conditions where lengthy queues of vehicles waiting to turn left at driveways saturate the left lane of a four-lane undivided roadway or the single through lane of a two-lane undivided roadway. However, as explained in the discussion of the simulation results, the combinations chosen for the 4U/5T comparison proved to be more appropriate than those chosen for the 2U/3T comparison.

The directional split used for the model runs was 50/50; i.e., the flow rates given in Table D-1 were the same in both directions of travel. A substudy performed using a 60/40 split found that more delay resulted with the 50/50 split; with a 60/40 split there are more gaps in the lightly traveled direction to serve the larger turning volume in the heavily traveled direction. Thus, it appears that a 50/50 directional split results in the maximum delay.

For each driveway density simulated, the driveway locations input to the model consisted of equally spaced driveways staggered on opposite sides of the roadway. In all model runs, a left- or right-turning vehicle was equally likely to turn into any of the driveways on the appropriate side of the road, so all of the simulated driveways had the same turning volumes.

All model runs included 10 percent right turns into driveways and 10 percent right turns out of driveways in the 1,000-ft section. As noted earlier, these right-turn maneuvers have no direct impact on left-turn delay, but they do create gaps for use by left-turning vehicles and/or fill gaps that could otherwise be used by left-turning vehicles.

The travel speed on the arterial was assumed to be 40 mph for flow rates of 400 and 650 vph in each direction and 35 mph for flow rates of 900 and 1,000 vph; this difference in travel speed was intended to approximate the volume-density relationships known to exist for arterial highways.

Simulation Results

The simulation results for the comparison of two-lane undivided (2U) and three-lane TWLTL sections are given in Table D-3. Similar results for the comparison of four-lane undivided (4U) and five-lane TWLTL (5T) sections are presented in Table D-4. Each line in the table represents the average results obtained from three to five pairs of simulation runs for the same flow rate, driveway density, and left-turn volume. The model results presented in the table are the average delay reduction for through vehicles due to installation of a TWLTL, the average reduction in the number of stops by through vehicles due to installation of a TWLTL, and the average waiting time for left-turn vehicles, all expressed in units of vehicle-seconds per hour and vehicle-seconds per left-turn vehicle.

The simulation results presented in the tables should be interpreted in the following manner. Consider, for example, the first line of Table D-3, which represents a 2U/3T comparison for a flow rate of 400 vph in each direction, with 15 percent left turns in a 1,000-ft section and 30 driveways per mile. Under these conditions, the installation of a TWLTL results in 1,073 veh-sec per hour of delay reduction for through vehicles in one direction of travel on the 1,000-ft section, or 17.88 sec of delay reduction to through vehicles for each of the 60 vehicles per hour that turn left. Similarly, TWLTL installation resulted in a reduction of 232 stops per hour by through vehicles in the 1,000-ft section or 3.86 stops per left-turn vehicle. Left-turn vehicles were themselves delayed 337 veh-sec, or an average of 5.62 sec per left-turn vehicle, waiting for a suitable gap in opposing traffic to turn left.

The average delay reduction was found to be much more consistent than the average reduction in number of stops, so measures of effectiveness for TWLWL installation were based on delay reduction alone.

The results obtained from the simulation runs showed that the combinations of input conditions chosen for the 4U/5T comparison were very appropriate, while those chosen for the 2U/3T comparison were less so. For example, simulation runs

Table D-3. Results of simulation runs comparing two-lane undivided and three-lane TWLTL sections.

		Left-Turn Volume[a] in 1,000-ft Section		Average Delay Reduction (veh-sec) for through vehicles		Average Reduction in Number of Stops by Through Vehicles		Average Waiting Time (veh-sec) for Left-Turn Vehicles	
Flow Rate[a] (vph)	Driveway Density[b] (driveways/mile)	% of Through Volume	Turns/hr	Per Hour[a]	Per Left-Turn Vehicle	Per Hour[a]	Per Left-Turn Vehicle	Per Hour[a]	Per Left-Turn Vehicle
400	30	15	60	1,073	17.88	232	3.87	337	5.62
		20	80	1,370	17.13	250	3.13	368	4.61
		25	100	2,203	22.03	287	2.87	460	4.60
	60	15	60	535	8.92	140	2.33	218	3.63
		20	80	967	12.09	208	2.60	267	3.34
		25	100	1,042	10.42	207	2.07	288	2.88
	90	15	60	741	12.35	169	2.82	184	3.06
		20	80	1,030	12.87	216	2.70	264	3.30
		25	100	1,841	18.41	249	2.49	301	3.01
650	30	10	65	22,551	346.94	780	12.00	1,853	28.51
		15	98	39,905	407.20	799	8.15	2,517	25.68
		20	130	45,819	352.45	705	5.42	2,899	22.30
	60	10	65	33,492	515.27	866	13.32	1,070	16.46
		15	98	35,857	365.89	907	9.26	1,854	18.92
		20	130	41,224	317.11	881	6.78	1,937	14.90
	90	10	65	25,337	389.81	785	12.08	741	11.40
		15	98	23,911	243.99	879	8.97	996	10.16
		20	130	32,566	250.21	872	6.71	1,873	14.41
900	30	5	45	62,426	1,387.26	188	4.18	18,866	419.24

[a] In each direction of travel.
[b] Driveways per mile including driveways on both sides of highway.

Table D-4. Results of simulation runs comparing four-lane undivided and five-lane TWLTL sections.

Flow Rate[a] (vph)	Driveway Density[b] (driveways/mile)	Left-Turn Volume[a] in 1,000-ft Section: % of Through Volume	Turns/hr	Average Delay Reduction (veh-sec) for Through Vehicles: Per Hour[a]	Per Left-Turn Vehicle	Average Reduction in Number of Stops by Through Vehicles: Per Hour[a]	Per Left-Turn Vehicle	Waiting Time (veh-sec) for Left-Turn Vehicles: Per Hour[a]	Per Left-Turn Vehicle
650	30	7.5	49	480	9.80	120	2.45	342	6.99
		10.0	65	715	11.00	111	1.71	461	7.10
		12.5	81	795	9.82	131	1.62	475	5.87
	60	7.5	49	372	7.59	89	1.82	284	5.80
		10.0	65	507	7.80	122	1.88	312	4.80
		12.5	81	780	9.63	122	1.51	338	4.17
	90	7.5	49	359	7.34	80	1.63	199	4.06
		10.0	65	648	9.97	110	1.69	301	4.64
		12.5	81	530	6.55	112	1.38	311	3.85
900	30	5.0	45	1,977	43.94	297	6.60	613	13.63
		7.5	68	4,800	70.58	423	6.22	970	14.26
		10.0	90	6,084	67.60	488	5.42	1,183	13.15
	60	5.0	45	713	15.84	206	4.58	529	11.76
		7.5	68	4,569	67.19	668	9.82	918	13.50
		10.0	90	5,407	60.08	459	5.10	1,090	12.11
	90	5.0	45	765	17.00	198	4.40	325	7.23
		7.5	68	1,779	26.17	264	3.88	536	7.88
		10.0	90	6,072	67.47	489	5.43	960	10.66
1,100	30	2.5	28	25,895	924.80	938	33.50	1,057	37.74
		5.0	55	45,245	822.63	1,165	21.18	1,675	30.47
		7.5	83	59,278	714.19	1,143	13.77	2,614	31.50
	60	2.5	28	16,631	593.95	855	30.54	1,345	48.04
		5.0	55	42,640	775.26	1,337	24.31	1,505	27.36
		7.5	83	52,465	632.10	1,208	14.55	2,261	27.24
	90	2.5	28	22,184	792.30	928	33.14	680	24.29
		5.0	55	30,236	549.75	1,157	21.04	1,176	21.38
		7.5	83	40,607	489.25	1,072	12.92	2,131	25.67

[a] In each direction of travel.
[b] Driveways per mile including driveways on both sides of highway.

could not be made for a 900-vph flow rate in each direction of travel on a two-lane undivided roadway with more than 5 percent left turns in 1,000 ft, because jammed conditions resulted. This result is consistent with current knowledge about the capacity of two-lane undivided highways, which is estimated to be 2,000 to 2,800 vph in both directions for *uninterrupted* flow conditions (*30, 31*). It is not surprising that under interrupted flow conditions, with vehicles waiting to turn left at driveways, traffic flow should break down at 1,800 vph. Even for through volumes of 650 vph in each direction of travel on two-lane undivided highways, the model did not jam, but the turning volumes simulated produced very high delay values indicative of overcapacitated conditions. It should be noted that the simulation runs did not allow through vehicles to bypass turning vehicles on the right.

For the reasons presented above, the only simulation results used for the 2U/3T comparison were those for a flow rate of 400 vph in each direction of travel. These results are presented above the line in the center of Table D-3 and are in good agreement with the results obtained from field data by Harwood and St. John (*6*) (see Chapter Two). While not quantitatively meaningful, the results presented below the line in Table D-3 illustrate that above a flow rate of 500 to 600 vph in each direction, a two-lane undivided roadway with even moderate left-turn demand will be overcapacitated and in need of an improved design alternative.

Figure D-2 presents the data for the 4U/5T comparison at a flow rate of 650 vph in each direction. Although the individual points in the figure do not show a completely consistent trend in the variation of delay with driveway density and left-turn volume for a given flow rate, such a trend is evident in the regression lines shown in the figure for each of the three driveway densities. The regression lines show that for a given flow rate and left-turn volume, there is more delay if that left-turn volume is concentrated at a few driveways than if that same volume is spread over many driveways.

Figure D-3 shows that these same trends as shown in Figure D-2 were observed for every volume level in the 4U/5T comparisons. For the 2U/3T comparison at 400 vph, a slightly larger delay reduction was found at 60 driveways per mile than at 90 driveways per mile. However, because the delay reduction for the 60 and 90 driveways per mile was less than the delay reduction for 30 driveways per mile, a decision was reached to combine the 60 and 90 levels into a single regression line so as not to provide a prediction of delay reduction inconsistent with the remainder of the findings. The combined regression line for

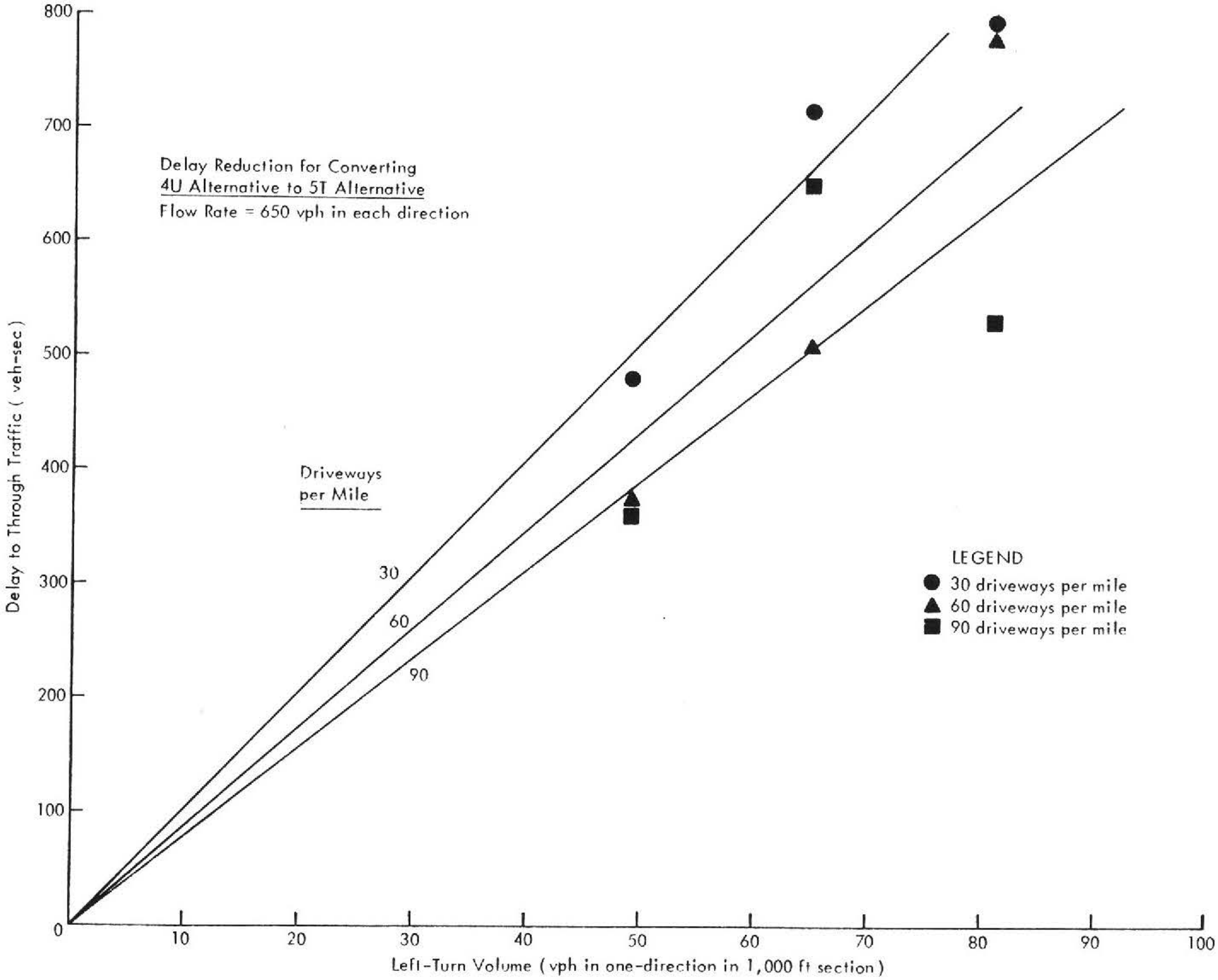

Figure D-2. Delay reduction for converting from 4U to 5T design alternative for flow rate of 650 vph.

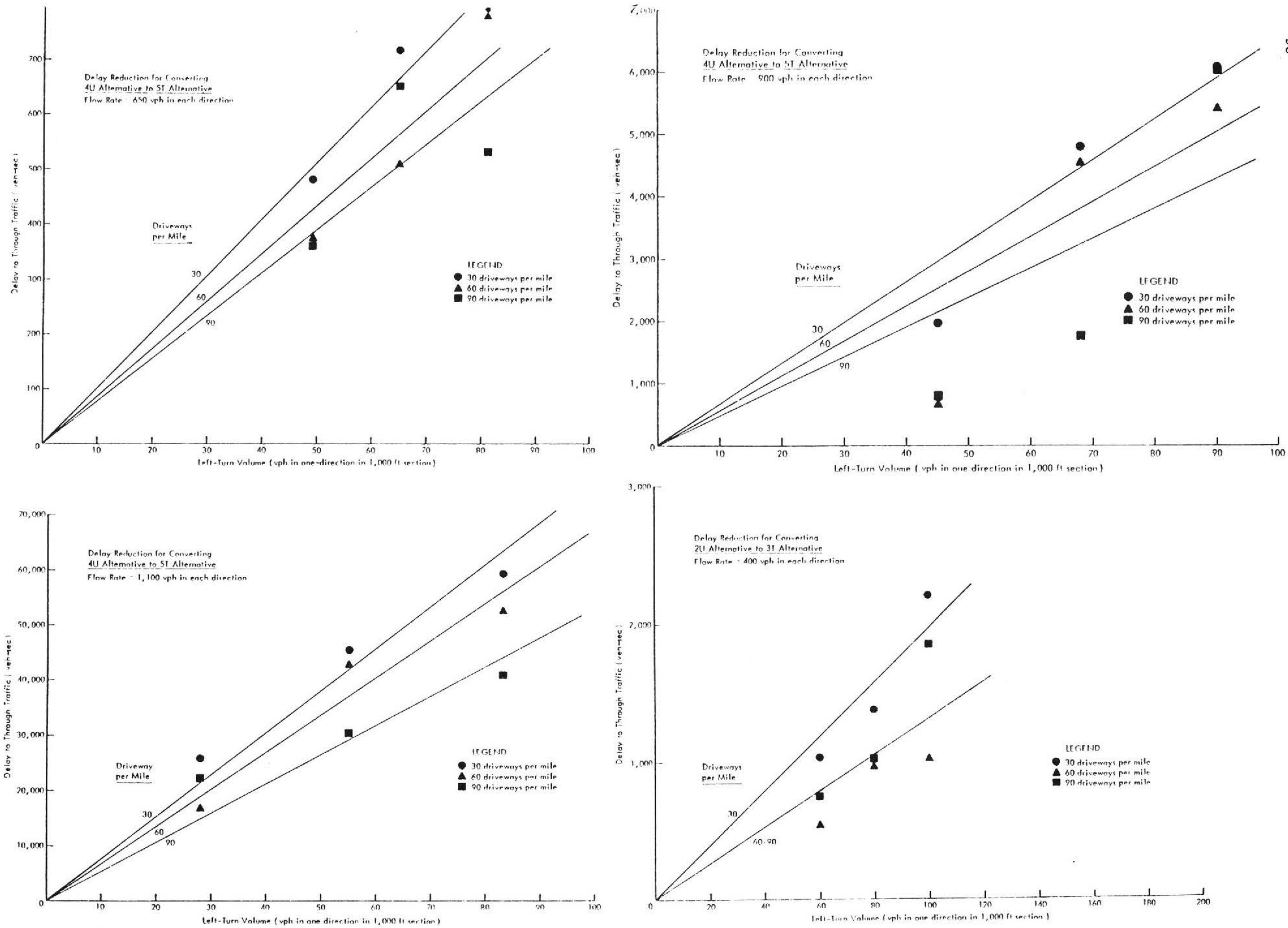

Figure D-3. Operational effectiveness measures for installation of TWLTLs.

the 2U/3T comparison at a flow rate of 400 vph is shown in Figure D-3.

The relationships between through traffic delay and left-turn volume shown in Figures D-2 and D-3 have been represented as straight lines passing through the origin. Theoretically, these relationships must pass through the origin, because no delay to through traffic would be expected at zero left-turn volume. There is no theoretical reason that these relationships are necessarily linear, and there is some suggestion from the data that they could be nonlinear with increasing slope at higher left-turn volumes. However, the limited amount of data available and the lack of precision in the simulation model results do not permit any interpretation of the model results more sophisticated than a simple linear fit.

The slope of each regression line in Figure D-3 is tabulated in Table D-5. These slopes represent delay reduction measures of effectiveness for TWLTL installation. The total delay to through vehicles in one direction for any length of section can be obtained by multiplying one of the values from Table D-5 by the traffic volume turning left in that section. Interpolation within the range of traffic volumes and driveway densities covered by the model runs and extrapolation within a limited span above that range is considered acceptable.

The use of these results to assess the operational effectiveness of highway improvements involving TWLTLs is addressed in Chapter Two of the report and in the design examples in Appendix F. The simulation results presented above have also been employed in Appendix E, together with analytical results, to assess the operational effectiveness of raised medians.

Table D-5. Delay reduction estimates for installation of TWLTLs on suburban highways.

Flow Rate (vph)[a]	Driveways per Mile	Delay Reduction (veh-sec per left-turn vehicle)[a] 2U vs. 3T	4U vs. 5T
400	30	+19.7	+6.3
	60	+13.1	+5.4
	90	+13.1	+4.8
650	30	-	+10.2
	60	-	+8.7
	90	-	+7.8
900	30	-	+65.4
	60	-	+56.3
	90	-	+47.8
1,100	30	-	+764.2
	60	-	+673.5
	90	-	+531.1

[a] In one direction of travel.

APPENDIX E

ESTIMATION OF OPERATIONAL EFFECTS OF RAISED MEDIANS

This appendix presents estimates of the operational effects of raised medians on suburban arterials. The results presented here represent an estimate of the reduction in delay that results from installing a raised median on a four-lane undivided arterial. The estimates are based in part on the results obtained with the TWLTL-SIM model presented in Appendix D and in part on estimates made using the signalized intersection analysis procedure in the 1985 *Highway Capacity Manual.* The results of the operational comparison between four-lane undivided and four-lane divided sections are compared with the effectiveness of five-lane TWLTL sections.

OPERATIONAL ANALYSIS APPROACH FOR RAISED MEDIANS

The operational effectiveness of a four-lane divided arterial with a raised median cannot be addressed directly with the TWLTL-SIM model because the model is intended to evaluate only undivided and TWLTL alternatives. However, indirect estimates of the effectiveness of a median divider can be developed combining the model results with estimates based on revised capacity analysis procedures.

The installation of a raised median provides an operational advantage to through vehicles similar to the installation of a TWLTL by eliminating delays due to left turns at most driveways, but it accomplishes this at the cost of forcing drivers that desire to turn left to reach their destinations by some other route. Several scenarios of how such drivers will reach their destinations are possible: some may go past their destination and make a U-turn at the next intersection with a median opening, while others may choose an entirely different route from their origin that brings them to their destination in the direction of travel from which they can make a right turn into the desired driveway. The first scenario is common and is used on many arterial streets where the left-turn lanes at signalized intersections are signed for use by left-turn and U-turn vehicles.

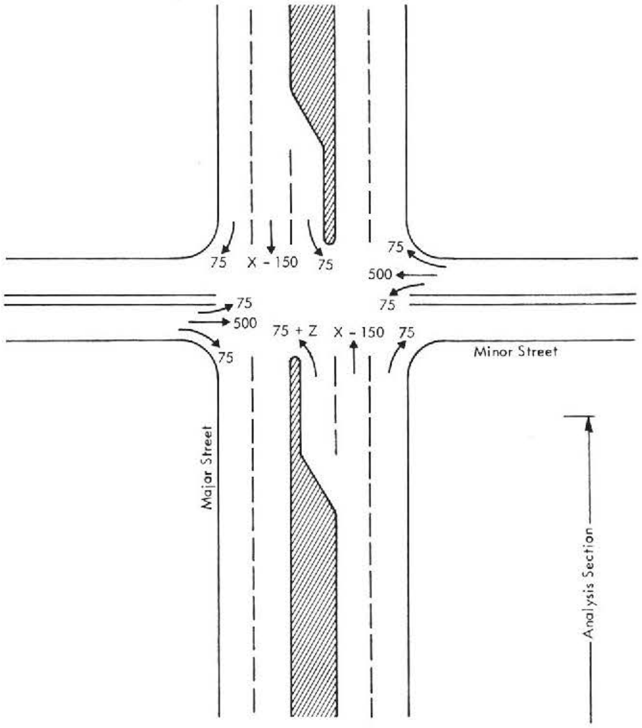

Figure E-1. Traffic flow rates and geometrics assumed for signalized intersection at boundary of analysis section.

This scenario can be analyzed in a relatively straightforward manner. The second scenario involves a myriad of possible routes to any given destination, but it can be assumed that a driver would not choose to use another route unless he perceived that route to involve less delay than the U-turn scenario.

The reduction in delay that results from upgrading a four-lane undivided arterial to a four-lane divided arterial with a raised median can be estimated from five components:

- Reduction in delay to through vehicles because they are not delayed by vehicles waiting to turn left at midblock driveways (C_1).
- Reduction in delay to left-turning vehicles by not having to wait for gaps in opposing traffic at a midblock driveway (C_2).
- Increase in travel time for left-turning vehicles as they proceed to the next intersection and return to their destination after making a U-turn (C_3).
- Increased delay to U-turning vehicles as they wait to make a left turn at a signalized intersection (C_4).
- Increased delay to all other vehicles at the signalized intersection due to increased left-turn volumes resulting from the U-turn demand (C_5).

Components C_1 and C_2 can be estimated with the results from the TWLTL-SIM model. Component C_3 can be estimated from the average running speed of the arterial and the average distance from a driveway to the next signalized intersection. Components C_4 and C_5 can be estimated using the signalized intersection procedure presented in Chapter 9 of the 1985 *Highway Capacity Manual* (HCM). A major advantage of the revised procedure over the existing HCM signalized intersection procedure is that the revised procedure provides an explicit measure of delay for use as a measure of effectiveness. Consideration was also given to estimation of the fourth and fifth components for unsignalized intersections as well, but the revised HCM procedures for unsignalized intersections provide only a general level of delay rather than an explicit estimate that can be used for this purpose.

The U-turn scenario was analyzed for each of the flow rate, left-turn volume, and driveway density levels considered on four-lane facilities in Appendix D. It was assumed that a driver denied the opportunity to turn left by the presence of a median would travel 500 ft to a signalized intersection, complete a U-turn during a separate left-turn signal phase, and return 500 ft to his destination. The same assumptions as were made in Appendix D concerning average running speed on the arterial highway were also made in this analysis; the assumed average running speed was 40 mph for a flow rate of 650 vph in each direction and 35 mph for flow rates of 900 and 1,100 vph in each direction. These assumptions concerning additional travel time are somewhat arbitrary, but not unreasonable, and the results obtained indicate that the additional travel time (Component C_3) is a relatively small part of the total operational effect of the median divider.

The analysis of the U-turn scenario also required assumptions concerning the traffic volumes and signal timing and phasing at the signalized intersection where the drivers make their left turns. All operational conditions at the signalized intersection (including the major street left-turn volumes, minor street volumes, signal cycle length and phasing) were kept constant and only the major street through volume and the increase in left-turn volume due to the U-turn demand were allowed to vary. Figure E-1 shows the assumed conditions at the signalized intersection. The major street through volume, X, was either 650, 900, or 1,100 vph in each direction, while the major street left-turn volume of 75 vph was increased by a U-turn volume, Z, of 45 to 90 vph depending on the left-turn demand in the upstream section.

The signalized intersection capacity procedure in Chapter 9 of the 1985 HCM allows the computation of delay estimates per vehicle for individual signal phases (including the left-turn phase) and for the intersection as a whole. The estimates for this analysis were developed by determining a signal timing for each level of through traffic volume, X, and applying the revised HCM procedure to that base condition with no U-turn volume ($Z = 0$ vph). Then, as each increment of U-turn volume, Z, was added to the left-turn phase, the signal timing was recomputed accordingly and the capacity analysis repeated. The differences between the base condition ($Z = 0$) and each subsequent case ($Z > 0$) in left-turn delay per left-turn vehicle and delay per entering vehicle on all other phases were used as measures of effectiveness. It should be noted that the delay estimates provided by the capacity analysis procedure in Chapter 9 are applicable to the peak 15-min period within a particular hour rather than to the entire hour. There is no established method to convert the peak 15-min delay per vehicle to an hourly delay per vehicle. We chose to make this conversion by reducing the peak 15-min delay per vehicle by the peak-hour factor (in this case, 0.85).

DELAY REDUCTION ESTIMATES FOR RAISED MEDIANS

Table E-1 illustrates the results of the comparison of the four-

Table E-1. Operational effectiveness of four-lane divided compared to four-lane undivided alternative for 30 driveways per mile.

Flow Rate[a] (vph)	Left-Turn Demand[a] in 1,000-Ft Section %	vph	Components of Delay Reduction[a] (veh-sec per hr) C_1	C_2	C_3	C_4	C_5	Net Delay Reduction (veh-sec) Per Hour[a]	Per Left-Turn Vehicle
650	7.5	49	499.3	342.4	-835.5	-1,137.5	-3,221.9	-4,353.2	-88.8
650	10.0	65	622.4	461.4	-1,108.3	-1,770.2	-3,408.9	-5,163.6	-79.4
650	12.5	81	825.4	475.3	-1,381.1	-2,293.4	-5,557.8	-7,931.6	-97.9
900	5.0	45	2,944.4	613.2	-876.6	-1,670.4	+527.6	+483.0	+10.7
900	7.5	68	4,449.2	970.3	-1,324.4	-3,174.4	-3,539.8	-2,619.3	-38.5
900	10.0	90	5,888.7	1,183.1	-1,753.2	-3,506.0	-10,906.6	-9,094.0	-101.0
1,100	2.5	28	21,397.6	1,056.8	-545.4	-1,160.7	-4,931.6	+15,816.7	+564.9
1,100	5.0	55	42,031.0	1,675.9	-1,071.4	-2,987.8	-7,145.6	+32,502.1	+590.9
1,100	7.5	83	63,428.6	2,614.3	-1,616.8	-4,336.7	+13,568.7	+46,520.7	+560.4

[a] In each direction of travel.

lane undivided and four-lane divided alternatives for selected levels of through-traffic volume and left-turn demand. The table presents the estimates for each of the five delay reduction components (C_1 through C_5) and their sum, the total delay reduction, expressed on a per hour basis and a per left-turn vehicle basis. It should be noted that all of the data in Table E-1 represent one direction of travel on an arterial highway and the delay reductions should be doubled to obtain estimates for both directions of travel. Increases in delay are represented in the table by negative values of delay reduction. Comparable data are presented in Tables E-2 and E-3 for driveway densities of 60 driveways per mile and 90 driveways per mile, respectively.

Table E-2. Operational effectiveness of four-lane divided compared to four-lane undivided alternative for 60 driveways per mile.

Flow Rate[a] (vph)	Left-Turn Demand[a] in 1,000-Ft Section %	vph	Components of Delay Reduction[a] (veh-sec per hr) C_1	C_2	C_3	C_4	C_5	Net Delay Reduction (veh-sec) Per Hour[a]	Per Left-Turn Vehicle
650	7.5	49	424.3	284.2	-835.5	-1,137.5	-3,221.9	-4,385.9	-91.5
650	10.0	65	563.6	311.9	-1,108.3	-1,770.2	-3,408.9	-5,411.9	-83.3
650	12.5	81	702.3	338.0	-1,381.1	-2,293.4	-5,557.8	-8,192.0	-101.1
900	5.0	45	2,535.3	529.1	-876.6	-1,670.4	+527.6	+10.2	+0.2
900	7.5	68	3,831.1	918.3	-1,324.4	-3,174.4	-3,539.8	-3,289.4	-48.4
900	10.0	90	5,070.6	1,190.0	-1,753.2	-3,506.0	-10,906.6	-10,005.2	-111.2
1,100	2.5	28	18,858.6	1,345.0	-545.4	-1,160.7	-4,931.6	+13,565.9	+484.5
1,100	5.0	55	37,043.6	1,505.1	-1,071.4	-2,987.8	-7,145.6	+27,343.9	+497.2
1,100	7.5	83	55,902.2	2,261.3	-1,616.8	-4,336.7	+13,568.7	+38,641.3	+465.6

[a] In each direction of travel.

Table E-3. Operational effectiveness of four-lane divided compared to four-lane undivided alternative for 90 driveways per mile.

Flow Rate[a] (vph)	Left-Turn Demand[a] in 1,000-Ft Section %	vph	Components of Delay Reduction[a] (veh-sec per hr) C_1	C_2	C_3	C_4	C_5	Net Delay Reduction (veh-sec) Per Hour[a]	Per Left-Turn Vehicle
650	7.5	49	381.2	199.1	-835.5	-1,137.5	-3,221.9	-4,614.6	-94.2
650	10.0	65	505.7	301.4	-1,108.3	-1,770.2	-3,408.9	-5,480.3	-84.3
650	12.5	81	630.2	311.5	-1,381.1	-2,293.4	-5,557.8	-8,290.6	-102.4
900	5.0	45	2,149.8	325.5	-876.6	-1,670.4	+527.6	+599.3	+13.3
900	7.5	68	3,247.0	536.1	-1,324.4	-3,174.4	-3,539.8	-4,255.7	-62.6
900	10.0	90	4,297.5	959.8	-1,753.2	-3,506.0	-10,906.6	-10,908.5	-121.2
1,100	2.5	28	14,871.1	680.1	-545.4	-1,160.7	-4,931.6	+8,913.5	+318.3
1,100	5.0	55	29,211.1	1,175.9	-1,071.4	-2,987.8	-7,145.6	+19,182.2	+348.8
1,100	7.5	83	44,082.1	2,131.0	-1,616.8	-4,336.7	+13,568.7	+26,690.9	+321.6

[a] In each direction of travel.

The delay reduction components in Tables E-1 through E-3 should be interpreted in the following manner. Component C_1 represents the reduction in delay to through vehicles because they are not delayed by vehicles waiting to turn left at midblock driveways. This is numerically the same as the effectiveness of a TWLTL in accomplishing this same objective and has been computed as the delay reduction per left-turn vehicle tabulated in Table D-5 for the appropriate flow rate and driveway density multiplied by the left-turn demand per hour.

Component C_2 represents the reduction in delay to left-turning vehicles by not having to wait for gaps in opposing traffic at a midblock driveway. This component is equal to the left-turn waiting time at an undivided or TWLTL section, which has been tabulated in Table D-4.

Component C_3 represents the increase in travel time for left-turning vehicles as they proceed to the next intersection and return to their destination after making a U-turn. This component was computed as the additional travel distance (assumed to be 1,000 ft) divided by the assumed average running speed and multiplied by the left-turn demand per hour. Component C_3 has a negative sign because it represents an increase rather than a reduction in delay.

Component C_4 is the increased delay to the U-turning vehicles as they wait to make a left turn at a signalized intersection. This component is the left-turn demand multiplied by the difference between the average delay per vehicle for the left-turn phase when serving both the left-turn and U-turn volumes and the average delay per vehicle for the left-turn phase when serving just the left-turn volume. The estimates of the average delay per vehicle were computed using the procedures of Chapter 9 of the 1985 HCM. Component C_4 has a negative sign because it represents an increase rather than a reduction in delay.

Component C_5 is the increase in delay to all other vehicles at the signalized intersection due to the increased left-turn volume resulting from the U-turn demand. This component is the difference between total intersection delay with and without the U-turn volume, reduced to eliminate Component C_4. Component C_5 was also computed with the Chapter 9 procedure and has a negative sign because it represents an increase, rather than a reduction, in delay.

OPERATIONAL COMPARISON OF RAISED MEDIANS AND TWLTLs

The operational analysis approach presented in Tables E-1 through E-3 can also be used to compare the 4D and 5T alternatives. Each of the five delay reduction components except Component C_1 is applicable to the 4D/5T comparison. Component C_1 is eliminated because both the 4U and 5T alternatives effectively eliminate delay to through vehicles caused by vehicles waiting to complete a left-turn and, therefore, do not differ in Component C_1.

The operational comparison between the 4D and 5T alternatives is presented in Tables E-4 through E-6, which are analogous to Tables E-1 through E-3, respectively, with Component C_1 set to zero.

SUMMARY OF OPERATIONAL COMPARISONS OF MULTILANE DESIGN ALTERNATIVES

Table E-7 presents a summary of each of the operational comparisons between design alternatives developed in Appendixes D and E. The comparisons presented in the table include four-lane undivided 4U vs. 4D, 4D vs. 5T, and 4U vs. 5T. In each design alternative comparison (e.g., 4U vs. 4D), a positive value indicates that the first design alternative is operationally preferable, while a negative sign indicates that the second design alternative is operationally preferable.

Two major conclusions are evident in Table E-7. First, at flow rates of 900 vph and below, median dividers generally result in an increase in delay. However, at flow rates of 1,100 vph and above, the installation of a median divider on an undivided street reduces delay, even for minimal levels of left-turn demand. These results suggest that the breakpoint where a median divider begins to provide operational benefits is a flow rate of approximately 1,000 vph in each direction of travel. This result does not mean that raised medians should not be used at lower flow rates, but it does imply that raised medians should be used only when there are other benefits that offset the operational disadvantages of the raised median.

Second, the 5T design alternative is preferable to both the 4U and 4D design alternatives for all levels of flow rate, left-turn demand, and driveway density. This result provides strong evidence that, strictly from an operational standpoint, the use of a TWLTL is a highly desirable alternative in a wide variety of design situations. Of course, as discussed in Chapter Two of this report, a wide variety of other factors including safety, right-of-way restrictions, land-use planning, and construction cost need to be considered.

Table E-4. Operational effectiveness of four-lane divided compared to five-lane TWLTL alternative for 30 driveways per mile.

Flow Rate[a] (vph)	Left-Turn Demand[a] in 1,000-Ft Section		Components of Delay Reduction[a] (veh-sec per hr)					Net Delay Reduction (veh-sec)	
	%	vph	C_1	C_2	C_3	C_4	C_5	Per Hour[a]	Per Left-Turn Vehicle
650	7.5	49	0	342.4	-835.5	-1,137.5	-3,221.9	-4,852.5	-99.0
650	10.0	65	0	461.4	-1,108.3	-1,770.2	-3,408.9	-5,826.0	-89.6
650	12.5	81	0	475.3	-1,381.1	-2,293.4	-5,557.8	-8,757.0	-108.1
900	5.0	45	0	613.2	-876.6	-1,670.4	+527.6	-2,461.4	-54.7
900	7.5	68	0	970.3	-1,324.4	-3,174.4	-3,539.8	-7,068.5	-103.9
900	10.0	90	0	1,183.1	-1,753.2	-3,506.0	-10,906.6	-14,982.7	-166.5
1,100	2.5	28	0	1,056.8	-545.4	-1,160.7	-4,931.6	-5,580.9	-199.3
1,100	5.0	55	0	1,675.9	-1,071.4	-2,987.8	-7,145.6	-9,528.9	-173.3
1,100	7.5	83	0	2,614.3	-1,616.8	-4,336.7	+13,568.7	-16,907.9	-203.7

[a] In each direction of travel.

Table E-5. Operational effectiveness of four-lane divided compared to five-lane TWLTL alternative for 60 driveways per mile.

Flow Rate[a] (vph)	Left-Turn Demand[a] in 1,000-Ft Section		Components of Delay Reduction[a] (veh-sec per hr)					Net Delay Reduction (veh-sec)	
	%	vph	C_1	C_2	C_3	C_4	C_5	Per Hour[a]	Per Left-Turn Vehicle
650	7.5	49	0	284.2	-835.5	-1,137.5	-3,221.9	-4,910.7	-100.2
650	10.0	65	0	311.9	-1,108.3	-1,770.2	-3,408.9	-5,975.5	-91.9
650	12.5	81	0	338.0	-1,381.1	-2,293.4	-5,557.8	-8,894.3	-109.8
900	5.0	45	0	529.1	-876.6	-1,670.4	+527.6	-2,545.5	+56.6
900	7.5	68	0	918.3	-1,324.4	-3,174.4	-3,539.8	-7,120.5	-104.7
900	10.0	90	0	1,090.0	-1,753.2	-3,506.0	-10,906.6	-15,075.8	-167.5
1,100	2.5	28	0	1,345.0	-545.4	-1,160.7	-4,931.6	-5,292.7	-189.0
1,100	5.0	55	0	1,505.1	-1,071.4	-2,987.8	-7,145.6	-9,699.7	-176.4
1,100	7.5	83	0	2,261.8	-1,616.8	-4,336.7	+13,568.7	-17,260.9	-208.0

[a] In each direction of travel.

Table E-6. Operational effectiveness of four-lane divided compared to five-lane TWLTL alternative for 90 driveways per mile.

Flow Rate[a] (vph)	Left-Turn Demand[a] in 1,000-Ft Section		Components of Delay Reduction[a] (veh-sec per hr)					Net Delay Reduction (veh-sec)	
	%	vph	C_1	C_2	C_3	C_4	C_5	Per Hour[a]	Per Left-Turn Vehicle
650	7.5	49	0	199.1	-835.5	-1,137.5	-3,221.9	-4,995.8	-102.0
650	10.0	65	0	301.4	-1,108.3	-1,770.2	-3,408.9	-5,986.0	-92.1
650	12.5	81	0	311.5	-1,381.1	-2,293.4	-5,557.8	-8,920.8	-110.2
900	5.0	45	0	325.5	-876.6	-1,670.4	+527.6	-2,749.1	+61.1
900	7.5	68	0	536.1	-1,324.4	-3,174.4	-3,539.8	-7,520.7	-110.3
900	10.0	90	0	959.8	-1,753.2	-3,506.0	-10,906.6	-15,206.0	-169.0
1,100	2.5	28	0	680.1	-545.4	-1,160.7	-4,931.6	-5,957.6	-212.8
1,100	5.0	55	0	1,175.9	-1,071.4	-2,987.8	-7,145.6	-10,028.9	-182.3
1,100	7.5	83	0	2,131.0	-1,616.8	-4,336.7	+13,568.7	-17,391.2	-209.5

[a] In each direction of travel.

Table E-7. Summary of operational comparisons of multilane design alternatives.

Flow Rate (vph)	Left-Turn Demand[a] in 1,000-ft Section %	Per Hour	4U vs. 4D Delay Reduction (veh-sec) Per Hour[a]	Per Left-Turn Veh.	4D vs. 5T Delay Reduction (veh-sec) Per Hour[a]	Per Left-Turn Veh.	4U vs. 5T Delay Reduction (veh-sec) Per Hour[a]	Per Left-Turn Veh.
				30 Driveways Per Mile				
650	7.5	49	-4,353.2	-88.8	-4,852.5	+99.0	+499.3	+10.2
650	10.0	65	-5,163.6	-79.4	+5,826.0	+89.6	+662.4	+10.2
650	12.5	81	-7,931.6	-97.9	+8,757.0	+108.1	+825.4	+10.2
900	5.0	45	+483.0	+10.7	+2,461.4	+54.7	+2,944.4	+65.4
900	7.5	68	-2,619.3	-38.5	+7,068.5	+103.9	+4,449.2	+65.4
900	10.0	90	-9,094.0	-101.0	+14,982.7	+166.5	+5,888.7	+65.4
1,100	2.5	28	+15,816.7	+564.9	+5,580.9	+199.3	+21,397.6	+764.2
1,100	5.0	55	+32,502.1	+590.9	+9,528.9	+173.3	+42,031.0	+764.2
1,100	7.5	83	+46,520.7	+560.4	+16,907.9	+203.7	+63,428.6	+764.2
				60 Driveways Per Mile				
650	7.5	49	-4,485.9	-91.5	-4,910.7	+100.2	+428.8	+8.7
650	10.0	65	-5,411.9	-83.3	+5,975.5	+91.9	+513.5	+8.7
650	12.5	81	-8,192.0	-101.1	+8,894.3	+109.8	+702.3	+8.7
900	5.0	45	+10.2	+0.2	+2,545.5	+56.6	+2,535.3	+56.3
900	7.5	68	-3,289.4	-48.4	+7,120.5	+104.7	+3,831.1	+56.3
900	10.0	90	-10,005.2	-111.2	+15,075.8	+167.5	+5,070.6	+56.3
1,100	2.5	28	+13,565.9	+484.5	+5,292.7	+189.0	+18,858.6	+673.5
1,100	5.0	55	+27,343.9	+497.2	+9,699.7	+176.4	+37,043.6	+673.5
1,100	7.5	83	+38,641.3	+465.6	+17,260.9	+208.0	+55,902.2	+673.4
				90 Driveways Per Mile				
650	7.5	49	-4,614.6	-94.2	-4,995.8	+102.0	+381.2	+7.8
650	10.0	65	-5,480.3	-84.3	+5,986.0	+92.1	+505.7	+7.8
650	12.5	81	-8,290.6	-102.4	+8,920.8	+110.2	+630.2	+7.8
900	5.0	45	-599.3	-13.3	+2,749.1	+61.1	+2,149.8	+47.8
900	7.5	68	-4,255.7	-62.6	+7,502.7	+110.3	+3,247.0	+47.8
900	10.0	90	-10,908.5	-121.2	+15,206.0	+169.0	+4,297.5	+47.8
1,100	2.5	28	+8,913.5	+318.3	+5,957.6	+212.8	+14,871.1	+531.1
1,100	5.0	55	+19,182.2	+348.8	+10,028.9	+182.3	+29,211.1	+531.1
1,100	7.5	83	+26,690.9	+321.6	+17,391.2	+209.5	+44,082.1	+531.1

[a] In each direction of travel.

APPENDIX F

DESIGN EXAMPLES

This appendix presents three design examples that illustrate the application of the procedure for selecting multilane design alternatives for suburban highways presented in this report. The design examples address the following situations:

- Improvement of a two-lane undivided (2U) design to the three-lane TWLTL (3T) design alternative.
- Improvement of a two-lane undivided (2U) design to the five-lane TWLTL (5T) design alternative.
- Improvement of a four-lane undivided (4U) design to the four-lane divided (4D) design alternative.

The design examples are based on actual sites, although some of the data have been changed to make the examples more illustrative. The design examples follow the outline of the design alternative selection procedure presented in Table 10. The examples recommend the ultimate design alternative for each site and any appropriate staged construction approaches.

DESIGN EXAMPLE 1

This design example illustrates a two-lane undivided (2U) suburban arterial appropriate for conversion to the three-lane TWLTL (3T) design alternative, with possible later upgrading to the five-lane TWLTL (5T) design alternative.

Existing Conditions

The highway section in question is 0.5 miles long and is located on the outskirts of a small city with a population of 23,000. The highway section has relatively dense strip commercial development with 41 driveways, three unsignalized intersections, and no signalized intersections. Table F-1 summarizes the existing geometrics, traffic control, and operational demand at the site.

Observation of traffic flow during the peak hour indicates an unacceptable level of service due to delays to through vehicles caused by vehicles waiting for a gap in opposing traffic to turn left. These delays are unavoidable with the current geometrics at the site because there is no provision for left turns in the highway median and no shoulder for through vehicles to bypass left-turning vehicles.

The site has substantial safety problems related to turning maneuvers. Table F-2 summarizes the accidents experienced for two calendar years—1981 and 1982. The table shows an average rate of 5.3 accidents per MVM for nonintersection locations and 14.2 accidents per MVM for unsignalized intersections, compared to the typical rates of 2.8 and 2.3 accidents per MVM, respectively, determined from Tables 1 and 2 (see Chapter Two). This comparison shows higher than expected accident experience for both nonintersection locations and unsignalized intersections, with the most serious problems associated with the three unsignalized intersections.

The accidents occurring at the site are not particularly severe; only 26 of the 85 accidents at the site (30.6 percent) involved a fatality or an injury, in comparison to 38 to 39 percent that is typical for commercial 2U sites (see Table 4). However, 67 of the 85 accidents (78.8 percent) involve multiple-vehicle head-on, rear-end, and angle collisions—types of accidents that are susceptible to correction through installation of a raised median or a TWLTL. This suggests that an upgrading project would be particularly effective in improving safety at this site since only 50 to 56 percent of accidents at commercial 2U sites typically involve head-on, rear-end, and angle collisions.

Table F-1. Existing conditions—Design Example 1.

Section length:	0.5 miles
Existing design alternative:	Two-lane undivided (2U)
Average daily traffic volume:	10,000 vpd
Peak hour flow rate (one-way):	450 vph
Left-turn demand (one-way):	90 left turns/hr/mile
Percent trucks:	8.6%
Type of development:	Commercial
Driveway density:	82 driveways per mile
Unsignalized intersections:	6 intersections per mile
Signalized intersections:	None
Speed limit:	35 mph
Lane width:	11 ft
Shoulders:	None
Pedestrian activity:	Low

Projected Future Conditions

The traffic operational demands at this site are not expected to change dramatically. The adjacent land is fully developed and no increase in the density of development is expected. The current traffic volumes are not growing. Over the last 5 years, the average daily traffic volume has increased in some years and decreased in others. No growth is expected over the next 5 years and only moderate growth over the next 20 years. The average daily traffic volume is expected to grow 50 percent to 15,000 vpd over the 20-year design period with proportional increases in the peak-hour flow rate and the left-turn demand.

Constraints

There are no right-of-way restrictions that will limit the choice of design alternative. Public opinion and adjacent landowners are supportive of an improvement that will ease congestion on this highway section.

Table F-2. Existing accident experience—Design Example 1.

	Accident Frequencies and Rates					
				(Accidents per million veh-miles)		
Year	Nonintersection Accidents	Unsignalized Intersection Accidents	Total Accidents	Nonintersection Accident Rate	Unsignalized Intersection Accident Rate	Total Accident Rate
1981	16	35	51	7.5	16.4	23.9
1982	7	27	34	3.3	12.6	15.9
Total	23	62	85	5.4	14.5	19.9

	Accident Type and Severity Distribution						
	Accident Type					Accident Severity	
Year	Single Vehicle	Head-on	Rear-end	Angle	Other Multiple Vehicle	Fatal and Injury	Property-Damage-Only
1981	5	4	14	20	8	17	34
1982	1	1	12	16	4	9	25
	6	5	26	36	12	26	59

Priorities

The highway section is classified as a major arterial, so an upgrading project to ease the movement of through traffic is appropriate. The site is already commercially developed, so an improvement to serve the existing development while reducing the interference with existing traffic is desirable.

Basic Number of Through Lanes

The current peak-hour flow rate of 450 vph can be served adequately with one through lane in each direction of travel if the interferences caused by left-turning traffic can be reduced. Two through lanes in each direction of travel will be needed to serve to a 20-year projected peak-hour flow rate of 675 vph.

Feasible Design Alternatives

There are three feasible design alternatives that provide two through-traffic lanes in each direction to serve the projected future traffic. These are: four-lane undivided (4U), four-lane divided (4D), and five-lane with TWLTL (5T). The three-lane TWLTL (3T) design alternative should also be considered as a possible first stage to the 5T alternative.

Geometric Variations

There are no right-of-way restrictions along the section that would limit the incorporation of a full shoulder in any of the design alternatives under consideration. Therefore, each design alternative will be considered both with and without a full shoulder.

Benefits and Disbenefits

Table F-3 compares the accident rate data for the existing condition to the typical accident rates for a 2U design and for each of the feasible design alternatives with and without a full shoulder. Data from Table 3 (Chapter Two) suggest that at an average or typical site conversion from the 2U to the 3T design alternative might reduce accident rate by 11 percent and conversion to the 4D or 5T alternatives could increase accident rate. However, the site considered in this example is not an average or typical site because of the peak-hour congestion, the high accident rate (four times the average for a 2U section), and the high proportion of accidents susceptible to correction. Thus, it is more reasonable to assume a much higher safety effectiveness estimate for this particular site.

The design alternatives that directly remedy head-on, rear-end, and angle accidents (3T, 4D, and 5T) are estimated to reduce the accident rate from its current level (19.5 accidents per MVM) to the average rate for each of those treatments. The 4U design alternative, which makes no special provision for left-turn movements, does not directly remedy head-on, rear-end, and angle accidents and would be expected to be much less effective than the 3T, 4D, or 5T alternatives in reducing the accident experience at this site. The expected accident rate reductions for 3T, 4U, and 5T design alternatives are quite com-

Table F-3. Estimated accident rates for design alternatives—Design Example 1.

	Accident Rate (accidents per million veh-miles)			
Design Alternatives	Nonintersection Accidents	Unsignalized Intersection Accidents	Total Accidents	Expected Accident Rate Reduction
2U - actual	5.3	14.2	19.5	-
2U - no shoulder	2.8	2.3	5.1	-
2U - full shoulder	2.5	2.3	4.8	-
3T - no shoulder	1.9	2.6	4.5	15.0
3T - full shoulder	1.8	2.6	4.4	15.1
4U - no shoulder	3.3	5.0	8.3	-
4U - full shoulder	3.0	5.0	8.0	-
4D - no shoulder	3.3	4.9	8.2	11.3
4D - full shoulder	3.0	4.9	7.9	11.6
5T - no shoulder	3.1	3.3	6.4	13.1
5T - full shoulder	2.8	3.3	6.1	13.4

parable, ranging from 11.3 to 15.1 accidents per MVM. This accident rate reduction corresponds to a reduction of 24 to 32 accidents per year at current traffic volume levels and 35 to 47 accidents per year at the projected future traffic volume level, so any of these design alternatives would be highly desirable from a safety viewpoint. Each of these design alternatives would also be expected to reduce the proportion of fatal and injury accidents at the site.

The operational effectiveness of the design alternatives is assessed in the following manner. Table 6 (see Chapter Two) shows the installation of the 3T alternative for a flow rate of 400 vph and 60 to 90 driveways per mile would be expected to reduce through-vehicle delay by 13.1 veh-sec per left-turn vehicle. For the actual flow rate of 450 vph at this site, the delay reduction estimate should be increased proportionally to 14.7 veh-sec per left-turn vehicle, or 2,650 veh-sec per hour in both directions combined during the peak period. For the projected future peak-hour volume, the capacity of the two-lane undivided roadway would be exceeded and a design alternative improvement would be absolutely essential. The future operational benefits of converting to the 3T design alternative would be large but unmeasurable. The 5T design alternative would be expected to reduce delay by the estimated effectiveness of the 3T alternative plus an additional reduction because the provision of two through lanes in each direction of travel will increase speeds at the site. Since the peak-hour flow rate is below 1,000 vph, even for projected future conditions, the 4D alternative would result in more delay than either the 4U or 5T alternatives.

Ultimate Design Alternative

Given the eventual need for two through lanes in each direction of travel to provide traffic service at the site, the 3T alternative is not appropriate as the ultimate design alternative. The 4U alternative is not considered appropriate either because, without a raised median or a TWLTL, the existing safety problems at the site are likely to continue. Thus, either the 4D or the 5T alternative is appropriate as the ultimate design alternative for this site.

The 5T design alternative is preferable to the 4D alternative at this site for a number of reasons. The 5T alternative provides greater operational and safety benefits than the 4D alternative. The installation of a TWLTL is more in keeping with the existing strip commercial character of the site. Installation of a raised median would restrict access to the existing development and increase total delay without any offsetting benefit. A raised median would not be appropriate to implement established land use policies at this site, because strip commercial development has already occurred. Therefore, the 5T alternative is most appropriate as the ultimate design alternative for this site.

The safety benefits from provision of a full shoulder at this site are relatively small in comparison to the safety benefits of the design alternative improvement. However, because there are no right-of-way restrictions, the provision of a full shoulder is recommended. However, if funding limitations constrain the decision, the construction of the 5T design alternative should take priority over the provision of a full shoulder.

Staged Construction Options

A staged construction approach is particularly suitable for this site because of the uncertainty about future traffic volume growth. The 3T alternative would adequately serve the current traffic volume and the 5-year traffic volume projection. The 3T alternative would be less expensive to construct than the 5T alternative, and it would provide immediate relief for the safety and delay problems at the site. Furthermore, the 3T alternative could easily be upgraded to the 5T alternative if the traffic volume growth projected for the medium- to long-term future actually occurs. Therefore, the construction of the 3T design alternative is recommended for the immediate future, while improvement to the 5T design alternative would occur when traffic volumes warrant.

DESIGN EXAMPLE 2

This design example illustrates a two-lane undivided (2U) suburban arterial appropriate for immediate upgrading to the five-lane TWLTL (5T) design alternative.

Existing Conditions

The highway section for the second design example is 0.3 miles long and is located in a rapidly growing suburban community within a metropolitan region with over one million population. The highway section has moderately dense commercial development with some interspersed residential properties; there are 20 driveways, two unsignalized intersections, and no signalized intersections on the section. Table F-4 summarizes the existing geometrics, traffic control, and operational demand at the site.

The two-lane undivided highway at this site creates a bottleneck because four-lane arterial streets are located at both ends of the section. The existing through traffic volume at the site (950 vph in each direction) during the evening peak hour is close to the capacity of a two-lane roadway for uninterupted flow conditions and is growing rapidly. Table F-5 illustrates the ADT growth in the 3 recent years. The left-turn demands on the section are also substantial (190 left turns/hr/mile). The only reason that the left turns are not causing a complete breakdown in traffic flow is that through vehicles frequently use the paved shoulder to bypass left-turning vehicles. However, with any further growth of either through volumes or left-turn volumes, jammed conditions are expected during the evening peak hour.

Table F-4. Existing conditions—Design Example 2.

Section length:	0.3 miles
Existing design alternative:	Two-lane undivided (2U)
Average daily traffic volume:	21,000 vpd
Peak hour flow rate (one-way):	950 vph
Left-turn demand (one-way):	190 left turns/hr/mile
Percent trucks:	2%
Type of development:	Commercial
Driveway density:	67 driveways per mile
Unsignalized intersections:	7 intersections per mile
Signalized intersections:	None
Speed limit:	35 mph
Pedestrian activity:	Low
Lane width:	12 ft
Shoulders:	8 ft paved shoulders

Table F-5. Recent traffic volume growth—Design Example 2.

Year	ADT (vpd)	Peak Hour Flow Rate (vph) (one-way)	Truck Percentage
1980	18,500	850	1.0
1981	19,300	900	2.0
1982	21,000	950	2.0

The site currently has a moderate safety problem, but this problem appears to be growing rapidly, especially for nonintersection accidents. Table F-6 summarizes the accident experience at the site for three calendar years—1980, 1981, and 1982. The expected accident rate for this type of site, based on Tables 1 and 2 (in Chapter Two), is 2.78 accidents per MVM for nonintersection locations and 2.61 accidents per MVM for unsignalized intersections. The actual nonintersection accident rate was only slightly higher than the expected accident rate in 1978 but has been increasing rapidly since, and is now nearly twice the expected rate. The unsignalized intersection accident rates are also higher than expected.

The accidents at the site are predominantly multiple-vehicle head-on, rear-end, and angle accidents that are susceptible to correction through installation of a raised median or a TWLTL. Of the 51 accidents that occurred at the site over a 3-year period, 46 (or 90 percent) are of these potentially correctable types.

The severity of the accidents at this site is not particularly high. A total of 18 of the 51 accidents (35 percent) at the site involved a fatality or personal injury compared to 38 to 39 percent fatal and injury accidents for commercial 2U sites as a whole.

Projected Future Conditions

Traffic volumes at the site have been increasing rapidly and this growth is expected to continue. Much of the traffic volume growth has been generated by construction of a major regional shopping center about 3 miles from the site, and further development adjacent to this shopping center, as well as in the

Table F-6. Existing accident experience—Design Example 2.

	Accident Frequencies and Rates					
				(Accidents per million veh-miles)		
Year	Nonintersection Accidents	Unsignalized Intersection Accidents	Total Accidents	Nonintersection Accident Rate	Unsignalized Intersection Accident Rate	Total Accident Rate
1980	6	9	15	3.0	4.4	7.4
1981	8	8	16	3.8	3.8	7.6
1982	12	8	20	5.2	3.5	8.7
Total	26	25	51	4.0	3.9	7.9

	Accident Type and Severity Distribution						
	Accident Type					Accident Severity	
Year	Single Vehicle	Head-on	Rear-end	Angle	Other Multiple Vehicle	Fatal and Injury	Property-Damage-Only
1980	1	0	10	4	0	6	10
1981	1	1	8	5	1	5	12
1982	2	1	11	6	0	7	14
	4	2	29	15	1	18	36

area as a whole, is expected to increase traffic volumes. The average daily traffic is expected to increase to 35,000 vpd over the 20-year design period, while the peak-hour flow rate is expected to grow to 1,700 vph. An increase in left-turn demand to 300 left-turns per hour per mile is also expected.

Constraints

Existing off-street parking arrangements and building setbacks along the section limit increases in roadway width to a maximum of five lanes with paved shoulders. Thus, because of right-of-way restrictions six- and seven-lane design alternatives with full shoulders are considered to be infeasible at this site.

Priorities

The priorities for Design Example 2 are similar to those for Design Example 1. The site is classified as a major arterial, so that priority is assigned to the movement of through traffic. There is no particular priority for control of development. It has been recognized and accepted in community land use planning that the remaining residential properties along this section are likely to be converted to commercial uses.

Basic Number of Through Lanes

The current peak-hour traffic is not adequately served with one through lane in each direction of travel. Two through lanes will be required in each direction to serve the 20-year traffic forecast of 1,700 vph per direction.

Feasible Design Alternatives

There are three feasible design alternatives that provide two through-traffic lanes in each direction to serve the projected future traffic. These are: four-lane undivided (4U), four-lane divided (4D), and five-lane with TWLTL (5T). Design alternatives with less than two through lanes in each direction of travel (2U and 3T) are not adequate to serve the current or projected traffic volumes.

Geometric Variations

The existing section has full paved shoulders and there are no right-of-way restrictions that would limit the incorporation of paved shoulders in a four- or five-lane site. The incorporation of full shoulders in the selected design alternative is recommended since the rapid traffic volume growth at this site might eventually require conversion of shoulders to travel lanes to achieve a six- or seven-lane design alternative.

Benefits and Disbenefits

Table F-7 compares the accident rate data for the existing 2U design alternative to the expected accident rates for the 4U, 4D, and 5T design alternatives with full shoulders. It is apparent from the table that the 4U and 4D design alternatives are unlikely to result in a substantial reduction in accident rate. On the other hand, if the 5T design alternative reduces the accident rate from its current level of 8.70 accidents per MVM to the average rate for similar commercial 5T sections, 6.68 accidents per MVM, a 23 percent reduction in accident rate would result. This decrease in accident rate corresponds to the elimination of five accidents per year at current traffic volume levels and eight accidents per year at projected future traffic volume levels. The percentage of accidents involving a fatality or injury would also be expected to decrease by approximately 6 percent.

Table F-7. Estimated accident rates for design alternatives—Design Example 2.

	Accident Rate (accidents per million veh-miles)			
Design Alternatives	Nonintersection Accidents	Unsignalized Intersection Accidents	Total Accidents	Expected Accident Rate Reduction
2U - actual	5.22	3.48	8.70	-
4U - full shoulder	3.21	5.27	8.48	0.22
4D - full shoulder	3.27	5.21	8.48	0.22
5T - full shoulder	3.07	3.61	6.68	2.02

A direct operational comparison of the 2U and 5T design alternatives is not possible using the simulation results in Table 6 (Chapter Two) because the simulation results indicate jammed flow conditions (i.e., infinite delay) for both the existing and projected future traffic volumes on the 2U design alternative. These jammed conditions have not, in fact, occurred under the existing conditions because the shoulder is being used as a bypass lane. However, jammed conditions are probable under the projected traffic volumes even with shoulder use. Thus, the operational benefits of improving the 2U design alternative to any of the multilane design alternatives are very large but not quantifiable.

The 5T design alternative has a clear operational advantage over the 4U design alternative. The estimated delay reduction for the 5T design alternative under current traffic conditions is 210.6 sec per left-turn vehicle or 24,000 veh-sec during the peak period. This delay reduction corresponds to an average time savings of 13 sec for each vehicle passing through the section and a delay reduction of at least 673.5 sec per left-turn vehicle, corresponding to 121,230 veh-sec per hour in the peak hour or 33 sec per through vehicle.

At the current peak-hour flow rate of 950 vph, the total delay experienced by motorists is nearly the same for the 4U and 4D design alternatives. However, as the peak-hour volume grows to 1,700 vph, the 4D design alternative will become operationally preferable to the 4U design alternative. However, even at this high flow rate, the 5T design alternative would provide less delay than the 4D design alternative.

Ultimate Design Alternative

The 5T design alternative is recommended as the ultimate design alternative for this highway section. Only alternatives with two through traffic lanes in each direction of travel can adequately serve the through traffic demand. Of the possible four- and five-lane alternatives, the 5T alternative has the greatest delay reduction and is also the only design alternative that can be expected to substantially reduce accident rates. The commercial development at the site is appropriate for a TWLTL and is consistent with current land use planning. There is a possible need for a six- or seven-lane design alternative at this site in the long-term future, although construction of this alternative would require narrow lanes and elimination of the full shoulder.

Staged Construction Options

There are no staged construction options that are appropriate for this site. The need for the 5T design alternative is immediate and neither the 3T or 4U design alternatives could adequately serve the through traffic and left-turn demands.

DESIGN EXAMPLE 3

This design example illustrates a four-lane undivided (4U) suburban arterial appropriate for conversion to a four-lane divided (4D) design.

Table F-8. Existing conditions—Design Example 3.

Section length:	0.8 miles
Existing design alternative:	Four-lane undivided (4U)
Average daily traffic volume:	15,000 vpd
Peak hour flow rate (one-way):	600 vph
Left-turn demand (one-way):	80 left turns/hr/mile
Percent trucks:	4%
Type of development:	Light commercial
Driveway density:	25 driveways per mile
Unsignalized intersections:	4 intersections per mile
Signalized intersections:	None
Speed limit:	40 mph
Lane width:	12 ft
Shoulders:	None
Pedestrian activity:	Low

Existing Conditions

The highway section for the final design example is 0.8 miles long and is located in a surburban community with a metropolitan area of several million population. The highway section serves a sparsely developed area with light commercial development interspersed with undeveloped areas. There are 20 driveways, three unsignalized intersections, and no signalized intersections on the section. The highway section is located on a major arterial street that serves a rapidly developing suburban area and connects that area with a nearby radial freeway. Table F-8 summarizes the existing geometrics, traffic control, and operational demand at the site.

The present 4U design alternative provides adequate traffic service to the existing traffic because the peak-hour through volume (600 vph) is moderate and the left-turn volume (64 left turns per hour in the 0.8-mile section) is low. However, substantial increases in traffic volume are expected from planned development, as explained below in the discussion of projected future conditions.

The safety conditions on the existing facility are illustrated for two recent years (1981 and 1982) in Table F-9. The accident rates at the site, 2.4 accidents per MVM for nonintersection locations and 3.5 accidents per MVM for unsignalized intersections, are slightly lower than the expected accident rates from Tables 1 and 2 (see Chapter Two) for such locations, 2.8 and 4.0 accidents per MVM, respectively.

Projected Future Conditions

An immediate increase in traffic volumes at the site is expected because of the opening, within the next year, of a major regional shopping center being constructed on undeveloped land at the eastern end of the section. The average daily traffic volume is expected to increase to 20,000 vpd and the peak-hour flow rate to 1,100 vph within the first year that the shopping center is open. Two signalized driveways with separate left-turn lanes are planned on the section to provide access to the shopping center.

The opening of the shopping center will not have a direct impact on left-turn volumes elsewhere on the section. However, the shopping center is expected to encourage new development along the entire section.

Table F-9. Existing accident experience—Design Example 3.

Year	Nonintersection Accidents	Unsignalized Intersection Accidents	Total Accidents	Nonintersection Accident Rate	Unsignalized Intersection Accident Rate	Total Accident Rate
	Accident Frequencies and Rates			(Accidents per million veh-miles)		
1981	10	14	24	2.3	3.2	5.0
1982	11	16	27	2.5	3.7	5.7
Total	21	30	51	2.4	3.5	5.4

Accident Type and Severity Distribution

Year	Single Vehicle	Head-on	Rear-end	Angle	Other Multiple Vehicle	Fatal and Injury	Property-Damage-Only
	Accident Type					Accident Severity	
1981	6	1	7	5	5	8	16
1982	8	2	5	6	6	9	18
	14	3	12	11	11	17	34

In the long term, the traffic volume on the section is expected to grow over the 20-year design period to 30,000 vpd with a peak-hour flow rate of 1,600 vph. The future turning volumes at the site are quite uncertain because they will depend on the type of development that occurs.

Constraints

There are no right-of-way restrictions at the end of the site adjacent to the new shopping center. At the western end of the site, existing development limits the maximum roadway width to four or five lanes with shoulders.

Priorities

The highway section is a major arterial and the preservation of its ability to move through traffic has been assigned a high priority. The community plans to limit strip commercial development of the section and encourage low-density office development with fewer access points.

Basic Number of Through Lanes

The projected peak-hour flow rate of 1,600 vph can be served adequately with two through lanes in each direction of travel under uninterrupted flow conditions. However, three through lanes will eventually be required on the approach to the signals at the shopping center driveways.

Feasible Design Alternatives

There are two feasible design alternatives that provide two through traffic lanes in addition to the existing four-lane undivided (4U) design alternative. These are four-lane divided (4D) and five-lane TWLTL (5T) alternatives. The six-lane divided (6D) and seven-lane TWLTL (7T) alternatives should also be considered because of the need for three through lanes in each direction on the shopping center signal approaches.

Geometric Variations

The existing site has a curb-and-gutter section with no shoulders. However, there are no right-of-way restrictions that would limit the construction of full shoulders for any alternative at the eastern end of the site or for any four- or five-lane alternative at the western end of the site.

Benefits and Disbenefits

The current accident rate at the site is below average, so the purpose of the improvement is to serve the traffic to be generated by the new development and to prevent the development of safety problems that might result from increased turning volumes.

The 4D design alternative would be operationally preferable to the existing 4U design because the expected peak-hour flow rate exceeds 1,000 vph both for the period immediately following the opening of the shopping center (1,100 vph) and for the long-term future (1,600 vph). However, the 5T design alternative would involve less delay than the 4D design alternative at all of the traffic volume levels under consideration.

The current accident rate for the existing 4U design is below average, but this rate would be expected to increase as development proceeds. Both the 4D and 5T design alternatives would be expected to alleviate any safety problems that result from increased development. Table F-10 compares the current accident rate at the site to the average rate for the current 4U design

Table F-10. Estimated accident rates for design alternatives—Design Example 3.

Design Alternatives	Nonintersection Accidents	Unsignalized Intersection Accidents	Total Accidents	Expected Accident Rate Reduction
	Accident Rate (accidents per million veh-miles)			
4U - actual	2.4	3.5	5.9	-
4U - no shoulder	2.8	4.0	6.8	-
4D - full shoulder	2.5	3.9	6.4	0.4
5T - full shoulder	2.3	2.3	4.6	2.2

and the expected rates for the 4D and 5T design alternatives. The table shows that if the existing accident rate were to increase even to the average for commercial 4U sections, both the 4D and 5T design alternatives with full shoulders would be likely to reduce that rate.

The 4D design alternative has an important benefit not provided by the 5T design alternative because of the priority placed by the community on controlling land use and development at the site. The 4D design alternative incorporating a raised median is most compatible with the type of office development desired by the community; this type of development has relatively few driveways at which median openings can be provided. The 5T design alternative would encourage strip commercial development and generate turning volumes that could degrade the through traffic movement function of this major arterial. It is possible that the apparent operational and safety benefits of the 5T alternative over the 4D alternative would be lost through increased development.

Ultimate Design Alternative

The ultimate design alternative that is most appropriate for this site is four-lane divided (4D) at the western end of the site and six-lane divided (6D) at the eastern end of the site (near the shopping center). The primary reason for choosing a raised median at this site is to influence future development and, thus, protect through-traffic capacity and safety of the site. The installation of full shoulders throughout the length of the project is recommended.

Staged Construction Options

The third through lane in each direction at the eastern end of the site is not needed immediately, so it is recommended that the entire site be reconstructed initially as four-lane divided until the need for the added lanes near the shopping center can be demonstrated.