NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP Report 395

Capacity and Operational Effects of Midblock Left-Turn Lanes

Transportation Research Board
National Research Council

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Capacity and Operational Effects of Midblock Left-Turn Lanes

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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FOREWORD

By Staff
Transportation Research Board

This report presents procedures for estimating the operational, safety, and access impacts of different midblock left-turn treatments and includes guidelines for selecting among raised-curb medians, two-way left-turn lanes, and undivided cross sections. The report will be useful to designers of multilane roads with unrestricted access.

Midblock left-turn lane treatments, which allow access to adjacent businesses and other properties, directly affect accident rates and roadway capacities. Although there is a general understanding of the impact of these treatments on safety, few, if any, studies provided explicit information on the capacity and operational effects of these facilities. Research was needed to provide guidelines and criteria for selecting appropriate midblock left-turn lane treatments.

Under NCHRP Project 3-49, the University of Nebraska-Lincoln evaluated current practice, collected data on the different types of left-turn treatments through field observations and simulations, and prepared this guide for practitioners to use in analyzing different treatments. The project included raised-curb median, two-way left-turn lane, and undivided cross sections.

The report presents detailed methods for determining the effects of different treatments on operations (including effects on speed and delay and the probability of queue spillback), safety, and adjacent businesses. These detailed models were used with typical construction costs to generate tables that can be used to quickly assess the cost-effectiveness of a left-turn treatment. The results are applicable to four-lane to seven-lane cross sections.

CONTENTS

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CAPACITY AND OPERATIONAL EFFECTS OF MIDBLOCK LEFT-TURN LANES

SUMMARY

The objective of this research project was to develop a methodology for evaluating alternative midblock left-turn treatments on urban and suburban arterials. The key requirements for the evaluation methodology were that it be quantitative and sensitive to the operational effects, safety effects, and access impacts relating to a midblock left-turn treatment. The methodology had to be applicable to three common midblock left-turn treatments: the raised-curb median, the flush median with two-way left-turn lane (TWLTL) delineation, and the undivided cross section. The methodology developed for this research focuses on the evaluation of midblock street segments on urban and suburban arterials. In this context, a midblock segment refers to the street segment between, but exclusive of, the bounding signalized intersections.

The approach taken to conduct this research was to develop a comprehensive methodology for evaluating midblock left-turn treatments, collect field data to calibrate the methodology, and use the calibrated methodology to develop treatment selection guidelines. The approach was applied to the parallel development of three models that comprise the evaluation methodology: the operations model, safety model, and access impact model. These models can be used to evaluate the operational effects, safety effects, and access impacts of a specific midblock left-turn treatment. The operations and safety models were used to develop midblock left-turn treatment selection guidelines based on a benefit-cost analysis approach.

Full-scale field studies were conducted to obtain the data necessary to refine and calibrate the operations, safety, and access impact models. Traffic flow data were collected during 32 field studies in eight cities and four states. Three-year accident histories for 189 street segments were obtained from cities in two states. Finally, 165 owners and managers of businesses located along four arterials in four cities and three states were surveyed to determine the effect on property access and business activity of a recent change in midblock left-turn treatment. In the case of the traffic data, 117 additional simulation runs were made to expand the range of field data.

The operations and safety models were used to develop guidelines for selecting a midblock left-turn treatment. The performance measures predicted by these models were used to compute the road user benefits associated with a change in left-turn treatment (e.g., from an undivided cross section to a TWLTL). This benefit was then compared with the con-

struction cost associated with the treatment conversion. Arterial conditions that were found to be cost-effective were identified in the selection guidelines. The guidelines are sensitive to the following conditions: traffic demand, access point density, number of traffic lanes, and land use.

The following conclusions were reached as a result of this research:

1. The performance of an unsignalized access point often is degraded by the close proximity of another intersection.
2. Traffic platoons created by upstream signalized intersections can affect the operation of an access point by increasing the capacity of its traffic movements.
3. The application of the operations model to a wide range of traffic demand and geometric conditions indicated that the raised-curb median and the TWLTL yield similar delays to arterial drivers. The undivided cross section yields significantly higher delays than the raised-curb median and TWLTL.
4. The operations model analysis indicates that any of the left-turn treatment types can function without causing congestion in arterial traffic movements at average daily traffic demands of 40,000 vehicles per day (vpd) or less.
5. Accidents are more frequent on street segments with higher traffic demands, driveway densities, and public street densities. Accidents also are more frequent when the land use is business or office as opposed to residential or industrial.
6. The undivided cross section has a significantly higher accident frequency than the TWLTL and raised-curb median treatments when parallel parking is allowed on the undivided street. When there is no parking allowed on either street, the difference between the undivided and TWLTL treatments generally is small and negligible for average daily traffic demands of less than 25,000 vpd. In general, the raised-curb median treatment appears to be associated with fewer accidents than the undivided cross section and TWLTL, especially for average daily traffic demands that exceed 20,000 vpd.
7. The majority of street reconstruction projects considered in this research that involve a left-turn treatment conversion resulted in no change in the level of access provided to adjacent properties. Two-thirds of these "no-change" projects involved a conversion from an undivided cross section to a TWLTL. Very few projects resulted in more property access, such as the conversion from a raised-curb median to a TWLTL.
8. Business owners believe that the conversion from an undivided cross section to either a raised-curb median (with openings every 330 ft) or a TWLTL *will* improve arterial traffic conditions and business conditions (i.e., property values, access, and sales). In contrast, business owners believe that the conversion from either a raised-curb median (with openings every 330 ft) or a TWLTL to a raised-curb median with openings every 660 ft *will not* improve business opportunities.
9. Business owners believe that customers rank property access much lower in importance than either service or quality. This finding indicates that the typical business may be able to overcome some reduction of access if it offers good, reliable service.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

RESEARCH PROBLEM STATEMENT

The continuing growth and development of our nation's cities has placed increasing pressure on the urban transportation system. This pressure is evidenced by increased traffic demand, longer periods of congestion, and reduced safety on most city streets. There are two primary sources of these operational and safety problems. First, there is an inadequate number of traffic lanes on arterials relative to the growing demand. Second, there is conflict between arterial through and turning vehicles. Turning vehicles at signalized intersections consume the capacity that otherwise would be available to through traffic movements. Turning vehicles at driveways also can be a major cause of turbulence in the arterial traffic stream.

Transportation agencies usually address operational and safety problems on existing arterials by using a combination of geometric and traffic control improvements. Geometric improvements involve the addition of arterial through lanes or midblock left- or right-turn treatments. Midblock left-turn treatments can be categorized as either direct or indirect, depending on the manner in which the arterial left-turn maneuver is completed. Direct left-turn treatments, typically found within the arterial cross section, include the raised-curb median with left-turn bays, flush median with two-way left-turn lane (TWLTL) delineation, flush median with delineated left-turn bays, and undivided cross section (i.e., no median). Indirect left-turn treatments, typically adjacent to the arterial, include a jug handle or an at-grade cloverleaf. Right-turn treatments include the provision of a right-turn bay or a larger turn radius. Traffic control improvements include signalization and turn prohibition.

Of these improvements, midblock left-turn treatments are believed by many practitioners to offer the most promise in terms of improved operations and safety. However, the extent of the improvement is difficult to assess with existing operational and safety evaluation procedures. This difficulty stems from the complexity of traffic operations on the arterial. In fact, this difficulty is attested to by the preponderance of publications discussing the operational effectiveness of one or more left-turn treatments, but offering little in the way of a comprehensive, quantitative assessment. Some of these publications offer broad-based, subjective insights into the relative merits of many midblock left-turn treatments, whereas others present quantitative findings from a limited study of only one or two treatments. Although the conclusions from these studies are generally consistent (i.e., "treatment A is generally better than treatment B under the conditions studied"), a rational method for evaluating alternative treatments under a wide range of conditions has yet to be developed using a large national database. Moreover, guidelines identifying conditions in which a specific midblock left-turn treatment offers the safest and most efficient operation do not exist.

In summary, a methodology is needed for quantitatively evaluating alternative midblock left-turn treatments on urban and suburban arterials. This methodology would provide the procedures and analytic tools needed to efficiently conduct the evaluation. This methodology would be comprehensive in its scope so that it includes the operational effects, safety effects, and access impacts of each left-turn treatment. The performance measures predicted by the methodology would be suitable for a cost-benefit analysis and thereby would facilitate the development of guidelines that identify the most cost-effective left-turn treatment for a given arterial.

RESEARCH OBJECTIVE AND SCOPE

The objective of this research project is to develop a quantitative methodology for evaluating alternative midblock left-turn treatments on urban and suburban arterials. The project results will be applicable to a full range of arterial cross sections, including raised-curb medians and cross sections with up to seven lanes. The product of this study will be in the form of a guide that allows the transportation practitioner to make decisions regarding the most appropriate midblock left-turn treatment based on available data. The research also will produce a better understanding of the relationship between the type of midblock left-turn treatment and adjacent traffic generators.

The key requirements of the evaluation methodology are that it be quantitative and that it be sensitive to the operational effects, safety effects, and access impacts relating to the midblock left-turn treatment. The operational effects relate primarily to the delays incurred by the arterial and access point traffic movements. The safety effects relate to the frequency and severity of vehicular accidents, although

unsafe and erratic maneuvers also are important safety clues. The access impacts relate to the number and function of access points provided to properties adjacent to the arterial.

The more common midblock left-turn treatments used on urban and suburban arterials are as follows:

- Flush median with TWLTL delineation;
- Raised-curb median with alternating left-turn bays (non-traversable median area);
- Flush median with alternating left-turn bays (traversable median area); and
- Undivided cross section (i.e., no median).

Other treatment types include the flush median with continuous parallel left-turn lanes, raised-curb median with acceleration lanes, mountable median with TWLTL delineation, and median channelization for U-turn or right-turn access only.

The methodology developed for this research focuses on the evaluation of midblock street segments on urban or suburban arterials. In this context, a midblock segment refers to the section of street between, but exclusive of, the bounding signalized intersections. Access everywhere along the midblock segment is by way of unsignalized access points. This segment has a constant cross section and one type of midblock left-turn treatment.

The following criteria were used to define an urban or suburban arterial segment, as related to the study objectives:

- Traffic volume exceeding 7,000 vehicles per day (vpd)
- Speed limit between 30 and 50 mph
- Spacing of at least 350 ft between signalized intersections
- Direct access from abutting properties
- No angle curb parking (parallel parking is acceptable)
- Located in or near a populated area (e.g., population of 20,000 or more)
- No more than six through traffic lanes (three each direction)
- Arterial length of at least 0.75 mi.

These criteria eliminated from consideration low-volume two-lane roadways, rural highways, expressways, roads through small towns, and low-speed collector streets.

Several terms are traditionally used to describe a location of unsignalized access to the arterial, including driveway, access point, unsignalized intersection approach, and public street approach. To eliminate confusion, the following terms are defined for this report:

- Access points—All unsignalized access locations. An access point can be either a driveway or a public street approach.
- Driveway—Any location on the arterial where the curb along the outside lane is removed or dropped for 10 or more ft to facilitate vehicular access to the adjacent property.
- Access point density—Total number of access points on both sides of the major-street segment (i.e., a two-way total) divided by the length of the segment (in mi). (Driveway density and public street approach density are defined in a similar manner.)

At this point, it is useful to clarify the meaning of other terms found in the literature, because they are used in this report. It is assumed that a major street is classified as an arterial and a minor street is classified as a collector, local street, or driveway. The through traffic movements on the arterial are referred to as priority movements; all other driveway-related movements are nonpriority movements.

The aforementioned definitions are used throughout the literature dealing with the safety and operational effects of midblock left-turn treatments. These definitions, however, are less rigidly applied in the literature on operational effects. Specifically, a distinction between access points and driveways is not made in this literature. Moreover, all driveways are generally assumed to have some minimum traffic volume (i.e., they are assumed to be "active"). In this latter sense, it is assumed that a driveway with negligible volume has negligible effect on traffic operations. Throughout this report, the term "access point" is used instead of driveway or public street approach; however, any such reference in an operational context infers that the access point is active. In this report, an active access point is defined as a driveway or street with an entering volume of 10 vehicles per hour (vph) or more.

RESEARCH APPROACH

The approach taken in conducting this research was to develop a comprehensive midblock left-turn treatment evaluation methodology, collect field data to calibrate this methodology, and use the calibrated methodology to develop treatment selection guidelines. This approach was applied to the parallel development of three models that comprise the evaluation methodology. These models, the operations model, safety model, and access impact model, can be used collectively or individually to evaluate the operational effects, safety effects, and access impacts associated with a specific midblock left-turn treatment. The operations and safety models were used to develop midblock left-turn treatment selection guidelines based on a cost-benefit analysis.

A survey of practitioners and a review of the literature were conducted to determine the current procedures, experiences, and needs with respect to evaluating alternative left-turn treatments. The survey addressed a variety of issues including the availability of existing treatment selection guidelines, volume or access point density thresholds that bound the effective range for a given treatment type, and the types of left-turn treatments being used currently. The insights obtained from the survey, literature review, and two

pilot field studies were used to identify traffic flow, safety, and access problems associated with common midblock left-turn treatments. Theoretic model forms were then developed to quantify the nature of the problem and its effect on relevant performance measures (e.g., delay, accident frequency, and property values).

Full-scale field studies were conducted to obtain the data necessary to refine and calibrate the theoretical model forms. Traffic flow data were collected during 32 field studies in eight cities and four states. Three-year accident histories for 189 street segments were obtained from cities in two states. Finally, 165 business owners or managers located along four arterials in four cities and three states were surveyed to determine the effect of a recent change in midblock left-turn treatment on property access and business activity. In the case of traffic data, 117 additional simulation runs were made to expand the range of field data.

The three models developed for this project predict quantitative performance measures that define the operational effect, safety effect, and access impact of a midblock left-turn treatment. The operations model predicts the delay to arterial left-turn and through movements. The safety model predicts the annual frequency of accidents along the midblock street segment. The access impact model predicts an index value that represents the proportion of business owners who would perceive a given left-turn treatment as having a favorable effect on business.

The operations and safety models were used to develop guidelines for selecting a midblock left-turn treatment. The performance measures predicted by these models were used to compute the road user benefit associated with a change in left-turn treatment (e.g., from an undivided cross section to a TWLTL). This benefit was then compared with the construction cost associated with the treatment conversion. Arterial conditions that were found to be cost-effective were identified in the selection guidelines. The guidelines are sensitive to the following conditions: traffic demands, access point density, number of traffic lanes, and land use.

CHAPTER 2

SELECTION GUIDELINES FOR MIDBLOCK LEFT-TURN TREATMENTS

This chapter describes the development of guidelines for selecting a midblock left-turn treatment. These guidelines were developed using the operations and safety models to compute the operational and safety effects of three midblock left-turn treatments: the raised-curb median, the two-way left-turn lane (TWLTL), and the undivided cross section. The operational effect of each treatment was computed as the annual delay incurred by arterial through and left-turn movements. The safety effect of each treatment was computed as the annual number of arterial accidents. The costs associated with these delays and accidents were used to compute the road user benefits associated with the conversion from one treatment to another. These benefits were then compared with the annualized cost of the conversion to determine traffic conditions that warranted a conversion on a cost-effective basis. These conditions formed the basis for the treatment selection guidelines provided at the end of this chapter.

The following two sections describe the procedures used to compute the operational and safety effects for a series of typical arterial traffic and geometric conditions. The last section describes the guidelines formed from the findings of the operational and safety analyses.

OPERATIONAL EFFECTS OF MIDBLOCK LEFT-TURN TREATMENTS

Approach

This section describes the effect of three alternative midblock left-turn treatments on the operational performance of an urban arterial. The operational performance of each treatment was evaluated using the operations model developed for this research project. This model embodies the operational analysis techniques described in Chapters 9 and 10 of the 1994 *Highway Capacity Manual* (HCM) (*1*). To facilitate this analysis, the operations model was reproduced in software form. A more detailed description of the operations model is provided in Chapter 3.

The performance assessment of each alternative midblock left-turn treatment was based on an examination of the delays incurred by the various traffic movements at each unsignalized intersection formed by the major street and a driveway or unsignalized public street. Delay was selected as the most appropriate measure of performance because it is recognized by the HCM (*1*) as the measure of effectiveness defining level of service. Moreover, delay is a desirable measure of performance because it can be incorporated into an economic assessment of performance that considers a facility's life-cycle costs.

The focus of a performance evaluation is the delays to two traffic movements at each unsignalized intersection. Of the many possible traffic movements associated with each intersection, the two movements most directly affected by alternative midblock left-turn treatments are those on the major-street approach to the intersection. The movement most directly affected is the left-turn from the major street. This movement's performance is affected by the presence of a median storage area.

The storage area provided by the raised-curb median and TWLTL treatments removes the left-turns from the through traffic stream, keeping the through traffic lanes open to through traffic only. This separation of flows increases left-turn capacity and reduces left-turn delay by increasing the frequency and size of available gaps in the opposing traffic stream, relative to the undivided cross section. Because the raised-curb median treatment always has less storage space than the TWLTL, differences between these two treatments also can emerge when left-turn demands are high enough to precipitate the overflow of the raised-curb median's bay storage area.

The other movement directly affected by midblock left-turn treatment is the major-street through movement. This movement is affected when one or more left-turning vehicles are queued in the inside through lane. Left-turn vehicles queue in the inside lane for the raised-curb median and TWLTL treatments when the left-turn queue exceeds the available median storage area. Similarly, left-turn vehicles frequently queue in the inside lane for the undivided treatment because of the lane's lack of a median storage area. Through vehicles in the inside lane approaching a left-turn queue will merge (i.e., change lanes while maintaining speed) into the adjacent through lane, if possible. However, as volume levels increase, some through vehicles will not be able to merge and will have to stop at the back of the left-turn queue. At this point, these through vehicles are delayed until they are able to change lanes or until the left-turn queue ahead of them dissipates.

Other traffic movements at the unsignalized arterial intersection are affected by midblock left-turn treatments; however, these effects are indirect. For example, traffic queues extending back from a downstream intersection (e.g., resulting from bay overflow) could impede the entry of these movements. Also, turbulence in the inside through lane resulting from major-street left-turn activity could degrade the structure of traffic platoons from the upstream signal and thereby reduce the capacity available to these movements.

Procedure

The effect of each midblock left-turn treatment on the operational performance of a typical urban arterial street segment was determined using the operations model. This model was used to evaluate treatment effects over a wide range of traffic demands and geometric configurations. Traffic demands included the major-street flow rate and left-turn percentage. Geometric configurations included the number of through traffic lanes and the density of access points along the major street. It should be noted that no distinction is made in this analysis between a driveway and an unsignalized public street approach; the term "access point" is used hereafter to denote either type of access.

The performance relating to each left-turn treatment was determined from a combination of major-street left-turn and through delays. Specifically, the delays to these two movements were computed for individual hourly flows representing those typically found during the course of a year for each average daily traffic and geometry case. The annual vehicle-hours of delay to the two affected movements were determined by aggregating their individual hourly delays. Guidance on the operational performance of any treatment and the relative performance difference between any two treatments can be obtained by comparing the annual delays computed in this manner.

Analysis Scenarios

The combinations of traffic demand and geometry considered for this performance evaluation were selected to be representative of typical urban arterials. To obtain this representation, an idealized street segment with all the attributes of a typical urban arterial was configured. To facilitate discussion of specific traffic movements, this street is defined as oriented in an east-west direction. The specific attributes of the study segment are as follows:

- The segment is ¼ mi in length.
- The cross section of the major street contains four or six through traffic lanes and is constant throughout the segment and symmetric about the centerline.
- The effect of nonideal conditions (e.g., trucks, lane width, area type, and buses) is incorporated in the saturation flow rate at the signalized and unsignalized (i.e., access point) intersections.
- Both ends of the segment have a signalized intersection.
- The phase sequence, phase duration, and traffic demands are equivalent at both signalized intersections.
- A minimum-delay signal offset is maintained between the two signalized intersections.
- Traffic demands in each direction along the major street are equivalent (including turn percentages).
- Turn percentages at each unsignalized intersection are balanced so that there is no net loss or gain in traffic demand along the major street.
- Left-turn and through (i.e., cross) movement volumes from the access points are negligible.
- Only "active" access points (i.e., access points with an entering volume of 10 vph or more) are considered on the street segment.
- Access points on opposite sides of the street are located so that they are directly opposite one another, thereby making a four-leg intersection with the major street.
- Access points are evenly spaced along the segment.
- Access points are not permitted within 300 ft of a signalized intersection approach.
- Median openings are provided at every access point; thus, U-turn maneuvers are negligible.

Segment Length

The ¼-mi segment length was selected because it represents a typical signal spacing for urban arterials. Experience with the operations model indicates that longer segment lengths have a negligible effect on the operational performance of the arterial (as it relates to the midblock left-turn treatment); hence, the findings reported in the next section can be extrapolated to longer segment lengths. The effect of shorter lengths can be significant depending on the type of signalization, density of the through movement platoon, ability to obtain efficient two-way progression, and length of the queues extending back from the signalized intersections into the study segment. The reported findings, therefore, should be used with caution for segments that are less than 1,000 ft in length.

Cross Section

The cross section of the study segment contains either four or six through traffic lanes. This range in lanes was determined to be typical of the sites found during the field studies to have operational problems and sufficient traffic demand to make these problems significant. Segments with only two through lanes were not included in the evaluation because arterials of this type with significant operational problems are believed to occur with much lower frequency than those with four or six lanes.

The cross section of the major street is constant throughout the length of the study segment and is symmetric about the centerline. In other words, one midblock left-turn treatment is applied throughout the length of the study segment. The symmetric attribute indicates that both travel directions will have either two lanes or three lanes; no unbalanced lane combinations were used. In addition, the study segment does not have bays for right-turn movements exiting the major street.

Saturation Flow Rate

A saturation flow rate of 1,700 vphgpl was used at the signalized and unsignalized intersections to account for nonideal traffic and geometric conditions. This variable, which is a well-recognized input into the operational evaluation of signalized intersections, also was incorporated into several of the component models used to enhance the unsignalized intersection analysis procedure. The saturation flow rate used for this evaluation represents a reduction from the ideal rate of 1,900 vphgpl to account for the effect of trucks, buses, narrow lane widths, area type, and so on in combinations typically found at real-world urban arterials.

Signalized Intersections

Signalized intersections have a very significant effect on the flow of traffic along the major street. Specifically, intersection signal timing promotes the cyclic flow of dense platoons of traffic, followed by periods of significantly lower flow. These alternating high and low flow rates tend to increase the capacity of the nonpriority traffic movements at the unsignalized access points beyond that found when arrival headways are more randomly distributed. Because signalized intersections are inherent to urban arterials, it is essential that they be included at the boundaries of the study segment so that the effect of traffic platooning is incorporated in the findings from the performance evaluation.

The signal timing characteristics for the signalized intersections are listed in Table 2-1. The phase durations reflect an equitable split of the available cycle time relative to the corresponding traffic demands and available lanes. Traffic movements entered the study segment as left-turn, through, or right-turn vehicles at the upstream signalized intersection. This approach was taken to replicate the effect of secondary platoons formed by the intersection turn movements (relative to the main platoon formed by the through movement). The minimum delay offset between the through signal phases at the two bounding intersections was determined using a time-space diagram.

Traffic Volumes

The traffic volume and turn percentages in each travel direction along the study segment were set at the same values. This approach resulted in a 50-50 directional split in arterial traffic flow. This split was found to be conservative in that other values would yield improved operational performance. Specifically, all other splits would increase volume in one direction and reduce it in the other direction. This imbalance matched an increased left-turn volume with an increased left-turn capacity in one direction and decreased left-turn volume and capacity in the other direction. The capacity increase generally exceeded the volume increase for the one direction, resulting in less delay and queueing for that direction. For the other direction, the volume reduction tended to mitigate the increase in total delay. The net result was that an unequal directional split would yield lower delays than would an equal split. Therefore, it was reasoned that a delay-based evaluation of the relative merits of alternative midblock treatments based on a 50-50 directional split would be conservative in the context of it being a worst-case assessment.

Turn Movements

The turn percentages were the same at each access point and balanced so that there was no net loss or gain in traffic

TABLE 2-1 Characteristics of the signalized intersections bounding the study segment

Characteristic	Entry Movements[1]			Exit Movements[1]	Total
Movement - East Intersection	NB Left	SB Right	WB Thru	EB Thru	
Movement - West Intersection	SB Left	NB Right	EB Thru	WB Thru	
Phase Sequence Number	1	2	3		
Phase Duration (G + Y), sec	17	19	54		90
Distribution of entry flow rate for the "Four Through Lanes" variation, %	20	10	70		100
Distribution of entry flow rate for the "Six Through Lanes" variation, %	10	10	80		100

Note:
1 - The study segment was oriented in an east-west direction.

demand along the street segment. This balance was achieved by specifying the major-street left- and right-turn movement percentages and then adjusting the access point right-turn movement percentage to achieve a balanced flow condition. The left-turn and through (cross) movements from the access points were assumed to be negligible based on observations made during the field studies. It is not expected that these latter two movements would have a direct impact on the magnitude of the major-street left-turn and through movement delays; hence, the findings reported in the next section also should be applicable to street segments having a small amount of left-turn or through movement volume exiting the access point.

The turn percentage for major-street left- and right-turn movements at each unsignalized intersection was determined by the number of access points in each direction of travel and the overall turn percentage assigned to the study segment. The following terms are defined for the eastbound direction; the same definitions apply to the westbound direction.

The left-turn percentage for the eastbound direction of the study segment is defined as follows:

$$P_L = \frac{V_L}{V} \tag{1}$$

where:

P_L = left-turn percentage for the eastbound segment
V_L = total number of vehicles turning left from the eastbound major street into an access point
V = total number of eastbound vehicles entering the segment.

A similar definition is applied to the right-turn percentage for the eastbound segment P_R.

The left-turn movement percentage at any one access point is defined as follows:

$$p_L = \frac{P_L}{n} \tag{2}$$

where:

p_L = average left-turn percentage at any one access point for the eastbound segment
n = number of access points located on the south side of the major street.

As before, a similar definition is applied to the right-turn percentage at any one access point for the eastbound segment p_R.

A review of turn percentages at several field study sites indicated that both P_L and P_R varied between 5 and 13 percent. Based on these data, it was determined that the left-turn percentage for the segment P_L would be varied between 0 and 30 percent and that the right-turn percentage P_R would be fixed at 10 percent. The left-turn percentage was varied because of its direct impact on the major-street left-turn delay and, hence, the major-street through movement delay. The right-turn percentage was fixed at a representative value to provide a realistic balance between resource requirements and the level of effort needed for the analysis of other variables.

Active Access Points

The access points included in the study segment represent access points with sufficient volume to have some effect on traffic operations. In this context, the access point density reported for the study segment represents the density of active access points (i.e., those with entering volumes of 10 vph or more). Inactive access points were excluded to better represent the traffic-related effect of access point density on arterial operations.

Access Point Locations

The access points on the study segment are evenly spaced and located opposite one another where they intersect the major street. This approach was taken to facilitate a direct comparison of the midblock left-turn treatments. Specifically, the raised-curb median treatment is most commonly associated with the aforementioned access point location scheme. Although this is not always true for the TWLTL or undivided cross section, a comparison of relative differences among all three treatments requires a common access point location scheme to preclude the effects of uneven spacing or staggered positioning from confounding the interpretation of the findings. Moreover, the scheme used in this evaluation is consistent with preferred engineering practice in the context of access point spacing and alignment (i.e., uneven spacing and staggered positioning can degrade traffic operations and typically result from a lack of effective access management). Hence, the findings from this evaluation will form an appropriate basis for guidelines for the selection of midblock treatments wherein accepted access management techniques will be applied.

Access points were not included in the vicinity of signalized intersection approaches (i.e., within 300 ft of the stop line). This approach was taken to be consistent with effective access management principles (in the context of the development of guidelines from the evaluation findings). Access points in the vicinity of signalized intersection approaches tend to degrade intersection capacity and increase the frequency of rear-end accidents. In fact, access points with volumes high enough to be called active were typically not found in the vicinity of the signalized intersection approaches at the sites visited during the field studies. Locations with access points near the intersection approach typically exhibit natural egress restrictions as a result of traffic queues extending back past and blocking the access point.

Median Openings

For this study, it was assumed that median openings exist at all active access points. This assumption is fairly consistent with the use of median openings at the field study sites with raised-curb median treatments, particularly for those with densities of less than 60 active access points per mile. Moreover, in addition to directly answering the question of differences in operational effects of left-turn treatments, this approach eliminated the criticism of associating an overly conservative estimate of delay with the raised-curb median treatment as a result of an assumed U-turn scenario. On the other hand, it was believed that it is impossible to accurately account for the effects of median closure on driver route choice without considering the surrounding street network, which was beyond the scope of this project. It is also believed that the predicted left-turn movement delays for the raised-curb median segments modeled in this study are more nearly equal to the delays actually incurred by these drivers (regardless of whether the median is open or closed) than the delays that would be obtained by assuming that all left-turn vehicles travel to the next downstream median opening, make a U-turn, and return.

Variables Included in the Evaluation

A wide range of traffic demands and geometric configurations were considered in the performance evaluation. Traffic demands included the major-street flow rate and left-turn percentage. Geometric configurations included the number of through traffic lanes and the density of access points along the major street. Table 2-2 lists the specific combinations of these variables used for the evaluation. The factorial combination of variable values listed in this table represents the 648 analyses considered for this evaluation.

The geometric layout that corresponds to one of the three access point densities is shown in Figure 2-1. Specifically, this figure illustrates the "90 access points per mile" scenario. This scenario includes T intersections for the first and last access points to avoid access points within 300 ft of the signalized intersection stop line. The figure also includes the traffic demands computed for one of the analysis scenarios.

Running speed on the arterial was allowed to vary with the through lane flow rate in the corresponding direction. The relationship between speed and volume used in the evaluation is based on the speed-volume relationship developed in Chapter 3 (Equation 21).

Analysis Results

The operational performance of the major-street traffic movements was evaluated for the range of conditions indicated in Table 2-2 using the operations model. Specifically, the delays to the major-street through and left-turn vehicles were computed for each midblock left-turn treatment for a range of traffic demand and geometric conditions. The computed delays have been tabulated for each of the two traffic movements studied. Regression analyses were conducted using the tabulated delays to provide an alternative means of estimating the computed delays.

The analysis scenarios and variables considered for this evaluation were selected to be representative of most urban arterials. Hence, the findings from this analysis can be used by practitioners to evaluate alternative midblock treatments for a wide range of urban arterial conditions.

Delays to Major-Street Through Vehicles

Operations Model Delay Estimates. The delays computed by the operations model for the major-street through movements are provided in Tables 2-3, 2-4, and 2-5. The delays reported in these tables represent the average delay to each through vehicle on any one intersection approach in the subject travel direction. These delays are in units of seconds per vehicle per approach (s/v/a). For each major-street approach,

TABLE 2-2 Range of evaluation variables for the operational effects evaluation

Variable	Range or Levels
Midblock Left-Turn Treatments	Raised-Curb, TWLTL, Undivided cross section
Through Traffic Lanes (both directions)	4, 6
Through Lane Flow Rate[1], vphpl	350, 450, 550, 650, 750, 850
Access Point Density[2] (both directions), access points / mile	30, 60, 90
Left-Turn Percent per 1,320-ft Segment Length[3] (P_L), %	0, 5, 10, 15, 20, 30

Notes:
1 - Traffic volume per lane on the major street (the count of lefts, throughs, and rights on the major-street approach to each access point and averaged for all access points).
2 - Access point density represents the total number of access points on both sides of the major street (i.e., a two-way total) divided by the length of the segment (in miles).
3 - Total number of left turns per hour exiting the major street into an access point in one direction of travel per 1,320-foot length of roadway divided by the total flow rate in that direction (expressed as a percentage).

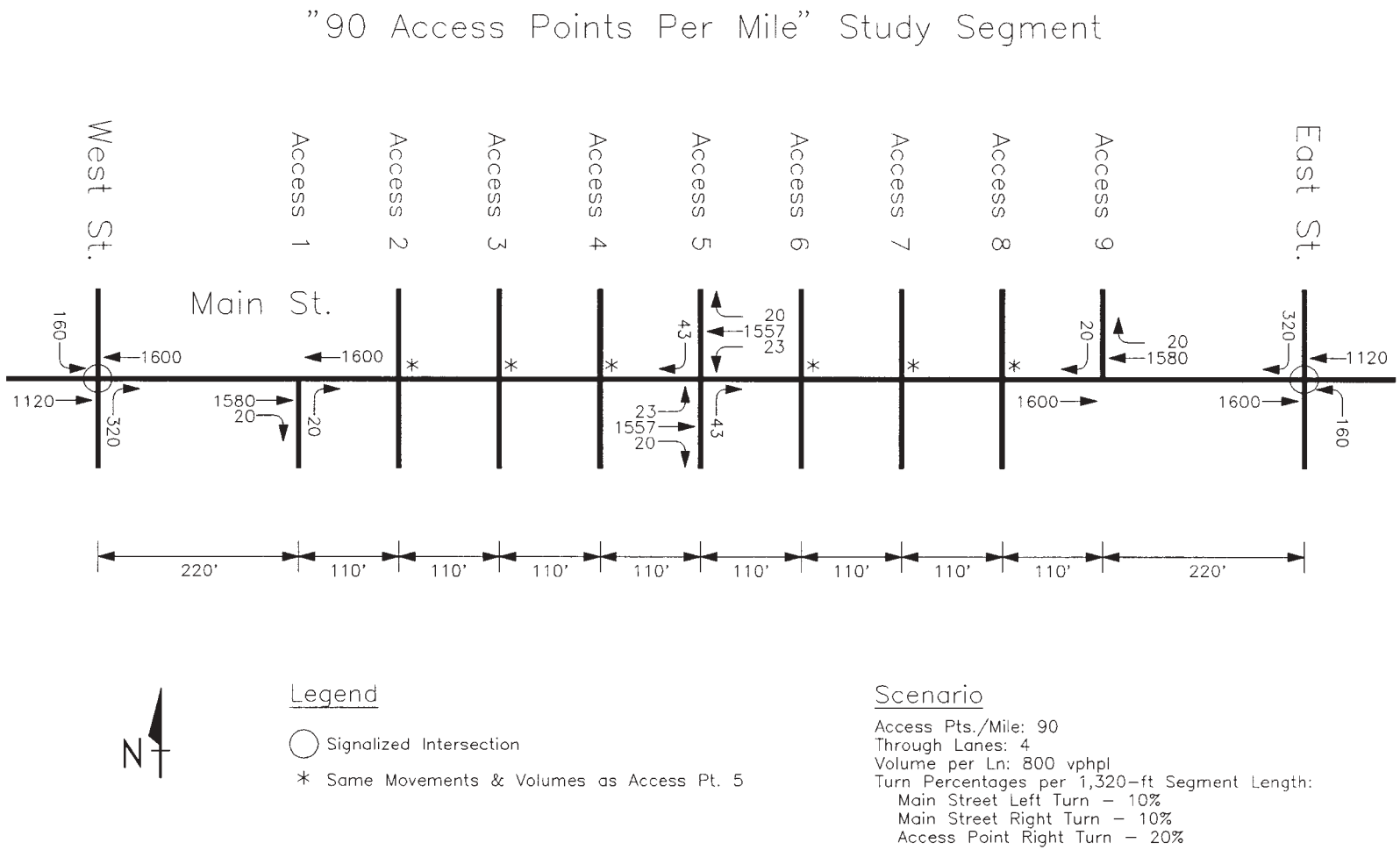

Figure 2-1. Study segment with 90 access points per mile.

TABLE 2-3 Through vehicle delay for the raised-curb median treatment (s/v/a)

Through Lanes	Thru Lane Flow Rate[1] (vphpl)	Access Pt. Density[2] (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length[3]					
			0.0	5.0	10.0	15.0	20.0	30.0
4	350	30			0.10	0.10	0.10	0.10
		60			0.06	0.06	0.06	0.06
		90			0.04	0.04	0.04	0.04
	450	30		0.10	0.10	0.10	0.10	0.13
		60		0.06	0.06	0.06	0.08	0.10
		90		0.04	0.04	0.04	0.06	0.06
	550	30	0.17	0.17	0.17	0.17	0.17	0.17
		60	0.10	0.10	0.10	0.12	0.14	0.16
		90	0.07	0.07	0.07	0.09	0.09	0.11
	650	30	0.20	0.20	0.20	0.20	0.20	0.23
		60	0.12	0.14	0.14	0.18	0.22	0.28
		90	0.10	0.10	0.10	0.11	0.14	0.19
	750	30	0.23	0.23	0.27	0.27	0.27	0.30
		60	0.16	0.16	0.24	0.32	0.38	0.52
		90	0.11	0.11	0.14	0.20	0.27	0.37
	850	30	0.30	0.30	0.30	0.33	0.40	0.47
		60	0.20	0.24	0.42	0.62	0.78	1.12
		90	0.14	0.14	0.27	0.41	0.56	0.80
6	350	30			0.07	0.07	0.07	0.07
		60			0.04	0.04	0.06	0.06
		90			0.03	0.03	0.03	0.04
	450	30		0.10	0.10	0.10	0.10	0.13
		60		0.08	0.08	0.08	0.10	0.12
		90		0.06	0.06	0.06	0.07	0.09
	550	30	0.13	0.13	0.13	0.13	0.13	0.17
		60	0.10	0.10	0.12	0.14	0.16	0.16
		90	0.07	0.07	0.09	0.11	0.13	0.14
	650	30	0.17	0.20	0.20	0.20	0.20	0.20
		60	0.12	0.14	0.20	0.24	0.26	0.40
		90	0.09	0.10	0.14	0.20	0.21	0.21
	750	30	0.23	0.23	0.23	0.23	cong	cong
		60	0.16	0.22	0.32	0.32	0.36	cong
		90	0.11	0.14	0.23	0.27	0.27	0.34
	850	30	0.27	0.27	0.30	cong	cong	cong
		60	0.18	0.34	0.44	0.44	0.64	cong
		90	0.13	0.23	0.33	0.34	0.36	0.63

Notes:

1 - Traffic volume per lane on the major street (averaged over each access point).

2 - Access point density represents the total number of access points on both sides of the street segment (i.e., a two-way total) divided by the length of the segment (in miles).

3 - Total number of left-turns per hour exiting the major street into an access point in one direction of travel per 1,320-ft length of roadway divided by the total flow rate in that direction (expressed as a percentage).

"cong" = Delays to one or more major-street left-turn movements are in excess of 40 s/v/a leading to congested flow conditions, queue spillback, and possible gridlock.

through vehicles are defined as those not turning left or right at that intersection; vehicles traveling through the subject intersection but turning at a downstream intersection are included in the count of through vehicles at the subject intersection. In contrast to other studies, through vehicles are not defined in this study as vehicles that enter and exit the study segment by means of a through movement at the bounding signalized intersections.

The delays computed include delays caused by turning vehicles (left or right) that slow down in a through lane to negotiate the turn. Delays caused by left-turning vehicles stopped in the inside through lane also are included. The

TABLE 2-4 Through vehicle delay for the TWLTL treatment (s/v/a)

Through Lanes	Thru Lane Flow Rate[1] (vphpl)	Access Pt. Density[2] (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length[3]					
			0.0	5.0	10.0	15.0	20.0	30.0
4	350	30			0.10	0.10	0.10	0.10
		60			0.06	0.06	0.06	0.06
		90			0.04	0.04	0.04	0.04
	450	30		0.10	0.10	0.10	0.10	0.13
		60		0.06	0.06	0.06	0.08	0.08
		90		0.04	0.04	0.04	0.06	0.06
	550	30	0.17	0.17	0.17	0.17	0.17	0.17
		60	0.10	0.10	0.10	0.12	0.12	0.12
		90	0.07	0.07	0.07	0.09	0.09	0.07
	650	30	0.20	0.20	0.20	0.20	0.20	0.20
		60	0.12	0.14	0.14	0.14	0.14	0.14
		90	0.10	0.10	0.10	0.09	0.09	0.10
	750	30	0.23	0.23	0.27	0.27	0.27	0.27
		60	0.16	0.16	0.16	0.16	0.16	0.16
		90	0.11	0.11	0.11	0.11	0.11	0.16
	850	30	0.30	0.30	0.30	0.30	0.33	0.33
		60	0.20	0.20	0.20	0.20	0.20	0.26
		90	0.14	0.14	0.14	0.16	0.23	0.36
6	350	30			0.07	0.07	0.07	0.07
		60			0.04	0.04	0.06	0.06
		90			0.03	0.03	0.03	0.03
	450	30		0.10	0.10	0.10	0.10	0.13
		60		0.08	0.08	0.08	0.08	0.08
		90		0.06	0.06	0.06	0.06	0.06
	550	30	0.13	0.13	0.13	0.13	0.13	0.17
		60	0.10	0.10	0.10	0.10	0.10	0.10
		90	0.07	0.07	0.07	0.07	0.07	0.09
	650	30	0.17	0.20	0.20	0.20	0.20	0.20
		60	0.12	0.12	0.12	0.12	0.12	0.14
		90	0.09	0.09	0.09	0.10	0.11	0.13
	750	30	0.23	0.23	0.23	0.23	cong	cong
		60	0.16	0.16	0.16	0.16	0.16	cong
		90	0.11	0.11	0.11	0.14	0.16	0.19
	850	30	0.27	0.27	0.27	cong	cong	cong
		60	0.18	0.18	0.18	0.20	0.20	cong
		90	0.13	0.13	0.17	0.20	0.23	0.31

Notes:

1 - Traffic volume per lane on the major street (averaged over each access point).

2 - Access point density represents the total number of access points on both sides of the street segment (i.e., a two-way total) divided by the length of the segment (in miles).

3 - Total number of left-turns per hour exiting the major street into an access point in one direction of travel per 1,320-ft length of roadway divided by the total flow rate in that direction (expressed as a percentage).

"cong" = Delays to one or more major-street left-turn movements are in excess of 40 s/v/a leading to congested flow conditions, queue spillback, and possible gridlock.

computed delays do not include the added travel time incurred as a result of lower running speeds during higher volume conditions nor any delay incurred while waiting for a traffic signal. These two delays were excluded because they are effectively the same among the three midblock treatments considered.

An examination of the delays reported in Tables 2-3, 2-4, and 2-5 indicated some interesting trends. One trend that is consistent among all three treatments is the increase in delay with the increase in number of through or left-turn vehicles. A second trend is that the raised-curb median and TWLTL treatments generally yield about the same delays; however,

TABLE 2-5 Through vehicle delay for the undivided cross section (s/v/a)

Through Lanes	Thru Lane Flow Rate[1] (vphpl)	Access Pt. Density[2] (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length[3]					
			0.0	5.0	10.0	15.0	20.0	30.0
4	350	30			0.20	0.27	0.30	0.40
		60			0.12	0.16	0.18	0.24
		90			0.09	0.11	0.14	0.19
	450	30		0.27	0.37	0.47	0.53	0.67
		60		0.16	0.22	0.28	0.34	0.44
		90		0.11	0.16	0.20	0.26	0.33
	550	30	0.17	0.40	0.60	0.77	0.87	1.00
		60	0.10	0.24	0.36	0.48	0.58	0.72
		90	0.07	0.17	0.27	0.34	0.43	0.56
	650	30	0.20	0.63	1.00	1.23	1.40	1.43
		60	0.14	0.40	0.62	0.82	1.00	1.18
		90	0.10	0.29	0.46	0.61	0.76	0.97
	750	30	0.23	1.03	1.63	1.97	2.00	1.63
		60	0.16	0.66	1.10	1.42	1.66	1.76
		90	0.11	0.49	0.81	1.11	1.37	1.67
	850	30	0.30	1.80	2.40	2.43	2.23	cong
		60	0.20	1.20	1.96	2.14	2.12	1.64
		90	0.14	0.87	1.56	1.90	1.99	1.80
6	350	30			0.23	0.27	0.27	0.20
		60			0.16	0.20	0.22	0.22
		90			0.11	0.14	0.17	0.20
	450	30		0.30	0.37	0.40	0.37	0.23
		60		0.18	0.28	0.32	0.34	0.30
		90		0.14	0.21	0.27	0.30	0.33
	550	30	0.13	0.50	0.60	0.53	0.40	0.33
		60	0.10	0.34	0.50	0.54	0.50	0.26
		90	0.07	0.26	0.40	0.49	0.50	0.43
	650	30	0.17	0.80	0.80	0.57	cong	cong
		60	0.12	0.58	0.76	0.66	0.48	cong
		90	0.09	0.43	0.67	0.73	0.60	0.34
	750	30	0.20	1.13	0.87	0.60	cong	cong
		60	0.16	0.92	0.94	0.68	0.46	cong
		90	0.11	0.71	0.94	0.77	0.59	0.31
	850	30	0.27	1.30	0.77	cong	cong	cong
		60	0.18	1.24	0.88	0.58	0.58	cong
		90	0.13	1.09	0.99	0.69	0.50	0.46

Notes:

1 - Traffic volume per lane on the major street (averaged over each access point).

2 - Access point density represents the total number of access points on both sides of the street segment (i.e., a two-way total) divided by the length of the segment (in miles).

3 - Total number of left-turns per hour exiting the major street into an access point in one direction of travel per 1,320-ft length of roadway divided by the total flow rate in that direction (expressed as a percentage).

"cong" = Delays to one or more major-street left-turn movements are in excess of 40 s/v/a leading to congested flow conditions, queue spillback, and possible gridlock.

the delays associated with the raised-curb treatment are slightly higher than the TWLTL at the highest volume levels. This trend is consistent with the increased frequency of bay overflow associated with the raised-curb treatment, relative to the TWLTL. The delays associated with the undivided cross section are higher than those of either the raised-curb or TWLTL treatments for all conditions except the "zero left-turn percentage" condition. This increase is a result of the lack of bay storage available to left-turn movements associated with the undivided cross section.

In general, it appears that the range of delays is larger for the four-lane major street than for the six-lane major street.

On the other hand, the six-lane street tends to become congested as a result of high left-turn delays when both the left-turn and through volumes are high, whereas, with one exception, the four-lane street does not become congested.

The delays reported in Tables 2-3, 2-4, and 2-5 were compared with those reported by Harwood (*2*). Harwood quantified the delay reduction attributable to a TWLTL relative to an undivided cross section on a four-lane suburban arterial. His delay data were obtained from the first version of TWLTL-SIM, as applied to an unsignalized street segment with staggered access points.

To facilitate this comparison, the delays reported in Tables 2-4 and 2-5 were used to compute the delay reduction (in vehicle-seconds) resulting from the conversion of an undivided cross section to a TWLTL. This comparison indicated that the delay reductions found in this research are about one-half to one-third of those reported by Harwood for similar volumes and driveway densities. Moreover, Harwood's data indicate that the maximum through lane flow rate for the undivided cross section is about 550 vphpl. In contrast, the findings of this research indicate that through lane flow rates of 850 vphpl will yield acceptable operation for the undivided cross section.

Three factors can be attributed to the differences between the results reported in this research and those reported by Harwood (*2*). First, the results of this research are based on an operations model calibrated using TWLTL-SIM III, which has undergone significant enhancement since the original version used by Harwood. Second, one of the TWLTL-SIM III enhancements is the capability of modeling signalized intersections at the beginning and end of the study segment. The effect of these traffic signals is to increase the capacity and lower the delays to access point traffic movements.

The third factor relates to the alignment of access points on opposite sides of the arterial. As mentioned previously, for the study segment for this analysis, unstaggered access point locations were used to facilitate the equitable comparison of all three midblock left-turn treatments. Research by Batra (*3*) has shown that delays are higher when access point locations are staggered along the street. This type of positioning creates a few areas of concentrated, opposite-direction left-turn activity that can amplify the frequency and duration of left-turn queues in the inside through lane. These queues will precipitate congestion and increase through vehicle delay at lower volume levels.

In summary, the differences between the findings of this research and those of Harwood (*2*) point out the wide range of operating conditions that may be encountered and the delays that can be incurred. The magnitude of the delay difference may depend on the nature of the study segment traffic control and the access point orientation.

Regression Models. SAS (*4*) was used to calibrate a predictive model of through vehicle delay. Specifically, the SAS nonlinear regression procedure (NLIN) was used to determine the best model fit to the delay data in Tables 2-3, 2-4, and 2-5. The form of the model was based on an examination of the data in the tables and the observance of logical boundary conditions (e.g., no delay when there is no volume). Several candidate model forms were hypothesized and examined; the following model form was found to yield the best fit to the data:

$$d_T = b_0 \times f_L \times f_T \tag{3}$$

with:

$$f_L = b_1 + (1 - x)x^{b_2} \tag{4}$$

$$f_T = \frac{y}{1 - y} \tag{5}$$

$$x = \frac{v_L}{1,800\left(1 - \frac{V_{N,o}}{C_m}\right)} \leq 1.0 \tag{6}$$

$$y = \frac{V - v_L - v_R}{1,800\left[\frac{N}{2} - x(1 - I_{TR})\right]} \tag{7}$$

where:

d_T = average through vehicle delay per approach in the subject direction of travel, s/v/a
f_L = effect of left-turn volume on through delay
f_T = effect of through volume on through delay
x = variable relating left-turn volume to opposing flow rate
y = variable relating through volume to number of traffic lanes
v_L = average left-turn volume per access point ($= V_L/n$), vph/access point
V_L = total number of vehicles turning left from the major street in the subject direction of travel (at all access points), vph
v_R = average right-turn volume per access point ($= V_R/n$), vph/access point
V_R = total number of vehicles turning right from the major street in the subject direction of travel (at all access points), vph
V = total volume in subject direction ($= V_N * N/2$), vph
V_N = through lane flow rate in subject direction (i.e., the count of lefts, throughs, and rights on the major-street approach to each access point divided by the number of approach lanes—averaged for all access points), vphpl
$V_{N,o}$ = through lane flow rate in the opposing direction, vphpl
N = number of through traffic lanes on the major street (both directions)

n = number of access points located on the outside (right side) of the major street in the subject direction of travel

C_m = constant (900 for undivided cross sections; 1,000 otherwise)

I_{TR} = indicator variable (1.0 for raised-curb or TWLTL treatments; 0.0 otherwise)

b_0, b_1, b_2 = regression parameter coefficients.

The results of the regression analysis are provided in Table 2-6 for each of the three treatments. As the statistics in this table indicate, the model provides a relatively good fit to the data. However, the standard error (i.e., root mean square error) is large enough to suggest that the fit is not perfect. Thus, the values in Tables 2-3, 2-4, and 2-5 should be preferred to those predicted by Equation 3 whenever the application allows a choice. The quality of fit of the calibrated model is shown in Figures 2-2, 2-3, and 2-4.

Delays to Major-Street Left-Turn Vehicles

Operations Model Delay Estimates. The delays computed by the operations model for the major-street left-turn movements are provided in Tables 2-7, 2-8, and 2-9. The delays reported in these tables represent the average total delay to each left-turning vehicle, as defined in Chapter 10 of the HCM (*1*). The units of the delay are seconds per vehicle per approach (s/v/a).

The left-turn delays represent an overall average delay relative to the many possible left-turn locations (i.e., access points) along the study segment. Left-turn delays at unsignalized intersections closer to the downstream signalized intersection are lower, whereas those at intersections more distant from the signalized intersection are higher. The delays reported represent only the delays incurred by the major-street left-turning drivers in the vicinity of the turn location; delays that drivers may incur one or more intersections upstream from the turn location are attributed to through vehicle delay, as discussed in the preceding section.

An examination of the delays reported in Tables 2-7, 2-8, and 2-9 indicated several interesting trends. One trend is the increasing left-turn delay with increasing left-turn or through volume. Another trend is that left-turn delays for the four-lane major street are always lower than those for the six-lane major street for the same combination of through lane flow rate, access point density, and left-turn percentage. The latter trend is a result of the higher total flow rate associated with the six-lane streets and the corresponding lower capacity.

In general, the left-turn delays for the TWLTL and raised-curb treatments are similar. However, the delays for the raised-curb treatment are slightly larger than those of the TWLTL at the highest combinations of left-turn and through volume. The delays for the undivided cross section tend to be a little higher than those of the TWLTL or raised-curb median treatment. These higher delays result primarily from the greater dispersion of the traffic platoon (caused by the increased frequency of blockage in the inside through lane) and a corresponding decrease in left-turn capacity. As noted previously, the higher left-turn and through volume combinations for the six-lane street tended to experience delays of sufficient magnitude to precipitate extensive queueing and the likelihood of congestion.

Regression Models. The SAS system (*4*) was used to calibrate a predictive model of left-turn vehicle delay. Specifically, the SAS nonlinear regression procedure (NLIN) was used to determine the best model fit to the delay data in Tables 2-7, 2-8, and 2-9. The form of the model was based

TABLE 2-6 Calibrated through delay regression model

Model Statistics		Midblock Left-Turn Treatment		
		Raised-Curb	TWLTL	Undivided
Overall Statistics	R^2	0.76	0.63	0.73
	Root Mean Square Error (s/v/a)	0.071	0.044	0.29
	Observations	231	231	227
Coefficient Values	b_0	2.48	1.13	1.89
	b_1	0.0903	0.203	0.215
	b_2	1.13	1.54	0.271
Std. Dev. of Coefficients	b_0	0.29	0.32	0.14
	b_1	0.011	0.055	0.023
	b_2	0.092	0.25	0.042
t-statistic of Coefficients	b_0	8.6	3.5	13.5
	b_1	8.2	3.7	9.3
	b_2	12.3	6.2	6.5

```
                Predicted Through Vehicle Delay, s/v/a

     |           (Legend: A = 1 obs, B = 2 obs, etc.)
0.6 +                 AA
     |           A                  A
     |         AB AA
     |                     AA
     |          A
     |         A  A  A A
0.4 +        BB A
     |       A  C B
     |        ACAAA  A
     |       BFA B
     |     AABDD A
     |     AFDA
0.2 +    AFFEBA
     |    HIHBB
     |   BMLB
     |  BKQB
     |  SSI
     |  AA
0.0 +
     --+------------+------------+------------+---------
      0.0          0.5          1.0          1.5

Through Vehicle Delay from Operations Model, s/v/a
```

Figure 2-2. Comparison of through delays for the raised-curb median treatment.

on an examination of the data in the tables and the observance of logical boundary conditions (e.g., no delay when there is no volume). Several candidate model forms were hypothesized and examined; the following model form was found to yield the best fit to the data:

$$d_L = \begin{cases} b_0 u(1 + x^{b_2}) & : if\ v_L > 0 \\ 0.0 & : if\ v_L = 0 \end{cases} \tag{8}$$

$$u = \frac{3,600}{V_o e^{(-V_o g/3,600)}} \tag{9}$$

```
                Predicted Through Vehicle Delay, s/v/a

     |           (Legend: A = 1 obs, B = 2 obs, etc.)
     |
0.4 +
     |
     |
     |                                        A
0.3 +                        A      A        B
     |                       B  A    A   A      A
     |                     A B  A        A
     |                 A A AB A       B   A
0.2 +               C C B AE B   A   D   C
     |             ED B J  B     B   D   A
     |             DA A C    B   D
     |          E E F  E  B   G
0.1 +         IE J D E   G   A
     |     B K  G H   B
     |   C K JF   F
     |
0.0 +
     |
     -+-----------+-----------+-----------+-----------+-
     0.0         0.1         0.2         0.3         0.4

Through Vehicle Delay from Operations Model, s/v/a
```

Figure 2-3. Comparison of through delays for the TWLTL treatment.

```
                Predicted Through Vehicle Delay, s/v/a

   |           (Legend: A = 1 obs, B = 2 obs, etc.)
 3 +
   |
   |
   |                      A
   |
   |
 2 +                        A    A   A
   |                           A A  A
   |                          B
   |          A A A      A  A
   |        A   A  A BA      A  A A
   |         AABA BAAA A BAAA  A
 1 +     A A CAAAD BA  B  AA   A
   |     A BA B AAB A  AA         A
   |      AB FCDA B  A
   |    EEDEGCBBA B
   |  CPUMEBA A
   |  LPG
 0 +
   --+------------+------------+------------+---------
     0            1            2            3

Through Vehicle Delay from Operations Model, s/v/a
```

Figure 2-4. Comparison of through delays for the undivided cross section.

$$g = \begin{cases} b_1 & : if\ N = 4 \\ b_3 & : if\ N = 6 \end{cases} \tag{10}$$

where:

d_L = average total left-turn vehicle delay per approach in the subject direction of travel, s/v/a

x = variable relating left-turn volume to opposing flow rate (= $v_L * u/3{,}600$)

v_L = average left-turn volume per access point (= V_L/n), vph/access point

V_L = total number of vehicles turning left from the major street in the subject direction of travel (at all access points), vph

n = number of access points located on the outside (right side) of the major street in the subject direction of travel

V_o = total volume in the opposing direction (= $V_{N,o} * N/2$), vph

$V_{N,o}$ = through lane flow rate in the opposing direction, vphpl

N = number of through traffic lanes on the major street (both directions)

b_0, b_1, b_2, b_3 = regression parameter coefficients.

The results of the regression analysis are provided in Table 2-10 for each of the three treatments. As the statistics in this table indicate, the model provides a relatively good fit to the data. However, the standard error (i.e., root mean square error) is large enough to suggest that the fit is not perfect. Thus, the values in Tables 2-7, 2-8, and 2-9 should be preferred to those predicted by Equation 8 whenever the application allows a choice. The quality of fit of the calibrated model is shown in Figures 2-5, 2-6, and 2-7.

Annual Delay to Major-Street Left-Turn and Through Vehicles

The results of the preceding analysis were used to compute the delay incurred by the traffic stream during 1 year. This computation was accomplished by determining the left-turn and through movement delays for each hour of a representative subset of all hours during a typical year. The delays corresponding to each of these representative hour intervals were then aggregated to represent the total vehicle-hours of delay experienced by the major-street left-turn and through movements during a typical year. This type of annualized aggregation of the delay data is intended to make the results described in the preceding section more useful for planning-level assessments of operational performance.

As mentioned previously, the computation of annual delay required an estimate of delay incurred during each hour of the year. Because the hourly traffic volume varies widely during the year and recognizing the significant effort required to analyze each hour, it was determined that a reasonable level of accuracy could be obtained by discretizing the distribution of hourly traffic flows into five representative intervals and conducting an hourly analysis for the average hourly flow within each interval. The total delays for the five intervals could then be added, as weighted by their frequency

TABLE 2-7 Major-street left-turn vehicle delay for the raised-curb median treatment (s/v/a)

Through Lanes	Thru Lane Flow Rate[1] (vphpl)	Access Pt. Density[2] (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length[3]					
			0.0	5.0	10.0	15.0	20.0	30.0
4	350	30	0.0	4.2	4.2	4.1	4.1	4.0
		60	0.0	4.2	4.2	4.2	4.2	4.2
		90	0.0	4.2	4.2	4.3	4.3	4.2
	450	30	0.0	5.4	5.3	5.2	5.2	5.1
		60	0.0	5.4	5.4	5.4	5.4	5.3
		90	0.0	5.4	5.4	5.4	5.4	5.4
	550	30	0.0	6.8	6.8	6.7	6.7	6.5
		60	0.0	6.9	6.9	6.9	7.0	6.9
		90	0.0	7.0	7.0	7.0	7.0	7.0
	650	30	0.0	8.7	8.8	8.9	8.8	8.7
		60	0.0	6.9	9.1	9.1	9.1	9.3
		90	0.0	9.0	9.1	9.2	9.3	9.3
	750	30	0.0	11.6	11.8	12.4	12.4	12.6
		60	0.0	12.0	12.3	12.6	12.7	13.3
		90	0.0	12.0	12.4	12.6	13.0	13.1
	850	30	0.0	15.8	16.3	17.9	19.5	23.2
		60	0.0	16.4	17.2	18.3	19.0	23.4
		90	0.0	16.3	17.2	18.0	19.0	20.9
6	350	30	0.0	5.6	5.6	5.6	5.6	5.6
		60	0.0	5.7	5.7	5.8	5.8	5.8
		90	0.0	5.8	5.8	5.8	5.8	5.9
	450	30	0.0	7.5	7.6	7.6	7.7	8.0
		60	0.0	7.6	7.8	7.9	8.0	8.2
		90	0.0	7.6	7.8	8.0	8.1	8.3
	550	30	0.0	9.6	10.1	10.7	10.9	11.9
		60	0.0	9.8	10.5	11.0	11.4	12.5
		90	0.0	9.8	10.5	11.0	11.5	12.3
	650	30	0.0	12.8	13.8	17.1	19.2	cong
		60	0.0	13.1	14.3	16.0	17.5	22.6
		90	0.0	13.0	14.3	15.9	17.2	20.3
	750	30	0.0	17.0	19.1	24.2	cong	cong
		60	0.0	17.3	18.6	21.0	24.0	cong
		90	0.0	17.3	18.5	19.8	21.8	28.2
	850	30	0.0	22.0	25.2	cong	cong	cong
		60	0.0	21.9	23.8	27.3	33.8	cong
		90	0.0	21.6	22.8	24.6	26.9	37.6

Notes:

1 - Traffic volume per lane on the major street (averaged over each access point).

2 - Access point density represents the total number of access points on both sides of the street segment (i.e., a two-way total) divided by the length of the segment (in miles).

3 - Total number of left-turns per hour exiting the major street into an access point in one direction of travel per 1,320-ft length of roadway divided by the total flow rate in that direction (expressed as a percentage).

"cong" = Delays to one or more major-street left-turn movements are in excess of 40 s/v/a leading to congested flow conditions, queue spillback, and possible gridlock.

of occurrence, to yield the annual total delay for a particular midblock treatment.

The frequency distribution of annual hourly volumes used for this analysis is shown in Table 2-11. The volume intervals were narrowest for the peak hours of the year and broadest for the nonpeak hours. This technique improves the representation of the peak traffic hours in which most of the traffic delay typically occurs. The values reported in Table 2-11 represent the hourly volumes found at several typical urban arterials in Nebraska (*5*). They are reported as a percentage of the total daily traffic because percentages tend to be fairly constant among facility types. Therefore, their use in com-

TABLE 2-8 Major-street left-turn vehicle delay for the TWLTL treatment (s/v/a)

Through Lanes	Thru Lane Flow Rate[1] (vphpl)	Access Pt. Density[2] (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length[3]					
			0.0	5.0	10.0	15.0	20.0	30.0
4	350	30	0.0	4.2	4.2	4.1	4.1	4.0
		60	0.0	4.2	4.2	4.2	4.2	4.1
		90	0.0	4.2	4.2	4.2	4.2	4.2
	450	30	0.0	5.4	5.3	5.2	5.2	5.1
		60	0.0	5.4	5.4	5.4	5.4	5.3
		90	0.0	5.4	5.4	5.4	5.4	5.4
	550	30	0.0	6.7	6.7	6.7	6.6	6.5
		60	0.0	6.8	6.9	6.9	6.9	6.8
		90	0.0	6.9	6.9	6.9	7.0	6.9
	650	30	0.0	8.5	8.6	8.9	8.8	8.6
		60	0.0	8.8	9.0	9.0	9.0	9.2
		90	0.0	8.8	9.0	9.1	9.2	9.2
	750	30	0.0	11.4	11.6	12.2	12.4	12.5
		60	0.0	11.8	12.1	12.4	12.5	13.1
		90	0.0	11.8	12.2	12.5	12.8	12.9
	850	30	0.0	15.6	16.0	17.3	19.4	22.6
		60	0.0	15.8	16.7	17.8	18.3	21.1
		90	0.0	15.9	16.8	17.7	18.4	19.9
6	350	30	0.0	5.5	5.5	5.5	5.5	5.6
		60	0.0	5.7	5.7	5.7	5.7	5.7
		90	0.0	5.7	5.7	5.7	5.7	5.8
	450	30	0.0	7.2	7.5	7.5	7.5	7.7
		60	0.0	7.3	7.6	7.8	7.9	8.2
		90	0.0	7.3	7.6	7.8	8.0	8.2
	550	30	0.0	9.2	9.6	10.6	10.7	11.5
		60	0.0	9.4	10.0	10.6	11.2	12.1
		90	0.0	9.4	10.1	10.7	11.1	11.9
	650	30	0.0	12.2	12.9	15.3	18.8	cong
		60	0.0	12.4	13.4	15.3	17.6	21.3
		90	0.0	12.4	13.5	15.1	16.5	19.5
	750	30	0.0	16.2	17.7	23.5	cong	cong
		60	0.0	16.4	17.8	19.5	22.6	cong
		90	0.0	16.4	17.6	18.9	20.8	27.2
	850	30	0.0	21.5	24.3	cong	cong	cong
		60	0.0	21.4	22.7	25.2	30.3	cong
		90	0.0	21.1	22.1	23.7	26.0	36.8

Notes:

1 - Traffic volume per lane on the major street (averaged over each access point).

2 - Access point density represents the total number of access points on both sides of the street segment (i.e., a two-way total) divided by the length of the segment (in miles).

3 - Total number of left-turns per hour exiting the major street into an access point in one direction of travel per 1,320-ft length of roadway divided by the total flow rate in that direction (expressed as a percentage).

"cong" = Delays to one or more major-street left-turn movements are in excess of 40 s/v/a leading to congested flow conditions, queue spillback, and possible gridlock.

puting the annual delay is expected to yield results applicable to a wide range of urban arterial traffic conditions.

A sample calculation of total annual delay to the major-street left-turn and through movements is shown in Table 2-12. This calculation applies to the TWLTL treatment on a four-lane cross section. The study segment has an access point density of 30 access points per mile, yielding a total of six access points (three on each side) in its 1,320-ft length. Fifteen percent of the traffic stream is distributed evenly among the access points as left-turn volume. The average daily traffic demand on the study segment is 32,500 vpd. Using the five hourly volume intervals from Table 2-11, the analysis proceeds as shown in Table 2-12 for one travel direction. Because of the symmetry of the analysis scenarios,

TABLE 2-9 Major-street left-turn vehicle delay for the undivided cross section (s/v/a)

Through Lanes	Thru Lane Flow Rate[1] (vphpl)	Access Pt. Density[2] (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length[3]					
			0.0	5.0	10.0	15.0	20.0	30.0
4	350	30	0.0	4.3	4.3	4.3	4.3	4.4
		60	0.0	4.4	4.4	4.4	4.4	4.4
		90	0.0	4.4	4.4	4.4	4.4	4.4
	450	30	0.0	5.5	5.5	5.6	5.6	5.8
		60	0.0	5.6	5.6	5.6	5.7	5.7
		90	0.0	5.5	5.6	5.6	5.6	5.7
	550	30	0.0	7.1	7.3	7.4	7.5	7.9
		60	0.0	7.2	7.3	7.4	7.5	7.7
		90	0.0	7.2	7.2	7.3	7.4	7.6
	650	30	0.0	9.3	9.8	10.1	10.4	11.4
		60	0.0	9.4	9.6	9.9	10.3	10.8
		90	0.0	9.4	9.6	9.9	10.1	10.6
	750	30	0.0	12.7	13.8	14.9	16.2	20.6
		60	0.0	12.8	13.5	14.2	15.0	17.5
		90	0.0	12.8	13.4	14.1	14.9	16.6
	850	30	0.0	17.8	20.7	27.4	39.0	cong
		60	0.0	17.9	20.1	22.4	26.3	35.4
		90	0.0	17.9	19.7	21.6	23.7	28.4
6	350	30	0.0	6.0	6.0	6.1	6.3	6.5
		60	0.0	6.0	6.0	6.1	6.1	6.4
		90	0.0	6.0	6.0	6.0	6.2	6.3
	450	30	0.0	8.0	8.3	8.6	9.0	10.2
		60	0.0	8.0	8.3	8.6	8.8	9.4
		90	0.0	8.0	8.2	8.5	8.7	9.2
	550	30	0.0	10.5	11.8	12.9	14.0	19.0
		60	0.0	10.5	11.4	12.4	13.1	15.3
		90	0.0	10.4	11.3	12.1	12.8	14.3
	650	30	0.0	14.4	17.2	22.4	cong	cong
		60	0.0	14.3	16.5	18.7	21.7	cong
		90	0.0	14.1	16.0	17.9	20.0	24.9
	750	30	0.0	19.8	24.1	35.2	cong	cong
		60	0.0	19.0	21.5	24.8	29.3	cong
		90	0.0	18.8	20.7	22.8	25.2	32.6
	850	30	0.0	24.5	30.8	cong	cong	cong
		60	0.0	23.2	25.7	29.3	35.4	cong
		90	0.0	22.6	24.2	26.0	28.4	37.6

Notes:

1 - Traffic volume per lane on the major street (averaged over each access point).

2 - Access point density represents the total number of access points on both sides of the street segment (i.e., a two-way total) divided by the length of the segment (in miles).

3 - Total number of left-turns per hour exiting the major street into an access point in one direction of travel per 1,320-ft length of roadway divided by the total flow rate in that direction (expressed as a percentage).

"cong" = Delays to one or more major-street left-turn movements are in excess of 40 s/v/a leading to congested flow conditions, queue spillback, and possible gridlock.

the delay for each travel direction is the same. As shown in the last row of the table, the total delay for both travel directions is 4,000 hours per year.

The computations shown in Table 2-12 were repeated for each of the three midblock left-turn treatments with each combination of through lanes, traffic demand, access point density, and left-turn percentage. The traffic demands used in this analysis represent the range of daily demands typically found at four- and six-lane urban arterials that yield a level of service B or worse during peak periods. The results of this analysis are provided in Tables 2-13, 2-14, and 2-15.

The trends shown in Tables 2-13, 2-14, and 2-15 are consistent with those discussed for the individual movement delays in the preceding section. Specifically, the data indicate

TABLE 2-10 Calibrated left-turn delay regression model

Model Statistics		Midblock Left-Turn Treatment		
		Raised-Curb	TWLTL	Undivided
Overall Statistics	R^2	0.92	0.92	0.89
	Root Mean Square Error (s/v/a)	1.89	1.78	2.73
	Observations	201	201	197
Coefficient Values	b_0	0.237	0.231	0.292
	b_1	6.21	6.20	6.35
	b_2	0.410	0.391	0.667
	b_3	5.30	5.27	5.21
Std. Dev. of Coefficients	b_0	0.017	0.016	0.027
	b_1	0.13	0.13	0.159
	b_2	0.048	0.046	0.082
	b_3	0.092	0.090	0.107
t-statistic of Coefficients	b_0	13.9	14.4	10.8
	b_1	47.8	47.7	39.9
	b_2	8.5	8.5	8.1
	b_3	57.6	58.6	48.7

that there is no difference among treatment types when there is no left-turn volume. However, as the left-turn volume increases, differences begin to emerge. The raised-curb treatment has slightly higher delays than the TWLTL treatment at the highest left-turn and through volume ranges; this trend results from the greater likelihood of bay overflow for the raised-curb median treatment under high-volume conditions. The undivided cross section has significantly higher delays than the raised-curb treatment for all nonzero combinations of left-turn and through volume. The difference increases exponentially with an increase in left-turn or through volume. This trend is the result of the added turbulence in the undivided treatment's through traffic stream that stems from left-turns being made from the inside lane.

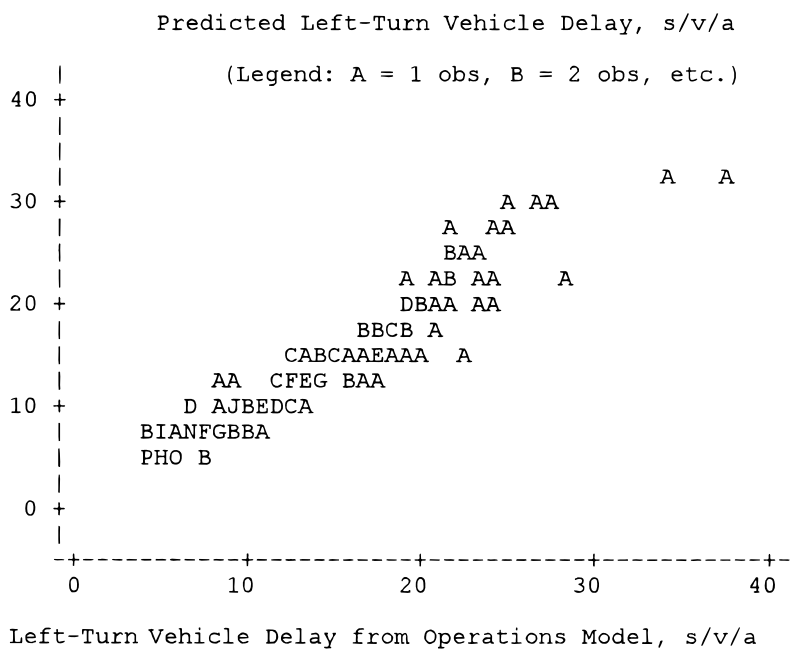

Figure 2-5. Comparison of left-turn delays for the raised-curb median treatment.

```
                Predicted Left-Turn Vehicle Delay, s/v/a

     |           (Legend: A = 1 obs, B = 2 obs, etc.)
  40 +
     |
     |
     |
     |                                     A       A
     |                              AAA
     |                             AA
     |                           ABA
     |                          ABA A
  20 +                       ABC A B     A
     |                     B CBAAA
     |                 AAA ADBAA  A
     |                AEECEBAA A
     |            AE BDIC
     |          JBBICDA
     |       IMGHJCA
     |       IGF
     |
   0 +
     -+-----------+-----------+-----------+-----------+-
      0          10          20          30          40

Left-Turn Vehicle Delay from Operations Model, s/v/a
```

Figure 2-6. Comparison of left-turn delays for the TWLTL treatment.

Another trend apparent in Tables 2-13, 2-14, and 2-15 is that the extra lanes associated with the six-lane major street (relative to the four-lane street) tend to reduce delays for the same daily traffic demand levels. The reduction is typically from 30 to 40 percent for the raised-curb median and TWLTL treatments and 40 to 60 percent for the undivided cross section.

Finally, the analysis indicates that any treatment type can function without causing congestion to the major-street movements at average daily traffic demands of 40,000 vpd or less. Demands in excess of 40,000 vpd are possible for both the four- and six-lane streets; however, congested conditions are likely to occur. When demand exceeds 40,000 vpd, congestion on a four-lane street is more likely to be found at the signal-

```
                Predicted Left-Turn Vehicle Delay, s/v/a

     |           (Legend: A = 1 obs, B = 2 obs, etc.)
  40 +                                          A  A
     |
     |                                     AA A
     |
     |                                  B
     |                               B   AA
     |                              A             B    A
     |                            AA AA  AAA   A
     |                           BAA AB A
  20 +                     A   ABBBAA
     |                   A BCAAAA A
     |                A  BCAACAAA A   A
     |            A BC BBCBAAB A
     |           DAAF ADACAA
     |         D KAFACC
     |       R RDAI
     |         G
     |
   0 +
     -+-----------+-----------+-----------+-----------+-
      0          10          20          30          40

Left-Turn Vehicle Delay from Operations Model, s/v/a
```

Figure 2-7. Comparison of left-turn delays for the undivided cross section.

TABLE 2-11 Frequency distribution of annual hourly flow percentages

Variable	Units	Hourly Volume Interval					Total
		1	2	3	4	5	
Intervals of annual hourly volumes (ranked from highest to lowest volume)	hours	0-250	250-500	500-1,500	1,500-4,000	4,000-8,760	
Frequency of occurrence	hours	250	250	1,000	2,500	4,760	8,760
Average hourly volume of interval (as a percent of ADT)	%	9.6	8.5	7.4	5.7	1.6	

ized intersections than at the major-street left-turn movement of an access point. In contrast, congestion on a six-lane street is more likely to be found at the major-street left-turn movement of an access point than at the signalized intersections.

SAFETY EFFECTS OF MIDBLOCK LEFT-TURN TREATMENTS

Approach

This section describes an evaluation of the effect of three alternative midblock left-turn treatments on the safety of traffic operations on a typical urban arterial. These treatments include the raised-curb median, TWLTL, and undivided cross section. The traffic safety afforded by each treatment was evaluated using the safety model developed for this study. This model was developed using nonlinear regression techniques that account for the nonconstant variance associated with accident data. A complete description of the model and its calibration is provided in Chapter 4.

The safety evaluation of each alternative left-turn treatment is based on the estimation of the average number of accidents that would be expected to occur on a typical urban arterial street segment having one of three treatment types.

TABLE 2-12 Sample calculation of annual delay to major-street left-turn and through vehicles

Midblock Left-Turn Treatment:	Two-Way Left-Turn Lane
Through Lanes (N):	4
Access Point Density:	30 access points / mile
Average Daily Traffic Demand (ADT):	32,500 vpd
Left-Turn Percent per 1,320-ft Segment Length (P_L):	15%
Access Points per 1,320-ft Segment Length (one-way total) (n):	3

Variable[1] 211	Units	Hourly Volume Interval					Total
		1	2	3	4	5	
a. Frequency of occurrence	hours	250	250	1,000	2,500	4,760	8,760
b. Average hourly volume as a % of ADT	%	9.6	8.5	7.4	5.7	1.6	
c. Average hourly volume (= b ADT / 200)	vph	1,560	1,381	1,203	926	260	
d. Hourly volume/lane (thru lane flow rate) (= 2 c / N)	vphpl	780	691	601	463	130	
e. Total lefts from major street (= c P_L / 100)	vph	234	207	180	139	39	
f. Total rights from major street (10%) (= c / 10)	vph	156	138	120	93	26	
g. Lefts from major street at each access point (= e / n)	vph	78	69	60	46	13	
h. Rights from major st. at each access point (= f / n)	vph	52	46	40	31	9	
i. Throughs on major st. at each access point (= c - g - h)	vph	1,430	1,266	1,102	849	238	
j. Through vehicle delay (interpolation of Table 2-4)	s/v/a	0.27	0.23	0.19	0.13	0.027	
k. Major-street left-turn delay (interpolation of Table 2-8)	s/v/a	13.5	10.5	8.1	5.5	2.2	
l. Annual through vehicle delay (= a i j n / 3,600)	hrs/yr	79	60	175	241	25	581
m. Annual left-turn vehicle delay (= a g k n / 3,600)	hrs/yr	219	151	407	531	111	1,419
n. Annual through and left-turn delay (= l + m)	hrs/yr	298	211	582	772	136	2,000
o. Annual delay for *both* travel directions	hrs/yr	596	422	1,164	1,544	272	4,000

Notes:
1 - Unless specifically noted, the computations represent one direction of flow along the street segment.

TABLE 2-13 Annual delay to major-street left-turn and through vehicles for the raised-curb median treatment (hr/yr)

Through Lanes	ADT	Access Pt. Density[1] (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length[2]					
			0	5	10	15	20	30
4	17,500	30	300	400	800	1,000	1,200	1,600
		60	300	400	800	1,000	1,300	1,700
		90	300	400	800	1,000	1,300	1,700
	22,500	30	500	800	1,300	1,700	2,000	2,700
		60	500	800	1,400	1,800	2,200	2,900
		90	500	900	1,400	1,800	2,200	2,900
	27,500	30	800	1,300	2,100	2,700	3,200	4,400
		60	800	1,300	2,300	3,000	3,600	5,000
		90	800	1,500	2,300	3,000	3,600	5,000
	32,500	30	1,200	2,000	3,100	4,000	4,900	6,900
		60	1,200	2,100	3,500	4,800	5,900	8,500
		90	1,200	2,200	3,400	4,700	5,900	8,400
	37,500	30	1,600	2,900	4,400	5,900	7,300	10,600
		60	1,700	3,100	5,300	7,300	9,300	13,800
		90	1,800	3,200	5,100	7,200	9,300	13,500
	42,500	30	2,200	4,100	6,100	8,400	10,700	16,100
		60	2,400	4,600	7,600	10,900	14,200	21,800
		90	2,500	4,500	7,300	10,600	14,100	21,200
6	26,250	30	300	800	1,300	1,800	2,100	3,200
		60	400	900	1,400	2,000	2,400	3,200
		90	400	900	1,400	2,100	2,500	3,500
	33,750	30	500	1,400	2,300	3,200	3,900	5,800
		60	700	1,500	2,600	3,500	4,400	6,200
		90	700	1,500	2,600	3,700	4,500	6,500
	41,250	30	900	2,200	3,700	5,300	6,700	9,800
		60	1,200	2,500	4,300	5,900	7,700	11,500
		90	1,200	2,500	4,300	6,100	7,500	11,300
	48,750	30	1,400	3,400	5,600	8,500	11,200	16,200
		60	1,800	4,000	6,800	9,400	12,700	20,700
		90	1,800	4,000	6,900	9,700	12,200	19,400
	56,250	30	2,100	5,000	8,400	13,300	cong	cong
		60	2,500	6,100	10,400	14,500	20,400	cong
		90	2,600	6,100	10,500	14,800	19,100	32,000
	63,750	30	2,900	7,100	12,200	cong	cong	cong
		60	3,400	9,000	15,500	21,800	cong	cong
		90	3,500	8,900	15,600	22,000	29,200	cong

Notes:

1 - Access point density represents the total number of access points on both sides of the street segment (i.e., a two-way total) divided by the length of the segment (in miles).

2 - Total number of left-turns per hour exiting the major street into an access point in one direction of travel per 1,320-ft length of roadway divided by the total flow rate in that direction (expressed as a percentage).

"cong" = Delays to one or more major-street left-turn movements are in excess of 40 s/v/a leading to congested flow conditions, queue spillback, and possible gridlock.

"Total arterial accidents" was selected as the most appropriate measure of safety (as opposed to traffic conflicts, injury accidents, accident type, and so on) because it provides the most direct measure of overall safety and its use precludes the need to divide the accident database into subsets. In general, "subsetting" a database to focus on a specific accident severity or type reduces the sample size available for model calibration without necessarily reducing the variability in the data or yielding better safety estimators.

The focus of the safety evaluation is on the accidents occurring on an urban arterial between, but not including, the bounding signalized intersections. Arterial accidents on the approach to and within the intersections formed by the driveways or unsignalized public streets are considered in this evaluation. The evaluation does not include accidents occur-

TABLE 2-14 Annual delay to major-street left-turn and through vehicles for the TWLTL treatment (hr/yr)

Through Lanes	ADT	Access Pt. Density[1] (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length[2]					
			0	5	10	15	20	30
4	17,500	30	300	400	800	1,000	1,200	1,600
		60	300	400	800	1,000	1,300	1,700
		90	300	400	800	1,000	1,300	1,700
	22,500	30	500	800	1,300	1,700	2,000	2,700
		60	500	800	1,400	1,800	2,200	2,900
		90	500	900	1,400	1,800	2,200	2,900
	27,500	30	800	1,300	2,100	2,700	3,200	4,400
		60	800	1,300	2,200	2,800	3,400	4,600
		90	800	1,500	2,200	2,800	3,400	4,700
	32,500	30	1,200	2,000	3,000	4,000	4,900	6,800
		60	1,200	2,100	3,200	4,200	5,100	7,100
		90	1,200	2,200	3,200	4,200	5,200	7,400
	37,500	30	1,600	2,900	4,300	5,800	7,200	10,400
		60	1,700	3,000	4,600	6,000	7,500	10,700
		90	1,800	3,200	4,600	6,000	7,800	11,200
	42,500	30	2,200	4,000	6,000	8,200	10,500	15,500
		60	2,400	4,300	6,400	8,600	10,700	16,000
		90	2,500	4,400	6,400	8,600	11,200	16,600
6	26,250	30	300	800	1,300	1,800	2,100	3,200
		60	400	900	1,400	2,000	2,400	3,200
		90	400	900	1,400	2,100	2,500	3,400
	33,750	30	500	1,400	2,300	3,100	3,800	5,700
		60	700	1,500	2,500	3,400	4,300	6,000
		90	700	1,500	2,500	3,500	4,300	6,100
	41,250	30	900	2,200	3,600	5,100	6,600	9,600
		60	1,200	2,500	3,900	5,400	7,100	10,500
		90	1,200	2,500	3,900	5,600	7,000	10,400
	48,750	30	1,400	3,400	5,500	8,200	11,000	15,600
		60	1,800	3,700	5,800	8,200	11,100	18,000
		90	1,800	3,800	5,900	8,500	10,900	17,400
	56,250	30	2,100	4,900	8,000	12,700	cong	cong
		60	2,500	5,300	8,400	12,100	16,900	cong
		90	2,600	5,400	8,600	12,500	16,700	28,400
	63,750	30	2,900	6,900	11,600	cong	cong	cong
		60	3,400	7,400	11,900	17,600	cong	cong
		90	3,500	7,500	12,200	18,000	24,900	cong

Notes:

1 - Access point density represents the total number of access points on both sides of the street segment (i.e., a two-way total) divided by the length of the segment (in miles).

2 - Total number of left-turns per hour exiting the major street into an access point in one direction of travel per 1,320-ft length of roadway divided by the total flow rate in that direction (expressed as a percentage).

"cong" = Delays to one or more major-street left-turn movements are in excess of 40 s/v/a leading to congested flow conditions, queue spillback, and possible gridlock.

ring on the driveway or unsignalized public street approaches to the arterial, nor does it account for accidents that occur on adjacent streets involving vehicles that are rerouted because of the arterial's raised-curb median treatment. These accidents are not included primarily because of the significant increase in effort required to assemble such a comprehensive database relative to the marginal effect these accidents would have (in terms of their number and severity) on the determination of relative safety differences among left-turn treatments.

TABLE 2-15 Annual delay to major-street left-turn and through vehicles for the undivided cross section (hr/yr)

Through Lanes	ADT	Access Pt. Density[1] (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length[2]					
			0	5	10	15	20	30
4	17,500	30	300	500	1,000	1,400	1,600	2,300
		60	300	500	1,000	1,400	1,700	2,400
		90	300	500	1,000	1,400	1,700	2,400
	22,500	30	500	1,200	2,200	2,900	3,300	4,700
		60	500	1,200	2,200	3,000	3,500	4,800
		90	500	1,200	2,200	3,000	3,700	5,100
	27,500	30	800	2,300	4,100	5,300	6,100	8,200
		60	800	2,400	4,300	5,700	6,700	8,900
		90	800	2,400	4,400	5,900	7,200	9,700
	32,500	30	1,200	4,200	7,100	9,100	10,600	13,300
		60	1,200	4,400	7,800	10,200	12,000	15,400
		90	1,200	4,500	8,000	10,800	13,100	17,100
	37,500	30	1,600	7,300	11,600	14,800	17,500	20,900
		60	1,700	7,700	13,100	17,100	20,200	25,200
		90	1,800	7,800	13,700	18,500	22,200	28,400
	42,500	30	2,200	11,700	18,100	23,000	27,800	cong
		60	2,400	12,700	21,000	27,100	32,200	39,800
		90	2,500	12,900	22,100	30,000	35,900	45,200
6	26,250	30	300	1,000	2,200	2,800	3,500	3,900
		60	400	1,100	2,300	3,400	4,400	5,500
		90	400	1,100	2,300	3,400	4,700	6,600
	33,750	30	500	2,300	4,000	5,000	6,000	7,700
		60	700	2,500	4,400	6,000	7,400	9,200
		90	700	2,500	4,600	6,200	8,100	10,800
	41,250	30	900	4,500	6,500	8,400	9,800	14,600
		60	1,200	4,800	7,700	9,600	11,700	14,900
		90	1,200	5,100	8,500	10,600	13,000	16,900
	48,750	30	1,400	7,600	10,100	13,600	cong	cong
		60	1,800	8,800	12,500	14,700	17,800	cong
		90	1,800	9,400	14,500	17,000	19,700	25,800
	56,250	30	2,100	12,100	15,000	cong	cong	cong
		60	2,500	15,000	19,300	21,700	26,500	cong
		90	2,600	16,400	23,400	25,800	28,700	38,800
	63,750	30	2,900	18,300	cong	cong	cong	cong
		60	3,400	24,300	28,600	31,300	cong	cong
		90	3,500	27,000	36,000	37,800	41,100	cong

Notes:

1 - Access point density represents the total number of access points on both sides of the street segment (i.e., a two-way total) divided by the length of the segment (in miles).

2 - Total number of left-turns per hour exiting the major street into an access point in one direction of travel per 1,320-ft length of roadway divided by the total flow rate in that direction (expressed as a percentage).

"cong" = Delays to one or more major-street left-turn movements are in excess of 40 s/v/a leading to congested flow conditions, queue spillback, and possible gridlock.

Procedure

The effect of alternative midblock left-turn treatments on motorist safety was determined using the safety model. This model was used to predict the average annual accident frequency on a typical arterial segment for a wide range of traffic demands, geometric conditions, and adjacent land uses. Traffic demand is represented by the average daily traffic on the arterial segment. Geometric conditions include the provision of parallel parking and the density of access points along the major street. Finally, land use includes the residential, office, business, and industrial categories.

It should be noted that statistical analysis of the accident data indicated that there is no significant difference between the effect of a driveway and an unsignalized public street approach on accident frequency. Hence, there is no distinction made in this analysis between a driveway and an unsignalized public street approach; the term "access point" is used hereafter to denote either type of access.

Analysis Scenarios

The combinations of traffic demand and geometry considered for this safety evaluation were selected to be consistent with the idealized street segment configured for the operational effects evaluation (described in the preceding section). The one difference between the street segments analyzed in the operational and safety evaluations is their access point density. The density used in the safety evaluation is based on the total number of access points on both sides of the major-street segment. In contrast, only active access points were considered in the operational effects evaluation. To facilitate later aggregation of the operational and safety measures, an assumption was made regarding the relationship between total and active access points. Specifically, it was assumed that total access point densities of 40, 65, and 90 approaches (ap)/mi corresponded with the 30, 60, and 90 ap/mi scenarios, respectively, previously developed for the operational evaluation.

Table 2-16 indicates the specific variable combinations considered in the safety evaluation. The variable values included in this table represent 480 unique combinations considered for this evaluation.

Analysis Results

The accident frequencies predicted by the safety model for the range of variable values listed in Table 2-16 are provided in Tables 2-17, 2-18, and 2-19. The evaluation of other values should be performed by means of a direct application of the safety model, as described in Chapter 4.

The property-damage-only (PDO) accident percentage is included in these tables to facilitate their calibration to local conditions. This percentage represents the ratio of PDO accidents to all reported accidents. As such, it is a direct measure of the accident cost reporting threshold and the degree of compliance with accident reporting requirements in a given area. Areas with higher reporting thresholds or a larger number of unreported accidents are associated with lower PDO percentages.

The trend of a higher number of annual accidents associated with a higher PDO percentage indicated in Tables 2-17, 2-18, and 2-19 does not mean that more accidents occur in areas with higher PDO percentages; rather, it means that more accidents are *being reported* in these areas. The technique of including PDO percentage in the tables also facilitates a comparison of the relative safety of arterial streets in different cities and states through the use of a common PDO percentage.

Not all levels of each variable listed in Table 2-16 were evaluated for this analysis. Specifically, the effect of parking activity was not evaluated for the raised-curb and TWLTL treatments. This action resulted from the lack of street segments in the database that have one of these treatment types and parallel parking. In contrast, street segments with an undivided cross section, both with and without parallel parking, are included in the database. As such, the safety model has the ability to account for the effect of parking on streets with an undivided cross section. As shown in Table 2-19, parallel parking tends to increase accident frequency by about 80 to 90 percent.

Another variable listed in Table 2-16 that was not evaluated for all conditions is access point density. Specifically, it was found that access point density did not have a significant effect on accident frequency in the Residential/Industrial land use category. This finding does not necessarily mean that access point density does not have an effect on accident frequency; rather, it could mean that the effect of access point density is correlated with other model variables (e.g., daily traffic demand). In other words, when two variables are correlated and one is already included in the model, the other

TABLE 2-16 Range of evaluation variables for the safety evaluation

Variable	Range or Levels
Midblock Left-Turn Treatments	Raised-Curb, TWLTL, Undivided cross section
Land Use	Business/Office or Residential/Industrial
ADT (in 5,000 increments), vpd	17,500 to 62,500
Access Point Density[1] (both directions), access points / mile	40, 65, 90
On-Street Parallel Parking (undivided only)	Yes, No
Property-Damage-Only Accident Percentage[2]	55, 65, 75

Notes:

1 - Access point density represents the total number of access points on both sides of the major-street segment (i.e., a two-way total) divided by the length of the segment (in miles).

2 - Property-damage-only accident percentage is a surrogate measure of differences in accident reporting level among different cities and states. Lower percentages reflect locations with a higher accident cost reporting threshold or a higher number of unreported accidents.

TABLE 2-17 Annual accident frequency for the raised-curb median treatment (accidents/yr)

Land Use	ADT	Access Pt. Density[1] (ap/mi)	Property-Damage-Only Accident Percentage[2]		
			55	65	75
			No Parallel Parking		
Business or Office	17,500	40	3	4	5
		65	4	5	6
		90	4	5	7
	22,500	40	4	5	7
		65	4	6	7
		90	5	6	8
	27,500	40	5	6	8
		65	5	7	9
		90	6	8	10
	32,500	40	6	7	9
		65	6	8	10
		90	7	9	12
	37,500	40	6	8	10
		65	7	9	12
		90	8	10	13
	42,500	40	7	9	12
		65	8	10	13
		90	9	12	15
	47,500	40	8	10	13
		65	9	11	15
		90	10	13	17
	52,500	40	9	11	14
		65	10	12	16
		90	11	14	18
	57,500	40	9	12	15
		65	10	14	17
		90	12	15	20
	62,500	40	10	13	17
		65	11	15	19
		90	13	16	21
Residential or Industrial	17,500	< 100	2	2	3
	22,500	< 100	2	3	4
	27,500	< 100	3	4	5
	32,500	< 100	3	4	6
	37,500	< 100	4	5	6
	42,500	< 100	4	6	7
	47,500	< 100	5	6	8
	52,500	< 100	5	7	9
	57,500	< 100	6	7	9
	62,500	< 100	6	8	10

Notes:
1 - Access point density represents the total number of access points on both sides of the major-street segment (i.e., a two-way total) divided by the length of the segment (in miles).
2 - Number of property-damage-only accidents divided by the number of reported accidents for the region in which subject street segment is located (expressed as a percentage).
Shaded areas denote traffic volume levels that exceed the range of the database used to calibrate the safety model.

variable may not incrementally add a significant amount to the predictive ability of the model and, hence, it can be eliminated from the model without any loss in model performance.

An examination of the accident frequencies listed in Tables 2-17, 2-18, and 2-19 indicates some interesting trends. One trend that is consistent among all three treatments is the increase in accidents with increasing daily traffic demand and access point density. More important, it appears that the raised-curb median treatment is associated with the fewest accidents of the three treatment types.

TABLE 2-18 Annual accident frequency for the TWLTL treatment (accidents/yr)

Land Use	ADT	Access Pt. Density[1] (ap/mi)	Property-Damage-Only Accident Percentage[2]		
			55	65	75
			No Parallel Parking		
Business or Office	17,500	40	4	6	7
		65	5	6	8
		90	5	7	9
	22,500	40	5	7	9
		65	6	8	10
		90	7	9	11
	27,500	40	7	8	11
		65	7	9	12
		90	8	11	14
	32,500	40	8	10	13
		65	9	11	14
		90	10	12	16
	37,500	40	9	11	14
		65	10	13	16
		90	11	14	18
	42,500	40	10	12	16
		65	11	14	18
		90	12	16	20
	47,500	40	11	14	18
		65	12	16	20
		90	14	18	23
	52,500	40	12	15	20
		65	13	17	22
		90	15	19	25
	57,500	40	13	16	21
		65	14	19	24
		90	16	21	27
	62,500	40	14	18	23
		65	15	20	26
		90	17	23	29
Residential or Industrial	17,500	< 100	3	4	5
	22,500	< 100	4	5	7
	27,500	< 100	5	6	8
	32,500	< 100	6	7	9
	37,500	< 100	6	8	11
	42,500	< 100	7	9	12
	47,500	< 100	8	10	13
	52,500	< 100	9	11	14
	57,500	< 100	9	12	16
	62,500	< 100	10	13	17

Notes:
1 - Access point density represents the total number of access points on both sides of the major-street segment (i.e., a two-way total) divided by the length of the segment (in miles).
2 - Number of property-damage-only accidents divided by the number of reported accidents for the region in which subject street segment is located (expressed as a percentage).
Shaded areas denote variable combinations that exceed the range of the database used to calibrate the safety model.

Differences between the TWLTL treatment and undivided cross section, in terms of accident frequency, are not as distinct and must be discussed in the context of whether parallel parking is provided on the undivided cross section. When parallel parking is allowed on the undivided cross section, the undivided cross section is associated with significantly more accidents than the TWLTL treatment.

When parallel parking is not allowed, the TWLTL generally has about the same accident frequency as the undivided cross section at lower traffic volumes. At the higher

TABLE 2-19 Annual accident frequency for the undivided cross section (accidents/yr)

Land Use	ADT	Access Pt. Density[1] (ap/mi)	Property-Damage-Only Accident Percentage[2]					
			55	65	75	55	65	75
			No Parallel Parking			With Parallel Parking		
Business or Office	17,500	40	4	5	7	7	10	12
		65	5	6	8	8	11	14
		90	5	7	9	10	12	16
	22,500	40	5	7	9	9	12	16
		65	6	8	10	11	14	18
		90	7	9	11	12	15	20
	27,500	40	6	8	11	11	15	19
		65	7	9	12	13	16	21
		90	8	10	14	14	19	24
	32,500	40	7	10	12	13	17	22
		65	8	11	14	15	19	25
		90	9	12	16	17	22	28
	37,500	40	8	11	14	15	19	25
		65	10	12	16	17	22	28
		90	11	14	18	19	25	32
	42,500	40	9	12	16	17	22	28
		65	11	14	18	19	24	32
		90	12	16	20	21	28	36
	47,500	40	11	14	18	19	24	31
		65	12	15	20	21	27	35
		90	13	17	22	24	30	39
	52,500	40	12	15	19	20	26	34
		65	13	17	22	23	30	38
		90	15	19	24	26	33	43
	57,500	40	13	16	21	22	29	37
		65	14	18	23	25	32	42
		90	16	20	26	28	36	47
	62,500	40	13	17	22	24	31	40
		65	15	20	25	27	35	45
		90	17	22	29	30	39	50
Residential or Industrial	17,500	< 100	2	3	3	4	5	6
	22,500	< 100	3	4	6	6	8	10
	27,500	< 100	5	6	8	9	11	14
	32,500	< 100	7	9	11	12	15	20
	37,500	< 100	9	12	15	16	20	26
	42,500	< 100	11	15	19	20	26	33
	47,500	< 100	14	18	23	25	32	41
	52,500	< 100	17	22	28	30	39	50
	57,500	< 100	20	26	34	36	46	60
	62,500	< 100	24	31	40	42	55	70

Notes:

1 - Access point density represents the total number of access points on both sides of the major-street segment (i.e., a two-way total) divided by the length of the segment (in miles).

2 - Number of property-damage-only accidents divided by the number of reported accidents for the region in which subject street segment is located (expressed as a percentage)

Shaded areas denote variable combinations that exceed the range of the database used to calibrate the safety model.

volume levels in the Residential or Industrial land use category, the undivided cross section logically tends to be associated with more accidents than the TWLTL. This trend is not apparent with the Business or Office category and may be the result of the more uniform distribution of access point traffic activity throughout the day (as opposed to the residential and industrial land uses, in which trips probably are more concentrated during the morning and evening peak hours). This lack of concentrated access point activity is likely to yield shorter periods of inside through lane blockage and thus fewer accidents, relative to the Residential or Industrial land use.

Another trend that can be observed in the accident frequencies in these tables is that the Business or Office land use for the raised-curb and TWLTL treatments is associated with a higher accident frequency than the Residential or Industrial land use. This trend is consistent with the higher level of access point traffic activity generated by the Business or Office land use. This trend is also true for the undivided cross section with parallel parking activity and for the lower-to-moderate volume levels of the undivided cross section without parallel parking activity. The trend is not adhered to for the higher volume levels of the undivided-without-parking scenario; however, the predicted accident frequencies for these traffic levels exceed the range of values included in the accident database and illustrate the problems associated with extrapolation.

GUIDELINES FOR SELECTING A MIDBLOCK LEFT-TURN TREATMENT

Approach

This section describes the development of guidelines for selecting a midblock left-turn treatment for an urban or suburban arterial. The performance measures described in the preceding sections were used to develop these guidelines based on a cost-benefit analysis approach. First, the annual delay and accident measures were converted into road user costs and then into road user benefits. Next, the road user benefits for all reasonable pairwise combinations of midblock treatments were tabulated and compared with the estimated reconstruction costs. Finally, the cost-effectiveness of each treatment pair was used to identify traffic volume, land use, and access point density combinations in which the conversion from one treatment to another was clearly cost-effective. The graphic representation of these combinations forms the basis for the guidelines.

Economic Analysis

Operational Benefits

As described previously, the operations model was used to quantify the operational performance of three midblock left-turn treatments in terms of annual delay. The conversion of these delays into road user costs is described in this section. This conversion is based on the cost of a delayed motorist's time and fuel consumption during idling. The difference between the delay costs of two alternative left-turn treatments translates into a road user benefit if the treatment with the lower cost is implemented. In other words, the road user benefit is the reduction in delay costs when one treatment is used instead or in place of another.

Delay was converted into a road user cost based on an estimate of the value of a motorist's time and the cost of fuel. McFarland and Chui (*6*) estimated the value of time as $8.03 per person-hour in 1985 dollars. This value was updated to $15.24 per hour in 1996 dollars by using the consumer price index and allowing for 1.3 persons per vehicle. An additional $0.76 per hour was added to account for the cost of fuel consumption during idling. Thus, the total road user cost resulting from delay was estimated as $16.00 per hour. In the context of road user benefit, a delay reduction of 10,000 hours per year resulting from a conversion from one treatment type to another translates into a benefit of about $160,000 per year.

Safety Benefits

As also described previously, the safety model was used to quantify the safety of three midblock left-turn treatments in terms of annual accident frequency. The conversion of these accidents into road user costs is described in this section. This conversion is based on the average cost of a traffic accident. The difference between the accident costs of two alternative left-turn treatments translates into a road user benefit if the treatment with the lower cost is implemented. In other words, the road user benefit is the reduction in accident costs when one treatment is used instead or in place of another. The average cost of a traffic accident was based on the distribution of accidents by severity and the average cost of each severity class (i.e., fatality, injury, and PDO). A document from the Federal Highway Administration (*7*) indicates that the average costs of a fatal, injury, and PDO accident are $1,700,000, $11,000, and $3,000, respectively, in 1986 dollars. The distribution of accidents by severity was obtained from Table 4-9 in Chapter 4. Based on these two sources and the consumer price index, the average accident cost was estimated as $15,000 in 1996 dollars. Thus, in the context of road user benefit, an average annual accident reduction of two accidents resulting from a conversion from one treatment type to another translates into a road user benefit of about $30,000 per year.

Roadway Reconstruction Costs

For an alternative left-turn treatment to be financially feasible, the road user benefits must exceed the cost of reconstruction or conversion. The reconstruction cost considered in this analysis includes only those cost elements directly related to the left-turn treatment. When converting from an undivided cross section, it was assumed that the reconstruction was relatively major in scope and was part of a larger project involving the entire street. As a result, the incremental cost of converting to a raised-curb median or TWLTL was estimated as the cost of constructing one new traffic lane. This incremental cost included the cost of a right-of-way for an urban area. When converting from a TWLTL to a raised-curb median or vice versa, it was assumed that the reconstruction was relatively minor in scope and required some

renovation and resurfacing. The cost for this conversion was not assumed to include the cost of a new right-of-way.

Estimates of reconstruction costs vary widely from region to region because of variations in the cost of living, construction materials, and the like, making it difficult to determine an overall average cost of reconstruction. In recognition of this wide variability, it was determined that the guidelines would be based on the provision of a minimum cost-benefit ratio of 2.0. In this context, traffic volume and access point density conditions yielding road user benefits in excess of this ratio would likely be cost-effective even if some of the benefits or costs were over- or underestimated for a specific area. Therefore, when the cost-benefit ratio of a proposed conversion exceeds 2.0, the alternative left-turn treatment is recommended in the guidelines.

In a similar manner, conditions yielding a cost-benefit ratio less than 1.0 are not likely to be cost-effective for any reasonable over- or underestimate of benefits and costs. The recommendation in this case would be to stay with the existing left-turn treatment. Conditions yielding benefits that fall in between the cost-benefit ratio limits of 1.0 and 2.0 would require a more detailed, site-specific evaluation to determine the feasibility of upgrading to a different left-turn treatment.

The construction costs used for this analysis are based on those presented in Table 3-2 of Chapter 3. These costs were amortized over a 20-year design life using a conservative rate of return of 4 percent per year. Table 2-20 shows the range of costs for four typical left-turn treatment conversion combinations.

Guideline Tables

Computations

Prior to the creation of the guidelines, a table of performance indices was created for each midblock left-turn treatment. The performance index is defined as an economic indicator of the road user costs associated with a specific combination of traffic volume, traffic lanes, access point density, and land use. This performance index was computed using the following equation:

$$PI = 16D + 15{,}000A \tag{11}$$

where:

PI = performance index (i.e., road user cost), \$/quarter-mile/yr
D = annual hours of delay, hr/quarter-mile/yr
A = annual accidents for 65 percent PDO and no parallel parking, accidents/quarter-mile/yr.

The constants used in Equation 11 represent the road user costs discussed in the preceding section.

Equation 11 was used with the annual delays reported in Tables 2-13, 2-14, and 2-15 and the annual accident frequencies reported in Tables 2-17, 2-18, and 2-19 to compute the performance indices for each midblock left-turn treatment. The performance indices were then subtracted from one another for the four conversion combinations shown in Table 2-20 to obtain the corresponding road user benefit. This benefit would be realized if the midblock left-turn treatment associated with the lower performance index was implemented instead of the treatment associated with the higher index.

Because the nature of the operations and safety models dictated their sensitivity to different traffic, geometric, and land use variables, it was necessary to consolidate or eliminate some less influential variables in the safety model (i.e., parking and PDO percentage) to maintain a reasonable balance between simplicity and accuracy. The accident frequencies used to compute the performance index are representative of a city or region with a PDO percentage of 65 percent and no parallel parking. The PDO percentage was used as a representative value based on the PDO percentages reported in Table 4-9 in Chapter 4.

Finally, the road user benefit for a range of lanes, volumes, densities, land uses, and turn percentages were compared with the reconstruction cost ranges listed in Table 2-20. The results were tabulated and examined to identify traffic volume and geometric conditions in which conversion to another left-turn treatment was clearly cost-effective. The results of this examination are shown in Tables 2-21 through 2-26 for the range of conditions considered. The access point density referred to in these tables relates to active access points (i.e., those with an entering volume of 10 vph or more).

There are no tables provided for the conversion from a raised-curb median to a TWLTL because the benefits associ-

TABLE 2-20 Range of reconstruction costs for midblock left-turn treatments (1996 dollars)

Reconstruction (or Conversion) Combination	Lower Boundary (\$/quarter-mile/year)	Upper Boundary (\$/quarter-mile/year)
Undivided to Raised-Curb Median	27,000	54,000
Undivided to TWLTL	23,000	46,000
Raised-Curb Median to TWLTL	14,000	28,000
TWLTL to Raised-Curb Median	18,000	36,000

TABLE 2-21 Conversion from an undivided cross section to a raised-curb median (business and office land use)

Through Lanes	ADT	Access Pt. Density (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length					
			0	5	10	15	20	30
4	17,500	30	Stay with existing undivided					
		60	cross section					
		90						
	22,500	30		Site-specific examination required				
		60						
		90						
	27,500	30						
		60						
		90						
	32,500	30						
		60						
		90						
	37,500	30		Consider adding a raised-curb median				
		60						
		90						
	42,500	30						
		60						
		90						
6	26,250	30	Site-specific examination required					
		60						
		90						
	33,750	30						
		60						
		90						
	41,250	30						
		60			Consider adding a raised-curb median			
		90						
	48,750	30						
		60						
		90						
	56,250	30						
		60						
		90						
	63,750	30						
		60						
		90						

Note: Hatching denotes volume levels that may be associated with congested flow conditions.

ated with the TWLTL were not found to be large enough to offset the cost of removing the raised-curb median and installing the TWLTL, regardless of geometric configuration or traffic demand condition. In general, the TWLTL was found to offer improved operational performance relative to the raised-curb median; however, it was also found to be generally associated with a higher number of accidents. It is possible that the improvement in property access resulting from a conversion to TWLTL may provide the necessary justification for its construction (a tool for quantifying access impact is described in Chapter 5). However, the benefits associated with improved access are very difficult to quantify in a cost-benefit context and, as a result, have not been incorporated in the guidelines.

For the undivided cross section, the guidelines recommend that the analyst stay with this treatment for the lower volume

TABLE 2-22 Conversion from an undivided cross section to a raised-curb median (residential and industrial land use)

Through Lanes	ADT	Access Pt. Density (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length					
			0	5	10	15	20	30
4	17,500	30						
		60		Stay with existing undivided cross section				
		90						
	22,500	30						
		60			Site-specific examination required			
		90						
	27,500	30						
		60						
		90						
	32,500	30						
		60						
		90						
	37,500	30		Consider adding a raised-curb median				
		60						
		90						
	42,500	30						
		60						
		90						
6	26,250	30						
		60	Site-specific examination reqd.					
		90						
	33,750	30						
		60						
		90						
	41,250	30		Consider adding a raised-curb median				
		60						
		90						
	48,750	30						
		60						
		90						
	56,250	30						
		60						
		90						
	63,750	30						
		60						
		90						

Note: Hatching denotes volume levels that may be associated with congested flow conditions.

conditions. Because the guidelines are based on the accidents and delays to arterial traffic movements, they do not consider the delays to left-turn vehicles entering the arterial at unsignalized access points. It is recognized that the individual delays to these entering left-turn drivers may be high and that the raised-curb median or TWLTL may offer some relief through the two-stage crossing maneuver. However, regardless of the number of stages in the crossing maneuver, the total person-hours of delay to this movement is believed to be a small fraction of that incurred by the arterial movements. Hence, it is believed that consideration of the delays to these entering movements would not significantly alter the guidelines as shown.

Application of Guidelines

When considering a conversion in midblock left-turn treatment, select the table that corresponds to the existing

TABLE 2-23 Conversion from an undivided cross section to a TWLTL (business and office land use)

Through Lanes	ADT	Access Pt. Density (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length					
			0	5	10	15	20	30
4	17,500	30						
		60						
		90						
	22,500	30		Stay with existing undivided cross section				
		60						
		90						
	27,500	30			Site-specific examination reqd.			
		60						
		90						
	32,500	30						
		60						
		90						
	37,500	30						
		60			Consider adding a TWLTL			
		90						
	42,500	30						
		60						
		90						
6	26,250	30						
		60	Stay with existing undivided cross section					
		90						
	33,750	30			Site-specific examination required			
		60						
		90						
	41,250	30						
		60			Consider adding a TWLTL			
		90						
	48,750	30						
		60						
		90						
	56,250	30						
		60						
		90						
	63,750	30						
		60						
		90						

Note: Hatching denotes volume levels that may be associated with congested flow conditions.

and alternative treatments. Within this table, locate the cell that most closely matches the geometric configuration and traffic demand conditions that will exist for the design year. If the cell is white, the estimated cost-benefit ratio is either less than 1.0 or greater than 2.0 and a definitive recommendation can be made. The nature of this recommendation is specified in the table in the immediate vicinity of the other white cells. The two recommendations are "Stay with the existing treatment" when the cost-benefit ratio is less than 1.0 and "Consider adding the alternative treatment" when the cost-benefit ratio is greater than 2.0.

If the cell is light gray, the estimated costs are nearly equal to the benefits and a site-specific examination will be needed to ascertain the actual cost-effectiveness of the proposed conversion. The "hatched" cells indicate

TABLE 2-24 Conversion from an undivided cross section to a TWLTL (residential and industrial land use)

Through Lanes	ADT	Access Pt. Density (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length					
			0	5	10	15	20	30
4	17,500	30						
		60						
		90						
	22,500	30		Stay with existing undivided cross section				
		60						
		90						
	27,500	30			Site-specific exam.			
		60			required			
		90						
	32,500	30						
		60						
		90						
	37,500	30			Consider adding a TWLTL			
		60						
		90						
	42,500	30						
		60						
		90						
6	26,250	30					Site-specific exam.	
		60	Stay with existing undivided cross section				required	
		90						
	33,750	30						
		60						
		90						
	41,250	30						
		60		Consider adding a TWLTL				
		90						
	48,750	30						
		60						
		90						
	56,250	30						
		60						
		90						
	63,750	30						
		60						
		90						

Note: Hatching denotes volume levels that may be associated with congested flow conditions.

areas where one or both of the treatments are likely to experience congestion. In this situation, the road user benefits are highly variable and strongly dependent on the duration of congestion. As with the light gray cells, a site-specific examination is recommended for congested conditions.

Finally, it should be noted that these guidelines are based on certain assumptions that may not be representative of a specific arterial or urban area. In this regard, the guidelines are most applicable to areas where the percentage of PDO accidents is in the range of 60 to 75 percent, signal spacings are 1,000 ft or more, signals are coordinated, there is no parallel parking, the arterial has four or six through lanes, the access points are aligned to form four-leg intersections, and there are no exclusive right-turn bays. The guidelines can be used outside these ranges but

TABLE 2-25 Conversion from a TWLTL to a raised-curb median (business and office land use)

Through Lanes	ADT	Access Pt. Density (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length					
			0	5	10	15	20	30
4	17,500	30						
		60						
		90		Site-specific examination required				
	22,500	30						
		60						
		90						
	27,500	30						
		60						
		90						
	32,500	30						
		60						
		90	Consider adding a raised-curb median					
	37,500	30						
		60						SWET
		90						
	42,500	30						
		60						SWET
		90						
6	26,250	30						
		60						
		90						
	33,750	30						
		60						
		90		Consider adding a raised-curb median				
	41,250	30						
		60						
		90						
	48,750	30						
		60						
		90						
	56,250	30						
		60						
		90						
	63,750	30						
		60						
		90					SWET	

Note: Hatching denotes volume levels that may be associated with congested flow conditions.
SWET = Stay with existing TWLTL.

become more questionable as the amount of deviation increases.

In general, average or typical values were used whenever possible to provide guidelines that are applicable to the most commonly found conditions. However, if the road user costs for a specific location are significantly different from those previously assumed, Tables 2-13, 2-14, and 2-15 (or Equations 2-3 through 2-10) should be used to compute the operational effects of the existing and proposed left-turn treatments. Similarly, Tables 2-17, 2-18, and 2-19 (or Equations 21, 22, and 23 in Chapter 4) should be used to compute the safety effects of these left-turn treatments. In any case, the access impact effect should be assessed using the access impact model described in Chapter 5.

TABLE 2-26 Conversion from a TWLTL to a raised-curb median (residential and industrial land use)

Through Lanes	ADT	Access Pt. Density (ap/mi)	Left-Turn Percent per 1,320-ft Segment Length					
			0	5	10	15	20	30
4	17,500	30						
		60						
		90		Site-specific examination required				
	22,500	30						
		60						
		90						
	27,500	30						
		60						
		90						
	32,500	30	Consider adding a raised-curb median					
		60						
		90						
	37,500	30						
		60						Stay with TWLTL
		90						
	42,500	30						
		60					Stay with existing TWLTL	
		90						
6	26,250	30						
		60						
		90						
	33,750	30						
		60		Consider adding a raised-curb median				
		90						
	41,250	30						
		60						
		90						
	48,750	30						
		60						
		90						
	56,250	30						
		60						
		90				Site-specific examination reqd.		
	63,750	30						
		60				Stay with existing TWLTL		
		90						

Note: Hatching denotes volume levels that may be associated with congested flow conditions.

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CHAPTER 3

OPERATIONAL EFFECTS OF MIDBLOCK LEFT-TURN TREATMENTS

This chapter describes the development of a model for predicting the operational effects of midblock left-turn treatments. In this context, a treatment's operational effects are defined as the delays to drivers traveling along an arterial street segment, which may be directly or indirectly caused by the left-turn treatment. The treatments considered are raised-curb median, two-way left-turn lane (TWLTL), and undivided cross section. The following sections describe a review of the literature on the operational effects of these treatments, a database assembled for calibrating the operations model, and the formulation of and statistical foundation for this model.

LITERATURE REVIEW

The literature review and survey of practitioners indicated that the four most common types of midblock left-turn treatments on urban and suburban arterials are (1) flush median with TWLTL delineation, (2) raised-curb median with alternating left-turn bays, (3) flush median with alternating left-turn bays, and (4) undivided cross section. These treatments are shown in Figure 3-1.

Treatments other than those listed here are used by some agencies; however, the choice depends on geographic location, topography, and agency preference. These other, less-frequently-used left-turn treatments include flush median with continuous parallel left-turn lanes, raised-curb median with acceleration lanes, mountable median with TWLTL delineation, and median channelization for U-turn or for right-turn access only.

Several terms are used in the literature to describe a location of unsignalized access to a major street, including driveway, access point, unsignalized intersection approach, and public street approach. All unsignalized access locations are defined as "access points." An access point can be either a driveway or a public street approach. A driveway is any location on the arterial where the curb along the outside lane is removed or dropped for 10 ft or more to facilitate vehicular access to the adjacent property. "Access point density" is defined as the total number of access points on both sides of the major-street segment (i.e., a two-way total) divided by the length of the segment (in miles). Driveway density and public street approach density are defined in a similar manner.

At this point it is useful to clarify the meaning of other terms found in the literature because they are used in this report. It is assumed that the major street is functionally classified as an arterial and the minor street is classified as a collector, local street, or driveway. Hence, the major street is also referred to as an "arterial" in this report. The through traffic movements on this arterial are referred to as "priority" movements; all other driveway-related movements are "non-priority" movements.

The aforementioned definitions are used throughout the literature dealing with the safety and operational effects of midblock left-turn treatments; however, they are less rigidly applied in the operational effects literature. Specifically, a distinction between access points and driveways is not made in this literature. Moreover, all driveways are generally assumed to have some minimum traffic volume (i.e., they are assumed to be "active"). In this sense, it is assumed that a driveway with negligible volume has negligible effect on traffic flow. Throughout this chapter, the term "access point" is used instead of driveway or public street approach; however, any such reference infers that the access point is active (i.e., it has an entering volume of 10 vph or more).

Operational Effects of Midblock Left-Turn Treatments

Major Street Through Movement Speed and Capacity

The 1994 *Highway Capacity Manual* (HCM) (*1*) includes procedures for determining the capacity and level of service of urban and suburban arterials. These procedures, which are presented in Chapters 7 and 11, respectively, are sensitive to the type of left-turn treatment and the density of access points; however, the degree of sensitivity is not the same. The procedure in Chapter 11 accounts for treatment type and access point density in an indirect manner by including these factors with other general arterial descriptors in a table of attributes for three basic arterial classifications. This table identifies applicable free-flow speeds (i.e., speeds at zero flow) for each arterial class but does not give an indication of the arterial class's capacity. The table indicates that a suburban arterial with a low access point density and a divided cross section has a free-flow speed of 35 to 45 mph, whereas

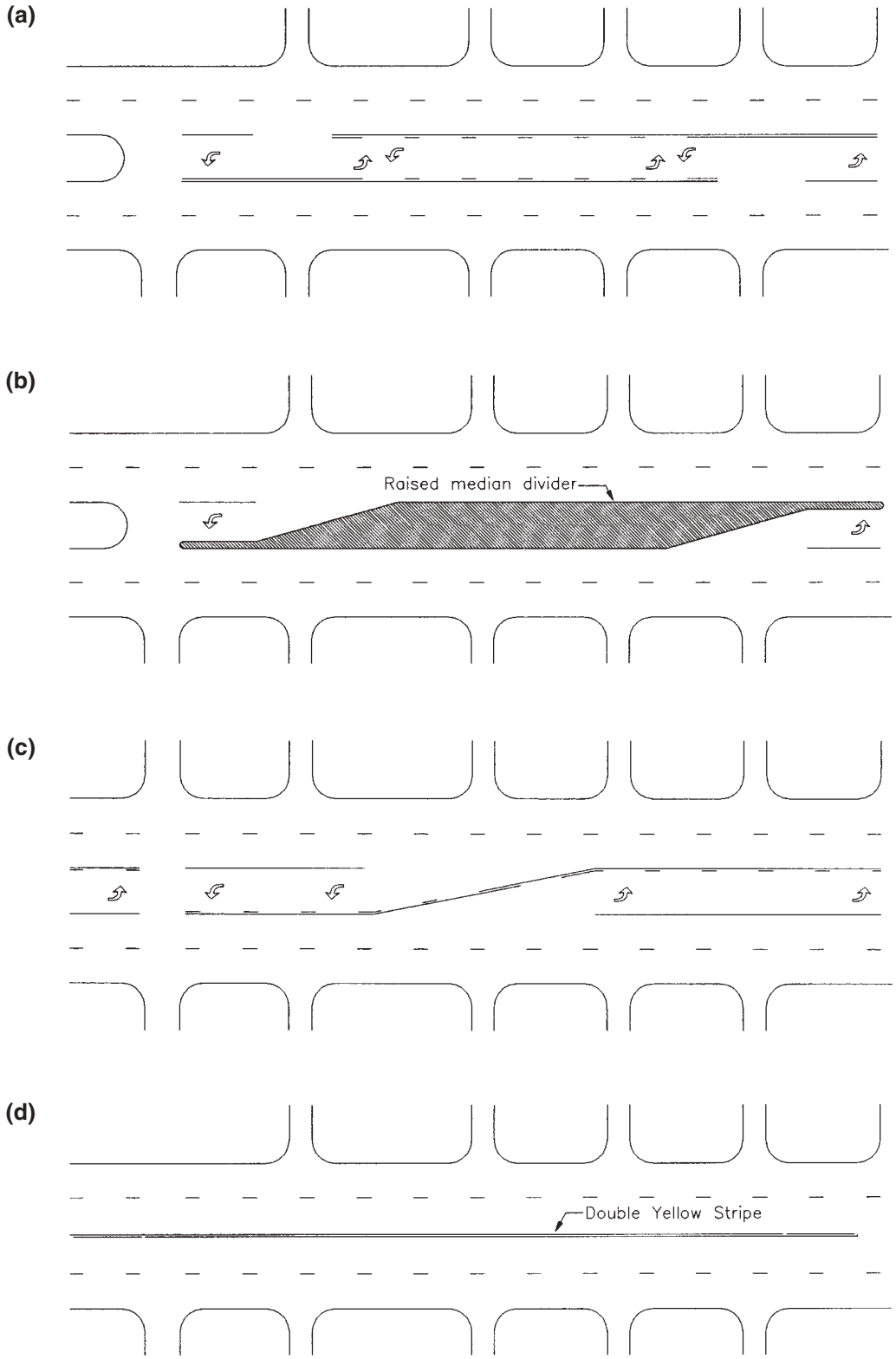

Figure 3-1. Common midblock left-turn treatments: (from top) flush median with two-way left-turn lane (TWLTL) delineation; raised-curb median with alternating left-turn bays (nontraversable median area); flush median with alternating left-turn bays (traversable median area); undivided cross section (i.e., no median).

an urban arterial with high access point density and undivided cross section has a free-flow speed of 25 mph to 35 mph.

The procedure described in Chapter 7 of the HCM relates arterial flow rate with free-flow speed to estimate the arterial running speed, capacity, and level of service. If free-flow speed is unknown, procedures are provided for estimating it based on median type, lane width, and access point density. In general, the effects of each factor are accounted for as a reduction in speed from the ideal free-flow speed. The "median type" factor indicates that an undivided cross section reduces the free-flow speed by 1.6 mph; however, it does not distinguish between a TWLTL and a raised-curb median. It should be noted that this procedure is intended for rural and suburban highways with four or six through lanes, speed limits between 40 and 55 mph, and signal spacing greater than 2 mi. Hence, it is not directly applicable to most urban arterials.

The relationships among arterial flow rate, running speed, and capacity, as defined in Chapter 7 of the HCM, is based on the speed-flow curves shown in Figure 3-2. These curves show the speed-flow relationships for multilane highways with free-flow speeds between 40 and 60 mph. According to these curves, the maximum service flow rates for level of service E are 2,200, 2,100, 2,000, and 1,900 pcphpl for free-flow speeds of 60, 55, 50, and 45 mph, respectively.

The effect of access point density (a one-way total in this case) is accounted for by reducing the free-flow speed by 0.25 mph per access point per mile. Thus, the free-flow speed is reduced by 2.5 mph if the density is 10 access points per mile. The combined speed reductions for the effects of median type and access point density are listed in Table 3-1.

As mentioned previously, the HCM Chapter 7 procedure links the capacity of the arterial to its free-flow speed. Thus, any reduction in free-flow speed correlates with a reduction in capacity. The magnitude of this reduction is shown in Table 3-1 in terms of a factor, f_c, that would be multiplied by the maximum service flow rate at level of service E, MSF_E, to compute the arterial capacity c (i.e., $c = f_c * MSF_E$). The

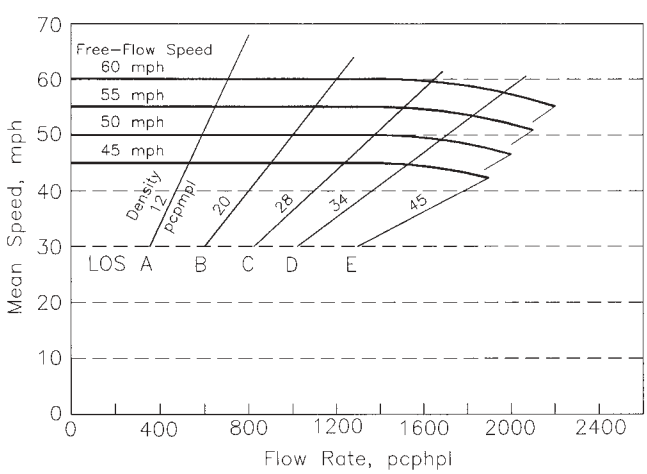

Figure 3-2. Speed-flow curves for multilane highways (1, Chap. 7).

reduction factors in this table are based on an ideal free-flow speed of 45 mph. In general, the reduction factors suggest that access point density has a significant effect on capacity, whereas median type has a relatively small effect.

Major Street Left-Turn and Through Movement Delay

Traffic Movements. Of the many possible traffic movements associated with each access point, the two movements most directly affected by alternative left-turn treatments are those on the major-street approach to the access point. Specifically, these are the major-street left-turn and through movements.

The major-street movement most directly affected by treatment type is the left-turn movement. The extent of this effect is based on the presence of a median left-turn vehicle storage area and its capacity. The storage area provided by the raised-curb and TWLTL treatments removes the left-turning vehicles from the through traffic lanes, thereby leaving these lanes available to serve through traffic without interruption. This separation of flows increases left-turn capacity and reduces left-turn delay by increasing the frequency and size of available gaps in the opposing traffic stream, relative to the undivided cross section. Because the raised-curb median always has less storage space than the TWLTL, differences between these two treatments also can emerge when left-turn demands are high enough to precipitate the overflow of the raised-curb median's bay storage area.

The other movement directly affected by midblock left-turn treatment is the major-street through movement. This movement is affected when one or more left-turning vehicles are queued in the inside through lane (hereafter, this queueing is referred to as "bay overflow"). Left-turning vehicles will queue in the inside lane for the raised-curb and TWLTL treatments whenever the left-turn queue exceeds the available median storage area. Similarly, the equivalent of bay overflow occurs whenever left-turning vehicles queue in the inside lane for the undivided treatment (even though there is no physical storage bay). Through vehicles in the inside lane approaching a left-turn queue will merge (i.e., change lanes while maintaining speed) into the adjacent through lane, if possible. However, as volume levels increase, the number of through vehicles able to merge will decrease. Through vehicles that are unable to merge are delayed until they are able to change lanes or until the left-turn queue ahead of them dissipates.

The major-street through movement also is affected by left- and right-turning vehicle deceleration occurring in the through traffic lanes. These turning vehicles need not form a queue or even stop in order to delay the through vehicles. The delay stems from the deceleration and subsequent acceleration the through vehicles must undergo as they slow to accommodate the turning vehicle and then return to the arterial running speed.

Two Through Lane Cross Sections. Harwood and St. John (*2*) evaluated the effects of the TWLTL treatment on two-lane highway operations, relative to an undivided cross section.

TABLE 3-1 Speed and capacity reductions due to left-turn treatment type and access point density (*1*, Chap. 7)

One-Way Access Point Density[1] (points/mile)	Speed Reduction, mph		Capacity Reduction Factor, f_c	
	Divided Median	Undivided	Divided Median	Undivided
0	0.0	1.6	1.00	0.96
10	2.5	4.1	0.94	0.91
20	5.0	6.6	0.89	0.85
30	7.5	9.1	0.83	0.80
40	10.0	11.6	0.78	0.74

Notes:
1 - One-way access point density is based on the count of active access points along the right side of the major street in the direction of travel divided by the length of the street (in miles).
2 - Capacity reduction factor is based on an ideal free-flow speed of 45 mph (f_c = 1 - speed reduction/45).

Based on a regression analysis of data collected at three sites, they derived the following equation for predicting the delay reduction resulting from the use of a TWLTL treatment:

$$D = -6.87 + 0.058V_o \quad (1)$$

where:

D = through vehicle delay reduction per left-turning vehicle, sec/veh
V_o = opposing flow rate, vph.

This equation indicates that the delay to through vehicles is reduced by the addition of a TWLTL to a two-lane highway. The amount of reduction ranges from 0.0 sec/veh at an opposing flow of 120 vph to about 51 sec/veh at 1,000 vph.

Because of the complexity of the arterial–access point interaction, computer simulation models have been used by some researchers to study the operational effects of midblock left-turn treatments. The first reported simulation studies were conducted by McCoy and Ballard and their colleagues (*3–5*). They developed a simulation model, TWLTL-SIM, to evaluate two-lane and four-lane streets with either a TWLTL or an undivided cross section. More recently, TWLTL-SIM was used by Harwood (*6*) to evaluate the operational effects of the TWLTL treatment on two-lane highways. Based on the simulation results, Harwood (*6*, Table 6) found that the through vehicle delay reduction per left-turning vehicle ranged from 13.1 to 19.7 veh/sec for access point densities of 90 to 30 access points per mile, respectively, and with an opposing flow rate of 400 vph. This range of delays is consistent with that predicted by Equation 1 for a 400 vph flow rate. Harwood (*6*) considered flow rates of 650 vph and higher but found them to yield overcapacity conditions and excessive delays.

Four Through Lane Cross Sections. In addition to two-lane highways, Harwood (*6*) used TWLTL-SIM to evaluate the performance of three midblock left-turn treatments on four-lane arterials. The three treatments considered were the raised-curb median, TWLTL, and undivided cross section.

Because TWLTL-SIM did not explicitly model U-turn activity, Harwood (*6*) evaluated the raised-curb median treatment by quantifying the added delay resulting from U-turns, relative to the TWLTL treatment. Specifically, the raised-curb treatment was assumed to have no median openings along the 1,000-ft simulation study segment. As a result, drivers on the raised-curb segment, who otherwise turned left at the midsegment access points with the undivided or TWLTL treatments, were assumed to continue to the signalized intersection bounding the study segment and make a U-turn during the left-turn signal phase. The added delays considered by this approach were those to the rerouted left-turning vehicles (i.e., added travel time and delay at the signalized intersection) as well as those to all other vehicles at the signalized intersection as a result of increased left-turn volume.

Harwood (*6*) recognized that this method of accounting for the operational costs of a raised-curb treatment was conservative because many drivers would likely avoid the U-turn and approach their destination from an alternative direction (so that access from the arterial was by means of a right turn rather than a left turn). Presumably, drivers using an alternative route would incur less delay than that predicted by Harwood's U-turn scenario. It should be noted that Harwood's results may be even more conservative because of his assumption that there were no median openings on the simulated street segment. Many arterials with raised-curb treatments do include midsegment median openings at unsignalized public street intersections and high-volume driveways.

Based on his analysis of operational effects, Harwood (*6*) concluded that a TWLTL was operationally more efficient than a raised-curb median or an undivided cross section over the range of left-turn and through volumes considered. In addition, his analysis indicated that raised-curb medians were operationally superior to undivided cross sections when the arterial flow rate exceeded 1,000 vph in each direction of travel; below 1,000 vph, the undivided treatment yielded lower total delay.

More recently, NETSIM was used by Modur et al. (*7*) to determine the effect of left-turn treatment on delay. Specifically, they examined the raised-curb median, TWLTL, and undivided cross section on a 600-ft four-lane street with

access points at 200-ft spacings (≈ 50 drives/mi). The arterial was bounded by signalized intersections. A median opening was provided for each access point; hence, there was no U-turning activity associated with the raised-curb treatment. Based on their analysis, Modur et al. (*7*) determined that there was no practical difference among the three medians at low arterial volume levels (i.e., 300 vph). However, at moderate volumes (i.e., 900 vph), the TWLTL was found to have 20 percent less delay than the raised-curb median and 50 percent less delay than the undivided cross section.

Modur et al. (*7*) overcame NETSIM's inability to explicitly model TWLTL operation by considering only major-street left-turn activity in one travel direction. The TWLTL scenario was modeled as a continuous lane for the length of each street segment or link between adjacent access points. As a consequence, the delay differences between the raised-curb median and TWLTL treatments stem from the TWLTL scenario having a larger left-turn storage area.

NETSIM also was used by Venigalla et al. (*8*) to compare the operational effects of the raised-curb median with those of the TWLTL. The researchers examined each treatment type on a 2,640-ft street with access points at 165 ft and at 330 ft. Median openings for the raised-curb treatment were provided only every 660 ft. They overcame NETSIM's inability to model the TWLTL by using numerous unsignalized intersections with short left-turn bays. The raised-curb treatment was modeled by removing the left-turn volume at selected access points (equivalent to closing the median) and manually reassigning it to the next downstream intersection having a median opening.

Based on their simulation results, Venigalla et al. (*8*) found total delay for the TWLTL to be more than 32 percent lower than that for the raised-curb median. The magnitude of this delay reduction is much larger than that found by Modur et al. (*7*). This likely is due to differences in the two researchers' approaches to modeling the raised-curb median (in terms of the frequency of median openings and the need to reroute left-turning vehicles).

Minor-Street Left-Turn and Through Movement Delay

Some drivers entering the major street from an access point use the median area to facilitate a two-stage crossing maneuver. This maneuver consists of two separate gap acceptance (or crossing) actions. In the first stage, the driver assesses gaps in the major-street traffic stream approaching from the left. Then, after crossing to the median area, the driver assesses gaps in the major-street traffic stream approaching from the right. By this action, the driver eliminates the need to find one large, simultaneously occurring gap in both major-street streams and thereby reduces his or her crossing time.

Kyte et al. (*9*) documented the two-stage crossing behavior and its capacity benefits in a recent study of unsignalized intersection capacity and delay. The researchers also have developed a deterministic model that predicts the capacity for this type of maneuver. Based on field data analysis, they found that the two-stage maneuver increases left-turn capacity and that the increase was slightly larger at streets with a TWLTL than at streets with a raised-curb median. This latter finding stems from observations of left-turning drivers using the TWLTL as an acceleration lane for the second stage of the entry maneuver. The acceleration maneuver typically requires a shorter gap than a stopped entry (as would be required for the second stage at a raised-curb median).

There is some evidence that the frequency of the two-stage crossing maneuver, especially when the TWLTL is used as an acceleration lane, is influenced by regional driver behavior and laws. The experience of the authors is that two-stage crossing maneuvers are found more commonly in larger cities where arterial traffic demands are high throughout the day. These conditions tend to yield high delays that have caused some drivers to adopt the two-stage maneuver to reduce this delay. Regarding TWLTL use as an acceleration lane, Bretherton (*10*) reports that this maneuver is considered undesirable in Georgia because it creates "head-on" conflicts that could lead to accidents. On the other hand, Sparks (*11*) reports that Arizona encourages the use of the TWLTL as an acceleration lane because it yields a large improvement in efficiency *without* degrading safety.

Pedestrian Refuge Area

The raised-curb median has an advantage over the TWLTL and undivided treatments in that it provides an area of refuge for pedestrians crossing the arterial. The primary benefit of a refuge area is safety; however, such as area also can influence operational effects. Specifically, a refuge area (1) may eliminate (or at least postpone) the need for a signal at an unsignalized intersection and (2) could reduce the minimum green requirement for the side-street approach at arterial signalized intersections. Both of these effects can reduce the delay to arterial through traffic.

The TWLTL has an advantage over the raised-curb and undivided treatments in that it provides an area of refuge for disabled vehicles. This use of the TWLTL can yield obvious safety and operational benefits. However, as noted by Parker (*12*), this advantage tends not to be as significant in terms of safety or operational benefits as it might initially seem because the disabled vehicles often occupy space otherwise needed to store left-turning vehicles.

Operational Flexibility

The TWLTL and undivided treatments have the advantage of providing direct left-turn access at all access points along an arterial. In contrast, the raised-curb treatment may be used to prohibit left-turn activity at some or all of these access points. As discussed previously, the prohibited left-turning

vehicles will incur additional delay if they travel to a downstream intersection, make a U-turn, and return to enter by means of a right turn.

The TWLTL and undivided treatment also offer more room for emergency vehicles to maneuver than the raised-curb median. The raised-curb median may impede emergency vehicle travel time along the arterial by making it difficult to pass or avoid stopped traffic queues.

The TWLTL treatment also has the operational advantage of providing an additional through traffic lane for special situations, such as when street maintenance and construction activities require closure of one of the through lanes, when an additional lane is needed to serve highly directional traffic flows associated with the daily peak hour or special events, and when an exclusive transit or high occupancy vehicle lane is needed to serve daily peak travel demand periods.

Through Volume Thresholds

The literature includes several reports in which authors have discussed or identified maximum volume levels that generally identify conditions in which one left-turn treatment is better than another. For example, Harwood (*6*) suggests that the raised-curb treatment performs better than the undivided treatment when the through flow rate exceeds 1,000 vph in each arterial travel direction. He also suggests that the TWLTL treatment operates better than either the raised-curb or undivided treatments, regardless of traffic demand. In contrast, Bretherton (*10*) suggests that the raised-curb treatment will operate more safely and efficiently than the TWLTL (and presumably the undivided) treatment when the average daily traffic demand exceeds 24,000 vpd. The Washington State Department of Highways (*14*) recommends that TWLTL treatments be used in the range of 10,000 to 25,000 vpd.

Parker (*12*) made an extensive review of the literature on maximum volume levels for specific treatment types. He concluded that there is no convincing evidence that such volume levels exist or that one treatment is consistently better than another at a given volume level. Sparks (*11*) indicates that the city of Phoenix, Arizona, has reached this same conclusion based on its experience with all three treatment types.

Speed Limit Thresholds

Parker (*12*) also reviewed the literature on maximum speed limits for specific treatment types. He noted that some early investigations indicated that TWLTL treatments were believed to operate more safely when limited to lower speed urban arterials. Presumably this limitation would extend to the undivided cross section, leaving the raised-curb treatment suitable for higher speed arterials. However, he found that TWLTL treatments were being operated successfully at speed limits as high as 55 mph. Based on this finding, Parker (*12*) concluded that there was no evidence to suggest that TWLTL treatments should be limited to low-speed arterials.

Access Point Density Thresholds

Many researchers have found that the density of access points along an arterial affect traffic operations in the context of turning vehicles disrupting the smooth flow of through traffic. The nature and magnitude of delays incurred by through drivers was discussed previously.

To examine the effect of access point density, Harwood (*6*) conducted simulation studies of a street segment at three density levels. He maintained similar through and left-turn volume conditions for each access-point-density scenario to isolate the effects of access point density on delay. It should be noted that the total left-turn volume was the same for the entire study segment; however, this volume was distributed equally among the access points within each scenario. As a result, the access point left-turn volume varied with each scenario.

Based on the results of his simulation analysis, Harwood (*6*) found that the through movement delay was higher on arterials with lower access point densities. The reason for this trend was explained by the concentration of more left-turning vehicles at each access point for the lower density scenarios. Harwood concluded that the TWLTL was most appropriate for suburban highways with commercial development, access point densities greater than 45 access points per mile, low to moderate through volumes, high left-turn volumes, and high rates of left-turn-related accidents. It should be noted that Bretherton (*10*) suggested that the TWLTL is safer than the raised-curb median treatment when the commercial access point density is less than 60 access points per mile. Thus, in terms of operations and safety, the TWLTL may be the better left-turn treatment when access point density is between 45 and 60 access points per mile.

Other Issues Related to Midblock Left-Turn Treatments

In addition to the operational differences among midblock left-turn treatments, there are many differences related to safety, access, cost, and aesthetics. The safety and access issues are addressed in Chapters 4 and 5, respectively. The cost and aesthetic issues are discussed in the remainder of this section.

Life-Cycle Costs

Initial Construction Costs. The cost of constructing a TWLTL or raised-curb median typically is viewed as an incremental cost, beyond that necessary to construct through traffic lanes (i.e., the cost of an undivided cross section). Harwood and Glennon (*15*) computed the construction cost

of a TWLTL and a raised-curb median under two conditions: (1) where the existing pavement is wide enough and (2) where widening and additional rights-of-way are needed. Their estimate of the cost of installing a TWLTL where the pavement is wide enough was $8,200 per mile in 1975 dollars ($24,000 in 1996 dollars). Although not explicitly reported, their data indicate that the raised-curb median would cost $89,400 per mile more than the TWLTL ($265,000 in 1996 dollars). In contrast, Parker (*12*) estimated that the raised-curb median would cost about $100,000 per mile more than the TWLTL in 1983 dollars ($158,000 in 1996 dollars).

These incremental costs were combined with the urban highway reconstruction costs reported by Cohen and Reno (*16*, Table 4-16) to determine the construction costs for three midblock left-turn treatments. These costs are reported in Table 3-2.

Modification Costs. As with construction costs, the costs of modifying left-turn treatments can vary. As Parker (*12*) noted, the raised-curb median is more costly to modify as land use and access needs change. The modification costs in this case would be related to opening the median to facilitate full access to all driveway traffic movements. To avoid the ongoing and incrementally expensive costs of opening the median one access point at a time, many agencies have been replacing the raised-curb median with a TWLTL for the entire length of the street segment. In a recent survey of 141 state and local agencies, Harwood (*13*) noted that 60 percent had removed raised-curb medians and, of these, 74 percent (for a total of 44 percent surveyed) did so for the purpose of TWLTL installation.

Maintenance Costs. The maintenance costs associated with a left-turn treatment are related to the annual cost of pavement repair, markings, and snow removal. McCoy et al. (*4*) found that these costs are higher for the TWLTL and raised-curb median treatments than for the undivided cross section. They reported that the annual maintenance costs for the TWLTL and raised-curb treatment were approximately $800 per mile more than the undivided cross section ($1,100 in 1996 dollars). It should be noted that in northern climates, there may be an annual cost associated with snow removal for the raised-curb median. Specifically, the raised-curb median presents an obstacle to snowplow equipment, and if the median is struck, damage to the equipment and the median could result.

Aesthetics

The raised-curb median offers the opportunity for an aesthetically pleasing treatment of the median area. In particular, the median area's appearance can be enhanced by introducing low-growth vegetation, grass, and brick or colored pavement into the central portion of the median. This attribute represents an advantage of the raised-curb treatment over the TWLTL and undivided treatments, provided that the median landscaping does not pose a traffic hazard or block sight lines. Trees, bushes, and poles should not be placed in the median area if they are of sufficient size to be a contributing factor to an accident or its severity.

Summary of Operational Effects and Other Issues

Findings of the literature review are summarized in Table 3-3. This table indicates the left-turn treatment of any pair of treatments that yields better operating conditions or has better qualities. Situations in which the differences are negligi-

TABLE 3-2 Construction costs associated with alternative midblock left-turn treatments

Cost Item	Area Type:	Built-Up Urban Areas			Outlying Urban Area		
	Lane Type:	Undivided	TWLTL	Raised-Curb[3]	Undivided	TWLTL	Raised-Curb[3]
Unit Costs (thousands of dollars per lane-mile)[1, 2]							
Construction		745	769	980	901	925	1,136
Right-of-Way		472	472	472	191	191	191
Total		1,217	1,241	1,452	1,092	1,116	1,327
Cost for a Street with Four Through Lanes (thousands of dollars per mile)[1]							
Construction		2,980	3,749	3,960	3,604	4,529	4,740
Right-of-Way		1,888	2,360	2,360	764	955	955
Total		4,868	6,109	6,320	4,368	5,484	5,695

Notes:
1 - Costs are updated to 1996 values using the Consumer Price Index.
2 - Costs from the "Undivided Highways, Pavement Reconstruction" category of Table 4-16 in Reference *16*.
3 - Incremental cost of Raised-curb over TWLTL was based on the average of values reported by Harwood and Glennon *(15)* and Parker *(12)* (i.e., (158,000 + 265,000)/2 = 211,000).

TABLE 3-3 Comparison of operational and other effects of three left-turn treatment types

Comparison Factor	"Preferred" Midblock Left-Turn Treatment[1]		
	Raised-Curb vs. TWLTL	Raised-Curb vs. Undivided	TWLTL vs. Undivided
Operational Effects			
1. Major-street through movement delay	n.d.[2]	Raised-Curb	TWLTL
2. Major-street left-turn movement delay	n.d.	Raised-Curb	TWLTL
3. Minor-street left & thru delay (two-stage entry)	n.d.	Raised-Curb	TWLTL
4. Pedestrian refuge area	Raised-Curb	Raised-Curb	n.d.
5. Operational flexibility	TWLTL	Undivided	n.d.
Other Effects			
1. Cost of maintaining delineation	n.d.	Undivided	Undivided
2. Median reconstruction cost	TWLTL	Undivided	Undivided
3. Facilitate snow removal (i.e., impediment to plowing)	TWLTL	Undivided	n.d.
4. Visibility of delineation	Raised-Curb	Raised-Curb	n.d.
5. Aesthetic potential	Raised-Curb	Raised-Curb	n.d.
6. Location for signs and signal poles	Raised-Curb	Raised-Curb	n.d.

Notes:
1 - The "Preferred" left-turn treatment is based on the findings of this research and the more commonly found opinion during the review of the literature.
2 - n.d.: negligible difference or lack of a consensus of opinion on this factor.

ble or lack a consensus in the literature are indicated by "n.d.," representing "negligible difference."

VALIDATION OF TWLTL-SIM SIMULATION MODEL

Model Overview

The TWLTL-SIM computer model was developed by Ballard and McCoy (*5*) for evaluating the operational effects of alternative midblock left-turn treatments. It is a microscopic, stochastic simulation model that can be used to evaluate traffic operations on two-, four-, and six-lane street segments with a raised-curb median, a TWLTL, or an undivided cross section. TWLTL-SIM computes several measures of traffic performance, including average delay and number of stops. These measures can be computed for a wide variety of arterial traffic movements. This computer model has undergone a series of revisions since its original inception; the current version is TWLTL-SIM III.

TWLTL-SIM was used for this research to supplement the data collected during field studies. In general, field data were used to calibrate the models that comprise the operations model and to validate the TWLTL-SIM model. Then TWLTL-SIM was used to verify the overall operations model by comparing the predicted delays for a series of selected arterial geometries and traffic volume combinations. Calibration and verification activities are described in more detail later in the chapter.

Model Description

The TWLTL-SIM III model is written in General Purpose Simulation System Version H (GPSS/H) language (*17, 18*), a general purpose computer language designed to model discrete systems. There are five separate programs within the TWLTL-SIM III model, each of which must be executed in sequence to complete the evaluation of a street segment. These programs, listed in the order of execution, are as follows:

1. Road Program (ROADPGM)
2. Speed Program (SPEEDPGM)
3. System Parameter Program (PARMPGM)
4. Main Simulation Program (MAINPGM)
5. Statistical Summarization Program (SUMMPGM).

Three of the five TWLTL-SIM programs facilitate the input of relevant model data, including street geometries, traffic characteristics, traffic control, and run control parameters. The fourth program combines the input data files and then simulates the described geometric/traffic scenario. A fifth program is then used to summarize the output from the simulation.

TWLTL-SIM output can be animated using the PROOF Animation System (*19*). This software generates a plan view of the street segment geometry, adds the vehicles, and then moves them along the street from entry to exit point.

A flowchart showing how the five programs interact with each other and the animation software appears in Figure 3-3.

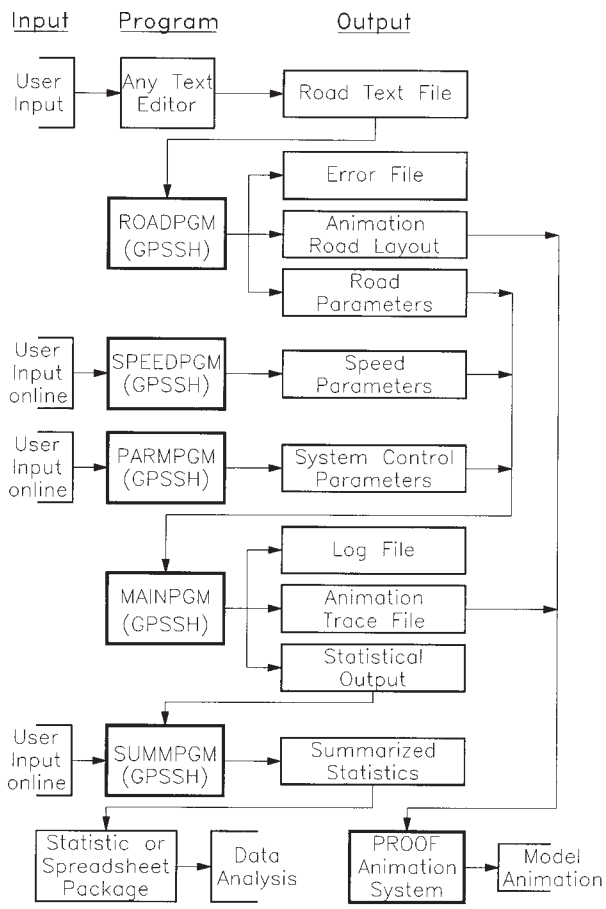

Figure 3-3. TWLTL-SIM III model flowchart.

The Road Program is used to validate the street geometry, traffic path, and traffic volume data coded in the Road Text File. This validation includes a check for value consistency and adherence to range limits. If no errors are found in the Road Text File, the Road Program creates a Road Parameters File for use by the Main Simulation Program.

In contrast to the Road Program, the Speed and System Parameter Programs prompt the analyst, in an interactive manner, for the necessary input data. The Speed Program collects relevant traffic characteristics and traffic signal controller settings, checks them for validity, and then creates the Speed Parameters File. The System Parameter Program primarily gathers the various run control parameters that define the character of the simulation run and the types of statistics desired. Following data entry, the System Parameter Program creates the System Control Parameters File. The input data needed by the Road, Speed, and System Parameter Programs are listed in Table 3-4.

The Main Simulation Program contains the simulation routines used by TWLTL-SIM III. This program reads the input data files (i.e., the Road, Speed, and System Control Parameters Files), simulates traffic flow, and creates the Statistical Output File. The output data in this file are then categorized and aggregated in the Statistical Summarization Program. The output summarization can be in terms of direction, node, or movement-specific categories. The output measures of performance include delays, number of stops, and travel times. The output data provided by the Summarization Program are listed in Table 3-5.

Two additional output files are used to create the event animation: (1) the Road Layout File created by the Road Program and (2) the Animation Trace File created by the Main Simulation Program. These files are used by the animation software to display a re-creation of the sequence of traffic and signal indication events that took place during the simulation.

Model Enhancements

Since its original development in the mid-1980s, TWLTL-SIM has undergone a series of enhancements to improve its ability to model driver behavior and to allow it to be applied to a wider range of geometric and traffic control conditions. Several of these enhancements were added to TWLTL-SIM for this research project, including the following:

- Addition of the raised-curb median as a midblock left-turn treatment option
- Incorporation of driver impatience (i.e., delay) into the gap acceptance logic
- Improvement of the lane-change logic
- Improvement of the traffic signal control logic
- An increase in the number of through lanes to include six-lane cross sections.

The original version of TWLTL-SIM would only allow through vehicle lane changes. Vehicles traveling in the through lanes that would ultimately make a left turn at a downstream location were not allowed to change lanes. As a consequence, these "restricted" vehicles would often incur excessive (and unrealistic) delay while waiting behind left-turning vehicles at access points upstream of the restricted vehicle's final turn location. This deficiency has been eliminated in TWLTL-SIM III so that any vehicle can change lanes to go around a stopped left- or right-turning vehicle and then go back to the lane, if needed, to make a downstream turn.

Hardware Requirements

The TWLTL-SIM program is written in GPSS/H language, which is supported on VAX/VMS and IBM mainframe computers as well as DOS-based personal computers. When used on a personal computer, the GPSS/H software requires DOS, Version 3.0 or higher; an 80836 or higher microprocessor; a math coprocessor; 640K RAM; 100MB of hard drive storage space; and a VGA monitor.

Simulation run time is determined by the processor speed and the amount of random access memory. Typical run times with a 60-MHz Pentium processor and 24MB of RAM are

TABLE 3-4 TWLTL-SIM III input data

Input Database Element	Program
Street Geometrics	
Left-Turn Treatment Type	PARMPGM
Number of Through Lanes (both directions)	ROADPGM
Length of Street Segment	ROADPGM
Location and Width of Access points / Cross Streets	ROADPGM
Length of Left-Turn Bays	ROADPGM
Traffic Characteristics	
Entering Movement Volumes at Both Signalized Intersections	ROADPGM
Left or Right Turn Volume from Major Street (for each access point)	ROADPGM
Left or Right Turn Volume from Minor Street (for each access point)	ROADPGM
U-Turn Volume at each Median Opening	ROADPGM
Origin-Destination Matrix of Traffic Movement Linkages Through Street Segment	ROADPGM
Minimum Discharge Headway	SPEEDPGM
Left, Right, & U-Turn Speed	SPEEDPGM
Average Free-Flow Speed (both directions)	SPEEDPGM
Signal Controller Settings	
Cycle Length	SPEEDPGM
Phase Splits / Duration	SPEEDPGM
Coordination Offsets	SPEEDPGM
Run Control Parameters	
Output Statistic Categories (i.e., group by travel direction, movement, intersection)	ROADPGM
Random Number Seed	PARMPGM
Simulation Run Time	PARMPGM
Statistic Reporting Interval	PARMPGM

about 2 to 10 min for 1 hr of simulated time. The exact run time depends primarily on the number of vehicles in the street system, the length of the street, and the number of access points. For example, a four-lane street segment 2,000 ft in length, with traffic demands of 1,500 vph in each travel direction and 20 access points, would require about 10 min of computer time to simulate 1 hr of traffic activity.

Model Validation

Validation Study Sites

Three of the 32 field study sites were used to validate TWLTL-SIM (more details on the field data collection effort are provided elsewhere in this chapter). This validation focused on demonstrating TWLTL-SIM's ability to accurately predict the delays to the major-street left-turn and through movements at the three sites. To demonstrate this ability over a wide range of conditions, each validation site had a different left-turn treatment and speed limit and a relatively high major-street left-turn volume at one access point. Some basic characteristics of each site are provided in Table 3-6.

Measures of Performance

A database of performance measures was established for the three validation sites using the TWLTL-SIM model and field study data. The performance measures considered

TABLE 3-5 TWLTL-SIM III output data

Output Database Elements	Units
Number of vehicles to exit the street segment during the simulation time period	veh
Average delay and stops to vehicles at the entry and exit signalized intersections	sec/veh, stops/veh
Average delay and stops to vehicles on the street between the signalized intersections	sec/veh, stops/veh
Number of lane changes by major-street through vehicles	changes/hour

TABLE 3-6 Characteristics of the study sites used in the TWLTL-SIM validation

Characteristic	Study Site		
	Metcalf Avenue	72nd Street	Harlem Avenue
Midblock Left-Turn Treatment Type	Raised-curb	TWLTL	Undivided
State	Kansas	Nebraska	Illinois
City	Overland Park	Omaha	Chicago
Major Street	Metcalf Ave.	72nd Street	Harlem Ave.
Bounding Streets	91st to 93rd	Jones to Pacific	Wilson to Montrose
Average Daily Traffic	35,000	38,700	34,000
Major Street Segment Length, feet	1,320	2,010	940
Number of Through Lanes	4	4	4
Speed Limit, mph	45	35	30
Highest Observed Left-Turn Volume, vph	92	151	62
Cycle Length during PM-Peak Hour, sec[1]	140/140	90/90	130/88
Number of Access Points (two-way total)[2]	5	19	14

Notes:
1 - Cycle length at the bounding signalized intersections.
2 - A two-way total represents the count of all active access points located on both sides of the major street.

included (1) the average through vehicle travel time and (2) the average left-turn delay for vehicles turning at the "high-volume" access point.

Though vehicle travel time is defined as the time required to travel from the stop line at the upstream signalized intersection to the stop line at the downstream signalized intersection as a through movement. Vehicles that turn onto or off of the major street are not included in the travel time measurement. In addition to segment length, travel time can be affected by three factors:

1. The delay due to the traffic signal at the downstream end of the segment
2. The volume of access point activity (i.e., vehicles turning into or out of access points) along the street segment
3. The volume of traffic (as shown by Figure 3-2).

The through vehicle travel times obtained from the field data and those predicted by TWLTL-SIM include each of these effects.

Traffic volume and signal timing data from the field studies were combined with the street-segment geometry (e.g., treatment type and length) to develop a TWLTL-SIM input file for each of the three sites. The simulation runs for each site included five replications for each analysis period. All inputs remained the same for each replication; the only change between runs was in the random number seed. Traffic conditions were simulated for a minimum of 1 hr per replication to obtain the desired confidence interval for the sample means.

Two time periods were established for each of the three validation sites. One period coincided with the noon peak traffic demand period and the other with the evening peak period. Within each peak period, the 15-min period with the highest through traffic demand was analyzed. Thus, for each 15-min analysis period, average traffic volumes, signal phase durations, and through vehicle travel times were obtained from the field data. One exception to this approach was the left-turn delay, which was averaged over a 1-hr peak period because of the significant random variability in delay data. In all cases, the peak hours used for the delay average coincided with the noon and evening peak periods and included the 15-min analysis period used to compute the other statistics.

Findings and Conclusions

The statistics describing the performance measures obtained from the field data and TWLTL-SIM are summarized in Tables 3-7, 3-8, and 3-9. One trend that can be found when comparing Tables 3-7 and 3-8 is the large difference in travel time between the northbound and southbound directions during the evening (or PM) peak period for both the 72nd Street and Harlem Avenue sites. Specifically, southbound vehicles at both sites incur significantly more travel time than northbound vehicles. This difference is due to the high volume-to-capacity (v/c) ratio on the southbound approaches during the evening peak period and the corresponding high signal delay.

A statistical comparison of the measured and predicted travel times in Tables 3-7 and 3-8 suggests that TWLTL-SIM is able to accurately predict through travel time. In all but two cases, the difference between the TWLTL-SIM-predicted and the measured travel times is less than 10 percent. Moreover, the difference is not statistically significant for half of

TABLE 3-7 Comparison of field study and TWLTL-SIM data (northbound travel time)

Study Period	Field Data[a]			TWLTL-SIM[a]			Error[b]		z^c
	μ_f	σ_f	n_f	μ_t	σ_t	n_t	Δ	%	
Metcalf Avenue Noon-Peak	40.0	23.2	194	42.5	23.4	1124	-2.5	-5.9	-1.4
Metcalf Avenue PM-Peak	37.9	23.4	110	41.6	24.1	1321	-3.7	-8.9	-1.6
72nd Street Noon-Peak	50.2	14.2	112	49.1	21.1	1139	1.1	2.2	0.74
72nd Street PM-Peak	52.6	16.4	105	49.6	15.4	1106	3.0	6.0	1.8
Harlem Avenue Noon-Peak	35.0	14.3	112	31.9	12.5	845	3.1	9.7	2.2
Harlem Avenue PM-Peak	33.6	14.3	105	33.3	14.6	755	0.3	0.9	0.20

Notes:
a - μ: sample mean, seconds per vehicle; σ: sample standard deviation, seconds per vehicle; n: sample size.
b - $\Delta = \mu_f - \mu_t$; $\% = (\mu_f - \mu_t)/\mu_t * 100$
c - $z = (\mu_f - \mu_t)/(\sigma_f^2/n_f + \sigma_t^2/n_t)^{0.5}$, z is a standardized normal variate. Underlined values denote tests where the null hypothesis (i.e., Ho: $\mu_f = \mu_t$) is rejected.

TABLE 3-8 Comparison of field study and TWLTL-SIM data (southbound travel time)

Study Period	Field Data[a]			TWLTL-SIM[a]			Error[b]		z^c
	μ_f	σ_f	n_f	μ_t	σ_t	n_t	Δ	%	
Metcalf Avenue Noon-Peak	30.9	14.6	106	30.9	11.4	1234	0.0	0.0	0.00
Metcalf Avenue PM-Peak	28.6	13.4	108	29.8	13.4	1315	-1.2	-4.0	-0.90
72nd Street Noon-Peak	66.5	18.7	112	67.4	17.3	1133	-0.9	-1.3	-0.49
72nd Street PM-Peak	91.2	31.8	103	83.0	29.7	1121	8.2	9.9	2.5
Harlem Avenue Noon-Peak	43.1	22.6	105	36.9	17.8	815	6.2	16.8	2.7
Harlem Avenue PM-Peak	78.4	38.0	108	38.7	18.7	947	39.7	102.6	10.7

Notes:
a - μ: sample mean, seconds per vehicle; σ: sample standard deviation, seconds per vehicle; n: sample size.
b - $\Delta = \mu_f - \mu_t$; $\% = (\mu_f - \mu_t)/\mu_t * 100$
c - $z = (\mu_f - \mu_t)/(\sigma_f^2/n_f + \sigma_t^2/n_t)^{0.5}$, z is a standardized normal variate. Underlined values denote tests where the null hypothesis (i.e., Ho: $\mu_f = \mu_t$) is rejected.

TABLE 3-9 Comparison of field study and TWLTL-SIM data (left-turn delay)

Study Period	Field Data[a]			TWLTL-SIM[a]			Error[b]		z^c
	μ_f	σ_f	n_f	μ_t	σ_t	n_t	Δ	%	
Metcalf Avenue Noon-Peak	23.6	35.2	88	21.2	21.5	94	2.4	11.3	0.55
Metcalf Avenue PM-Peak	20.2	23.5	73	25.2	23.9	55	-5.0	-19.8	-1.18
72nd Street Noon-Peak	22.0	42.9	124	20.8	18.4	137	1.2	5.8	0.29
72nd Street PM-Peak	28.4	34.7	114	25.8	21.9	123	2.6	10.1	0.68
Harlem Avenue Noon-Peak	14.6	17.2	64	10.0	13.4	84	4.6	46.0	1.77
Harlem Avenue PM-Peak	6.1	9.3	51	8.7	8.4	52	-2.6	-29.9	-1.49

Notes:
a - μ: sample mean, seconds per vehicle; σ: sample standard deviation, seconds per vehicle; n: sample size.
b - $\Delta = \mu_f - \mu_t$; $\% = (\mu_f - \mu_t)/\mu_t * 100$
c - $z = (\mu_f - \mu_t)/(\sigma_f^2/n_f + \sigma_t^2/n_t)^{0.5}$, z is a standardized normal variate. Underlined values denote tests where the null hypothesis (i.e., Ho: $\mu_f = \mu_t$) is rejected.

the cases listed in Tables 3-7 and 3-8. The two cases in which the travel time error is large were found in the southbound direction at the Harlem Avenue site. The error is attributable to the inability of TWLTL-SIM to model shared-lane (i.e., left and through vehicles in the same approach lane) operation for signalized intersections.

The data in Table 3-9 suggest that TWLTL-SIM is able to accurately predict left-turn delay at access points along the street segment. In general, the error in delay prediction is less than 5 sec per vehicle. Although the error, as a percentage, is large in some instances, the magnitude of the error is quite small relative to the large variability in delay data. The effect of this variability can be seen in the statistical test shown in the last column of Table 3-9. Specifically, the high variability has produced relatively small normal variates, indicating that the observed differences are not statistically significant relative to the uncertainty we have about the true mean of the two delay statistics.

DEVELOPMENT OF AN OPERATIONAL EFFECTS DATABASE

Database Composition

Midblock left-turn treatments affect the performance of through and turning traffic in a variety of complex ways. In recognition of this complexity, it was determined that any quantitative evaluation procedure would require a cohesive set of models that collectively replicate the interaction of these through and turning vehicles. The calibration of these models would require the assembly of a large database with a wide range of operational data. This section describes the complex traffic flow problems that stem from through and turn vehicle interactions, as they are affected by midblock treatment. It also describes the model framework needed to evaluate these problems, the data needed for each model, and the data collection effort and presents an overview of the database composition.

Traffic Flow Problems Directly Related to Midblock Left-Turn Treatment

Through Lane Blockage. A flow problem that occurs in varying degrees in all midblock left-turn treatments is blockage of the inside through lane. This problem occurs when major-street left-turn demand exceeds the available storage area and vehicles spill back into the adjacent through lane. This problem does not generally occur with significant frequency on streets with divided cross sections; however, it frequently occurs on undivided streets because of their lack of left-turn storage.

There are two consequences of through lane blockage, depending on the size of the queue that forms in the through lane. If the queue spills back into an upstream unsignalized intersection (formed by an access point and the major street), an additional impedance to traffic flow at that intersection and a corresponding reduction in intersection capacity results. If the queue overflows the bay storage but does not spill back, a situation in which through drivers in the inside lane will try to merge with vehicles in the adjacent through lane is created, providing there is an adequate gap for drivers to safely do so. If there is no merge opportunity, the through drivers in the inside lane will be delayed until the queue ahead dissipates or until they can safely merge.

Three types of data were collected to calibrate the component models needed to quantify the effects of lane blockage:

1. Data to calibrate a model for predicting the frequency of lane blockage
2. Data to calibrate a merge-capacity model
3. Data to calibrate a model for predicting the flow rate in each lane during bay overflow.

This last model is important because the lane flow rates directly relate to the number of vehicles delayed as well as to the capacity of the merge maneuver.

Through Vehicle Slow-Down Resulting from Turns. This flow problem essentially is a delay incurred by through drivers as they follow a left- or right-turning vehicle as it enters the turn bay or lane. This problem is most prevalent when the turning vehicle is at or near the front of a platoon and the turn bay is either relatively full or too short to accommodate the required deceleration distance. Through vehicles that follow the slowing vehicle are delayed because they also must decelerate with the turning vehicle and then accelerate back to the arterial running speed.

Two types of data were collected to calibrate the component models needed to quantify the effect of through lane slow-down: (1) data describing the bay entry speed and deceleration rate to calibrate a model for predicting the speed reduction to through vehicles and (2) data to calibrate a model for predicting the flow rate in each lane prior to bay overflow. This model is important for the same reasons as mentioned for the previous flow problem.

Through Vehicle Slow-Down Resulting from Traffic Volume. This problem is more fundamentally rooted than the preceding two problems. Specifically, the problem relates to the reduction in speed caused primarily by increasing traffic volume (or density). As noted previously, Chapter 7 of the HCM describes a relationship between volume level and speed on rural and suburban highways. Recognizing the possible existence of such a relationship on urban arterials, speed and volume data were collected for this project. This relationship would be used to predict the added travel time resulting from traffic volume and any other relevant factors.

Traffic Flow Problems Indirectly Related to Midblock Left-Turn Treatment

Three flow problems were identified as being indirectly related to midblock left-turn treatment type. These problems

relate to the interaction of a bounding signalized intersection with relatively near access points. They are described in the next few paragraphs because the magnitude of their effect is influenced by left-turn treatment type; however, data were not specifically collected to quantify these problems because they were beyond the scope of the research project.

Spillback from a Downstream Signalized Intersection. When the queue from a signalized intersection spills back into an upstream unsignalized intersection (formed by an access point and the major street), departures are blocked and most intersection movements are impeded. The capacity of the upstream intersection is effectively reduced in proportion to the period of time in which spillback occurs. This effect can be significant when the signalized intersection has such a long cycle length or a high volume that long traffic queues form each cycle. The magnitude of the impact also is determined by the location of the upstream intersection relative to the signal.

Reduced Lane Capacity at a Downstream Signalized Intersection. The capacity of a signalized intersection approach can be restricted by turning vehicles exiting the major street at an upstream access point. The magnitude of the effect depends on whether the movement is a left or a right turn and whether the turning vehicle slows or stops in the through lane. At worst, these turning vehicles block the through lane while the phase is in service and severely reduce the lane's traffic capacity. At best, the turning vehicles create large headways in the discharging traffic queue, thereby reducing the saturation flow rate of the signalized approach. This latter effect has been quantified by McCoy (*20*), who found that the magnitude of the reduction was determined by the proximity of the access point to the signalized intersection and the turn volume at the access point.

Effect of Upstream Signals on Nonpriority Movement Capacity. Appendix I in Chapter 10 of the HCM documents a procedure for estimating the capacity of a nonpriority movement at an unsignalized intersection in the presence of upstream signalized intersections. This HCM appendix states that the traffic platoons created by an upstream signal will block a nonpriority movement as they pass through the intersection, but leave large gaps afterward. These large gaps tend to increase the capacity of the nonpriority movements.

Operations Model Framework

Methodologies for evaluating all of the aforementioned flow problems were developed and combined into a larger model of arterial traffic flow. The larger model is referred to as the "operations model"; the models comprising the flow problem methodologies are referred to as the "component models." The formulation of the operations model, its various analysis methodologies, and the associated component models are discussed in more detail later in this chapter.

Five component models were identified in the section that described flow problems that are directly related to treatment type:

1. Probability of left-turn bay overflow
2. Lane flow rate approaching the left-turn point during downstream blocked/unblocked conditions
3. Through movement speed and volume
4. Bay entry speed and deceleration
5. Lane-change gap acceptance

Data collection activities and the resulting databases were defined by the data needs of these component models. Each database is described in a later section on data collection.

Database Elements

The data needed to calibrate the component models were categorized as (1) basic traffic characteristics, (2) traffic performance measures, (3) signal controller settings, (4) traffic control features, and (5) geometric data. The elements that comprise the first two categories are dynamic and were collected continuously during the field study. These elements are listed in Table 3-10.

The latter three categories represent static data types. With one exception, these data were measured before the field study of a particular site. The exception is phase duration at interchanges operating in a traffic-actuated mode. This variable was measured for each cycle and averaged. Elements of each of the latter three categories include the following:

- *Signal Controller Settings.* This category included the traffic signal control settings and operation of the midblock segment's upstream and downstream signalized intersections. Specifically, this included the cycle times, coordination offsets, signal phase sequence, change interval, and phase durations for each intersection.
- *Traffic Control Features.* This category included speed limit, traffic control signs, and pavement markings. This information was used to describe the general character of the arterial.
- *Geometric Data.* This category included geometric information along the arterial and at each intersection. Arterial geometric data included cross section, lane width, grade, lane assignments, and distance between the centerlines of adjacent access points or signalized intersections. Intersection geometric data included approach width, curb return radii, and bay lengths.

Study Site Descriptions

A list of desirable characteristics for the field study sites was prepared using information from the survey of practitioners and the literature. For this research, a study site was

TABLE 3-10 Operations model database elements

Database Elements	Component Models[2]				
	1	2	3	4	5
Major-street left-turn queue length distribution	V[1]				
Major-street left-turn movement volume	V				
Competing (or opposing) major-street through movement volume	V				
Major-street through movement speed (during unblocked periods)	V				
Major-street approach lane flow rates		C			
Presence of a left-turn vehicle in the inside through lane (i.e., a block)		V			
Major-street through movement headways at a midblock point			C		
Major-street through movement speed at a midblock point			C		
Major-street through vehicle wheelbase (for classification)			C		
Major-street left-turn bay entry speed				C	
Major-street left-turn queue length at time of bay entry				V	
Major-street left-turn bay entry deceleration rate				C	
Headway accepted for lane-change					V
Lane-changing vehicle speed					V

Notes:
1 - Data collection systems:
C - computer-monitored tape switch sensors placed in street, V - video cameras and tape recorders.
2 - Database types for models that constitute the Operations Model:
1. Probability of left-turn bay overflow
2. Lane flow rate approaching the left-turn point during downstream blocked/unblocked conditions
3. Through movement speed and volume
4. Bay entry speed and deceleration
5. Lane-change gap acceptance

defined as an urban or suburban arterial street segment bounded by signalized intersections but having only unsignalized access points along its length. This segment has a constant cross section and one type of midblock left-turn treatment. The following criteria were used to define an urban or suburban arterial segment:

- Traffic volume of more than 7,000 vpd
- Speed limit between 30 and 50 mph
- Spacing of at least 350 ft between signalized intersections
- Direct access from abutting properties
- No angle curb parking (parallel parking is acceptable)
- Location in or near a populated area (e.g., 20,000 or more)
- No more than six through traffic lanes (three each direction)
- Arterial length of at least 0.75 mi.

The application of these criteria in selecting the study sites was intended to ensure that low-volume two-lane roadways, rural highways, expressways, roads through small towns, and low-speed collector streets were not included in the candidate list of field study sites.

Study Site Characteristics

Segment Types. Based on the results of the survey of practitioners, it was determined that each study site must have one of the four most commonly used midblock left-turn treatments. These four types are shown in Figure 3-1. Because the survey indicated that the four- and six-through-lane cross sections constituted the largest number of lane-miles of urban arterial, it was determined that most of the study sites should have four or six through lanes.

Study Types. The field studies were categorized as either microscopic or macroscopic. In general, the former relates to the study of a specific access point along a street segment, whereas the latter relates to the study of traffic flow along the street. The microscopic (or access point) study procedure examined the major-street left-turn movement at an access point. In all cases, the left-turn movement studied had the highest volume of all left-turn movements on the study segment. This study focused on left-turn gap acceptance, left-turn queue length, and turbulence to through traffic caused by major-street left-turn movements. Twenty-two of the 32 field studies conducted used the microscopic study procedure.

The macroscopic (or traffic flow) study procedure was intended to examine the effect of a left-turn treatment on through movement traffic flow. This examination focused on the effect of turning movements on through vehicle travel time and headway distribution. Ten of the 32 field studies conducted used the macroscopic study procedure. Of these 10 studies, 2 were included in a pilot study and 8 were included in the subsequent full-scale field study.

Geometric and Traffic Demand Criteria. The selection of specific study sites was based on the sites' degree of compliance with the following geometric and traffic demand criteria. Most of these criteria were established as "desired" rather than "required" attributes in recognition of the difficulty of finding 32 study sites that satisfied all criteria. The geometric criteria are as follows:

- Functional Class of Major Street—Urban or suburban arterial (as defined previously)
- Midblock Left-Turn Treatment
  - Flush median with two-way left-turn lane (TWLTL)
  - Raised-curb median with alternating left-turn bays (RM)
  - Flush median with alternating left-turn bays (FM)
  - Undivided cross section (no median, NM)
- Through Lanes in Cross Section—two, four, or six
- Segment Length—600 to 2,600 ft between signalized intersections
- Other Desirable Attributes
  - Average intersection skew angle, 10 deg or less
  - Major street horizontal curvature, 2 deg or less
  - Major street vertical grade, 3 percent or less

Following are the traffic demand criteria:

- Major Street Peak Left-Turn Volume (highest volume location)—Microscopic study
  - Divided section (TWLTL, RM, FM), 200 vph or more
  - Undivided section (NM), 50 vph or more
- Major Street Peak Left-Turn Volume (highest volume location)—Macroscopic study
  - Divided section (TWLTL, RM, FM), 150 vph or more
  - Undivided section (NM), 40 vph or more
- Major Street ADT
  - Two-lane, 11,000 vpd or more
  - Four- or six-lane, 33,000 vpd or more.

It should be noted that the left-turn and through traffic demands are relatively high. These levels were specifically set at high values for two reasons. First, it was believed that high-volume conditions were necessary to distinguish among alternative midblock left-turn treatments in terms of their operational performance. Second, higher volumes would provide a larger number of observations in a given time period, which was desired from a statistical standpoint.

Study Site Locations

The street segments studied are listed in Table 3-11. As this table indicates, there were eight macroscopic studies and 22 microscopic studies conducted at 30 unique street segments. Two additional study sites were included in the pilot study for the project; however, these sites are not shown because the pilot study was used primarily to refine the data collection methods that were ultimately used in the other 30 studies.

With one exception, the street segments listed in Table 3-11 have either four or six through lanes. Street segments with two through lanes were sought during the site selection process, but those satisfying the desired criteria generally were not found.

In general, the field study sites satisfied almost all of the desired study site criteria. The most challenging criterion to satisfy was the major-street left-turn traffic volume. Streets with peak-hour left-turn volumes in excess of 100 vph were not found as frequently as anticipated. It is believed that the high daily traffic volumes also desired tended to yield low left-turn capacity and therefore low left-turn volumes at many locations. Nevertheless, many sites with relatively high left-turn volumes were found, and several of these had high v/c ratios. This latter characteristic typically yielded the desired degree of left-turn queueing.

Data Collection

Approach

Equipment used to collect field data included video cameras and computer-monitored tape switch sensors placed in the traffic lanes. In all cases, only one direction of travel was studied at each site. The tape switch sensors were adhered to the pavement surface to record the passage time of individual vehicles. These times, which were recorded by a computer that monitored the tape switches, were used to compute vehicle headway and speed. Video cameras were positioned so that their collective fields of view included the beginning and end of the high-volume major-street left-turn location. Typical camera positions are shown in Figure 3-4 for one macroscopic study site. The corresponding fields of view obtained with video cameras are shown in Figure 3-5.

The field studies were conducted in the fall of 1994. It should be noted that all data were collected during weekday daytime periods between 7:00 a.m. and 7:00 p.m. The study period included the hours of peak traffic demand at each study site. Data were not collected during inclement weather or during unusual traffic conditions (e.g., a traffic accident).

Left-Turn Bay Overflow Data

The data collected for the bay overflow equation include the characteristics necessary to predict the portion of

TABLE 3-11 Study site characteristics

City, State	Study Site Location	Median Type[1]	Thru Lanes	Section Length (feet)	ADT	Peak LT Volume[2] (vph)	Speed Limit (mph)
Macroscopic Study							
Lincoln, NE	**27th St.** - Vine to Holdrege St.	RM	4	2,650	30,000	65	35
Overland Park, KS	**Metcalf Ave.** - 91st to 93rd St.	RM	4	1,320	35,000	92	45
Phoenix, AZ	**Camelback Rd.** - 22nd to 24th St.	RM	6	1,350	52,000	15	35
Lincoln, NE	**Holdrege St.** - 33rd to 38th St.	TWLTL	2	1,680	19,000	112	35
Omaha, NE	**72nd St.** - Pacific to Jones St.	TWLTL	4	2,010	38,700	151	35
Chandler, AZ	**Arizona Ave.** - Elliot to Warner Rd.	TWLTL	6	5,280	36,000	65	45
Chicago, IL	**Roosevelt Rd.** - Westmore/Meyers to Mall	FM	4	2,800	42,000	24	35
Chicago, IL	**Harlem Ave.** - Montrose/Agatite to Wilson	NM	4	940	34,000	62	30
Microscopic Study							
Omaha, NE	**W. Center Rd.** - 129th to 132nd St.	RM	4	1,360	45,000	90	45
Overland Park, KS	**Roe St.** - 110th to I-435	RM	4	1,050	36,000	373	40
Kansas City, KS	**State Ave.** - 74th to 78th St.	RM	4	3,100	30,000	70	na
Chicago, IL	**Dempster St.** - Luther to Greenwood	RM	4	1,760	40,000	133	na
Chicago, IL	**Harlem Ave.** - Cullom to Garage Ent.	RM	4	1,010	43,000	78	30
Omaha, NE	**W. Dodge Rd.** - 78th to 84th St.	RM	6	2,250	50,000	247	40
Chicago, IL	**Dempster St.** - Ozark to Harlem	RM	6	1,520	44,000	47	35
Phoenix, AZ	**Indian School Rd.** - 27th to 31st Ave.	RM	6	2,630	44,000	80	40
Chandler, AZ	**Chandler Blvd.** - Beck to Kyrene Mall	RM	6	3,280	31,000	338	50
Lincoln, NE	**48th St.** - O to R St.	TWLTL	4	1,260	28,000	190	35
Lincoln, NE	**48th St.** - Normal to Van Dorn	TWLTL	4	1,400	22,000	249	35
Lawrence, KS	**23rd St.** - Alabama to Louisiana	TWLTL	4	1,360	30,000	57	35
Chandler, AZ	**Alma School Rd.** - Elliot to Warner	TWLTL	4	5,280	38,000	50	na
Phoenix, AZ	**Thomas Rd.** - 11th to 15th Ave.	TWLTL	4	1,324	30,000	62	35
Phoenix, AZ	**Camelback Rd.** - 12th to 15th St.	TWLTL	6	1,990	49,000	21	35
Chicago, IL	**Roosevelt Rd.** - Finley to Main St.	FM	4	1,080	41,000	58	35
Chicago, IL	**Roosevelt Rd.** - Warrenville/West to Main	FM	4	1,570	26,000	43	35
Chicago, IL	**Harlem Ave.** - Archer to I-55	FM	6	2,820	43,000	201	40
Lincoln, NE	**48th St.** - Vine to Holdrege	NM	4	2,650	27,000	33	35
Omaha, NE	**84th St.** - F to L St.	NM	4	2,500	36,000	58	40
Overland Park, KS	**87th Pkwy.** - Grant to Grandview	NM	4	1,320	27,000	197	40
Kansas City, KS	**Rainbow Blvd.** - 40th to 43rd St.	NM	4	780	24,000	58	30

Notes:
1 - Midblock left-turn treatment type: RM = raised-curb median, TWLTL = flush median with two-way left-turn lane delineation, FM = flush median with alternating left-turn bays, NM = undivided cross section (no median).
2 - Left-turn volume during the hour of peak turning activity.

time in which the queue exceeds the storage area. Because this problem is fundamentally related to left-turn volume and capacity, data were collected for quantifying both of these characteristics. Specifically, data collected for estimating capacity included opposing through movement flow rate and speed, left-turn queue length, and frequency and duration of bay overflow. These data were collected during the peak traffic demand hour at each of 16 study sites. The 16 sites chosen represent sites that had the highest left-turn demands and the necessary camera fields of view.

Lane Flow Rate Data

Lane flow rate data were obtained from the tape switch sensors and the videotape recordings. The tape switches were used to count traffic in each lane, whereas the video cameras were used to determine whether the inside through lane was blocked at the high-volume left-turn location. Blocking, as it is defined for this research, was observed to occur primarily at sites with an undivided cross section. Sites with raised-curb, TWLTL, or flush median treatments rarely had any blockage in the inside through lane (i.e., bay overflow). Lane

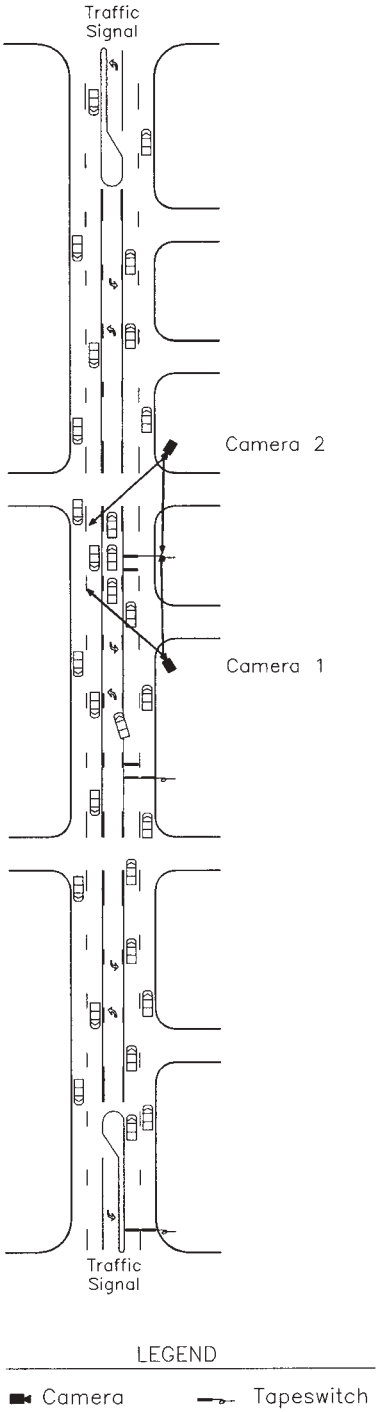

Figure 3-4. Typical data collection equipment locations.

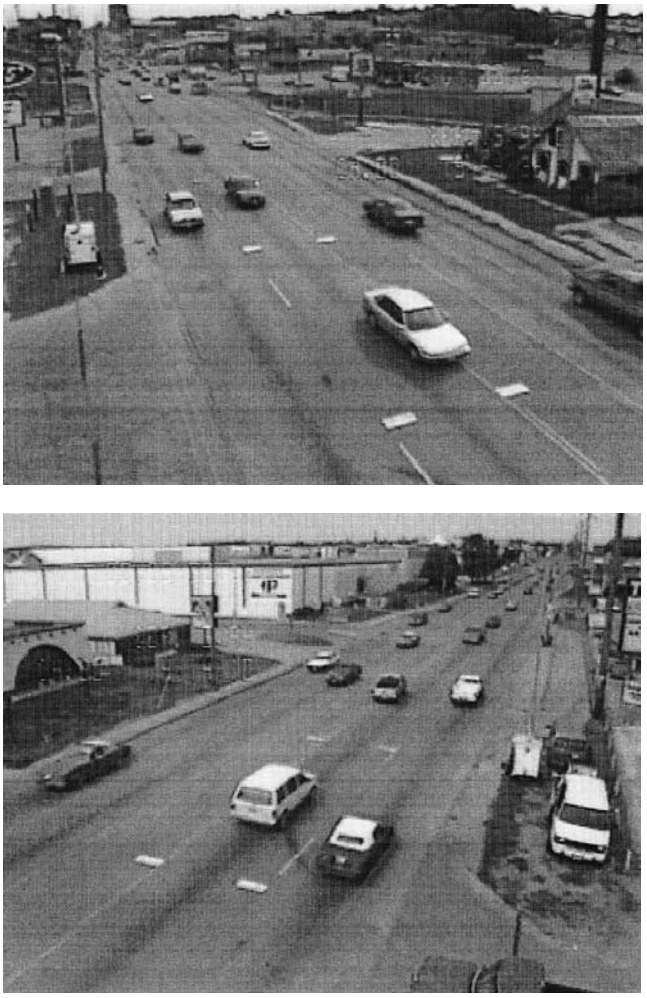

Figure 3-5. Typical fields of view for the camera locations shown in Figure 3-4: (above) Camera 2; (below) Camera 1.

flow rate and blockage status data were collected during the 2-hr peak traffic demand period at each of seven study sites.

Through Movement Speed and Volume Data

Through lane speed and volume data were obtained from the tape switch sensors located upstream of the high-volume left-turn location. Data collected included the speed and headway of each through vehicle in the inside through lane. These data were collected during the peak traffic demand hour at seven study sites.

Bay Entry Speed and Deceleration Data

Data describing the left-turning vehicle's speed and deceleration just prior to bay entry were obtained from the video cameras and tape switch sensors located upstream of the bay entrance (or its equivalent). The data collected for this database included characteristics necessary to predict the speed and deceleration rate of the left-turning vehicle as it enters the left-turn bay (or lane). Specific data collected included the speed and deceleration rate of each left-turning vehicle and the number of vehicles queued in the left-turn bay. The speed and deceleration data were collected using the tape switch sensors, whereas the number of queued vehicles was obtained

from the videotape recordings. These data were collected during the peak traffic demand hour at seven study sites.

Lane-Change Gap Acceptance Data

Lane-change gap acceptance data were obtained using the videotape recordings. Video cameras were used to record the size of the gap accepted by a vehicle changing lanes. Only lane changes stemming from left-turn-related blockage in the inside lane were studied. Sites with divided cross sections rarely had any blockage in the inside through lane and hence very little lane changing. As a result, data collection focused on sites with undivided cross sections.

The data collected include a measure of the duration of the accepted gap and an indication of the speed of the vehicle accepting the gap. The method of gap measurement was based on a technique described by Worrall and Bullen (*21*). The accepted gap was divided into two parts: (1) the time between the leading vehicle and the vehicle accepting the gap and (2) the time between the accepting vehicle and the following vehicle. In general, one of these two parts was critical during each lane change, whereas the other was excessively long and inconsequential. These parts were then combined to determine the critical gap. Data were collected during the peak traffic demand hour at four study sites. The four sites chosen represent sites with an undivided cross section and the highest left-turn demands.

Data Reduction

Approach

Data reduction focused on assembling three separate databases. Collectively, these three databases include the data needed to calibrate the five models identified in Table 3-10. The three databases are the (1) left-turn queue length database, (2) through lane characteristics database, and (3) lane-change gap acceptance database. The nature and status of the data reduction effort for each database is discussed in the following paragraphs.

Left-Turn Queue Length Database

Traffic events recorded on videotape were used to create a database of traffic characteristics needed to calibrate the probability of left-turn bay overflow model. The method of data extraction from the videotape followed that used to measure delay on intersection approaches, as described in Appendix III of Chapter 9 of the HCM. The method described in the HCM was revised slightly to accommodate the queue dynamics of an unsignalized movement. Specifically, the requirement that the vehicle be completely stopped (i.e., wheels locked) was relaxed in recognition of the continual state of motion in many unsignalized queues. This motion is particularly noticeable in the left-turn movement from the major street because it has the highest rank of all nonpriority movements; hence, it has the first opportunity to accept a gap in the opposing traffic stream.

For this study, the left-turning vehicle was defined as "in queue" when it arrived at the back of queue (or stop line, if no queue existed) and its speed was less than 3 ft/sec (fps). Departure from the queue was defined as the moment the vehicle accepted a gap and "initiated" the turn maneuver. Turn initiation was defined as the instant the back axle crossed the left edge line of the lane from which the turn was made. It should be noted that this turn-initiation crossing point was generally about 25 ft downstream of the stop line, whereas the back-of-queue line varied from the stop line (when there was no queue) to about 25 to 50 ft upstream of the rear of the last stopped vehicle.

Data reduction required a videotape that provided a view of traffic activity in the vicinity of the left-turn bay (or lane). During video playback, several types of data that could be used to describe the character of the left-turn queue were extracted. Specific data collected included the number of left-turning vehicles queued, left-turn volume, and volume of traffic opposing the left-turn queue. Left-turn queue counts were obtained at 10-sec intervals, whereas traffic volumes were measured in 15-min intervals.

Queue length data were extracted from 1 hr of videotape for each of 16 study sites. The sites collectively included the raised-curb median, TWLTL, and undivided treatments. The number of through lanes at the sites ranged from two to six. This approach yielded queue data for 1,829 left-turning vehicles. These vehicles were opposed by about 18,900 vehicles (or 1,200 vph per site on average).

Through Lane Characteristics Database

This database contains the traffic characteristics needed for calibrating three of the models listed in Table 3-10. Collectively, these three equations predict (1) lane flow rates approaching the left-turn point when the inside through lane is blocked, (2) through movement speed and headway, and (3) bay entry speed and deceleration.

Data reduction for this database was the most complicated and time-consuming of all three databases. The complication stemmed from the combined use of tape switch and video data. The tape switch data were the primary source of vehicle characteristic data (e.g., flow rate, headway, speed, and acceleration). The videotapes were the source of information on left-turn queue length, turn movement, and blockage of the inside through lane. The primary focus of the data collection was on traffic behavior in the inside (or median) through lane; however, traffic flow rates and headways also were obtained for the other through lanes. In all cases, only one travel direction was studied at each site.

The tape switch data files were processed first and then supplemented with data extracted from the video recordings. Processing included extraction of vehicle characteristics at two reference lines. These reference lines were perpendicular to the traffic lanes and located before and after the left-turn bay entry. Specifically, one reference line (i.e., Line B) was located just before the beginning of the left-turn bay taper (or its equivalent for the TWLTL based on observation of left-turn vehicle trajectories). The second reference line (i.e., Line A) was located at the end of the left-turn bay (or lane). Computer-monitored tape switch sensors were located at each reference line to facilitate measurement of the desired characteristics.

Processing of the tape switch data files focused on a lane-by-lane and reference-line-by-reference-line basis. Initially, the tape switches in each lane at each reference line were processed to obtain the desired traffic characteristics at that location. Then, the characteristics measured at both reference lines in a given lane were combined to create a record of each vehicle's characteristics as it proceeded through the study boundaries (i.e., between Lines A and B) in that lane. Finally, these lane-specific files were combined to create one file containing the attributes of all vehicles in all traffic lanes during the study period. The tape switch data files were supplemented with video data to provide additional information regarding traffic behavior (e.g., queueing) in the vicinity of the study boundaries.

The combined database has a wide range of data describing the travel characteristics of both left-turning and through vehicles. The database can be described as "vehicle-specific" because each line entry represents a vehicle that entered the study boundary (i.e., crossing Line B). The vehicle's speed, headway, and acceleration were recorded as it crossed this line. The video data provided information about the number of vehicles in the left-turn bay (or lane) as well as the existence of a blocking queue in the inside lane.

The format of the combined database is shown in Table 3-12. The sample data shown represent the characteristics of 16 vehicles passing through the study boundaries at the 27th Street site. The first vehicle crossed Line B at 14:24:49.56 (about 25 min after 2:00 p.m.). The vehicle was in Lane 1, its two axles were spaced 8.8 ft apart, its speed was 53.5 fps, and it was about 3.25 sec behind the previous vehicle. This vehicle was destined to make a left turn from the bay (which is currently empty) and had a deceleration rate of 1.2 fpss to facilitate this turn.

The second vehicle shown in the sample data in Table 3-12 is a through vehicle; therefore, its speed was measured at both Line B (i.e., 55.2 fps) and Line A. The vehicle's speed at Line A (54.5 fps) was estimated using the difference in crossing times recorded in Fields F and M (i.e., 0.22 sec) and the speed trap length recorded in Field N (i.e., 12.0 ft). Similar times are not recorded for the third vehicle because speeds in Lane 2 (i.e., the curb lane) were not important for this study. The fields coded with multiple nines (e.g., 99.9) were intended to be ignored during processing because these codes were used whenever aberrant data or events precluded measurement of some characteristics.

The combined database includes measurements for more than 17,200 vehicles. These vehicles were observed during a 2-hr period in 16 traffic lanes at seven sites. The 2-hr period included the period of peak left-turn demand at each site. The sites collectively included the raised-curb median, TWLTL, and undivided left-turn treatments. In addition, they included four and six through-lane cross sections.

Lane-Change Gap Acceptance Database

Traffic events recorded on videotape were used to create the lane-change gap acceptance database. This database was used to calibrate the model of lane-change capacity. The theoretical form of this model follows that used to predict unsignalized movement capacity in Chapter 10 of the HCM. This capacity equation is based on the distribution of gaps in the conflicting traffic stream and the number of vehicles that can safely enter these gaps. Thus, the focus of the data reduction was to determine the size of critical gap needed by a lane-changing vehicle as a function of its speed.

The reduction procedure required a camera view of the left-turn location and a short segment of the street in advance of this location. Sites with undivided cross sections were selected for study because they had the most frequent occurrence of blockage (resulting from left turns) in the inside lane. During replay of the videotape, the gap accepted by each vehicle changing from the inside to the adjacent through lane was recorded. The vehicle's speed was defined in terms of one of three categories: (1) starting from a stop, (2) starting from a slow-moving condition, or (3) changing lanes at speed.

The gap acceptance database contains data for 277 lane changes at four sites with undivided cross sections. About half of the observed vehicles were categorized as "slow-moving" at the start of their lane changes. One-fourth of the vehicles were categorized as "stopped," and one-fourth were categorized as "at speed." Between 4 and 6 hr of videotape were reviewed for each site to obtain the desired data.

Database Summary

Approach

This section focuses on the computation of summary statistics for each database and a graphical examination of trends within the data. Some preliminary comparisons are made between the component model predictions and observed measures. The details of the model calibration and predictive ability of the calibrated models are discussed later in this chapter.

TABLE 3-12 Through lane characteristics database format

Sample Data (27th Street - raised-curb cross section)

field:

A_	B	C_____	D	E	F_____	G	H	I	J	K_	L__	M_____	N___	O___	P____	Q____	R_____
27	2	14 24 49.56	1	2	0 0 0.00	L	0	N	N	0	211	0 0 0.00	12.0	8.8	53.5	-1.2	3.25
27	2	14 24 52.20	1	2	14 24 56.03	T	1	N	N	0	211	14 24 55.81	12.0	8.5	55.2	-2.3	2.64
27	2	14 24 55.02	2	2	0 0 0.00	T	2	P	N	0	211	0 0 0.00	0.0	9.0	60.6	0.0	8.43
27	2	14 25 3.46	2	2	0 0 0.00	T	2	P	N	0	211	0 0 0.00	0.0	99.9	999.9	999.9	999.99
27	2	14 25 3.67	2	2	0 0 0.00	T	2	P	N	0	211	0 0 0.00	0.0	99.9	999.9	999.9	999.99
27	2	14 25 4.31	1	2	14 25 8.64	T	1	P	N	0	211	14 25 8.40	12.0	9.8	48.7	-0.4	12.13
27	2	14 25 14.15	2	2	0 0 0.00	T	2	N	N	0	211	0 0 0.00	0.0	9.0	37.7	0.0	999.00
27	2	14 25 21.28	1	2	0 0 0.00	L	0	N	N	0	211	0 0 0.00	12.0	12.9	33.1	-1.1	17.04
27	2	14 25 27.69	1	2	14 25 31.71	T	1	P	N	0	211	14 25 31.48	12.0	10.0	52.7	-1.2	6.34
27	2	14 25 29.84	2	2	0 0 0.00	T	2	N	N	0	211	0 0 0.00	0.0	9.0	47.3	0.0	15.64
27	2	14 25 32.10	1	2	14 25 36.32	T	1	N	N	0	211	14 25 36.08	12.0	8.0	49.8	-1.5	4.37
27	2	14 25 35.17	1	2	0 0 0.00	L	0	N	N	0	211	0 0 0.00	12.0	11.0	45.4	-3.0	3.13
27	2	14 25 39.05	1	2	14 25 43.82	T	1	P	N	0	211	14 25 43.55	12.0	10.0	44.6	-0.1	3.86
27	2	14 25 41.72	1	2	14 25 46.68	T	1	P	N	0	211	14 25 46.40	12.0	8.8	41.4	-0.0	2.63
27	2	14 25 43.77	1	2	14 25 48.85	T	1	P	N	0	211	14 25 48.57	12.0	7.3	38.2	-1.0	2.00
27	2	14 25 45.18	1	2	14 25 49.91	T	1	P	N	0	211	14 25 49.63	12.0	9.9	45.7	-1.9	1.49

Field	Description	Units	Line[3]
A	**Site code** - indicates city and street location, median type, etc.	na	B
B	**File sequence number** (1, 2, or 3)	na	B
C	**Time front axle crosses Line B** (i.e., line just before left-turn lane entry point)	MT[1]	B
D	**Traffic lane[2] occupied while crossing Line B**	na	B
E	**Number of axles**	ea	B
F	**Time front axle crosses Reference Line A** (throughs only)	MT[1]	A
G	**Movement type upon crossing Line A** - L=left-turn, T=through, R=right-turn	na	A
H	**Traffic lane[2] occupied while crossing Line A** (throughs only)	na	A
I	**Left-Turn Lane Status** - P = one or more vehicles stopped in left-turn bay or lane N = no vehicles stopped in left-turn lane	na	A
J	**Lane[2] 1 Status** - B = blocked by stopped vehicle (stopped due to left turn) N = approaching vehicle not blocked C = congestion due to spillback or other condition invalidating data	na	B
K	**No. of vehicles stopped in left-turn bay** and/or Lane[2] 1 between end of bay and Line B	ea	B
L	**Distance between Lines A and B**	feet	B
M	**Time front axle crosses second line that forms speed trap[4] with Line A** (see Field F)	MT[1]	A
N	**Trap[4] length** (i.e., distance between Line A and second line, see Fields F and M)	feet	A
O	**Wheelbase** (computed for Lane 1, 9.0 ft entered as estimate for Lanes 2 and 3)	feet	B
P	**Speed at Line B** (computed for Lane 1, based on 9.0-ft wheelbase for Lanes 2 and 3)	fps	B
Q	**Acceleration at Line B** (computed for Lane 1, 0.0 fpss entered for Lanes 2 and 3)	fpss	B
R	**Headway at Line B** (measured between back axles of subject and preceding vehicles)	sec	B

Notes:

1 - MT (Military Time): successive columns in this field contain Hours, Minutes, and Seconds.

2 - Through lanes are numbered from 1 to 3 starting with the inside (or median) lane and increasing to the curb lane.

3 - Reference Line B is located in advance of the bay taper (or equivalent). Line A is located at the end of the bay.

4 - A short speed trap was located near the end of the left-turn bay in the through lane adjacent to the lane from which left-turns were made (i.e., Lane 2 for undivided cross sections and Lane 1 for all others).

Left-Turn Queue Length Database

The analysis of the left-turn queue length database focused on the computation of summary queueing statistics and the distribution of queue length. The summary queue statistics are shown in Table 3-13. As this table indicates, there is a wide range in left-turn demand and queue length at the 16 study sites. The number of left turns observed ranged from 18 to 394 during the study hour, with an average of 114 left turns per site. The average queue length ranged from 0.04 to 1.80 vehicles, with a maximum queue of 13 observed at one site.

The nature of the data extraction method facilitated the computation of percent stopping and delay statistics. As mentioned previously, a stopped or delayed vehicle was defined as a vehicle in queue or moving up in queue at a speed of less than 3.0 fps. The percent stopping ranged from 31 to 86 percent, with an average of 61

TABLE 3-13 Left-turn queue length database summary

Study Type	Study Site Location	Median Type[1]	Left-Turn Obs.	Avg. Queue Length (veh)	Max. Queue Length (veh)	Percent Stops (%)	Total Delay[2] (s/v)
Macro-scopic	**27th St.** - Vine to Holdrege St.	RM	47	0.07	2	51	5.5
	Metcalf Ave. - 91st to 93rd St.	RM	88	0.58	4	86	23.9
	Camelback Rd. - 22nd to 24th St.	RM	18	0.06	2	83	12.8
	Holdrege St. - 33rd to 38th St.	TWLTL	64	0.04	2	31	2.2
	72nd St. - Pacific to Jones St.	TWLTL	124	0.76	5	85	21.9
	Arizona Ave. - Elliot to Warner Rd.	TWLTL	55	0.14	2	55	8.9
	Roosevelt Rd. - Westmore/Meyers to Mall	FM	20	0.10	2	70	17.5
	Harlem Ave. - Montrose/Agatite to Wilson	NM	64	0.26	5	61	14.8
Micro-scopic	**Roe St.** - 110th to I-435	RM	394	1.80	9	75	16.5
	Chandler Blvd. - Beck to Kyrene Mall	RM	340	1.36	13	63	14.4
	48th St. - Normal to Van Dorn	TWLTL	221	0.53	5	58	8.6
	Thomas Rd. - 11th to 15th Ave.	TWLTL	52	0.07	2	54	5.0
	Roosevelt Rd. - Finley to Main St.	FM	78	0.38	4	79	17.6
	Roosevelt Rd. - Warrenville/West to Main	FM	45	0.06	3	38	5.1
	84th St. - F to L St.	NM	48	0.11	2	50	7.9
	87th Pkwy. - Grant to Grandview	NM	171	0.18	4	36	3.7
		Average:	114	0.41	4	61	11.6

Notes:
1 - Midblock left-turn treatment type: RM = raised-curb median, TWLTL = flush median with two-way left-turn lane delineation, FM = flush median with alternating left-turn bays, NM = undivided cross section (no median).
2 - Delay based on the count of vehicles in queue that were either stopped or moving up in queue at speeds of 3.0 fps or less.

percent. The delay ranged from 2.2 to 23.9 sec per vehicle. This range of delays corresponds to levels of service ranging from A to D, based on Table 10-3 of the HCM.

The queue length distributions for four of the study sites are shown in Figure 3-6. Collectively, these sites illustrate the queue length distribution for left-turn movements with a low to moderate degree of queueing and a v/c ratio as high as 0.50. The predicted queue length distribution for the 72nd Street site also is shown in Figure 3-6. This trend line is provided to illustrate the predictive ability of the probability of bay overflow model.

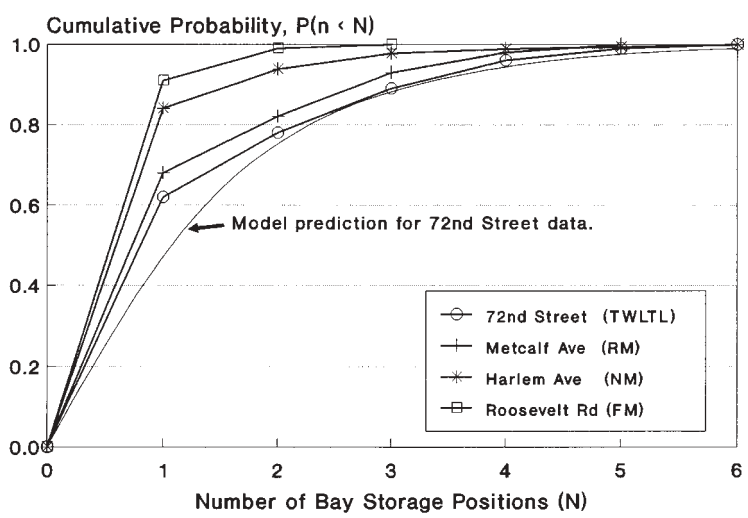

Figure 3-6. Queue length distribution at selected sites.

Through Lane Characteristics Database

Analysis of the through lane characteristics database was divided into three topic areas, which relate to the component models defined in Table 3-10:

1. Lane flow rate approaching the left-turn point during downstream blocked/unblocked conditions
2. Through movement speed and volume
3. Bay (or lane) entry speed and deceleration.

Relevant summary statistics of the data corresponding to each topic area as well as findings from a preliminary analysis of these data are provided in the following paragraphs.

Lane Flow Rates. The analysis of lane flow rates focused on the distribution of major-street vehicles among the through lanes approaching an unsignalized intersection (this intersection is formed by an access point and the major street). It was theorized that the distribution of vehicles among the approach lanes would be sensitive to the type of midblock left-turn treatment, left- and right-turn percentage, number of through lanes, left-turn capacity, and frequency of bay overflow.

Sites with undivided cross sections were found to experience "bay overflow" most frequently. This term is applied figuratively in the sense that every queued left-turning vehicle on an undivided cross section is equivalent to a bay overflow condition. Observations at the sites with this left-turn treatment indicated that the portion of through vehicles in the outside lane depended strongly on the frequency and extent of bay overflow. Through drivers in the inside lane tended to shift to outside lanes during overflowed (or blocked) conditions to avoid lengthy delays.

Table 3-14 summarizes the vehicle counts for a 2-hr period at seven sites. The inside through lane was blocked by left-turning vehicles at only one site—the site with an undivided cross section. Traffic demands were not sufficiently high to precipitate frequent bay overflow at the other six sites. Nevertheless, there were sufficient data at the undivided site to quantify the effect of blockage on the distribution of vehicles among the through lanes.

The data in Table 3-14 indicate that the percentage of vehicles in the inside lane does vary with lane status (i.e., blocked or not blocked). Specifically, during unblocked conditions, the data indicate that vehicles are almost evenly distributed among the available lanes. If anything, there is a slight trend toward more vehicles in the inside lane at the sites with a divided cross section (i.e., RM, TWLTL, and FM). This trend suggests that through drivers may prefer the inside lane at divided sites (perhaps because they want to avoid delays resulting from right-turning vehicles). In contrast, the data indicate that many drivers shift to the outside lane during blocked conditions, presumably to avoid delays resulting from left-turning vehicles.

The issue of lane flow distribution was more closely examined by segregating the database into flow characteristics for 15-min intervals. The results of this analysis are shown in Figure 3-7. This figure illustrates the relationship between the inside lane and left-turn movement percentages during unblocked and blocked conditions at sites with four through lanes.

TABLE 3-14 Through lane characteristics database (lane volume summary)

Study Site Location	Median[1] Type	Thru Lanes	Lane[2] Status	Vehicles[3]			Percent	
				V_L	V_I	V_T	V_I/V_T	V_L/V_T
27th St. - Vine to Holdrege St.	RM	4	N	76	911	1,738	52.4	4.4
Metcalf Ave. - 91st to 93rd St.	RM	4	N	140	1,195	2,454	48.7	5.7
Camelback Rd. - 22nd to 24th St.	RM	6	N	52	1,199	3,423	35.0	1.5
72nd St. - Pacific to Jones St.	TWLTL	4	N	177	1,251	2,292	54.6	7.7
Arizona Ave. - Elliot to Warner Rd.	TWLTL	6	N	88	737	1,922	38.3	4.6
Roosevelt Rd. - Westmore/Meyers to Mall	FM	4	N	46	1,691	3,371	50.2	1.4
Harlem Av . - Montrose/Agatite to Wilson	NM	4	N	75	818	1,667	49.1	4.5
			B	21	106	360	29.4	5.8
Average (four-lane sites, not blocked):				103	1,173	2,304	51.0	4.7
Average (six-lane sites, not blocked):				70	968	2,673	36.7	3.0

Notes:

1 - Midblock left-turn treatment type: RM = raised-curb median, TWLTL = flush median with two-way left-turn lane delineation, FM = flush median with alternating left-turn bays, NM = undivided cross section (no median).

2 - Inside through lane status: N = not blocked by left-turning vehicles; B = blocked by left-turning vehicles.

3 - Traffic counts on the major-street approach to the unsignalized intersection studied during the two-hour study period. V_L = left-turn count, V_I = count of left-turn and through vehicles in inside through lane, V_T = count of all vehicles in all lanes.

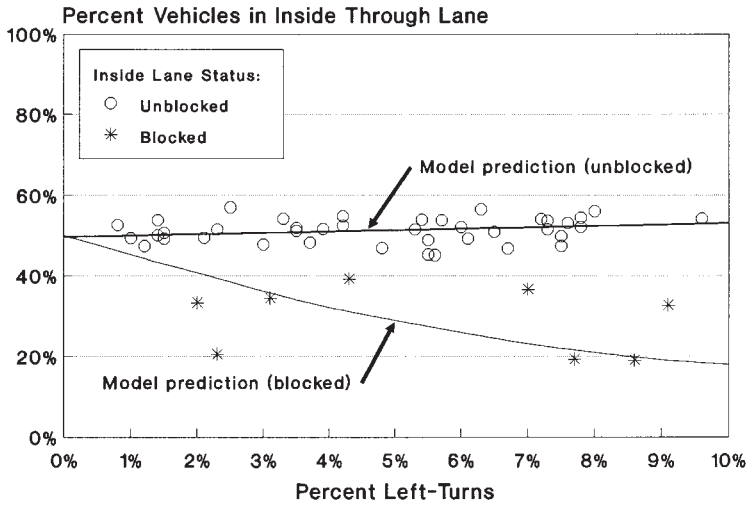

Figure 3-7. Effect of left-turn percentage and blockage on the distribution of traffic to the inside through lane.

As the data in Figure 3-7 indicate, the percentage of vehicles in the inside lane is relatively insensitive to left-turn percentage during unblocked conditions. This trend suggests that drivers distribute themselves evenly among the lanes when there are no potential disruptions (or delays) resulting from turning vehicles. In contrast, the data suggest that traffic is unevenly distributed during blocked conditions. Under these conditions, through drivers change from the inside lane to the outside lane to avoid the delay associated with left-turn-related queues.

The lines shown in Figure 3-7 represent the inside lane percentage predicted using a model developed for this research. Specifically, this equation predicts the flow rate in each through lane based on the assumption that drivers want to minimize their travel time. A description of this model and an assessment of its predictive ability is provided later in this chapter.

Through Movement Speed. The through lane characteristics database contains data on the relationship between speed and headway in the inside through lane at seven sites. These data were used to examine the relationship between speed and flow rate during unblocked conditions.

The speed, headway, and flow rate data in the through lane characteristics database are summarized in Table 3-15. As this table indicates, a wide range of speeds and flow rates were found at these sites. In particular, the average speed ranged from 40.9 to 62.8 fps, with an average of 52.0 fps. The average flow rate ranged from 474 to 878 vphpl, with an average of 632 vphpl.

The relationship between speed and flow rate at two sites is illustrated in Figure 3-8. Data for the Camelback Road site were collected at a point 450 ft downstream of the traffic signal. The average vehicle at this site tended to have a small

TABLE 3-15 Through lane characteristics database (speed and flow rate summary)

Study Site Location	Median[1] Type	Thru Lanes	No. Obs.	Speed, fps		Headway, sec		Flow Rate, vphpl
				Avg.	S.D.[2]	Avg.	S.D.[2]	
27th St. - Vine to Holdrege St.	RM	4	816	48.1	7.3	5.9	6.8	610
Metcalf Ave. - 91st to 93rd St.	RM	4	1,035	53.4	7.7	5.7	8.9	632
Camelback Rd. - 22nd to 24th St.	RM	6	1,111	51.4	5.9	5.9	9.5	610
72nd St. - Pacific to Jones St.	TWLTL	4	1,024	50.3	6.6	4.3	5.5	837
Arizona Ave. - Elliot to Warner Rd.	TWLTL	6	614	62.8	9.2	7.6	9.4	474
Roosevelt Rd. - Westmore/Meyers to Mall	FM	4	1,585	57.0	7.1	4.1	5.5	878
Harlem Ave. - Montrose/Agatite to Wilson	NM	4	668	40.9	7.6	6.4	9.2	563
Average (Not blocked only):			979	52.0	7.3	5.7	7.8	632

Notes:

1 - Midblock left-turn treatment type: RM = raised-curb median, TWLTL = flush median with two-way left-turn lane delineation, FM = flush median with alternating left-turn bays, NM = undivided cross section (no median).

2 - Standard deviation.

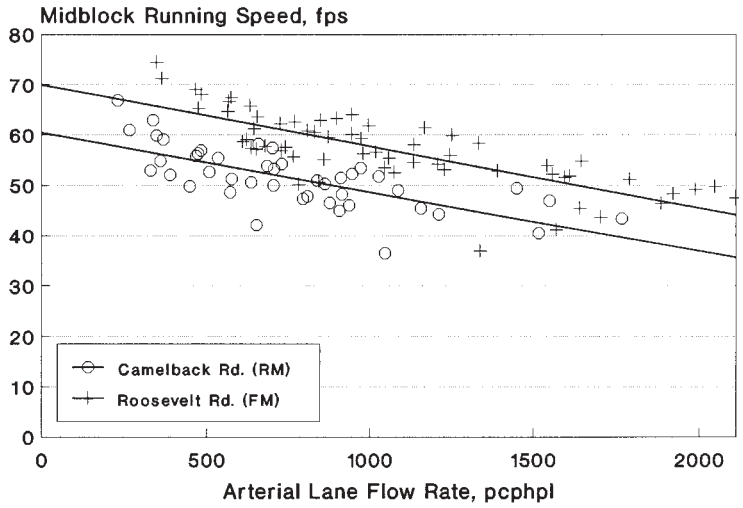

Figure 3-8. Effect of flow rate on midblock running speed at two study sites.

acceleration, indicating that it had not reached the desired running speed by the point of measurement. The data for the Roosevelt Road site was measured about 1,460 ft downstream of the traffic signal and exhibited no overall trend toward acceleration by the average vehicle. It should be noted that the data points shown represent the average of 25 observations. The technique of plotting average values was used to remove some of the scatter inherent in the individual observations and thereby facilitate the examination of trends.

The data shown in Figure 3-8 indicate a definite trend toward reduced speed with increasing volume. This trend is consistent with that found for multilane highways (as shown in Figure 3-2), although the speed reduction found in Figure 3-8 data is more significant and generally occurs over the entire range of flow rates. The data in Figure 3-8 also suggest that a maximum flow rate of about 2,000 pcphpl is possible on urban arterials, especially if the distance between traffic signals is sufficiently long to allow the platoon to reach its desired running speed.

An examination of speed and flow rate data at the other study sites indicated that the correlation between speed and flow rate was not always as strong as that shown in Figure 3-8. Specifically, sites with higher levels of access point activity were found to have more variability in their speed-flow data, yielding less clarity in the linear-decreasing trend. This additional variability is attributed to the turbulence associated with the higher degree of turning activity at these sites.

Bay Entry Speed and Deceleration. One of the methodologies in the operations model predicts the effect of turning vehicles on major-street through traffic. This effect stems from the turning vehicle's speed reduction in the through lane prior to completing its turn from the through lane or its entry into the turn bay. In the latter case, the impact can be significant if the bay is relatively short or frequently filled with queued vehicles so that some deceleration in the through lanes is necessary. Quantification of this effect requires information on the basic speed and deceleration behavior of drivers as they enter the turn bay or lane. These data were collected at seven of the study sites.

The deceleration behavior of the left-turning drivers at the seven sites is shown in Table 3-16. As this table indicates, average deceleration rates ranged from 0.47 to 3.89 fpss at the study sites. At first glance, this range would seem relatively large; however, it must be paired with the approach speed and remaining distance available for deceleration for proper interpretation. The available deceleration distance was computed as the distance between the point of deceleration measurement (i.e., Line B) and the stopping point. The stopping point was defined as the end of the bay or the back of the last queued vehicle, whichever was farther upstream. Using these definitions, the driver's deceleration rate at the bay entry point was statistically and graphically evaluated relative to speed and distance to queue. The results of this evaluation are shown in Figure 3-9.

As Figure 3-9 indicates, drivers do not choose a constant deceleration rate as they enter the left-turn bay or lane. Rather, they adopt a deceleration rate based primarily on their bay entry speed and, to a lesser extent, on the remaining distance within which they have to stop. More specifically, Figure 3-9a indicates that drivers adopt a deceleration rate that varies from 0.5 to 3.5 fpss for speeds of 45 to 60 fps. At speeds below 45 fps, it appears that a value of about 0.5 fpss is acceptable (of course, a higher rate would have to be adopted subsequent to bay entry to stop the vehicle in any reasonable remaining distance). It should be noted that the data points in this figure each represent averages of 10 or more observations. This technique of plotting average values was used to remove some variability in the data and

TABLE 3-16 Through lane characteristics database (left-turn vehicle deceleration summary)

Study Site Location	Median[1] Type	Thru Lanes	Left-Turns	Queue[2], veh	Decel.[3] Dist, ft	Speed, fps	Decel, fpss
27th St. - Vine to Holdrege St.	RM	4	72	0.06	270	48	0.83
Metcalf Ave. - 91st to 93rd St.	RM	4	141	0.14	315	53	0.47
Camelback Rd. - 22nd to 24th St.	RM	6	47	0.04	210	51	0.77
72nd St. - Pacific to Jones St.	TWLTL	4	175	0.71	337	50	1.12
Arizona Ave. - Elliot to Warner Rd.	TWLTL	6	86	0.10	250	63	3.89
Roosevelt Rd. - Westmore/Meyers to Mall	FM	4	45	0.11	270	57	2.19
Harlem Ave. - Montrose/Agatite to Wilson	NM	4	126	3.46	315	35	0.95
		Average:	99	0.66	281	51	1.46

Notes:
1 - Midblock left-turn treatment type: RM = raised-curb median, TWLTL = flush median with two-way left-turn lane delineation, FM = flush median with alternating left-turn bays, NM = undivided cross section (no median).
2 - Average number of vehicles queued in the left-turn bay when a left-turn vehicle arrives at the bay entrance.
3 - Distance between the tape switch at Reference Line B (i.e., bay entry point) and the end of the bay or lane.

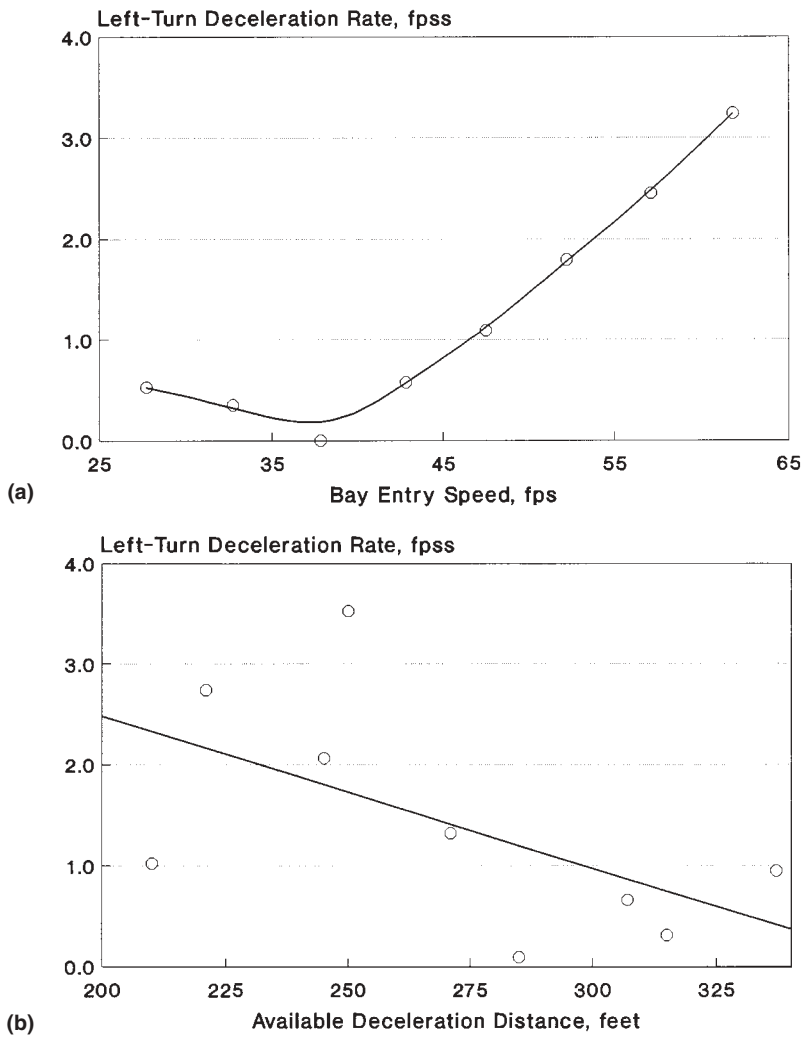

Figure 3-9. Effect of speed and deceleration distance on deceleration rate: (a) effect of bay entry speed on deceleration rate; (b) effect of available deceleration distance on deceleration rate.

thereby to facilitate the graphical examination of underlying trends.

Figure 3-9b indicates that drivers also modify their deceleration rate based on the remaining stopping distance. Specifically, drivers accept higher deceleration rates when they have shorter distances. There is considerably more variation in the relationship between distance and deceleration than between speed and deceleration. This additional variability suggests that distance is not as dominant, nor as uniformly applied, a criterion as is speed in selecting a deceleration rate.

Lane-Change Gap Acceptance Database

The analysis of the lane-change database focused on determining the magnitude of the critical gap needed by a lane-changing driver. In this context, the driver evaluates headways in the adjacent through traffic lane, factors in the relative speed between his or her vehicle and those in the adjacent lane, and determines the adequacy of each headway in terms of a safe lane change. The impetus for this lane change is the presence of a left-turning vehicle blocking the inside lane.

As mentioned previously, the method of gap measurement was based on a technique described by Worrall and Bullen (*21*). Specifically, the accepted gap was divided into two parts: (1) the time between the leading vehicle and the vehicle accepting the gap and (2) the time between the accepting vehicle and the following vehicle. In general, one of these two times is critical during each lane change, whereas the other is excessively long and inconsequential. These critical times can then be combined to determine the critical gap.

This type of study differs from the traditional gap acceptance study because it is impossible to determine which gaps a driver rejects. The determination of critical gap can be based only on the size of the accepted gaps. Many accepted gaps can be quite large as a result of random headways in the traffic stream; therefore, critical gap must be established as a minimum value at which headways below this value would be acceptable to relatively few drivers. Worrall and Bullen (*21*) suggested the use of the fifth percentile value. Traditional gap acceptance studies that define critical gap as the value that is rejected by as many drivers as accept it tend to yield values that equate to about the 15th percentile of accepted values.

The combined distribution of accepted gaps measured at four study sites is shown in Figure 3-10. Separate distributions are shown for the lead, lag, and headway values that were accepted by drivers. In all cases, these distributions contain many small values and a few large values because of occasional large headways occurring in the traffic stream. The small values are fairly stable because they are dictated by driver behavior as opposed to random traffic flows.

The data in Figure 3-10 combine all observed data, regardless of the speed of the lane-changing vehicle at the start of the lane change. As a second part of this analysis, distributions were developed for each of the three speed ranges: "stopped," "slowed," or "at running speed." The results of this analysis are shown in Table 3-17.

As the data in Table 3-17 indicate, lead times tend to be shorter than lag times, suggesting that merging drivers want more time between themselves and the vehicle behind. The data also indicate that drivers merging from the stopped condition tend to require the largest time interval, whereas those merging at a slowed speed condition require the shortest time. Moreover, it appears that the lead value increases with speed, whereas the lag value decreases with increasing speed. Both these trends appear reasonable in the context of drivers' desire to provide more of a buffer distance between their vehicles and the one in front of them at higher speeds and between their vehicles and the one behind when their speeds are low. These trends are consistent with drivers' perceptions of the greatest risk at the respective speeds. In com-

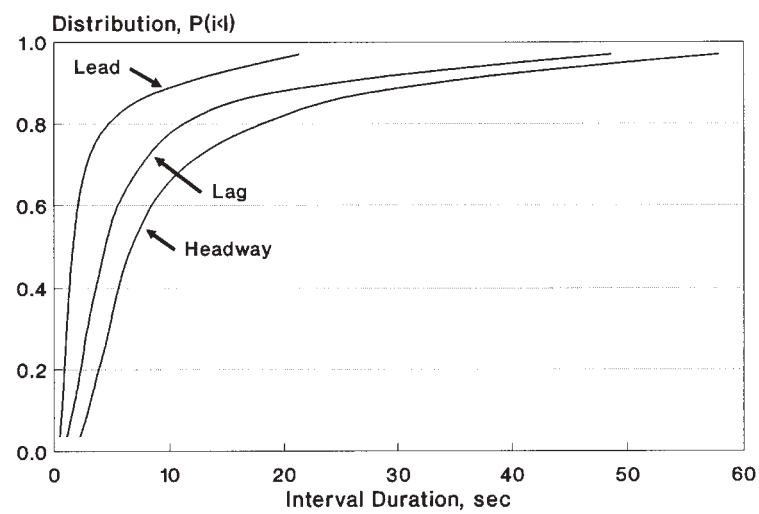

Figure 3-10. Distribution of accepted headways at four sites.

TABLE 3-17 Lane change gap acceptance database summary

Percentile Value	Speed Condition[1]	Interval, seconds			
		Lead	Lag	Lead+Lag	Headway
5th Percentile	Stopped	0.5	1.7	2.2	3.1
	Slow	0.5	1.2	1.7	2.2
	Running Speed	0.7	1.2	1.9	2.5
	Overall	0.5	1.2	1.7	2.3
15th Percentile	Stopped	0.8	2.9	3.7	4.1
	Slow	0.7	1.9	2.6	3.0
	Running Speed	1.0	1.6	2.6	3.8
	Overall	0.8	2.0	2.8	3.5

Note:
1 - Indicator of the speed of the subject vehicle just prior to changing lanes.

bination, these trends tend to offset one another. In fact, the combined lead/lag values are very similar for the slowed and at running speed conditions.

It should be noted that the combined lead and lag times in Table 3-17 are always shorter than the headway values. This trend results because most of the accepted headways had either a lead time or lag time, but not both, that was critically short. Thus, the combined lead and lag times would be a more conservative estimate of the critical gap than the headway value because it represents the sum of both critical gap portions.

OVERVIEW OF OPERATIONS MODEL

The operations model was developed to demonstrate how two HCM analysis procedures (i.e., those in Chapters 9 and 10) could be used to evaluate the operational effects of alternative midblock left-turn treatments. Several models were developed for this research to supplement and extend the HCM procedures so that they could be used to evaluate the interaction of several access points along an urban arterial. Unfortunately, at the conclusion of operations model development, it was found to be too complex for manual applications. As a result, the operations model was implemented as a software program. The model was calibrated using the field data, and its predictive capability was verified by comparing it with other traffic models. The calibrated operations model was used to evaluate the numerous analysis scenarios that formed the basis for the midblock treatment selection guidelines described in Chapter 2.

Model Description

The operations model is based on the HCM procedures for analyzing unsignalized and signalized intersections. As such, the model can be defined as a deterministic, macroscopic model. It focuses on assessing the operation of nonpriority movements at unsignalized intersections along an arterial and the impact of these movements on the travel time of the arterial through movement. The arterial segment is assumed to be bounded by signalized intersections and to have any reasonable number of access points along its length. In the context of the HCM procedures, each of these access points is assumed to form an unsignalized intersection with the major street.

The delay to any nonpriority traffic movement is strongly influenced by the distance between the subject intersection and an adjacent unsignalized or signalized intersection. When this distance is short, relative to the length of queue that forms at the adjacent intersection, a complex series of interactions can occur that can directly or indirectly increase the delay to the movements at the subject intersection. Moreover, these interactions can be particularly disruptive when the adjacent intersection is signalized. The queues formed at signalized intersections can be lengthy and can block an upstream intersection for a sustained length of time. The operations model includes several supplemental models that account for the effect of access point spacing on arterial operations.

In addition to the distance between access points, the operations model is sensitive to several important geometric and traffic characteristics that can lead to disruption and delay to arterial traffic movements. Specifically, the model is sensitive to left-turn treatment type, intersection signal timing, traffic volumes at each access point, frequency of left-turn bay (or lane) overflow, and platoons formed by upstream signals. The left-turn treatments considered were the raised-curb median, TWLTL, and undivided cross section. The primary model outputs were the major-street left-turn delay and through movement travel speed.

The general framework of the operations model is shown in Figure 3-11. As this figure indicates, the evaluation process consists of two analysis iterations. The first iteration is performed as if the intersections are isolated; the second iteration uses the queue information from the first iteration to account for the effects of left-turn bay overflow and queue spillback from the downstream unsignalized or signalized intersection.

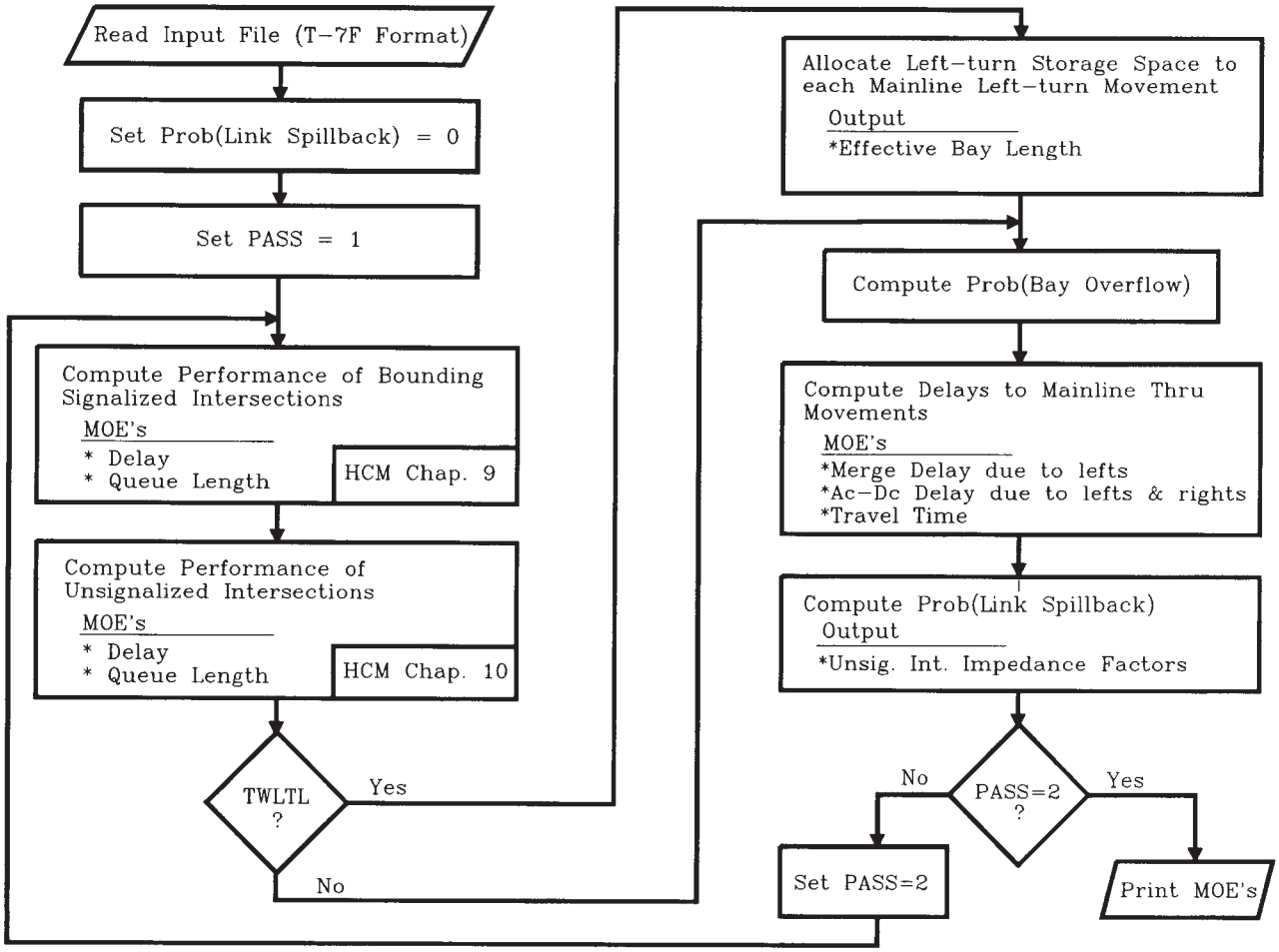

Figure 3-11. Framework of the operations model.

The operations model is based on three main modules, their associated models, and a series of component models. These modules and models follow.

Signalized Intersection Module

- Through capacity model
- Through delay model
- Queue length model.

Unsignalized Intersection Module

- Left-turn capacity model
- Left-turn delay model
- Queue length model.

Major Street Through Lane Module

- Through merge/lane-change capacity model
- Through merge/lane-change delay model
- Through slow-down delay model.

Component Models

1. Probability of left-turn bay overflow
2. Lane flow rate approaching the left-turn point for downstream blocked/unblocked conditions
3. Through movement speed and volume
4. Bay entry speed and deceleration
5. Lane-change gap acceptance
6. Probability of outbound signalized intersection approach lane blocked
7. Probability of spillback from downstream intersection (signalized or unsignalized)
8. Signalized intersection approach lane capacity during upstream blocked conditions
9. Left-turn lane allocation (for the TWLTL)
10. Traffic platoon dispersion.

The capacity and delay models listed for the signalized and unsignalized modules are based on the models recommended in Chapters 9 and 10, respectively, of the HCM. The remaining models were developed for this project and are described in the following paragraphs.

Models Associated with Main Modules

Queue Length Model. The prediction of average queue length was based on a fundamental relationship developed by Little (*22*) for queueing systems. This relationship states that the average queue length is equal to the product of the average delay and the arrival flow rate. The delay used in this product is that obtained from the delay equations provided in Chapters 9 and 10 of the HCM. This approach to queue length estimation (i.e., extending the delay estimate from the HCM) was considered preferable to deriving a queue length equation because it provides continuity with the well-founded HCM delay equations and because the predicted queues will have the same time-dependent sensitivity as the HCM equations.

Through Merge/Lane-Change Capacity Model. This model predicts the lane-change capacity available to through drivers in the inside lane. The lane-change maneuver that is modeled is that by a through driver who is traveling in the inside lane and is caught in a stopped queue associated with left-turn bay overflow (or blockage, in the case of an undivided cross section). The capacity of this maneuver is based on the distribution of available gaps in the through lane adjacent to the inside lane. This distribution is based on the flow rate in the adjacent lane when the inside lane is blocked (this flow rate is typically higher during blocked than unblocked conditions). The lane flow rate model (i.e., Component Model 2 in the previous list) is used to estimate this flow rate.

Through Merge/Lane-Change Delay Model. This model is an extension of the HCM delay equation for unsignalized traffic movements. It combines an estimate of the portion of the approaching traffic flow that will change lanes with the estimated lane-change capacity to predict the delay to lane-changing vehicles. The portion of traffic changing lanes is estimated using one of the component models (i.e., Component Model 2). The delay actually incurred is the lesser of the delay required to change lanes and the delay incurred as a vehicle waits in the left-turn-related queue.

Through Slow-Down Delay Model. This model predicts the delay incurred by through vehicles caused by the slowing (or stopping) of a turning vehicle. The model is applied to both the left-turn and right-turn movements from the major street. It incorporates probabilistic methods to estimate the number of through vehicles following each turning vehicle. An expected value formulation combines event probabilities to predict the incremental impact of the slowing (or stopping) "wave" on each following through vehicle. The sum of these incremental impacts represents the added travel time, or delay, to the through vehicles. This model is sensitive to the turning speed of the right-turning vehicle and the bay-entry speed of the left-turning vehicle (the latter parameter is obtained from Component Model 4). The model also requires an estimate of the lane flow rates approaching the turning point during unblocked conditions (i.e., Component Model 2).

Component Models

Probability of Left-Turn Bay Overflow. This model is based on a queueing theory formulation of the queue length distribution. The model relates the left-turn volume and capacity to the length of left-turn storage area to predict the probability of left-turn bay overflow. In the special case of an undivided cross section (where storage area is nonexistent), the model predicts the probability of a left-turning vehicle being queued.

Lane Flow Rate Approaching Left-Turn Point. This model predicts the flow rate in each through lane approaching the left-turn point. The model is based on the assumption that drivers choose their lanes based on a desire to minimize their travel time. In this equation, the minimum travel time is presumed to be achieved when the demand-to-saturation flow ratio of each lane is as low as possible. This condition is satisfied when the ratios for each lane are equal. This approach is similar to that used in Chapter 9 of the HCM (i.e., Equation 9-18) for computing the portion of left-turning vehicles in the inside lane of a shared-lane intersection approach. However, the equation developed for this research is derived to account for both the blocked and unblocked condition of the inside through lane as well as the percentage of right-turning vehicles.

Through Movement Speed and Volume. This model predicts the running speed of through traffic as affected by arterial volume level. The predicted running speed does not include delays caused by turning vehicles (as defined in the previous section) or those caused by the signalized intersections that bound the arterial segment.

Bay Entry Speed and Deceleration. This model predicts the bay entry speed and average "equivalent" constant deceleration rate needed by the through slow-down delay model. These characteristics are estimated using kinematic equations of motion combined with the bay length and average left-turn queue length.

Lane-Change Gap Acceptance. This model is a parametric representation of the shortest intervehicle gap accepted by the majority of drivers making a lane change. Traditional techniques of estimating the critical gap of drivers at unsignalized intersections are not applicable to the lane-change maneuver because of the impossibility of knowing when a driver rejects a gap. The measurement process also is complicated by the frequent occurrence of very large gaps as a result of randomness in the traffic stream. To overcome these measurement problems, the accepted headways have been subdivided into a lead and a lag time (one of which may

be critical). The combination of the critical lead and lag times is assumed to represent the critical lane-change gap.

Probability of Downstream Signalized Intersection Approach Lane Blocked. This model predicts the probability that the inside through lane of the downstream signalized intersection approach will be blocked by left-turn activity at an upstream unsignalized intersection.The equation is sensitive to the distance between the unsignalized intersection and the downstream signal and to the major-street left-turn volume at the unsignalized intersection. The predicted probability is used to estimate the effective number of through lanes available on the signalized intersection approach and its resulting queue length.

Probability of Spillback from Downstream Intersection. This model predicts the probability of an upstream unsignalized intersection being blocked by the spillback from a downstream intersection. This spillback effectively impedes the flow of the nonpriority movements at the subject intersection and results in a reduction in their movement capacity. The predicted probability effectively represents the union of spillback probabilities for all intersections downstream of the subject intersection (including the downstream signalized intersection).

Signalized Intersection Approach Lane Capacity During Upstream Blocked Conditions. This model predicts the capacity of the outbound through movement at the bounding signalized intersections. As discussed previously, the inside through lane on a signalized intersection approach can be blocked by left-turning vehicles at an upstream unsignalized intersection. This model computes the effective saturation flow rate for the signalized through movement based on the portion of time the inside lane is blocked. The model is applicable to signalized approaches with exclusive left-turn lanes or shared through and left-turn lanes. In the latter case, an equation was developed for estimating approach lane flow rates. This equation is based on Equation 9-18 of the HCM; however, it was modified for application to situations involving upstream lane blockage.

Left-Turn Lane Allocation. This model predicts the portion of the TWLTL that is available to each of the two major-street left-turn movements that share it. Specifically, these are the left-turn movements that occur at adjacent unsignalized intersections and whose queues extend backward toward one another, effectively competing for use of the same storage space. This model allocates available storage space based on the queue lengths of the respective left-turn movements. One interesting attribute of this model is that the sum of the effective storage space predicted for each movement typically exceeds total storage space. This characteristic results from the shared use of available space.

Traffic Platoon Dispersion. This model predicts the portion of time the major-street traffic stream is platooned as it passes through an unsignalized intersection. In this context, a platooned vehicle is defined as a vehicle having a headway shorter than the critical gap acceptable to a major-street left-turning driver. Hence, the model predicts the time required for the platoon to pass through the intersection relative to the upstream signal cycle time. During this portion of time, left-turn capacity is assumed to be negligible. Subsequent to this time, left-turn capacity is computed using the left-turn capacity model and a lesser flow rate reflecting the flow of secondary (i.e., nonplatooned) movements.

Component Model Calibration

This section describes the formulation and calibration of 5 of the 10 component models that support the various modules that comprise the operations model. The models described in this section are as follows:

1. Probability of left-turn bay overflow model
2. Lane flow rate model
3. Through movement speed model
4. Bay entry speed and deceleration model
5. Lane-change gap acceptance parameter.

The remaining five component models were developed for the operations model but were not calibrated with field data. These models were excluded from the data collection effort primarily because of the high cost of assembling a database for each model. In many instances, the corresponding flow problems tended to occur infrequently at some sites or frequently at a very small percentage of widely scattered sites. When these sites were identified, they tended not to satisfy the desired site criteria or offered no option for a combined study of other flow problems. The five component models for which calibration data were collected were deemed to represent a reasonable balance between model priorities, database assembly cost, and available project resources.

Probability of Left-Turn Bay Overflow Model

Model Development. A major-street left-turn movement that overflows its allocated storage space causes significant disruption to through traffic in the adjacent lanes. Therefore, an equation for predicting the frequency of overflow is essential to the evaluation of midblock left-turn treatments. In a probabilistic context, the frequency of overflow equates to the probability that the left-turn queue exceeds the given storage space. Hence, the equation developed to quantify this probability is based on a mathematical representation of the queue length distribution.

Analysis of the queue length distributions observed at the field study sites indicated an exponential trend of increasing probability with an increasing number of vehicles. The distribution found at four of the study sites was shown previously in Figure 3-6. The probability that there are fewer than N vehicles in queue can be read directly from the y-axis of this figure. Or, in the context of a bay with N storage positions, the probability of bay overflow is equal to the probability that there are $N + 1$ or more vehicles. This probability

can be computed by subtracting the probability associated with $N + 1$ (on the y-axis) from 1.0. For example, the Harlem Avenue site has zero storage positions as a result of its undivided cross section (i.e., $N = 0$); thus, the probability of bay overflow equals the probability that there are one or more vehicles or 0.16 (= 1.0 – 0.84).

The probability of bay overflow model developed for this research is based on the queue length distribution equation traditionally applied to queueing systems with exponential arrival and service times.The queue length distribution describes the probability of a queue being less than a prescribed number of vehicles. For the purposes of this research, the probability of bay overflow is defined as the probability of a queue exceeding available bay storage N. Based on these definitions, the queue length distribution is as follows:

$$P(n < N) = 1 - \left(\frac{v_l}{c_l}\right)^N \tag{2}$$

where:

$P(n < N)$ = probability of a queue length less than N;
n = observed queue length at any instant in time;
v_l = left-turn flow rate, vph;
c_l = left-turn capacity, vph; and
N = number of vehicles that can be stored in an exclusive turn lane without overflow.

The probability of bay overflow P_{ov} (= $P(n \geq N + 1)$) is derived from Equation 2 as follows:

$$P_{ov} = \left(\frac{v_l}{c_l}\right)^{N+1} \tag{3}$$

The capacity required by Equations 2 and 3 was predicted using a capacity model similar to that recommended in Chapter 10 of the HCM. This capacity model was modified to account for the effects of traffic platoons formed by upstream signalized intersections.

Model Calibration. The data collected during the field study were used to calibrate the queue length distribution model in Equation 2. The queue length distributions for the study sites were used for this purpose. The capacity for each site was estimated using the observed opposing through traffic volumes in combination with the modified left-turn capacity model.

The fit of the proposed model to the distribution of the queue lengths at the study sites is shown in Figure 3-12. This figure compares the predicted and measured queue length probabilities. As the data in this figure indicate, the model predictions are in good agreement with the observed probabilities over a wide range of values. Based on the quality of this fit, no empirical calibration constants were deemed necessary for Equation 2 or 3.

Sensitivity Analysis. Equation 3 can be used to examine the effect of bay storage length and left-turn v/c ratio on the probability of bay overflow. This relationship is shown in Figure 3-13. As this figure indicates, the probability of overflow increases with increasing v/c ratio; it also increases with a decreasing number of available queue storage positions.

Lane Flow Rate Model

Model Development. The delay to major-street through vehicles caused by turning activity is highly dependent on the

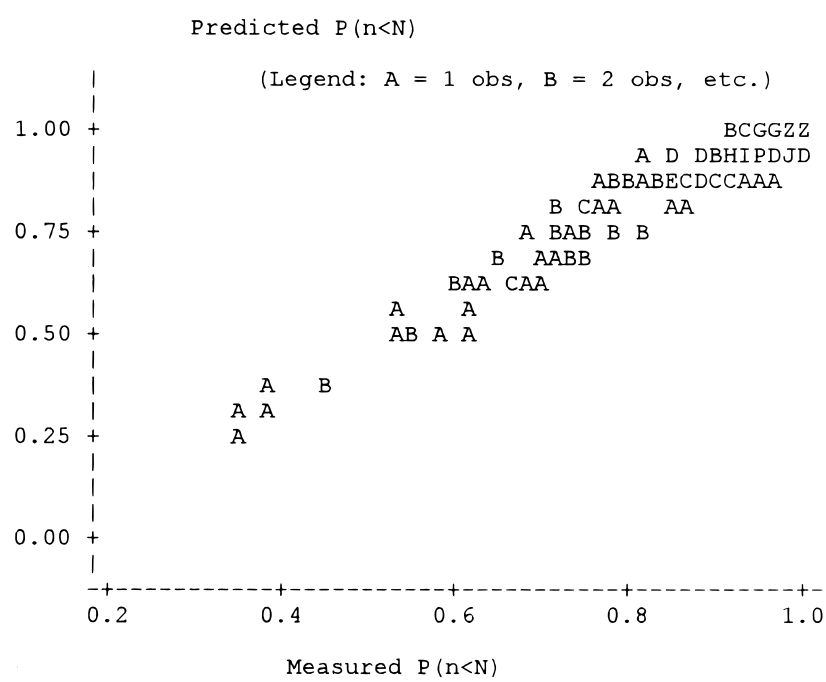

Figure 3-12. Comparison of predicted and measured queue length probabilities.

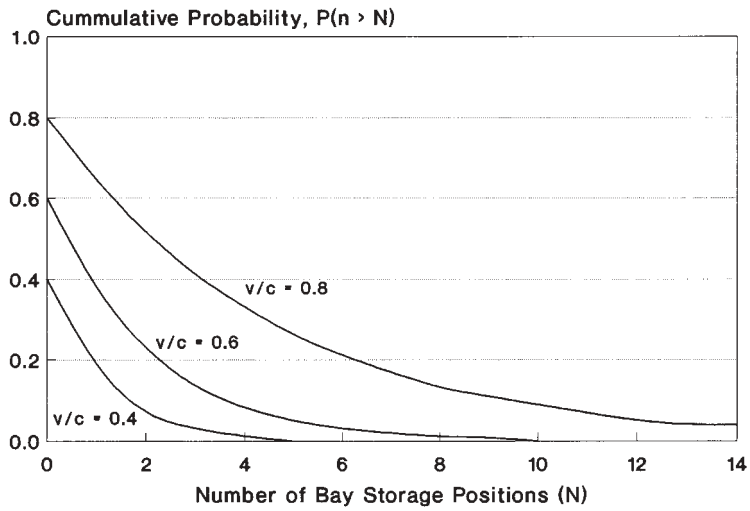

Figure 3-13. Predicted effect of volume-to-capacity ratio and bay storage length on the probability of bay overflow.

distribution of through vehicles among the available lanes in advance of the access point. The nature and extent of this delay is determined by whether a turn bay or lane is provided and whether the queue of turning vehicles overflows the available storage area and the speed the turning vehicle slows to prior to entering the bay.

Consider the case where no left-turn bay is provided (or a bay is provided but is filled). Prior to the arrival of a blocking left-turning vehicle, lane flow rates tend to be somewhat balanced as drivers attempt to minimize their travel time through the unsignalized intersection. An exact balance may not be possible because some through drivers will choose to avoid the threat of delay caused by turning vehicles by concentrating in any exclusive through lanes that may be available.

Just after the arrival of a blocking left-turning vehicle, the following through vehicles will attempt to merge into an adjacent through lane. Many through drivers will be able to complete this merge without stopping; drivers who stop will have to wait until the turning vehicle departs or until it is safe to merge into the adjacent lane. The capacity of the stopped merge maneuver depends on the flow rate in the adjacent traffic lane while the turning vehicle is present or is blocking the inside through lane. The effect of left-turn activity and blockage on the distribution of vehicles among the through lanes was shown previously in Figure 3-7.

Consider now the case where there are turn bays of adequate storage capacity to prevent overflow. In this case, delays are possible to following through traffic if the deceleration distance in the storage bay is not sufficient for the driver to slow completely within the bay. The extreme of this situation is a right-turn movement that has no bay at all. In this situation, the right-turning driver will slow to the turning speed entirely in the through lane. When there is the potential for this type of delay, through drivers more frequently may choose to position themselves in the nonturning lanes on the intersection approach to avoid possible delays caused by turning vehicles.

The modeling approach used to replicate the distribution of through drivers among traffic lanes is based on the hypothesis that through drivers will choose the traffic lane that minimizes their travel time. One mathematical representation of this hypothesis is the equalization of the demand-to-saturation flow ratios among alternative traffic lanes. In this context, turning vehicles are represented as "equivalent" through vehicles when computing the saturation flow rate of the shared through lane. This representation was adopted in the development of Equation 9-18 of the HCM for computing the portion of left-turning vehicles in the inside lane of a shared-lane intersection approach. It is also applicable to traffic flow on an unsignalized intersection approach and can be formulated as follows:

$$\frac{v_i}{s_i} = \frac{\sum_{i=1}^{n} v_i}{\sum_{i=1}^{n} s_i} \tag{4}$$

where:

v_i = flow rate in lane i ($i = 1, 2, ..., n$; $i = 1$ for the inside or median lane), vphpl;
s_i = saturation flow rate in lane i, vphpl; and
n = number of through lanes on the subject lane ($n \geq 2$).

The proportion of left-turning vehicles in the inside through lane (i.e., Lane 1) can be quantified as follows:

$$P_L = \frac{P_{LT} \sum v_i}{v_1} \tag{5}$$

where:

P_L = proportion of left-turning vehicles in the inside lane flow (Lane 1); and
P_{LT} = proportion of left-turning vehicles in the approach flow.

Similarly, the proportion of right turns in the outside through lane (i.e., Lane n) can be quantified as follows:

$$P_R = \frac{P_{RT} \sum v_i}{v_n} \tag{6}$$

where:

P_R = proportion of right-turning vehicles in the outside lane flow (Lane n); and
P_{RT} = proportion of right-turning vehicles in the approach flow.

Combining Equations 4 and 5 yields the desired relationship for predicting the proportion of left-turning vehicles in the inside lane under "equilibrium" conditions (i.e., $v_1/s_1 = v_2/s_2 = \ldots v_n/s_n$):

$$P_L = P_{LT} \frac{\sum s_i}{s_1} \tag{7}$$

Using a similar approach, the proportion of right turns in the right-turn lane can be computed as follows:

$$P_R = P_{RT} \frac{\sum s_i}{s_n} \tag{8}$$

For a mixture of through and turning vehicles, the effective saturation flow rate of the inside lane, outside lane, and middle lane can be computed using the following three equations:

$$s_1 = \frac{s_0(1 + P_L I_t)}{1 + P_L(E_L - 1) + (P_L E_L I_t)} \tag{9}$$

$$s_n = \frac{s_0}{1 + P_R(E_R - 1)} \tag{10}$$

$$s_m = s_0(n - 2) \tag{11}$$

where:

s_1 = saturation flow rate for the inside lane, vphpl;
s_n = saturation flow rate for the outside lane, vphpl;
s_m = saturation flow rate for *all* middle lanes (if any), vph;
s_0 = saturation flow rate for a through stream (e.g., 1,800 vphpl);
I_t = indicator variable (1.0 when left-turning vehicles block the through lane; 0.0 otherwise);
E_L = through vehicle equivalent for a left-turning vehicle ($= s_0/c_l$);
c_l = left-turn capacity, vph; and
E_R = through vehicle equivalent for a right-turning vehicle.

Equation 10 is a variation of Equation 9-13 in the HCM. It yields the effective saturation flow rate of a mixed (i.e., through and right-turn) traffic lane using a proportion-based weighting of the headways of the two vehicle types. Equation 9 is a slight variation of Equation 10. It represents a more general version in which the effect of a left turn blocking the through lane can be considered (i.e., $I_t = 1.0$) as well as the more common case of no left turn blocking the through lane (i.e., $I_t = 0.0$). As mentioned previously, the "blocking" case predicts the lane flow rate *given that a blockage exists*. The "nonblocking" case requires no preconditions.

Equations 7 through 11 can be combined to yield equations for predicting the proportion of left- and right-turning vehicles in each lane (i.e., P_L and P_R). The simultaneous solution of these equations yields the following equations for predicting P_L and P_R:

$$P_L = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \leq 1.0 \tag{12}$$

with

$$a = RI_t \tag{13}$$

$$b = R - P_{LT}[I_t + (n - 1)(E_L[1 + I_t] - 1)] \tag{14}$$

$$c = -P_{LT} n \tag{15}$$

$$R = 1 + P_{RT}(E_R - 1) \tag{16}$$

and

$$R_R = P_{RT} \frac{\frac{s_1}{s_0} + n - 1}{1 - P_{RT}\left(\frac{s_1}{s_0} + n - 2\right)(E_R - 1)} \leq 1.0 \tag{17}$$

Equations 12 and 17 can be used to estimate the proportion of through vehicles in each lane. These equations then can be combined with Equations 5 and 6 to estimate the through flow rate in each lane under equilibrium conditions during blocked and unblocked states.

The left- and right-turn equivalency factors (i.e., E_L and E_R) can be computed as the ratio of the headway of the respective turn movement with that of the through movement. For the right-turn movement, the right-turn headway includes the time required to turn that exceeds the travel time if the right-turning vehicle had not turned. This incremental time headway can be computed as follows:

$$h_R = h_0 + \frac{(u_r - u_t)^2}{2du_r} \tag{18}$$

where:

h_R = right-turn headway, sec;
h_0 = minimum discharge headway for through vehicles (= 3,600/s_0), sec;
u_r = midblock running speed, fps;
u_t = speed of turning vehicle (e.g., 15 fps); and
d = average turn vehicle deceleration rate, fpss.

The right-turn equivalency factor E_R can be computed by dividing Equation 18 by h_0 yielding

$$E_R = 1 + \frac{(u_r - u_t)^2}{2du_r h_0} I_R \tag{19}$$

where:

I_R = indicator variable (1.0 if there is no right-turn bay; 0.0 if there is a bay).

The equation above includes an indicator variable to account for the effect of a right-turn bay. This variable yields an equivalency factor of 1.0 when the deceleration occurs in a right-turn bay.

The left-turn equivalency factor E_L is based on a similar ratio of left-turn and through headways. However, the left-turn headway is sensitive to both the deceleration in the through lane and the capacity of the left-turn movement. The additional time resulting from deceleration is estimated using a similar approach as that used in Equation 18. Thus, the left-turn equivalency factor E_L is computed as follows:

$$E_L = \frac{h_L}{h_0} + \frac{(u_r - u_t)^2}{2du_r h_0} I_L \tag{20}$$

where:

h_L = left-turn headway (= 3,600/c_l), sec;
c_l = left-turn capacity, vph; and
I_L = indicator variable (1.0 if there is no left-turn bay or if a blockage/bay overflow condition exists; 0.0 otherwise).

Model Calibration. The calibration effort for the lane flow rate model consisted of a comparison of the predicted and measured proportions of left- and right-turning vehicles in their respective inside and outside lanes. For this calibration, the deceleration component of the left- and right-turn equivalency factors (i.e., the part multiplied by the indicator variable) was treated as the empirical adjustment constant. A range of alternative values were iteratively applied to obtain the single value that yielded the best overall agreement.

The quality of fit of the model to the measured lane flow rate data was quite good, as evidenced by the R^2 values of 0.90 and 0.98 that were obtained for the left- and right-turn proportions, respectively. The predictive ability of the lane flow rate model can be observed in Figure 3-14.

The empirical adjustment constant that yielded the best fit was 1.2. Substitution of this value in Equation 19 yields an E_R of 2.2 (= 1 + 1.2). This value is consistent with a vehicle decelerating from a running speed of 56 fps at an equivalent constant rate of 6.3 fpss (where the saturation flow rate is 1,800 vphpl and the turn speed is 15 fps).

Sensitivity Analysis. The calibrated lane flow rate model can be used to examine the relationship between the left-turn percentage and provision of a turn bay on the proportion of through vehicles in the inside through lane. The results of this examination are shown in Figure 3-15. This figure was developed using a left-turn equivalency E_L of 6.0, 10 percent right turns, and two through lanes (i.e., n = 2).

As Figure 3-15 indicates, the percentage of vehicles in the inside lane is evenly balanced with the outside lane when there is negligible left-turn demand, regardless of whether a bay is provided or blockage occurs. In addition, the percentage is evenly balanced over the full range of left-turn percentages when there is bay storage for the left-turn movements (e.g., a TWLTL).

If a left-turn storage area is not available (i.e., at an undivided cross section), the percentage of vehicles in the inside lane decreases with increasing left-turn percentages. When blockage occurs (i.e., the left-turn bay overflows or a left turn arrives on an undivided street), the percentage of vehicles in the inside lane decreases even more rapidly.

Through Movement Speed Model

Model Development. The relationship between speed and flow rate is well-documented and typically follows the trend shown in Figure 3-2 for uninterrupted flow facilities. The speed and flow data collected for this project were used to determine the nature and extent of any such relationship between speed and flow rate on urban arterials. The relationship that was found was shown previously in Figure 3-8 for two study sites. Further examination of data from several other sites indicates similar trends.

In general, a trend toward decreasing speed with increasing flow rate was found at all sites. The free-flow speed (i.e., speed at zero flow) for all sites with divided cross sections ranges between 60 and 70 fps. There is some evidence that the differences in speed within this range can be explained by the length of the street segment studied (as measured between the bounding signalized intersections); however, there also appears to be some correlation between free-flow speed and speed limit. The free-flow speed for the site with the undivided cross section is about 42 fps.

A maximum flow rate of approximately 2,000 pcphpl was found at three of the sites; the other sites had much lower flow rates. These high flow rates generally were found in the platoons created by the upstream signals. It is likely that this value is representative of capacity flow; however, because breakdown (or forced-flow) conditions were never observed

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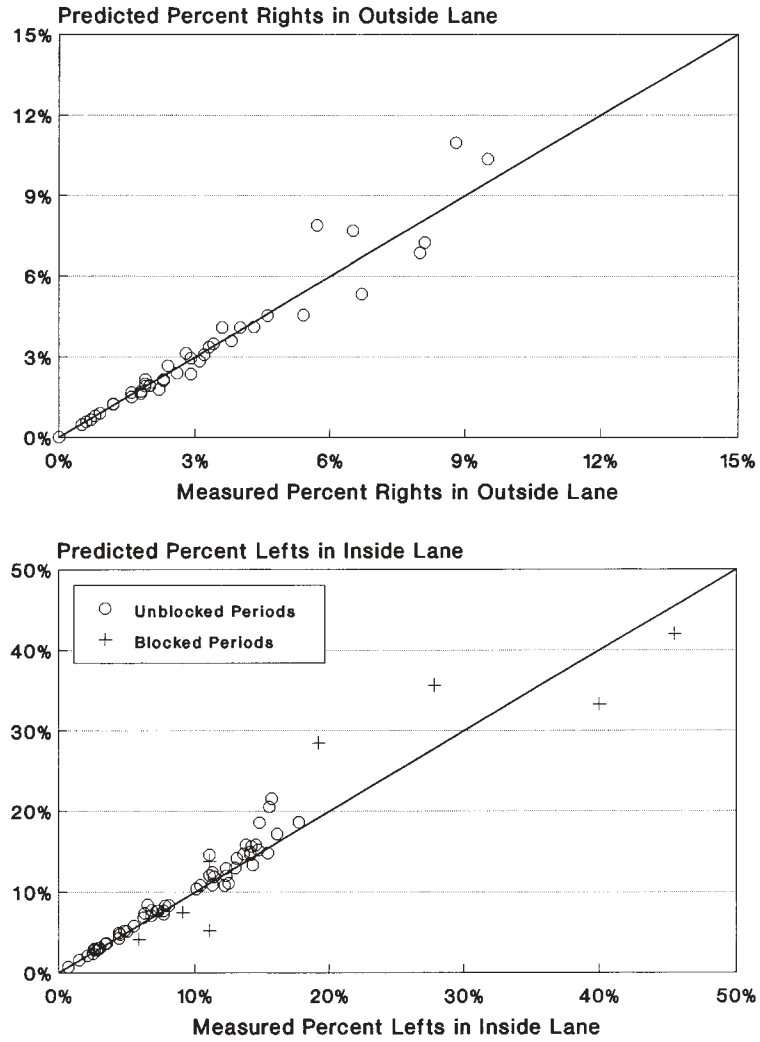

Figure 3-14. Comparison of predicted and measured left- and right-turn percentages: (above) comparison of right-turn percentages; (below) comparison of left-turn percentages.

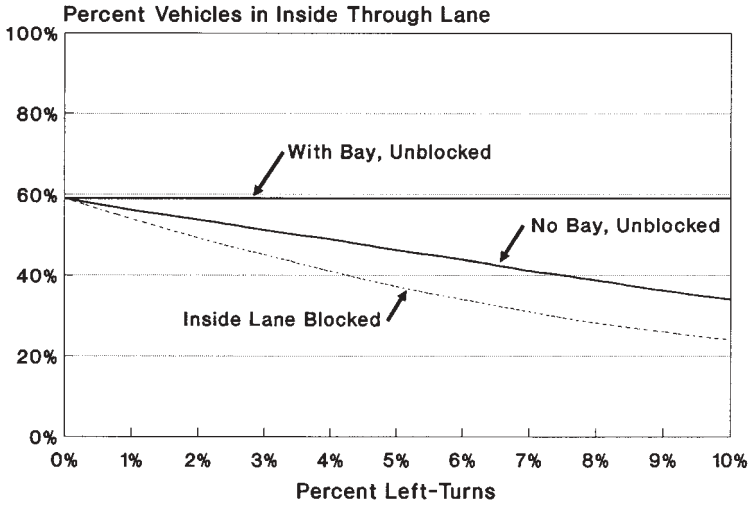

Figure 3-15. Predicted effect of left-turn percentage and blockage on the distribution of traffic to the inside through lane.

at these sites, it is possible that higher flow rates were achievable.

With one exception, the speed and flow rates observed were well-defined and followed a linear-decreasing trend. All sites that followed this linear-decreasing trend have divided cross sections. An exception to this trend was observed at the site with an undivided cross section. At this site, the data were varied and did not clearly follow a linear trend. It is believed that frequent midblock turning activity (especially left turns from the major street) disrupted traffic flow and created the observed wide variability in speeds and flow rates.

Based on this examination and some preliminary analysis of the effect of distance and speed limit, the following model was developed using the data from the sites with divided cross sections:

$$u_r = u_f + b_2 v \tag{21}$$

with

$$u_f = b_0 + b_1 I_D \tag{22}$$

where:

u_r = midblock running speed, fps;
u_f = midblock free-flow speed, fps;
I_D = indicator variable (1.0 for segment lengths in excess of ½ mi; 0.0 otherwise); and
v = arterial lane flow rate, pcphpl.

The effect of both segment length and speed limit were considered in the development of this model. However, only one factor could be used because segment length and speed limit were strongly correlated. An examination of their individual performances indicated that distance yielded a stronger relationship than did speed limit. It also should be noted that the speed and flow rate data used to calibrate Equation 22 were collected for passenger cars only. Hence, the flow rate for mixed passenger car and truck streams should be converted to that for an equivalent passenger-car-only stream before using Equation 22.

Model Calibration. Before calibration, the data collected for this model were aggregated into groups of 25 observations each. This aggregation was performed to overcome the variability in the speed and flow rate of individual vehicles. It does not bias the resultant model calibration parameters; however, it does yield higher R^2 values as a result of reduced variability. As urban arterial traffic flows are "pulsed" as a result of upstream signals, there is little justification for aggregating the observations by common time intervals (as is often done for flows on uninterrupted facilities). Rather, it was reasoned that the flows should be grouped in accordance with a common flow characteristic such as speed. Thus, the technique used was to first sort the data by speed, aggregate them into contiguous groups of 25 observations, and compute the average speed for the group and its average flow rate. The average group flow rate was computed as the reciprocal of the group's average headway.

The quality of fit of the calibrated through movement speed model is indicated in Table 3-18. The statistics reported in this table demonstrate the ability of the calibrated equation to predict the *average* speed for a given *average* flow rate; the R^2 and root mean square error reflect the precision of this average speed estimate (relative to the observation of 25 vehicles). The quality of fit to the averaged data also is shown in Figure 3-16.

Sensitivity Analysis. The calibrated equation can be used to predict midblock running speed as a function of flow rate and street segment length. Figure 3-17 illustrates these relationships and indicates that midblock speed decreases with increasing flow rate. The rate of decrease is the same for both short (i.e., less than 2,600 ft) and long street segments. However, the short street segments have a free-flow speed of 42 mph (62.4 fps), whereas the long segments have a free-flow speed of 48 mph (70 fps). These trends were found to be weakly correlated with speed limit; however, the range of speed limits in the database was relatively narrow (i.e., 35 to 45 mph), which made it difficult to establish a significant and meaningful trend.

Bay Entry Speed and Deceleration Model

Model Development. As discussed previously with regard to the lane flow rate model, turn vehicles decelerating in a through lane can delay following through drivers. The amount of delay is determined by the speed at which turning vehicles exit the through lane. When no turn bay is provided, the exit speed is about 15 fps. When a bay is provided, the exit speed can range from zero (e.g., when the turn bay overflows) to the running speed of through traffic (e.g., when the bay is relatively long), in which case there is no delay to following vehicles.

The exit (or bay entry) speed can be computed using Equation 23, which follows. This equation is based on an assumed constant deceleration rate during the bay entry maneuver.

$$u_{exit} = \sqrt{2dL_{avail}} \leq u_r \tag{23}$$

with

$$L_{avail} = L_s - N_q L_v \geq 0.0 \tag{24}$$

where:

u_{exit} = speed of turning vehicle when entering the bay (i.e., exiting the through lane), fps;
u_r = midblock running speed, fps;

TABLE 3-18 Calibrated through movement speed model

Statistic	Value
R^2	0.61
Root Mean Square Error:	5.75 fps
Observations:	271 (averages of 25 vehicles)

Range of Model Variables

Variable	Variable Name	Units	Minimum	Maximum
u_r	Midblock running speed	fps	24	82
I_D	Arterial segment length indicator variable	feet	940	5,280
v	Arterial lane flow rate	pcphpl	228	2,109

Calibrated Regression Coefficient Values

Variable	Interpretation	Value	Std.Dev.	t-statistic
b_o	Free-flow speed for segments < ½ mile long	62.4	5.2	12.00
b_1	Increase in free-flow speed for segments > ½ mile long	7.7	3.8	2.03
b_2	Reduction in speed for each additional vehicle	-0.0125	0.0009	-13.89

L_{avail} = length of bay available for deceleration, ft;
L_s = length of turn bay, ft;
N_q = average queue length for the subject turn movement, veh;
L_v = average storage length occupied by a queued vehicle (e.g., 25 ft/veh), ft/veh; and
d = average turning vehicle deceleration rate, fpss.

The bay entry maneuver described in Equations 23 and 24 was studied for the left-turn movement for this research (although it is recognized that the manuever is also applicable to right-turn movements). Specifically, data were collected to determine the average left-turning vehicle deceleration rate d and exit speed u_{exit}. These data were obtained from a short speed trap placed in the inside through traffic lane immediately before the bay entry location. Examination of the deceleration data indicated that left-turning drivers did not adopt a constant deceleration rate, as assumed for Equation 23. Rather, their deceleration rates varied with speed; higher speeds were associated with higher

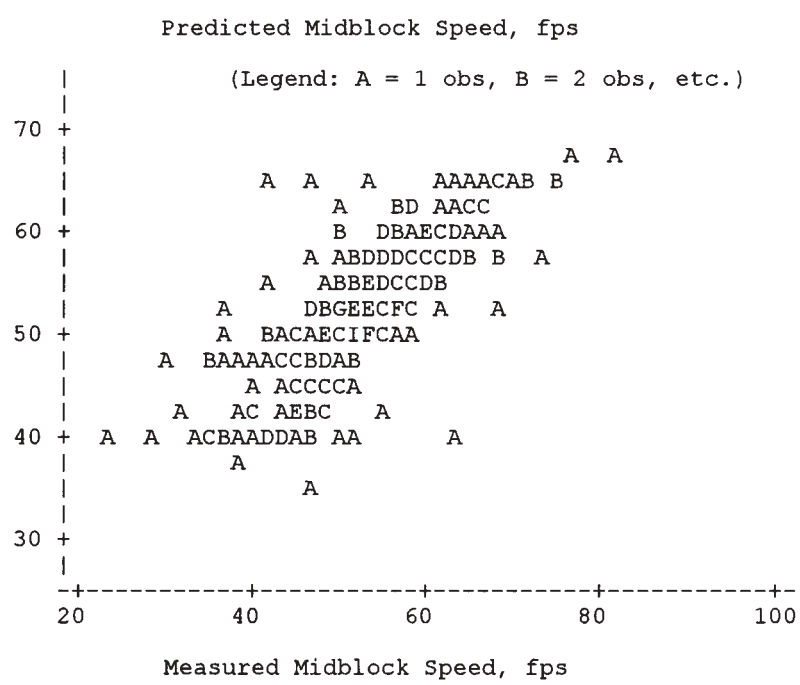

Figure 3-16. Comparison of predicted and measured midblock speeds.

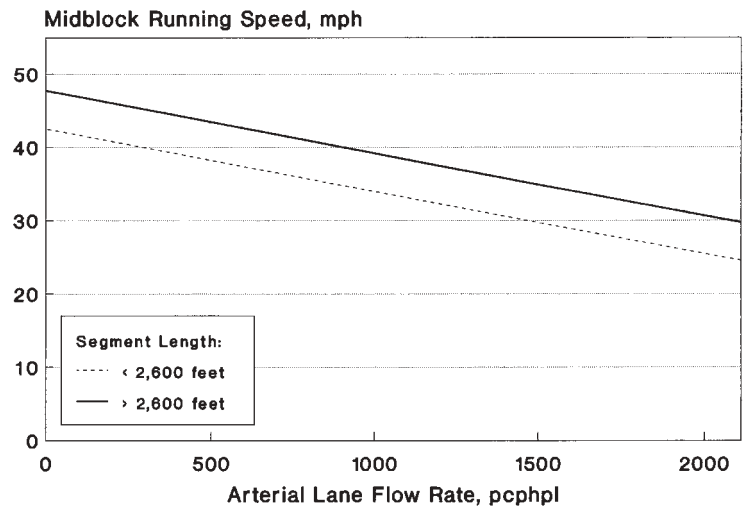

Figure 3-17. Predicted effect of flow rate and segment length on midblock running speed.

deceleration rates. This examination also indicated that driver deceleration decreased with increasing available deceleration distance. These effects were shown previously in Figure 3-9.

Based on an examination of the data, it was determined that the deceleration rate varies with speed and distance to the stopping point. The trend shown in Figure 3-9a suggests that the following relationship exists between deceleration rate and speed:

$$d(u) = d_{\min} e^{\frac{u}{b_0}} \tag{25}$$

where:

$d(u)$ = instantaneous deceleration rate relative to speed u, fpss;
$d_{\min}$ = minimum deceleration rate, fpss;
u = speed, fps; and
b_0 = calibration constant.

The relationship in Equation 25 can be integrated over distance with respect to speed (i.e., $\int \partial x = \int u/d(u)\, \partial u$) to obtain the following equation for predicting the deceleration distance X:

$$X = \frac{b_0^2}{d_{\min}} \left[e^{-\frac{u_2}{b_0}} \left(\frac{u_2}{b_0} + 1 \right) - e^{-\frac{u_1}{b_0}} \left(\frac{u_1}{b_0} + 1 \right) \right] \tag{26}$$

where:

X = distance needed to decelerate from speed u_1 to speed u_2, sec;
u_1 = initial speed, fps; and
u_2 = final speed, fps.

A similar integration, using a constant deceleration rate, yields the following equation for predicting deceleration distance:

$$X = \frac{u_1^2 - u_2^2}{2d_{eq}} \tag{27}$$

Equations 26 and 27 can be combined to determine the equivalent constant deceleration rate that would yield the same deceleration distance as the more realistic speed-varying deceleration relationship in Equation 26. This equivalent constant deceleration rate d_{eq} is as follows:

$$d_{eq} = \left(\frac{d_{\min}}{2b_0^2} \right) \frac{\left(u_1^2 - u_2^2\right)}{e^{-\frac{u_2}{b_0}} \left(\frac{u_2}{b_0} + 1 \right) - e^{-\frac{u_1}{b_0}} \left(\frac{u_1}{b_0} + 1 \right)} \tag{28}$$

In application, the deceleration rate predicted by Equation 28 would be used in Equation 23 to predict the bay entry speed.

Model Calibration. The equation calibration activity focused on the calibration of Equation 28 to the left-turn speed and deceleration data. This database includes the speed of the left-turning vehicles at a point immediately before bay entry and the number of vehicles queued in the left-turn bay at the time of entry (i.e., u_1 and N_q, respectively). Equation 24 was used to estimate the available deceleration distance L_{avail}. Equation 27 was then used with the speed and distance information to predict the constant deceleration rate necessary to ensure a safe stop in the turn bay. Finally, this "computed" constant deceleration rate was calibrated to Equation 28 using nonlinear statistical analysis procedures. Specifically, the nonlinear regression procedure (NLIN) in SAS (*23*) was

used to perform this calibration. The final speed u_2 was estimated as 0.0 fps.

The results of the calibration process are provided in Table 3-19. As the statistics in this table indicate, the calibrated deceleration equation is able to predict the equivalent constant deceleration rate with reasonable accuracy. The quality of fit also is shown in Figure 3-18.

Sensitivity Analysis. The calibrated bay entry deceleration model can be used to examine the effect of initial and final speed on equivalent constant deceleration rate. This effect is shown in Figure 3-19. As the trend lines in this figure indicate, the constant (or overall average) deceleration rate is higher when the initial speed is higher. The deceleration rates tend to be higher than those shown in Figure 3-9a because the model predicts the average rate necessary to come to a stop. In contrast, the decelerations shown in Figure 3-9a represent the decelerations measured at the point of bay entry; further examination of these data indicated that drivers adopted higher deceleration rates after they entered the bay.

The trend shown in Figure 3-19 also indicates that the equivalent constant deceleration rate is higher when the final speed is higher, for the same initial speed. This trend stems from a driver's desire to have an initially high deceleration rate at high speed and then to decrease this rate as speed drops (as shown in Figure 3-9a). Thus, a driver decelerating from a high speed to a slightly lower speed may vary his or her deceleration from 4.0 to 3.0 fpss (for an average of 3.5 fpss), whereas a driver decelerating from a high speed to a stop may vary his or her deceleration from 4.0 to 0.5 fpss (for an average of 2.25 fpss).

Lane-Change Gap Acceptance Parameter

Analysis of the lane-change gap acceptance database focused on determining the magnitude of critical gap needed by a driver stopped in a left-turn-related queue in the inside lane. In attempting to change lanes, this driver evaluates headways in the adjacent through traffic lane and determines the adequacy of each headway in terms of a safe lane change. The impetus for this lane change is a reduction in the through driver's delay. This driver also will weigh the likely delay he or she will incur if he or she chooses to wait for the left-turn queue ahead to dissipate when deciding whether to make a lane change.

From the standpoint of estimating lane-change capacity during blocked conditions, the stopped condition represents the most appropriate situation to consider because the through driver incurs the most delay once he or she is caught in the left-turn queue. Based on the data in Table 3-17 and the argument that the 15th percentile value is representative of the median driver, the critical gap for a lane-change maneuver was determined to be 3.7 sec.

Operations Model Verification

This section describes the verification of the operations model using output from two microscopic computer simula-

TABLE 3-19 Calibrated bay entry deceleration model

Statistic	Value
R^2	0.68
Root Mean Square Error:	0.77 fpss
Observations:	445

Range of Model Variables

Variable	Variable Name	Units	Minimum	Maximum
d_{eq}	Equivalent constant deceleration rate	fpss	0.74	8.3
u_1	Initial speed	fps	22	66
N_q	Queued length	veh	0	5
L_{avail}	Decel. distance (from speed trap to back of queue)	ft	185	337

Specified Parameter Values

Variable	Variable Name	Units	Value
L_v	Storage lane length occupied by a queued vehicle	ft	25

Calibrated Regression Coefficient Values

Variable	Interpretation	Value	Std.Dev.	t-statistic
b_o	Calibration coefficient	19.0	1.94	9.8
d_{min}	Minimum deceleration rate	0.95	0.06	15.8

```
Predicted Constant Deceleration Rate, fpss

    8 +     (Legend: A = 1 obs, B = 2 obs, etc.)
      |
      |
      |
      |                                AB
      |                              AA A    AA
    6 +                                   A B A
      |                        ACDA  BAACA A
      |                      BCAAABBCDDA  B
      |                    DEHABBAEDB  B B
      |                  DKLEEFHFG  C A  A
      |                BRNNDEIBC DAA
    4 +              AHKKJKGCACCA
      |            BHTGHJC BCB
      |           BCJFFAADC
      |         AGGCC  A
      |        DBB A
      |       B
    2 +    A
      --+--------+--------+--------+--------+--------+---
        0        2        4        6        8       10

      Computed Constant Deceleration Rate, fpss
```

Figure 3-18. Comparison of predicted and computed constant deceleration rates.

tion models: TWLTL-SIM III and NETSIM (Version 5.0) (*24*). These two models and the operations model have in common the ability to replicate traffic flow conditions on an urban street with both signalized and unsignalized intersections. Moreover, each model is able to evaluate (to varying degrees of realism) the effects of three midblock left-turn treatments: the raised-curb median, TWLTL, and undivided cross section. The verification process entailed using each model to simulate street segments with a range of traffic demands and treatment types; the output of the operations model was then compared with the output of the other two models.

Verification Process

The verification process was intended to demonstrate the ability of the operations model to predict traffic flow performance measures that are consistent in magnitude and trend with those of other, more thoroughly tested and validated traffic models. This process also was intended to demonstrate the robustness of the operations model to predict performance measures for a wide range of geometric and traffic conditions—a wider range than could be reasonably obtained from field study.

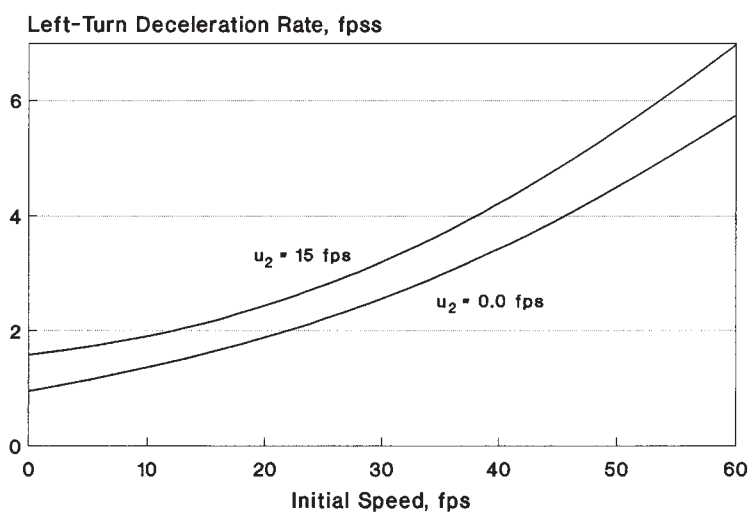

Figure 3-19. Predicted effect of initial and final speed on equivalent constant deceleration rate.

Performance Measures. The performance of a traffic movement can be quantified by its average delay, travel time, travel speed, and probability of stopping. NETSIM, TWLTL-SIM, and the operations model all report variations of the first three performance measures (at this time, the operations model does not predict the probability of stopping). Of the performance measures common to all three models, delay was selected as the most appropriate measure to verify because the HCM recognizes it as the measure of effectiveness for defining level of service at an intersection.

The operations model uses Equation 10-11 in the HCM (*1*, Chapter 10) to predict delay to nonpriority movements. The HCM states that this equation, which has its basis in queueing theory, predicts average total delay per vehicle. In this regard, the HCM defines total delay as the total elapsed time from when a vehicle stops at the back of the queue until the vehicle departs from the stop line. This delay also is (perhaps more correctly) referred to by many researchers as queueing delay because it includes the time the vehicle is in the queue, either stopped or slowly moving toward the stop line. The same researchers reserve the term "total delay" (some use "approach delay") for queueing delay and all other delay components (e.g., deceleration time and intersection negotiation time) associated with a nonpriority movement at an intersection. Nevertheless, the operations model is stated in this report to predict total delay to be consistent with the HCM; however, it must be remembered that the model is actually predicting queueing delay.

TWLTL-SIM tracks the travel paths of individual vehicles and uses associated travel time information to compute what is called "stopped delay" per vehicle. In contrast to queueing delay, stopped delay typically is defined to include only the time the vehicle is actually stopped in a queue. However, the nature of TWLTL-SIM's simulation modeling and statistics computation approach is such that its stopped delay is actually near that of the aforementioned queueing delay. Hence, the delay reported by TWLTL-SIM is generally comparable with that obtained from the operations model.

NETSIM also tracks the travel times of individual vehicles and uses this information to compute delay. However, NETSIM reports three different types of delay: stopped delay, queue delay, and total delay. NETSIM's stopped delay represents the time the vehicle is stopped in a queue. Queue delay is not clearly defined in the NETSIM documentation or in the literature; hence, it does not represent a reliable statistic for the verification process. NETSIM's total delay includes all components of a vehicle's actual travel time that exceed its potential travel time at free-flow speed. This total delay differs from the operation model's total delay because it includes the added travel time caused by the volume of the traffic stream and by the deceleration and acceleration associated with joining a queue. However, based on this assessment, it was determined that NETSIM's total delay would be the best measure to use in the verification, although it is recognized that NETSIM's total delay is always larger than that predicted by the operations model.

Total Delay. As mentioned in the preceding paragraph, there are two key differences between NETSIM's and the operations model's total delay. The first difference relates to NETSIM's use of free-flow speed to compute total delay. NETSIM computes travel time based on free-flow speed and subtracts this time from the actual travel time to compute total delay. As a result, NETSIM's total delay includes the added travel time caused by traffic volume. In contrast, the operations model does not include this added travel time in computing total delay. A comparison of the running and free-flow speeds predicted by Equations 21 and 22, respectively, indicates that this added time ranges from 0.0 to 5.0 sec for a 0.25-mi street segment with flow rates ranging from 0.0 to 1,000 vphpl.

The second difference in delay results from NETSIM's inclusion of the added travel time associated with joining a queue. Only a small portion of this added time is included indirectly in the operations model delay estimate through the critical gap parameter. In general, the added time associated with joining a queue has been estimated by Olszewski (*25*) to be about 8 to 10 sec per vehicle. Of course, it must be recognized that the major-street left-turn movement does not always stop. Based on these factors, it is reasoned that the total delay reported by NETSIM is an additional 1 to 3 sec higher per left-turning vehicle than the operations model. This amount will vary depending on the number of left-turning vehicles stopping.

In summary, NETSIM's total delay is always higher than that predicted by the operations model (and Equation 10-11 of the HCM) because of differences in travel time and in deceleration and acceleration time. It is believed that NETSIM reports major-street left-turn movement delays that are 1 to 8 sec higher per vehicle than the operations model, the exact amount depending on volume level and percent stopping.

Traffic Movements. The two traffic movements most directly affected by midblock left-turn treatments are those on the major-street approach to the access point. Of these, the movement that is most directly affected is the left turn from the major street. This movement's performance is affected by the presence or lack of a median storage area. The storage area provided by the raised-curb and TWLTL treatments removes the left-turning vehicles from the through traffic stream, leaving the through traffic lanes available to serve through traffic without interruption. This separation of flows increases left-turn capacity and reduces left-turn delay by increasing the frequency and size of available gaps in the opposing traffic stream, relative to the undivided cross section. Because the raised-curb treatment always has less storage space than the TWLTL, differences between these two treatments also can emerge when left-turn demands are high enough to precipitate the overflow of the raised-curb median's bay storage area.

The other movement directly affected by midblock left-turn treatment is the major-street through movement. This movement is affected when one or more left-turning vehicles

are queued in the inside through lane (i.e., bay overflow). The major-street through movement also is affected by turning vehicle deceleration in the through traffic lane. Through vehicle delay caused by turning vehicle deceleration occurs when through vehicles are slowed by vehicles in front of them preparing to turn left or right.

Analysis Scenarios

Three types of scenarios were devised for the verification process. One focuses on traffic operations on a typical street segment with a series of closely spaced access points. The other two were dictated by NETSIM's inability to model closely spaced access points (this limitation will be described in a later section).

Common Scenario Attributes. All three scenarios model a quarter-mile segment of urban arterial. This study segment has signalized intersections at each of its ends, with identical phase sequences and timing. A minimum delay offset between the through phases at each intersection was determined using a time-space diagram. The signalization details are provided in Table 3-20.

At all study segments, vehicles enter at a signalized intersection as a through, protected left-turn or right-turn movement; the distribution of the entry volume to these movements is listed in Table 3-20. All traffic that exits the study segment at the downstream signal does so as a through movement. Median openings are provided at each access point.

Three traffic movements were not included in the study scenarios: the U-turn, access point left-turn, and access point through movement. This approach was taken because these movements typically were found to be of very low volume at the 32 field study sites. On average, the access point left-turn movement was found to be about one-tenth of the volume of the major-street left-turn movement. The U-turn and access point through movements were rarely, if ever, observed during the 6-hr study of each site.

The exclusion of these movements reduced the number of simulation runs and simplified data analysis. It is not believed to have biased the results because the low volume of these movements resulted in them having no impact on the performance of the major-street left-turn and through movements. On the other hand, it is recognized that these minor movements do incur considerable delay (the likely reason they have low volumes) that may be affected by midblock left-turn treatment type. However, the total vehicle-hours of delay to these movements is believed to represent a small fraction of that incurred by the major-street movements studied.

The scenarios also are similar in that vehicles exiting the study segment at an access point (by turning left or right) are replaced by vehicles turning right into the study segment at that same access point. In this manner, the same arterial volume level is maintained throughout the study segment. For example, if it is specified that 10 vph turn left and 5 vph turn right off the major street at an access point, then 15 vph (5 + 10) were determined to turn right onto the major street at that access point to maintain a balance in the arterial traffic volume.

The three scenarios do differ in traffic volume, left-turn percentage, and access point density. These differences are described in more detail in the next three sections.

Type 1 Scenario. The Type 1 scenario includes three access point densities (30, 60, and 90 access points per mile), three left-turn treatment types, and two through lane combinations (four and six). This scenario resulted in a total of 18 geometric configurations. Figure 3-20 presents the access point configuration used for the "90 Access Points Per Mile" study segment. The study segments for the 30 and 60 access points per mile configurations were similar to the segment shown in Figure 3-20, except that the access points had 330-ft and 165-ft spacings, respectively.

TABLE 3-20 Characteristics of the signalized intersections bounding the study segment

Characteristic	Entry Movements[1]			Exit Movements[1]	Total
Movement - East Intersection	NB Left	SB Right	WB Thru	EB Thru	
Movement - West Intersection	SB Left	NB Right	EB Thru	WB Thru	
Phase Sequence Number	1	2	3		
Phase Duration (G+Y), sec	17	19	54		90
Distribution of entry flow rate for the "Four Through Lanes" variation, %	20	10	70		100
Distribution of entry flow rate for the "Six Through Lanes" variation, %	10	10	80		100

Note:
1 - All study segments were oriented in an east-west direction.

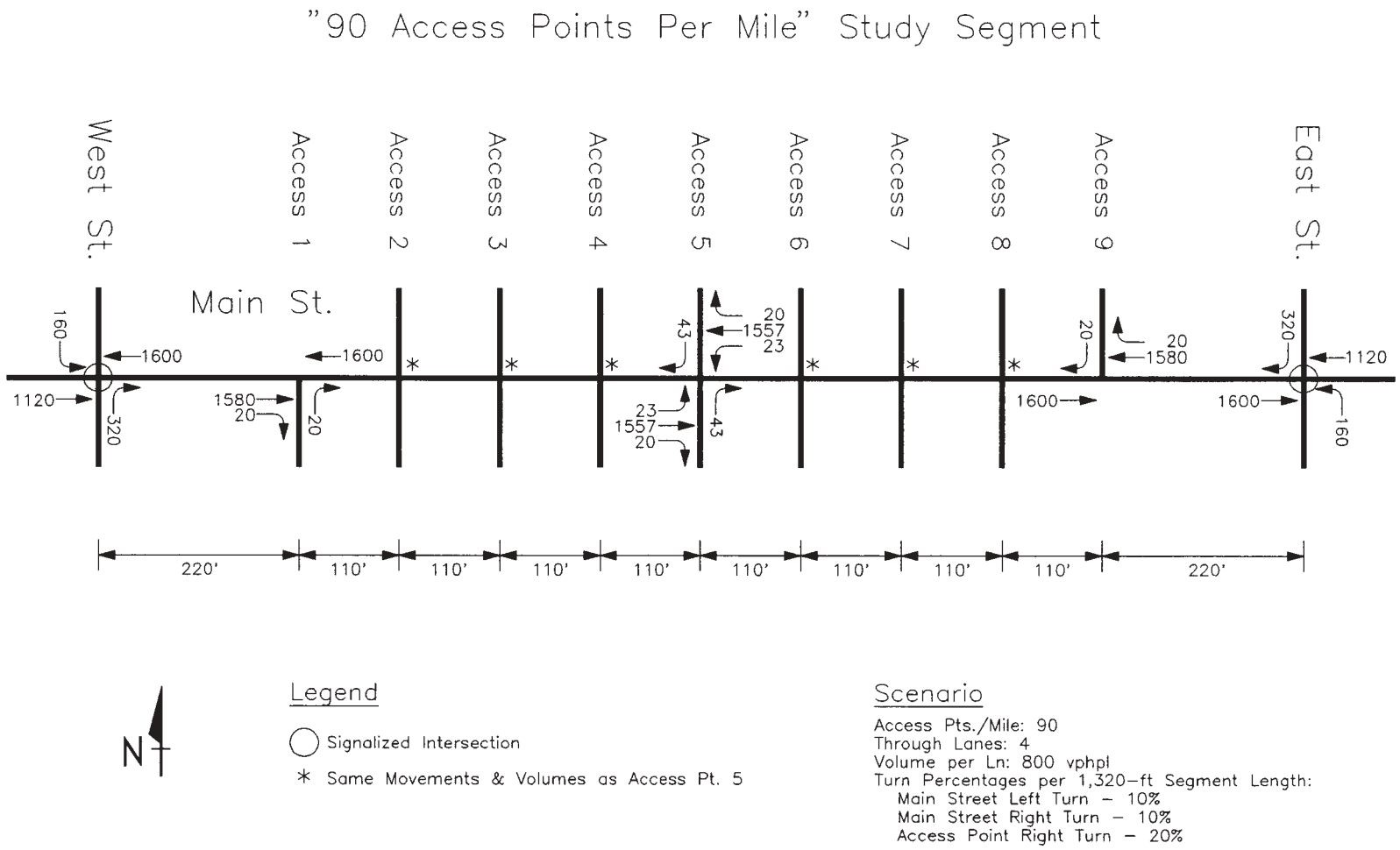

Figure 3-20. Type 1 scenario study segment with 90 access points per mile.

The access points were located for this scenario using three criteria, which required that the access points be (1) evenly spaced, (2) no closer than 330 ft to the nearest signalized intersection approach, and (3) located directly across from one another. The actual number of access points for each access point density was adjusted to be in compliance with these three criteria. At 60 and 90 access points per mile, the second criteria required that the first upstream access point encountered in either travel direction be a T intersection in which no left-turn movements are allowed; only right turns off of and onto the major street are permitted.

In the raised-curb median geometry, the median is always divided evenly into two left-turn bays—one for each of the bounding access points. The bay lengths were specified as being 30, 30, and 90 ft for the 90, 60, and 30 access points per mile configurations, respectively. An overlapped bay taper was used in all cases; its length varied from 20 to 120 ft.

Short bays were provided for the raised-curb median treatment, instead of closing some of the median openings, to facilitate the comparison of the three treatment types—given the same level of activity at each access point. It is believed that this approach more directly answers the question of differences in operational effects of alternative midblock left-turn treatments. Some researchers (*6,8*) have attempted to answer this question by conservatively assuming that the median openings would be closed at some or all of the access points and that displaced drivers would make U-turns at downstream intersections. However, these researchers also recognized the difficulties associated with their approach in terms of quantifying the true U-turn volume increase (if any) and the true increase in delay caused by median closure.

For all segments with a raised-curb median, it was assumed that median openings exist at all active access points. An active access point is defined as having an entering volume of 10 vph or more. This assumption is fairly consistent with the median openings found at the field study sites. Moreover, in addition to directly answering the question noted previously, this approach eliminated the criticism of associating an overly conservative estimate of delay with the raised-curb median treatment as a result of an assumed U-turn scenario. It was recognized early that it is impossible to accurately account for the effects of an unopen median on driver route choice without considering the surrounding street network. On the other hand, it can be argued that the modeling assumption noted above yields a delay to drivers accessing the adjacent property that is nearly equal to the delay actually incurred, regardless of whether they turn left (as modeled) or take an alternative route.

The geometric configurations associated with this scenario were modeled using TWLTL-SIM. In contrast to NETSIM, TWLTL-SIM was developed specifically for modeling arterial street segments that have many closely spaced unsignalized intersections and any of the three midblock left-turn treatments.

A wide range of left-turn, through, and right-turn volumes were evaluated for each of the 18 geometric configurations. The major-street left- and right-turn volumes at each access point were determined from the specified turn movement percentages and the access point density. Table 3-21 lists the volume variations and geometric configurations that comprise the Type 1 scenario.

Type 2 Scenario. The Type 2 scenario was created for two purposes. One purpose was to provide a means for isolating and examining the effect of left-turn bay overflow on through vehicle delay. To effect this purpose, this scenario's geometry and traffic demands were established so that bay overflow would occur frequently, with varying duration. In this regard, the only left-turn treatment considered for this scenario was the undivided cross section because this treatment precipitates frequent bay overflow.

A second purpose of the Type 2 scenario was to facilitate verification of left-turn delay using the NETSIM model. Satisfying this second purpose required that the access point spacing be large enough to prevent NETSIM's short-segment modeling limitations from significantly affecting the accuracy of the findings (this topic will be addressed in more detail in a later section). It should be noted that the TWLTL-SIM model also was used in this analysis to facilitate a three-way model verification.

As shown in Figure 3-21, the Type 2 scenario includes two access points on a quarter-mile study segment bounded by signalized intersections. For each travel direction, vehicles turn left off of the major street only at the downstream access point. Likewise, vehicles turn right onto the major street only at this same access point. There are no right turns off of the major street at either access point. Table 3-22 lists the volume variations and geometric configurations that comprise the Type 2 scenario.

Type 3 Scenario. The purpose of the Type 3 scenario was to provide a means of isolating and examining the effect of major-street right-turn movements on through vehicle delay. Thus, this scenario did not require any major-street left-turn volumes nor did it require an examination of various midblock left-turn treatments.

The NETSIM model was used for the analysis of the Type 3 scenario. It is believed that NETSIM's car-following model is slightly more robust than that used in TWLTL-SIM; hence, it was reasoned that the through vehicle delays (caused by right-turn deceleration) reported by NETSIM would be more accurate than those reported by TWLTL-SIM.

The Type 3 scenario geometry is almost identical to that of the Type 2 scenario. The only difference is that the major-street left turns at the downstream access points were replaced with major-street right turns.

Table 3-23 lists the volume variations and geometric configurations that comprise the Type 3 scenario. Six-lane scenarios were not modeled because it was thought that the extra lane would reduce the number of through vehicles in the out-

TABLE 3-21 Volume variations and geometric configurations for the Type 1 scenario

Right-Turn Percent per 1,320-ft Segment Length[1] (%)	Midblock Left-Turn Treatments	Access Point Density[2] (acc. pt./mi)	Through Traffic Lanes[3]	Through Lane Flow Rate[4] (vphpl)	Left-Turn Percent per 1,320-ft Segment Length[5] (%)	Left-Turn Volume per 1,320-ft Segment Length[6] (vph)
10	Undivided, TWLTL, Raised-Curb	30, 60, 90	4	450	5	45
				800	10	160
			6	450	5	68
				550	10	165
				800	10	240

Notes:
1 - Total number of right-turns per hour exiting the major street into an access point in one direction of travel per 1,320-foot length of roadway divided by the total flow rate in that direction (expressed as a percentage).
2 - Access point density represents the total number of access points on both sides of the major street (i.e., a two-way total) divided by the length of the segment (in miles).
3 - Total number of through lanes for both travel directions.
4 - Traffic volume per lane on the major street (the count of lefts, throughs, and rights on the major street approach to each access point and averaged for all access points).
5 - Total number of left-turns per hour exiting the major street into an access point in one direction of travel per 1,320-foot length of roadway divided by the total flow rate in that direction (expressed as a percentage).
6 - Total number of left-turns per hour exiting the major street into an access point in one direction of travel per 1,320-foot length of roadway (this total is divided evenly among the access points).

side lane and thereby minimize the turn vehicle delay effect being studied. Therefore, the database assembled for this scenario focuses only on four-lane configurations.

Analysis Results

Simulation Runs. A statistical analysis was conducted to determine the minimum total simulation time needed to ensure reasonable precision in the simulation results. This time was then partitioned into a desired number of replications (i.e., runs) and run durations.

The minimum total simulation time for each volume and geometric condition considered was based on the need for a minimum number of independent observations. In this context, an observation is defined as one vehicle with the potential to incur delay as a result of traffic events of interest. As a result of the symmetry of the traffic volumes and geometric conditions for each scenario, twice as many observations were obtained per replication. In other words, each movement of interest in one travel direction had an identical twin in the other travel direction. As a consequence, the number of replications for each combination was effectively reduced by one-half.

The minimum total simulation time was determined using a traditional statistical approach. Specifically, it was assumed that the desired precision in the average delay estimate could be obtained by running the simulation long enough to obtain a minimum number of delayed vehicle observations. The following sample size equation was developed based on an assumed normal distribution of vehicle delays:

$$n = \left(\frac{z\sigma}{e}\right)^2 \tag{29}$$

where:

n = number of observations (vehicles potentially incurring the delay of interest), veh/drive;
z = standard normal variate corresponding to a desired level of confidence;
σ = standard deviation of delay, sec/veh;
e = difference between the average and true mean delay ($= \bar{x} - \mu$), sec/veh;
$\bar{x}$ = average delay, sec/veh; and
μ = true mean delay, sec/veh.

It was assumed that the standard deviation of delay was equal to the true mean delay (i.e., $\sigma = \mu$) and that the difference between the average delay and the true mean delay could be expressed as a percentage P_e of the true mean (i.e., $e = P_e * \mu/100$). Using a 95 percent confidence level (i.e., $z = 2.0$), Equation 29 can be rewritten as follows:

$$n = \left(\frac{200}{P_e}\right)^2 \tag{30}$$

where:

P_e = acceptable error in mean delay estimate (expressed as a percentage of the mean error).

Based on a maximum acceptable error P_e of 20 percent, Equation 30 predicts a minimum sample size of 100 vehicles per access point per simulation run. Using this required minimum sample size, the simulation durations were derived

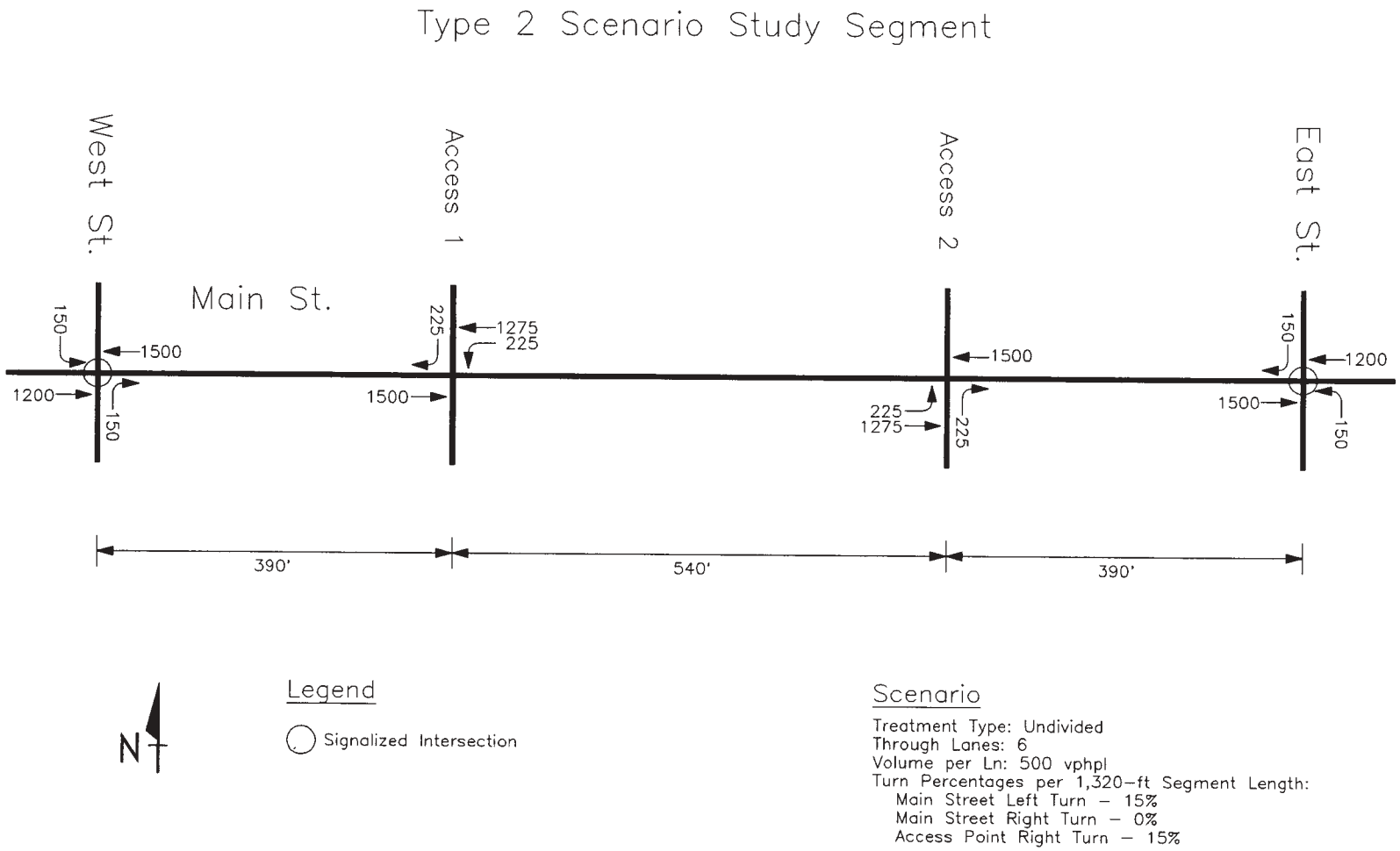

Figure 3-21. Type 2 scenario study segment.

TABLE 3-22 Volume variations and geometric configurations for the Type 2 scenario

Right-Turn Percent[1] (%)	Midblock Left-Turn Treatment	Through Traffic Lanes[2]	Through Lane Flow Rate[3] (vphpl)	Left-Turn Volume[4] (vph) by Left-Turn Percentage[5]			
				5%	10%	15%	20%
0	Undivided	4	500	50	100	150	200
			600	60	120	180	240
			700	70	140	210	280
			800	80	160	240	320
		6	400	60	120	180	240
			500	75	150	225	300
			600	90	180	270	360

Notes:
1 - Total number of right-turns per hour exiting the major street into the access point in one direction of travel divided by the total flow rate in that direction (expressed as a percentage).
2 - Total number of through lanes for both travel directions.
3 - Traffic volume per lane on the major street (the count of lefts, throughs, and rights on the major street approach to each access point and averaged for all access points).
4 - Total number of left-turns per hour exiting the major street into the access point in one direction of travel.
5 - Total number of left-turns per hour exiting the major street into the access point in one direction of travel divided by the total flow rate in that direction (expressed as a percentage).

based on the number of turns per access point that occur per hour in each scenario. This computation varied according to the scenario type and simulation model used. The results of this analysis are summarized in Table 3-24.

Simulation Model Limitations and Their Implications. Application of the TWLTL-SIM and NETSIM simulation models during the verification process revealed that each model had some limitations that could reduce the accuracy of its output. As a result of these limitations, some of the delays predicted by TWLTL-SIM and NETSIM differed from those predicted by the operations model. These limitations are addressed in the following paragraphs to provide the background necessary to discuss the results of the verification process.

Both TWLTL-SIM and NETSIM exhibited limitations in their ability to model driver behavior at or on the approach to an unsignalized intersection. Six behavioral problems were identified that relate to improper or illogical decisions being made for individual vehicles via the simulation logic:

1. A left-turning driver who stops in the through lane adjacent to the lane from which the turn is made because his or her entry into the turn lane is blocked by a left-turn-related queue. The driver waits in the adjacent through lane until the queue dissipates, regardless of the delays he or she causes to following drivers in this adjacent lane.

TABLE 3-23 Volume variations and geometric configurations for the Type 3 scenario

Left-Turn Percent[4] (%)	Midblock Left-Turn Treatment	Through Traffic Lanes[2]	Through Lane Flow Rate[3] (vphpl)	Right-Turn Volume[5] (vph) by Right-Turn Percentage[1]			
				5%	10%	15%	20%
0	Undivided	4	500	50	100	150	200
			600	60	120	180	240
			700	70	140	210	280
			800	80	160	240	320

Notes:
1 - Total number of right-turns per hour exiting the major street into the access point in one direction of travel divided by the total flow rate in that direction (expressed as a percentage).
2 - Total number of through lanes for both travel directions.
3 - Traffic volume per lane on the major street (the count of lefts, throughs, and rights on the major street approach to each access point and averaged for all access points).
4 - Total number of left-turns per hour exiting the major street into the access point in one direction of travel divided by the total flow rate in that direction (expressed as a percentage).
5 - Total number of right-turns per hour exiting the major street into the access point in one direction of travel.

TABLE 3-24 Simulation replication and duration by model and scenario

Model	Scenario Type	Simulation Replications	Through Lane Flow Rate (vphpl)	Simulation Run Duration (hours/rep)
TWLTL-SIM	Type 1	5	450	4
			550	3
			800	2
	Type 2	5	All	1
NETSIM	Type 2 and 3	1	All	1

2. A through driver who makes lane-choice decisions based only on traffic conditions in his or her immediate vicinity (rather than looking farther downstream), thereby waiting until the last minute to avoid left-turn-related delays by means of a lane change. This wait typically reduces the chance of a successful lane change and results in an unrealistically large number of through vehicles being caught in the left-turn queue.
3. A left- or right-turning driver who does not know that he or she will be turning until entering the "link" (i.e., arterial street segment between intersections) from which the turn is to be made. As a result, the driver does not position himself or herself in the appropriate inside or outside lane in advance of this link, resulting in an unrealistic distribution of traffic among the major-street traffic lanes.
4. A left- or right-turning driver who makes his or her turn maneuver at an unrealistically high speed. This behavior is related to Problem 3. Because the driver does not know that he or she will be making a turn until entering the link, it may be impossible for him or her to slow to the turn speed if the link is not long enough to allow for a realistic rate of deceleration. As a result, the delay to following through drivers is lower than would otherwise be realized in real-world traffic flows.
5. A left-turning driver who accepts a gap that is shorter than his or her minimum acceptable gap. This behavior stems from the simulated driver's inability to see beyond the length of the opposing link. To illustrate this point, consider a driver who is assigned a minimum acceptable gap of 6.0 sec. If the opposing link is only 4.0 sec long (in terms of link travel time), the driver will only be able to see gaps of 4.0 sec or less. Gaps greater than 4.0 sec (represented by an empty opposing link) will be evaluated as infinitely long gaps by the driver. Thus, if a gap of 4.1 sec occurs in the opposing stream, the left-turning driver will incorrectly accept this gap. The consequence of this modeling limitation is an unrealistically high left-turn capacity when the opposing link is short.
6. A left-turning driver who waits in the major street as long as it takes to make the turn, regardless of the length of delay he or she experiences. In the real world, drivers will modify their behavior (e.g., accept a shorter gap) or their desire to turn to limit the delay they incur to reasonable values. This behavior results in longer average delays being reported than would actually occur.

TWLTL-SIM exhibited the first two behavioral problems only, whereas NETSIM exhibited all six. The first two problems reflect inherent limitations in the analytic modeling of traffic flow, and overcoming these would require significant enhancements to the car-following and lane-changing logic in the respective programs. In general, the problems tend to result in unrealistically high through delays when link lengths are short or arterial volume levels are high. Volume and geometric combinations that produced this unrealistic behavior were identified using the animation features of each simulation model. These combinations were not included in the verification process. Review of the NETSIM animation output revealed that the first behavioral problem noted previously was very likely to occur when the product of the left-turn flow rate (in vph) and opposing flow rate per lane (in vphpl) exceeded 75,000.

The third through fifth behavioral problems stem from NETSIM's link-based decision-making process. NETSIM makes decisions for each driver based only on information available on the link on which the driver is traveling. One known exception to this generalization is for left-turn movements. In this case, left-turning drivers assess the adequacy of gaps based on traffic information on the opposing link. However, the point remains the same: NETSIM drivers can only "see" one link at time. The limitation of this "link myopia" surfaces when the length of the link is relatively short so that an inadequate amount of information is available to the driver. The consequences are that the simulated drivers make incorrect decisions based on imperfect information, as described in the aforementioned behavioral problems.

The sixth behavioral problem relates to NETSIM's modeling approach to the driver gap acceptance process. To its credit, NETSIM recognizes that the population of drivers is not homogenous in that they collectively have a range of minimum acceptable gaps. In this regard, NETSIM assigns a minimum gap to each driver in a random manner. On the

other hand, NETSIM models these drivers as being "consistent" by requiring them to search for this gap regardless of how long they have been delayed. Kyte et al. (*9*) have found that drivers reduce their acceptable gap based on the length of time they have been delayed. In fact, there is some evidence that they will abort the maneuver or force their way into the street if their delay becomes excessive.

A kinematic analysis was conducted to determine the minimum link length needed for NETSIM to overcome the fourth and fifth behavioral problems. Based on a description of the NETSIM deceleration process provided by Wong (*26*), it appears that NETSIM decelerates turning vehicles at a rate of 1.0 fpss until their speed has dropped 10 percent. Thereafter, the vehicle decelerates with a rate of 7.0 fpss until it reaches the target turning speed of 13 fps. Based on this description, the respective distances required to decelerate to the right-turn speed are shown in Table 3-25. The total deceleration distance is shown in Column 5. The minimum link lengths needed to provide a left-turning driver with a complete view of gaps in the opposing stream are listed in Column 6. These lengths are based on the running speed combined with an assumed maximum viewable gap of 6.0 sec. The larger distance required by either of these two turn movements represents the minimum link length necessary to overcome the aforementioned behavioral problems.

Table 3-25 was used in the development of the Type 2 and Type 3 scenarios. The fastest running speed assigned to either scenario was 40 mph; hence, the minimum link length needed for the right-turn maneuver was determined to be about 500 ft. Similarly, the minimum opposing link length needed for the left-turn maneuver was determined to be 350 ft. As shown in Figure 3-21, the actual lengths used for these scenarios were 540 and 390 ft for the approach and opposing links, respectively, at each access point.

Delays to Major Street Left-Turning Vehicles. As stated previously, the objective of this component of the verification process was to demonstrate the ability of the operations model to predict the average delay to the major-street left-turn movement, relative to the delay predicted by both TWLTL-SIM and NETSIM. The comparison with TWLTL-SIM is discussed in the context of the Type 1 scenario; the comparison with NETSIM is discussed in the context of the Type 2 scenario.

For the Type 1 scenario, the average left-turn delay for each traffic volume variation and geometric configuration listed in Table 3-21 were computed using both TWLTL-SIM and the operations model. Figure 3-22 compares the left-turn delay predicted by the operations model with that predicted by TWLTL-SIM for this scenario. As the data in this figure indicate, the delays predicted by the operations model are very similar to those predicted by TWLTL-SIM.

For the Type 2 scenario, the average left-turn delay for each traffic volume variation and geometric configuration listed in Table 3-22 were computed using both NETSIM and the operations model. Figure 3-23 compares the left-turn delay predicted by the operations model with that predicted by NETSIM for the Type 2 scenario. The delays shown represent the values obtained at the single access point where left turns are made, and the straight line represents the line of perfect agreement. As expected, the "total" delays predicted by NETSIM exceed those predicted by the operations model. This trend is a result of the differences in the delay definitions used by each model, as previously discussed.

In addition to differences in delay definitions, a portion of the NETSIM left-turn delay that exceeds the delay predicted by the operations model is likely the result of differences in how each model replicates left-turning driver behavior. As discussed previously, NETSIM drivers do not modify their behavior or desire to turn, regardless of the length of their delay. In contrast, this behavior was indirectly accounted for in the operations model by introducing a minimum, nonzero left-turn capacity. Because the reciprocal of capacity represents the average service time for a nonpriority movement,

TABLE 3-25 Minimum link lengths for NETSIM

Major-Street Maneuver:		Right-Turn			Left-Turn	All
Running Speed (mph)	Running Speed (fps)	First Deceleration Distance[1] (ft)	Second Deceleration Distance[2] (ft)	Total Deceleration Distance (ft)	Minimum Viewing Distance[3] (ft)	Minimum Link Length (ft)
25	36.8	128	66	194	221	221
30	44.1	185	100	285	265	285
35	51.5	251	141	393	309	393
40	58.8	328	188	516	353	516
45	66.2	416	241	657	397	657
50	73.5	513	300	814	441	814

Notes:
1 - Distance required to decelerate to 90% of the initial speed at 1.0 fpss.
2 - Distance required to decelerate from the 90% speed to the turn speed of 13 fps at 7.0 fpss.
3 - Distance required to "view" gaps in the opposing traffic stream of 6.0 seconds or less.

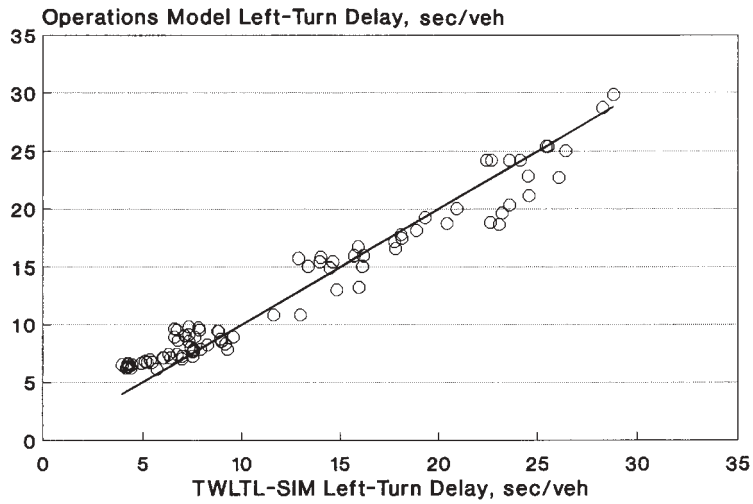

Figure 3-22. Comparison of left-turn delay predicted by the operations model and TWLTL-SIM.

the minimum capacity is argued to represent the maximum time the average driver will wait for service. In a sense, it somewhat crudely accounts for driver reduction of their minimum acceptable gap with increasing delay and their tendency to abort the turn or force their way across the priority stream when delay becomes excessive. The maximum service time used in the operations model for the left-turn movement is 22 sec (i.e., a minimum capacity of 164 vph).

NETSIM's total delay definition and modeling approach makes a direct comparison of delays between NETSIM and the operations model impossible. However, the data in Figure 3-23 do indicate that there is general agreement in delay trend and that NETSIM delay is consistently about 20 percent higher than the delay of the operations model. This increase is consistent with the differences in delay definition, as discussed in a preceding section. In general, the trends shown in Figures 3-22 and 3-23 suggest that the operations model is able to replicate the effects of opposing volume, left-turn percentage, and access point density on left-turn delay.

Delays to Major Street Through Vehicles Caused by Left-Turn Bay Overflow. As stated previously, the objective of this component of the verification process was to demonstrate the ability of the operations model to predict the average delay to major-street through movement caused by left-turn bay overflow. The volume variations and geometric configurations shown in Table 3-22 for the Type 2 scenario were used as the basis for this verification.

Figure 3-24 compares the through vehicle delay caused by left-turn bay overflow predicted by the operations model

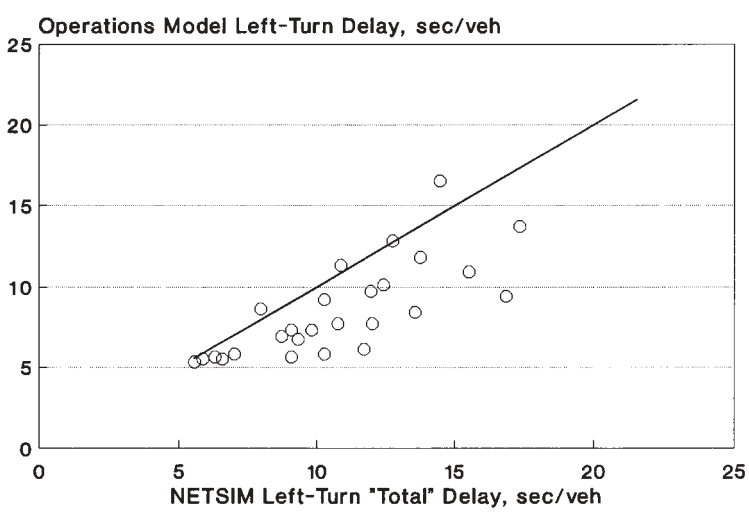

Figure 3-23. Comparison of left-turn delay predicted by the operations model and NETSIM.

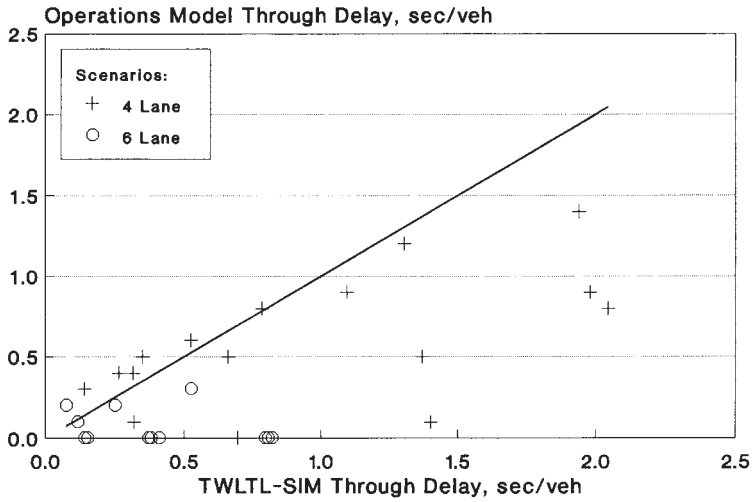

Figure 3-24. Comparison of through delay (due to bay overflow) predicted by the operations model and TWLTL-SIM.

with the delay predicted by TWLTL-SIM for the Type 2 scenario. The data in this figure suggest that the operations model tends to predict lower through vehicle delays than TWLTL-SIM, particularly for the six-lane geometry.

The difference between the delays predicted by the operations model and those predicted by TWLTL-SIM can be explained by the driver behavior modeling limitations discussed previously. Specifically, TWLTL-SIM drivers do not make lane-choice decisions based on traffic conditions several hundred feet downstream. Instead, they are constrained to react when they are within a few seconds' travel time to the back of a queue. As a result, they are more likely to get caught behind left-turn queues and incur larger delays than would real-world drivers. This limitation becomes more evident at higher volume levels; as a result, TWLTL-SIM delay probably exceeds the true delay by an amount that increases with volume level. The operations model does not share this behavioral limitation because it includes a calibrated lane flow rate model. Thus, this comparison suggests that the delays predicted by the operations model are much closer to the delays that are actually incurred.

The effect of TWLTL-SIM's inability to model lane-choice decisions is especially evident in the six-lane geometry. In this situation, the operations model distributes most of the through traffic into the outer two lanes, reflecting driver desire to avoid even the possibility of bay overflow. In contrast, TWLTL-SIM's drivers do not have this predisposition and unrealistically choose the inside lane as often as they choose either of the other two lanes.

Despite this limitation of TWLTL-SIM, the trends shown in Figure 3-24 suggest that the operations model is able to predict through vehicle delays caused by bay overflow. The disagreement between the two model predictions is likely a result of TWLTL-SIM's inability to accurately model driver lane-choice decisions well in advance of a bay overflow condition.

Figure 3-25 compares the through vehicle delay caused by left-turn bay overflow predicted by the operations model with that predicted by NETSIM for the Type 2 scenario. As the data in this figure indicate, the operations model tends to predict lower through vehicle delay than NETSIM, particularly for the six-lane geometry. This trend is similar to that found in the TWLTL-SIM results noted in the preceding paragraph, although it is lower in magnitude.

As discussed previously, both NETSIM and TWLTL-SIM drivers do not look relatively far downstream in an attempt to avoid congestion. However, NETSIM drivers appear to look a little farther than TWLTL-SIM drivers because they have a tendency not to get caught in queue as often. As a result, the delays to the average through driver predicted by NETSIM are higher than those likely incurred by real-world drivers (but have less error than those predicted by TWLTL-SIM). Based on this analysis, it appears reasonable to conclude that the operations model is able to replicate the effects of approach volume and left-turn percentage on through vehicle delay and that it does not share the behavioral limitations exhibited by TWLTL-SIM and NETSIM.

Delays to Major Street Through Vehicles Caused by Turning Vehicle Deceleration. As stated previously, the objective of this component of the verification process was to demonstrate the ability of the operations model to predict the average delay to the major-street through movement caused by a right-turn maneuver, relative to that predicted by NETSIM. The volume variations and geometric configurations shown in Table 3-23 for the Type 3 scenario were used as the basis for this verification.

Figure 3-26 compares the through vehicle delay caused by turning vehicle deceleration predicted by the operations model with that predicted by NETSIM for the Type 3 scenario. As the data in this figure indicate, the operations model

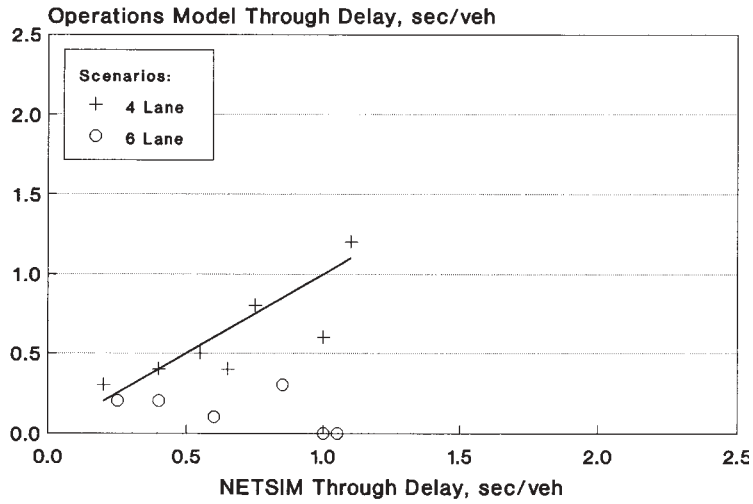

Figure 3-25. Comparison of through delay (due to bay overflow) predicted by the operations model and NETSIM.

is generally able to predict the delay to through vehicles as predicted by NETSIM.

As the data in Figure 3-26 indicate, there is a slight tendency for the operations model to predict lower delays than NETSIM at the higher delay values. However, the reason for this discrepancy stems from the NETSIM driver's inability to look sufficiently far downstream to avoid congestion, as previously discussed. In general, this behavioral limitation tends to increase the NETSIM-predicted through delays slightly beyond the delays that would be incurred by real-world drivers. The operations model has accounted for this behavior; hence, it does not share this limitation. Thus, the data shown in Figure 3-26 suggest that the operations model is able to replicate the effects of approach volume and right-turn percentage on through vehicle delay and that it does not share the behavioral limitations exhibited by NETSIM.

Conclusions

In verifying the operations model, several limitations were found in the two comparator models: TWLTL-SIM and NETSIM. These limitations relate to deficiencies in the ability of these models to accurately replicate driver behavior on urban arterials with closely spaced access points. Of the two models, TWLTL-SIM has fewer limitations than NETSIM.

These behavioral limitations tend to result in TWLTL-SIM and NETSIM predicting through and left-turn move-

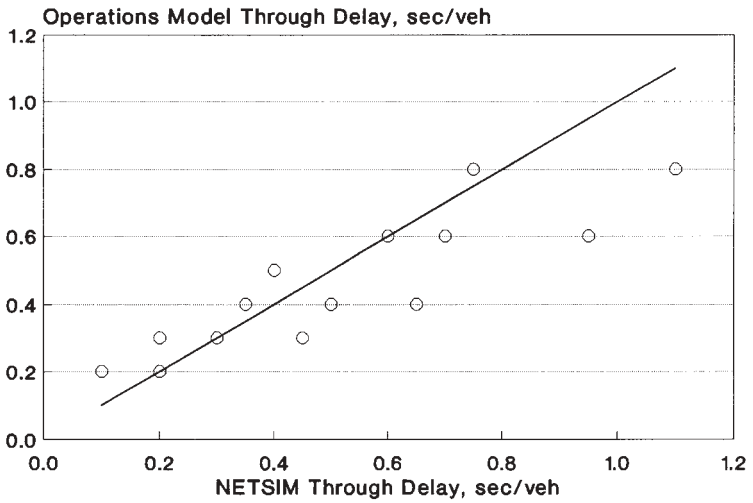

Figure 3-26. Comparison of through delay (due to right-turn deceleration) predicted by the operations model and NETSIM.

ment delays that are believed to be higher than those that would be incurred by real-world drivers. It was determined that some of the behavioral limitations of TWLTL-SIM and NETSIM could be overcome by adherence to specified minimum link lengths and elimination of certain high-volume scenarios. Through the use of these techniques, it was concluded that these two models could still be used to verify the predictive ability of the operations model.

Three measures of effectiveness predicted by the operations model were verified with the TWLTL-SIM and NETSIM models: major-street left-turn delay, major-street through delay caused by left-turn bay overflow, and major-street through delay caused by right-turn deceleration. The results of these verifications indicated that the operations model produces delays with the same sensitivities to traffic demand and geometry as TWLTL-SIM and NETSIM.

Although differences did exist between the delays predicted by the operations model and both TWLTL-SIM and NETSIM, the trends in these differences could be explained by differences in delay definition, various TWLTL-SIM and NETSIM modeling limitations, or both. Based on this verification, it was concluded that the operations model is able to predict delays to major-street vehicles caused by various midblock left-turn treatments with an accuracy that equals or exceeds that of TWLTL-SIM or NETSIM.

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CHAPTER 4

SAFETY EFFECTS OF MIDBLOCK LEFT-TURN TREATMENTS

This chapter describes the development of a model for predicting the safety of alternative midblock left-turn treatments. In this context, a treatment's safety is defined as the expected annual number of accidents that would occur on a street segment with a specific treatment type. The midblock left-turn treatments considered are raised-curb median, flush median delineated as a two-way left-turn lane (TWLTL), and undivided cross section (i.e., no median). The following sections describe a review of the literature on treatment safety, the details of a database assembled for calibrating a safety model, and the formulation of and statistical foundation for this model.

LITERATURE REVIEW

Several terms are used in the literature to describe a location of unsignalized access to a major street:

- Access points—All unsignalized access locations. An access point can be either a driveway or a public street approach.
- Driveway—Any location on the arterial where the curb along the outside lane is removed (or dropped) for 10 ft or more to facilitate vehicular access to the adjacent property.
- Access point density—Total number of access points on *both* sides of the major-street segment (i.e., a two-way total) divided by the length of the segment (in miles). Driveway density and public street approach density are defined in a similar manner.

At this point it is useful to clarify the meaning of other terms found in the literature, because they are used in this report. It is assumed that a major street is classified as an arterial and a minor street is classified as a collector, local street, or driveway. Hence, the major street is also referred to as the "arterial" in this report. The through traffic movements on this arterial are referred to as "priority" movements; all other driveway-related movements are "nonpriority" movements.

The review of the literature dealing with safety assessment of various midblock left-turn treatments revealed three different assessment approaches:

1. Before-and-after analysis of accident data
2. Comparative (or cross-section) analysis of accident data
3. Observation and analysis of traffic conflict data.

The first two approaches are more direct in their evaluation of safety, whereas the last approach is based on an assumed correlation between traffic conflicts and safety. Because the latter approach represents an indirect method of safety assessment, findings from studies of this type are generally difficult to interpret and apply. Consequently, the majority of research on left-turn treatment safety focuses on either the first or second approach.

Before-and-After Approach

In a before-and-after study, the effect of a treatment is assessed by comparing the accident frequency before and after the treatment's installation. An inherent assumption with the before-and-after study is that there is little or no change in the geometry or traffic characteristics of the roadway, other than the left-turn treatment. Thus, a change in accident frequency or type can be reasonably attributed to the new left-turn treatment. This type of study is sometimes supplemented with a control site to account for natural changes in traffic demand and accident trend during the study period.

The primary disadvantage of the before-and-after approach is that it typically includes some degree of regression-to-the-mean (RTTM). Hauer and Lovell (*1*) have shown that the bias from this effect can be quite high if the sites for treatment are not selected randomly. In general, data with RTTM artifacts will exhibit a strong tendency toward significant accident reduction in the "after" period; however, this reduction will be a consequence of the natural tendency of accident frequency to gravitate back to the true mean. Because most agencies select high-accident sites for treatment (i.e., nonrandom selection), the bias from agency-selected projects can be quite significant.

The literature review identified eight before-and-after studies of midblock left-turn treatments. The results of an additional study were obtained during the survey of practitioners. Of the nine studies, seven represent changes from an undivided cross section to a TWLTL. The findings reported

in these studies are shown in Table 4-1. All reduction percentages were reported to be statistically significant.

There are three trends suggested by the accident reduction percentages shown in this table. First, the conversion from undivided cross section to flush center lane (with either paint-delineated bays or a TWLTL) reduces all accidents by about one-third. Second, the conversion from TWLTL to raised-curb median also reduces all accidents by about one-third. Third, the percentages suggest that midblock accidents are reduced by nearly 50 percent after conversion from undivided cross section to flush center lane. Midblock accidents are accidents that occur on the major street but are not related to an intersection. In this context, an intersection is defined as the junction of the major street with any public street; this intersection can be signalized or unsignalized.

One conclusion that might be reached from these trends is that raised-curb median treatments are the safest. This conclusion was reached by Parsonson et al. (*2*). The conclusion has merit because the raised-curb median has the most positive delineation of all midblock treatments. The raised curb ensures that left-turn maneuvers will occur at specific locations and that the turns from the major street are protected by the exclusive bay design.

The magnitude of accident reduction must be interpreted cautiously. It is entirely likely that there are some RTTM artifacts in the data. These artifacts could easily account for as much as 15 percent of the reduction percentage, depending on the number of years of accident data available and the number of accidents that occurred.

Several factors discourage the generalization of accident reduction potential based on these types of studies. First, RTTM artifacts vary among the studies. Second, the types of accidents considered are not consistent between studies (three categories are shown in Table 4-1). Finally, differences in accident reporting threshold among cities and states may bias the count of some accident types.

Despite the aforementioned factors, Glennon et al. (*3*) have generalized the accident reduction potential of the TWLTL relative to the undivided cross section. Based on a review of previous studies, Glennon et al. determined that the TWLTL is effective in reducing accidents by 35 percent. This finding is consistent with that noted previously for Table 4-1. An extensive review of the literature and nationwide survey of TWLTL experience by Nemeth (*4*) revealed a similar finding. In addition, Nemeth found that the frequency of head-on collisions in TWLTLs was negligible.

Comparative Approach

In a comparative evaluation, accident histories for a cross section of sites with a given midblock left-turn treatment are statistically examined and compared with the histories of other sites with a different treatment. Differences between sites, such as traffic demand, speed, and segment length, are accounted for using regression-based procedures.

Several researchers have investigated the safety effects of both raised-curb medians and TWLTLs using the compara-

TABLE 4-1 Before-and-after study findings

Source	Before			After			No. of Sections	Total Section Length (mi)	Year Installed	Accident Category[2]		
	Thru Lanes	Median Type[1]	Duration (yrs)	Thru Lanes	Median Type[1]	Duration (yrs)				All	Affected	Mid-block
Parsonson (2)	6	TWLTL	1	6	RM	1	1	4.3	1990	-37%	na	-55%
Thomas (24)	4	NM	1	4	FM	1	1	4.0	1962	-28%	na	na
Thakkar (14)	2	NM	2	2	TWLTL	2	16	7.4	1970's	-32%	-46%	na
Kastner (25)	2	NM	2	2	TWLTL	2	1	0.8	1987	na	na	-50%
Thakkar (14)	4	NM	2	4	TWLTL	2	15	11.3	1970's	-28%	-40%	na
Kastner (25)	4	NM	2	4	TWLTL	2	1	2.2	1987	na	na	-45%
Harwood (26)	4	NM	1.5 - 3	4	TWLTL	1.5 - 3	17	10.9	1980's	-44%	na	-45%
Burritt (27)	4	NM	2	4	TWLTL	2	7	12.2	1977	-36%	na	na
Hoffman (13)	4	NM	1	4	TWLTL	1	4	6.6	<1973	na	-33%	na

Notes:

na - data not available.

1 - Midblock left-turn treatment type: RM = raised-curb median, TWLTL = flush median with two-way left-turn lane delineation, FM = flush median with alternating left-turn bays, NM = undivided cross section (no median).

2 - Accident category (a negative number denotes a reduction in accidents): All = all accidents occurring on the major street including those on the major-street approach to an intersection with a public street; Affected = left-turn, rear-end, and side swipe accidents occurring on the major street including those on the major-street approach to an intersection with a public street. Midblock = all accidents occurring on the major street excluding those on the major-street approach to an intersection with a public street.

tive approach. The most comprehensive model was developed by Harwood (*5*), who conducted a comparative analysis of 420 highway segments, each segment having one of the following combinations of left-turn treatments and number of through lanes:

- Two-lane with an undivided cross section
- Two-lane with a TWLTL
- Four-lane with an undivided cross section
- Four-lane with a divided cross section (i.e., raised-curb median)
- Four-lane with a TWLTL.

Based on a statistical analysis of the accident data for unsignalized and midblock locations, Harwood (*5*) developed a model for predicting the accident rate for the five treatment combinations studied. This model is presented in Table 4-2.

The accident rates listed in Table 4-2 indicate that the two-lane TWLTL design is safer than the two-lane undivided design and that the four-lane TWLTL design is safer than the four-lane undivided design. These trends are consistent with those shown in Table 4-1.

There are, however, several counterintuitive findings suggested by Table 4-2. First, it suggests that the four-lane undivided design is safer than the raised-curb median treatment in residential areas. It also suggests that there is a negligible difference between the two treatment types in commercial areas. Intuition suggests that a raised-curb median would be associated with fewer accidents than the undivided cross section.

Second, the data in Table 4-2 suggest that the TWLTL has a lower accident rate than a raised-curb median. A study by Squires and Parsonson (*6*) of 82 street segments indicated that raised-curb median segments have lower accident rates than TWLTL segments. A similar conclusion was reached by Hartman and Szplett (*7*) in a comparison of raised-curb median and TWLTL segments. The accident data assembled by Chatterjee et al. (*8*) and by Parker (*9*) also indicate that raised-curb median segments have lower accident rates than TWLTL segments.

Third, the data in Table 4-2 suggest that an increased truck percentage results in safer operation. Intuition would suggest that more accidents would occur as the proportion of trucks increases. Harwood (*5*) acknowledges this paradox and explains that it is likely the result of correlations between truck percentage and other model variables.

Other models for predicting the safety of specific midblock left-turn treatments also have been developed. To simplify their presentation and comparison, the following generalized model form is used.

$$A = B_0 ADT^{B_1} Len^{B_2} (linear\ terms) \tag{1}$$

with

$$\begin{aligned} linear\ terms = {} & C_0 + C_1 ADT + C_2 Pop + C_3 Drv \\ & + C_4 Sig + C_5 Unsig + C_6 Strt \\ & + C_7 Trk + C_8 Ltvol + C_9 Dev \end{aligned} \tag{2}$$

where:

A = annual accident frequency;
ADT = average daily traffic;
Len = street segment length, in miles;
Pop = area population;
Drv = driveway density, in driveways/mile;
Sig = signalized intersection density, in signals/mile;
$Unsig$ = unsignalized intersection approach density, in approaches/mile;

TABLE 4-2 Harwood safety model (*5*)

Accident Rates for Midblock Locations and Unsignalized Intersections Combined (accidents per million-vehicle-miles)						
Type of Development	Design Alternative					
	Through Lanes:	2		4		
	Median Type:	Undivided	TWLTL	Undivided	Raised-Curb	TWLTL
Commercial		4.50	3.99	7.62	7.61	5.80
Residential		4.76	3.55	4.00	4.10	3.24
Adjustment Factors						
Driveways/mile				Under 30 -0.41	30 - 60 -0.03	Over 60 +0.35
Intersections/mile				Under 5 -0.99	5 - 10 +0.28	Over 10 +1.55
Truck percentage				Under 5 +0.40	5 - 10 -0.15	Over 10 -0.71

Strt = public street approach density, in approaches/mile;
Trk = truck percentage;
Ltvol = average daily left-turn volume per driveway, in vpd/driveway;
Dev = development type (1 if commercial, 0 if residential); and
B_i,C_i = regression coefficients.

The general model combines linear and nonlinear regression terms. Linear terms consist of any environmental or geometric factors that may be correlated with accident frequency. The advantage of the general form is that it allows any combination of linear and nonlinear factors to be considered. In fact, by setting one or more of the B_i coefficients to 0 or 1, the general form defaults to an equivalent model for predicting accident rates (e.g., accidents/mile and accidents/million vehicle miles (mvm)).

The comparison of alternative models is shown in Tables 4-3, 4-4, and 4-5. The models described in these tables apply to arterials with raised-curb medians, TWLTLs, and undivided cross sections. As suggested by the B_1 parameter coefficients, most models were developed to predict accidents/mile; the others predict accidents/million vehicle miles. Some authors developed equations for two-, four-, and six-lane cross sections; however, only models for four-lane sections are shown. With one exception, the equations predict all accidents occurring on the segment (including those occurring at signalized intersections). The exception is the equation developed by Harwood (*5*). Harwood only developed accident rates for predicting accident frequency at midblock and at unsignalized intersection locations.

Several observations can be made by examining the parameter coefficients in Tables 4-3, 4-4, and 4-5. First, there is agreement that accident frequency increases with traffic demand. Second, all researchers considered driveway density but most found its effect to be insignificant. Those that found a significant effect do not appear to agree on whether accidents increase or decrease with an increasing number of

TABLE 4-3 TWLTL safety models using the general model form

Component	Parameter		Accidents/Mile Models						Accidents / MVM	
	Var	Name	Walton (11)	Parker (9)	McCoy (28)	Squires (6)	Parker (12)	Chatterjee (8)	Harwood[a] (5)	Squires (6)
Exposure (Nonlinear)	B_0	intercept	1	1	1	1	1	1	1	1
	B_1	ADT	0	0	0	0	0	0	1	1
	B_2	Segment Length	1	1	1	1	1	1	1	1
Explanatory (Linear)	C_0	intercept	-43.5	-28.8	9.44	-21.7	-22.3	19.7	1.69	4.01
	C_1	ADT	0.00203	0.00173	0.00214	0.00388	0.00153	0.0035	0[b]	0[b]
	C_2	Population	0.000175	-0.0000058	--	--	--	--	--	--
	C_3	Driveway Density	0.491	0[b]	0[b]	0[b]	0[b]	0[b]	0.013	0[b]
	C_4	Signal Density	9.20	5.43	0[b]	22.7	5.60	0[b]	--	2.29
	C_5	Unsig. App. Density	--	--	0[b]	-8.85	--	0[b]	0.127	0[b]
	C_6	Pub. St. App. Density[d]	--	2.16	--	--	1.94	--	--	--
	C_7	Truck Percentage	--	--	--	--	--	--	-0.111	--
	C_8	Left-turn volume	--	--	--	--	--	--	0[c]	--
	C_9	Development Type	--	--	--	--	--	commercial only	2.56	--
Database	Years of Accident Data		na	3	4	3	3	3-4	5	3
	Number of Sections		na	17	4	42	5	12	135	42
	Total Section Length (mile)		na	12.2	4.35	62.5	na	19.7	91.2	62.5
	R^2		0.75	0.71	0.84	0.60	0.73	0.65	na	0.44
	Through Lanes		4	4	4	4	4	4	4	4

Notes:
-- - Factor is not specifically included in model.
a - Estimated from tabular values provided in the final report.
b - Factor was considered but not found to be statistically significant.
c - Factor was significant but correlated with other, more significant model variables and was excluded from model.
d - Public street approaches include all minor street approaches at either signalized or unsignalized intersections.
na - Not available.

TABLE 4-4 Raised-curb median safety models using the general model form

Component	Parameter		Accidents/Mile Models				Accidents/MVM	
	Var	Name	Parker (9)	Squires (6)	Parker (12)	Chatterjee (8)	Harwood[a] (5)	Squires (6)
Exposure (Nonlinear)	B_0	intercept	1	1	1	1	1	1
	B_1	ADT	0	0	0	0	1	1
	B_2	Segment Length	1	1	1	1	1	1
Explanatory (Linear)	C_0	intercept	-12.7	-14.8	-12.6	11.0	2.55	1.92
	C_1	ADT	0.00155	0.00192	0.00137	0.0035	0[b]	0[b]
	C_2	Population	-0.0000093	--	--	--	--	--
	C_3	Driveway Density	-0.0228	0[b]	0[b]	0[b]	0.013	0[b]
	C_4	Signal Density	8.04	16.1	8.37	0[b]	--	2.72
	C_5	Unsig. Approach Density	--	0[b]	--	0[b]	0.127	0[b]
	C_6	Public St. Approach Density[d]	0[b]	--	0[b]	--	--	--
	C_7	Truck Percentage	--	--	--	--	-0.111	--
	C_8	Left-turn volume	--	--	--	--	0[c]	--
	C_9	Development Type	--	--	--	commercial only	3.51	--
Database		Years of Accident Data	3	3	3	3-4	5	3
		Number of Sections	19	15	3	11	44	15
		Total Section Length (miles)	28.2	24.7	na	19.9	21.8	24.7
		R^2	0.73	0.77	0.84	0.65	na	0.80
		Through Lanes	4	4	4	4	4	4

Notes:
-- - Factor is not specifically included in model.
a - Estimated from tabular values provided in the final report.
b - Factor was considered but not found to be statistically significant.
c - Factor was significant but correlated with other, more significant model variables and was excluded from model.
d - Public street approaches include all minor street approaches at either signalized or unsignalized intersections.
na - Not available.

driveways. Third, there is general agreement that accident frequency increases with increasing signal density.

Finally, most researchers considered some measure of the number of public street approaches and its effect on accident frequency. About one-half found that the effect was insignificant, whereas the remainder could not agree on the effect (increase or decrease). It is possible that correlations among driveway density, signal density, and public street approaches are obscuring the true relationship between cause and effect.

A safety model also has been developed by Bowman and Vecellio (*10*). This model predicts all accidents occurring along the segment, including signalized intersections. This model was not included in the preceding tables because it did not follow the general model form. The model form used by Bowman and Vecellio to predict all accidents is shown in Equations 3 and 4; the model coefficients are shown in Table 4-6.

$$A = B_0 ADT^{B_1} Len^{B_2} e^{(linear\ terms)} \quad (3)$$

with

$$linear\ terms = C_0 + C_1 Thr + C_2 Off + C_3 Bus + C_4 Area + C_5 Med + C_6 Unsig + C_7 Drv + C_8 Cross + C_9 Spd \quad (4)$$

where:

A = annual accident frequency;
ADT = average daily traffic;
Len = street segment length, in miles;
Thr = accident reporting threshold, in dollars;
Off = land use (1 if office, 0 if other);
Bus = land use (1 if business, 0 if other);
$Area$ = area type (1 if CBD, 0 if suburban);
Med = median width, in feet;
$Unsig$ = unsignalized intersection approach density, in approaches/mile;
Drv = driveway density, in driveways/mile;
$Cross$ = median crossover density, in crossovers/mile;
Spd = speed limit, in mph; and
B_i, C_i = regression coefficients.

TABLE 4-5 Undivided cross section safety models using the general model form

Component	Parameter		Accidents/Mile Models		Accidents/MVM
	Var	Name	Parker (9)	McCoy (28)	Harwood[a] (5)
Exposure (Nonlinear)	B_0	intercept	1	1	1
	B_1	ADT	0	0	1
	B_2	Segment Length	1	1	1
Explanatory (Linear)	C_0	intercept	-36.5	101.8	2.45
	C_1	ADT	0.00212	0.0149	0[b]
	C_2	Population	0[b]	--	--
	C_3	Driveway Density	0.557	-4.36	0.013
	C_4	Signal Density	3.06	0[b]	--
	C_5	Unsig. Approach Density	--	0[b]	0.127
	C_6	Public St. Approach Density[d]	-0.264	--	--
	C_7	Truck Percentage	--	--	-0.111
	C_8	Left-turn volume	--	--	0[c]
	C_9	Development Type	--	--	3.62
Database		Years of Accident Data	3	4	5
		Number of Sections	14	5	129
		Total Section Length (miles)	16.6	6.4	73.3
		R^2	0.79	0.82	na
		Through Lanes	4	4	4

Notes:
-- - Factor is not specifically included in model.
a - Estimated from tabular values provided in the final report.
b - Factor was considered but not found to be statistically significant.
c - Factor was significant but correlated with other, more significant model variables and was excluded from model.
d - Public street approaches include all minor street approaches at either signalized or unsignalized intersections.
na - Not available.

Three items can be noted when examining Table 4-6. One item is the reduction in accidents with increasing accident reporting threshold. This factor was included in the model because the data were obtained from three cities in three different states. The significance of this trend suggests that models developed with data from different cities or states may not be totally comparable if the accident reporting threshold is different. This conclusion was reached by Parker (*9*) after comparing his equation with that of Walton and Machemehl (*11*).

A second item worthy of note is the apparent omission of signalized intersection density. Bowman and Vecellio (*10*) explained that signal density was considered but was not found to be statistically significant. They explained that signal density is likely correlated with other model variables and, therefore, the effect of signals is accounted for by these variables.

A third item worth noting in the examination of Table 4-6 is the trend toward fewer accidents with increasing speed. Bowman and Vecellio explained this counterintuitive trend by noting that higher speeds usually occur in areas with lighter development intensity and a corresponding low level of vehicle interaction. It should be noted that speed limit also was considered by Harwood (*5*) and Walton and Machemehl (*11*) but was not found to be statistically significant.

The models described in Tables 4-3 through 4-6 were used to compare the expected accident frequency for each of the left-turn treatment types. The large number of variable combinations prevented an exhaustive analysis; however, a typical combination of variable values was established and used to compute the accident frequency predicted by each model for a range of daily traffic demands. The typical arterial was assumed to have the following attributes:

- Population: 100,000
- Driveway density: 50/mi
- Signal density: 2/mi
- Unsignalized intersection approach density: 8/mi
- Truck percentage: 5
- Development type: Commercial
- Through lanes: 4 (2 each direction)

Additional assumptions required by the Bowman and Vecellio safety model are as follows:

- Accident reporting threshold: $250
- Median width: 16 ft
- Speed limit: 40 mph

TABLE 4-6 Bowman and Vecellio safety model (*10*)

Component	Parameter		Midblock Left-Turn Treatment Type		
	Var.	Name	Raised-Curb Median	TWLTL	Undivided
Exposure (Nonlinear)	B_0	intercept	0.000365	0.000365	0.000365
	B_1	ADT	1	1	1
	B_2	Segment Length	1	1	1
Explanatory (Linear)	C_0	intercept	7.21	3.71	1.88
	C_1	Reporting Threshold	-0.00788	-0.00278	-0.00303
	C_2	Office Land Use	-0.448	0.0723	1.06
	C_3	Business Land Use	0[b]	0[b]	0.657
	C_4	Area Type	0[b]	0[b]	0.457
	C_5	Median Width, Med	-0.0276	0.0354	0[b]
	C_6	Unsig. Approach Density	0[b]	-0.0606	0[b]
	C_7	Driveway Density	0[b]	0.0129	0.0132
	C_8	Crossover Density	0.0962	0[b]	0[b]
	C_9	Speed Limit	-0.070	-0.0339	0[b]
Database		Years of Accident Data	3 - 5		
		Number of Sections	150	178	152
		Total Section Length (miles)	51.9	55.1	38.9
		R^2	na		
		Through Lanes	2, 4, 5, 6		

Notes:
b - Factor was considered but not found to be statistically significant.
na - Not available.

The public street approach density was computed as 12/mi (i.e., two approaches/signalized intersection plus the eight unsignalized intersection approaches). This value represents the critical density recommended by Parker (*9*). Specifically, Parker found that TWLTL treatments are generally safer than raised-curb median treatments if there are fewer than 12 public street approaches; the reverse is true if there are more than 12.

To facilitate the comparison of the Harwood safety model (which predicts only midblock and unsignalized accidents) with the other models, an adjustment factor, the ratio of total accidents to midblock accidents plus unsignalized accidents, was developed. Sufficient data were provided by Harwood (*5*) to make this computation; the resulting adjustment factor was 1.38. Thus, total accidents were computed by first using the Harwood safety model to estimate the midblock accidents plus unsignalized accidents and multiplying this value by 1.38.

The accident frequency predicted by each model for the typical arterial is listed in Table 4-7 for a range of arterial traffic demands. The average and standard deviation of these frequencies for the group of models also are listed in Table 4-7 by traffic demand and left-turn treatment type. The effect of traffic demand on the average accident frequency for each treatment type is shown in Figure 4-1.

Two researchers have two models listed in Tables 4-3, 4-4, and 4-5; however, only one model from each source was used in this comparison to avoid bias in the computed average. In particular, the "accidents/mile" model from Squires and Parsonson (*6*) was used because it had a better R^2 than his "accidents/MVM" model. The model described by Parker (*12*) was used because it is based on an expanded database that included data from his first model.

Examination of the data in Table 4-7 indicates several trends worth noting. First, the undivided treatment has the highest accident frequency over the range of traffic demands. These data support the 30 to 35 percent accident reduction found in the before-and-after studies discussed previously (for the conversion from undivided cross section to either raised-curb median or TWLTL). The TWLTL and raised-curb median tend to have about the same accident frequency, with the raised-curb median having a slightly lower frequency for moderate to high volumes.

It must be remembered that the predicted accident frequencies are based on a series of assumed arterial attributes; other attributes could change the magnitude of the observed relationships. In addition, there is considerable variability among the predicted values; thus, the magnitude of the difference between TWLTL and raised-curb median is probably not statistically nor practically significant.

TABLE 4-7 Safety model comparison

	Expected Accidents / Mile / Year											
ADT:	10,000			20,000			30,000			40,000		
Left-Turn Treatment:	TWLTL	Raised Median	Un-divided	TWLTL	Raised Median	Un-divided	TWLTL	Raised Median	Un-divided	TWLTL	Raised Median	Un-divided
Walton (11)	37	na	na	58	na	na	78	na	na	98	na	na
McCoy (28)	31	na	33	52	na	oor	oor	na	oor	oor	na	oor
Squires (6)	-8	37	na	31	56	na	69	75	na	108	94	na
Parker (12)	27	18	na	43	32	na	58	45	na	73	59	na
Chatterjee (8)	55	46	na	90	81	na	125	116	na	oor	oor	na
Harwood (5)	27	36	36	54	72	72	81	108	109	108	144	145
Bowman (10)	43	25	63	85	50	126	128	75	190	170	101	253
Average Freq.	30	32	44	59	58	99	90	84	149	112	100	199
Std. Deviation	7	5	10	8	9	27	12	13	41	16	18	54

Notes:
na - Model not available or developed for this midblock left-turn treatment type.
oor - Traffic demand exceeds range of data used to calibrate the model.

A second item worth noting in the examination of Table 4-7 is that some models tend to consistently predict to the extreme. In particular, the Parker (*12*) safety model often yields the lowest accident frequency. On the other hand, the Bowman and Vecellio (*10*) safety model often yields the highest accident frequency. Differences, particularly consistent differences, among models might be explained by differences in accident reporting threshold in the underlying accident database.

Pedestrian Accidents

Bowman and Vecellio (*10*) examined the frequency of pedestrian-vehicle accidents on arterials with different midblock left-turn treatments. Their database included 1,012 pedestrian-vehicle accidents. They found that pedestrian accidents were more frequent in central business districts (CBDs) than in suburban areas. About 7 percent of all accidents on CBD streets involved pedestrians, whereas only 2 percent of all accidents on suburban streets involved pedestrians.

Bowman and Vecellio converted the pedestrian-vehicle accident counts to accident rates by dividing by 1 million vehicle miles. These rates indicate that raised-curb medians have about 0.2 pedestrian accidents per mvm (pa/mvm) on CBD streets and 0.06 pa/mvm on suburban streets. Streets with TWLTLs were found to have about twice the number of pedestrian accidents; however, the difference was not statistically significant.

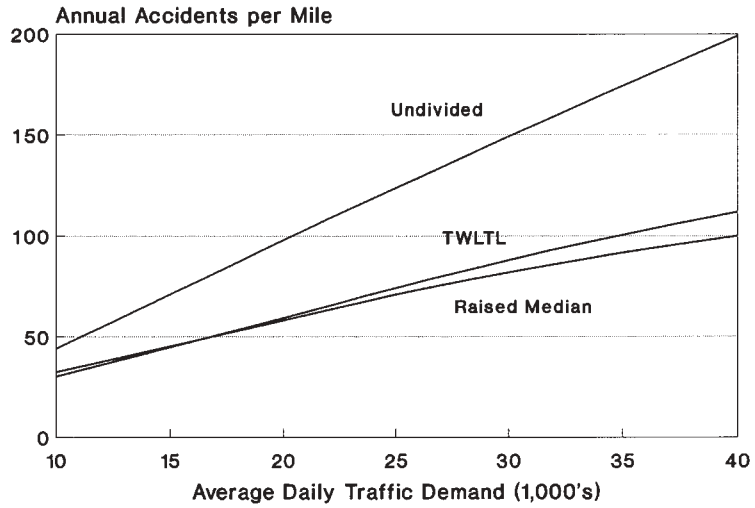

Figure 4-1. Predicted average accident frequency comparison.

Streets with undivided cross sections were found to have even higher pedestrian accident rates, particularly for CBD streets. The rate was found to be 0.87 pa/mvm for CBD streets and 0.14 pa/mvm for suburban streets. The difference between undivided and TWLTL treatments on CBD streets was statistically significant; however, the difference between these two treatments on suburban streets was not significant.

These rates suggest that raised-curb medians are safer for pedestrians than undivided cross sections. They also suggest that raised-curb medians are safer for pedestrians than TWLTLs, although the differences are not as distinct (nor statistically significant). It should be noted that Parker (*9*) conducted a study of pedestrian accidents and found that raised-curb medians are associated with about one-half as many pedestrian accidents as TWLTLs.

Accident Severity

Several studies have examined the effect of left-turn treatment on accident severity. Harwood (*5*) found that midblock fatal and injury accidents combined accounted for 38.4, 33.7, and 33.7 percent of all accidents on four-lane streets with undivided, raised-curb median, and TWLTL treatments, respectively. Hoffman (*13*) found that conversion from an undivided cross section to a TWLTL resulted in a 41 percent reduction in casualties. For a similar conversion, Thakkar (*14*) found that the total fatal and injury accident rate (including both midblock and intersection) decreased by 26 percent. On the other hand, Bowman and Vecellio (*10*) found that TWLTLs were associated with a slightly higher midblock injury accident rate than undivided cross sections; although the difference was not statistically significant. They also found that the raised-curb median had a significantly lower injury accident rate (about 30 percent lower) than either the TWLTL or undivided cross section.

The findings of these studies are somewhat contradictory and difficult to compare because of the wide variety of severity measures used by the researchers. It is likely that the contradictions may be due to a combination of (1) high variability in accident data, (2) small number of observations (injury accidents may account for only 30 to 40 percent of all accidents), (3) inconsistency between studies in the types of accidents included in the database, and (4) differences in analysis approach (e.g., Harwood and Bowman and Vecellio used the comparative approach whereas Hoffman and Thakkar used the before-and-after approach).

DEVELOPMENT OF A SAFETY DATABASE

The development of a safety model required the assembly of a database containing geometric, traffic, and accident data for typical urban and suburban arterials. The arterials included in the database have one of three midblock left-turn treatment types: raised-curb median, TWLTL, and undivided cross section. This database was assembled from the accident records of the city of Phoenix, Arizona, and the city of Omaha, Nebraska. It contains data on 189 street segments that total 78.6 mi in length.

Database Composition

Study Segment Attributes

A list of desirable characteristics for the study segments was prepared using information from the survey of practitioners and a review of the literature. For the purpose of this research, a study segment was defined as an urban or suburban arterial street segment bounded by signalized intersections but having only unsignalized access points along its length. This segment has a constant cross section and one type of midblock left-turn treatment. The following criteria were used to define an urban or suburban arterial segment, as related to the study objectives:

- Traffic volume more than 7,000 vpd
- Speed limit between 30 and 50 mph
- Spacing of at least 350 ft between signalized intersections
- Direct access from abutting properties
- No angle curb parking (parallel parking is acceptable)
- Located in or near a populated area (e.g., 20,000 or more)
- No more than six through traffic lanes (three each direction)
- Arterial length of at least 0.75 mi.

The application of these criteria in selecting the study sites was intended to ensure that low-volume two-lane roadways, rural highways, expressways, roads through small towns, and low-speed collector streets were not included in the candidate list of field study sites.

The arterials selected for inclusion in the database have one of the following three midblock left-turn lane treatment types: raised-curb median, TWLTL, or undivided cross section. These three treatment types were selected for two reasons. First, they represent the most distinctly different midblock left-turn treatment types (many types represent only slight variations from the three types considered). Second, these treatments are the most frequently used in practice. This reason is very important because it relates to the availability of street segments in sufficient number to provide some statistical reliability in the calibrated model.

Database Elements

The types of data needed for the safety model database include the geometric characteristics, traffic characteristics, and accident history of each study site. These data were col-

lected from agency accident records, traffic counts, aerial photography, and data obtained during site visits. A list of data elements and their sources is presented in Table 4-8.

The elements included in the database were selected for a variety of reasons. In most cases, the elements found to be helpful in previous models are included. A distinction between CBD and suburban streets (i.e., area type) is not included because it is likely to be positively correlated with speed limit. Bowman and Vecellio (*10*) considered both factors and found each to be significant, but were unable to put both factors in their models at the same time because of the factors' correlation. It is anticipated that speed limit will provide the necessary distinction between suburban and CBD areas.

Data Collection Approach

Traffic and accident data were obtained from the cities of Phoenix and Omaha. Accident data were obtained for 1991, 1992, and 1993. Staff from each city helped identify the study segments and provided annual average daily traffic estimates that were representative of the three study years. The selected study segments were videotaped in both directions while researchers drove their lengths. These videotapes provided important land use, traffic, and geometric information. Finally, portions of each segment were surveyed to obtain measurements of selected cross section components.

One element that was proposed for inclusion in the safety database was the accident-cost-reporting threshold. Bowman and Vecellio (*10*) and Parker (*9*) have found that the number of accidents reported is related to the damage cost threshold at which the law requires that an accident be reported. Specifically, accident frequency has been found to be lower in areas with higher reporting thresholds. This element is often essential when comparing accident frequencies between cities in different states. The drawback with using it is that it is not generally known or adhered to by the motoring public nor is it followed by police (*15*). This drawback typically leads to

TABLE 4-8 Safety database elements

Element	Source		
	Agency Computer Files	Field Survey/ Site Videolog	Agency Documents
Geometric Characteristics			
Midblock left-turn treatment type	--	P	S
Number of through lanes	--	P	S
Segment length	--	P	S
Cross section width	--	P	S
Median width	--	P	S
Number of driveways	--	P	--
Number of unsignalized public street approaches	--	P	--
Type of development/adjacent land use	--	P	S
Type of curb parking	--	P	S
Traffic Characteristics			
Average daily traffic (ADT)	--	--	P
Speed limit	--	P	S
Accident Data			
Location (milepost)	P	--	--
Date	P	--	--
Time of day	P	--	--
Severity (number of injuries and fatalities)	P	--	--
Driver condition (esp. influence of alcohol or drugs)	P	--	--
Street surface condition (esp. ice or snow)	P	--	--
Collision type (manner struck)	P	--	--
Accident type (entity collided with)	P	--	--
Intersection relationship (intersection or midblock acc.)	P	--	--
Intersection control (signal, no signal)	P	S	--

Notes:
P - primary data source
S - secondary data source

varying degrees of underreporting among cities and states. Consequently, correlations between reporting threshold and accident frequency are likely to be unstable and difficult to quantify on a regionwide basis. Because of this uncertainty, the accident reporting threshold was dropped from further consideration in the development of the safety model.

A surrogate measure, therefore, was used to account for differences in reporting levels among cities and states. This surrogate was defined as the percentage of all accidents that are described as property-damage-only (PDO) in the city of interest. This measure is related to the reporting threshold because PDO accidents typically are in question and generally go unreported. One advantage of using this measure is that it is a very tangible measure of the true behavior of drivers in a specific city or state in terms of their propensity to report an accident. As a result, it can be used to compare cities with different reporting thresholds or different degrees of underreporting. Another benefit of this measure is that it is represented as a statistic of the accident database instead of a value printed in some legal statute that is likely to be invoked to widely varying degrees. The PDO percentage typically varies from 60 to 75 percent for most cities and states based on the data reported by Bowman and Vecellio (*10*) and Harwood (*5*). Lower values would reflect regions with higher reporting thresholds, a larger number of unreported accidents, or both.

Database Summary

Summary statistics describing the safety database are provided in Tables 4-9 and 4-10. The arterial mileage for each city and treatment type was maintained at approximately equal values to avoid bias in the statistical analysis.

The midsignal accident frequency represents the total number of accidents actually occurring on the arterial segments. Accidents occurring at the signalized intersections that bound these segments are not included in the database. These accidents tend to have their own causes (e.g., change interval timing) that are not related to the midblock left-turn treatment. Hence, their inclusion in the database would only obfuscate the search for factors related to midsignal accident frequency. The term "midsignal" is used in Table 4-9 instead of "midblock" because it better conveys the fact that accidents occurring at midblock locations and at unsignalized public street intersections (major-street approaches only) are included in the midsignal accident frequency.

Whether an accident was associated with one of the bounding signalized intersections was determined from the accident report. Both the Omaha and Phoenix accident databases include a field that identifies the accident as "intersection-related." Discussions with the engineers responsible for maintaining the accident database in the two cities indicated that this field was coded by technicians per the engineer's instructions. These instructions were to (1) consider the accident report information (including time of day, accident type, path-to-collision sketch, distance from the intersection, and driver/officer comments), (2) combine this with other information about the intersection and its queueing potential, and (3) make a subjective determination about the intersection relationship of the accident.

The use of the "intersection-related" field is intuitively more defensible for identifying intersection accidents than

TABLE 4-9 Safety database summary—total mileage and accidents

Element	Location		Total
	City of Omaha, NE	City of Phoenix, AZ	
Total Mileage by Midblock Left-Turn Treatment Type			
Raised-Curb Median	12.6 miles	12.5 miles	25.1 miles
Two-Way Left-Turn Lane (TWLTL)	10.8 miles	16.7 miles	27.5 miles
Undivided Cross Section	10.7 miles	15.3 miles	26.0 miles
Total:	34.1 miles	44.5 miles	78.6 miles
Total Accident Frequency by Severity[1]			
Property Damage Only Accidents	2,510 (72%)	2,389 (64%)	4,899
Personal Injury Accidents	970 (28%)	1,321 (35%)	2,291
Fatal Accidents	8 (0%)	12 (1%)	20
Total:	3,488	3,722	7,210
Mid-Signal Accident Frequency[2]	2,988	3,403	6,391

Notes:
1 - Total accident frequency includes all accidents occurring at signalized intersections, unsignalized intersections, and midblock locations on the study segments.
2 - Mid-signal accidents include all accidents occurring on the major street excluding those occurring on the approach to the bounding signalized intersections.

TABLE 4-10 Safety database summary—range of individual database elements

Element	Range	
	City of Omaha, NE	City of Phoenix, AZ
Geometric Data		
Number of through traffic lanes	4 - 6	2 - 6
Segment length, feet	554 - 7,978	360 - 5,280
Cross section width, feet	40 - 92	22 - 100
Median width, feet	0 - 18	0 - 30
Driveway density, drives/mile	0 - 108	0 - 116
Unsignalized public street approach density, app./mile	0 - 31	0 - 24
Speed limit, mph	30 - 45	25 - 45
Traffic Data		
Average daily traffic	6,000 - 56,700	3,000 - 52,300

simply using only the "distance from the intersection" field, as is often done by researchers. A check of the Phoenix dataset, which has both the "distance" and "intersection-related" fields, indicated that 98 percent of all intersection-related accidents occurred within 100 ft of the intersection. This strong correlation indicates why distance is often used alone; nevertheless, the intersection-related field is considered preferable.

To facilitate the examination of accident cause and effect in the context of a statistical analysis of street geometry and traffic demand, accidents caused by extraordinary or special circumstances were excluded from the safety database. Specifically, accidents in which the driver was under the influence of drugs or alcohol were excluded, as were accidents occurring on ice- or snow-covered streets.

A more detailed summary of the safety database is provided in Tables 4-11 and 4-12. Table 4-11 presents statistics for six selected segment attributes; each statistic is categorized by treatment type and location. In general, the ranges of each attribute are quite wide and overlap among the three treatment types. The segments with the raised-curb median treatment tend to have the highest ADT, and those with the undivided treatment have the lowest ADT. Segments with the raised-curb treatment tend to have the lowest driveway and public street approach densities, whereas those with the undivided treatment tend to have the highest densities. Speed limit and segment length have about the same range of values for all treatments.

Table 4-12 summarizes the total number and length of arterial segments included in the safety database. As mentioned previously, control over site selection was exercised to ensure a general equality in both number and length among the three treatment types and two cities.

The land use categories included in this database refer to the facilities generating vehicle-related trips into and out of the land adjacent to the study segment. The four categories are residential, office, business, and industrial. Residential indicates land use that varies from single-family dwellings to apartment complexes. Weekday trips from this land use category occur most frequently during the morning and evening peak traffic hours. Office refers to land use in which weekday trips are made primarily by professional employees (and associated staff) arriving in the morning and leaving in the evening. The office land use category also incorporates customer trips occurring throughout the day, but at a lower rate than that of employee trips during the morning and evening. Business land use is associated with trips made primarily by customers on a stop-by-and-shop basis; these trips occur throughout the business day. Industrial refers to land use in which nonprofessional employees constitute the largest number of trips, primarily taking place during shift changes in the morning and late afternoon. In situations in which land use varies along the segment, the segment's land use was categorized based on the most dominant land use (measured in terms of trip activity observed during the field survey).

Parallel parking was found only on segments with an undivided cross section. This distinction between the undivided and the TWLTL or raised-curb segments was not intended; however, this is a characteristic of the two cities chosen for the study.

DEVELOPMENT OF A SAFETY MODEL

This section describes the development and calibration of the safety model. The findings presented are the result of a formal statistical analysis of the safety database described in the preceding section. This section includes the development of the safety model, the statistical approach used to calibrate the model, an examination of the accident rates for the treat-

TABLE 4-11 Safety database summary—statistics for selected elements

Treatment Type:		Raised-Curb			TWLTL			Undivided			Overall
	Location:	Omaha	Phoenix	Both	Omaha	Phoenix	Both	Omaha	Phoenix	Both	
Average daily traffic	Avg.	32,336	34,915	33,516	22,466	34,214	28,858	22,946	16,460	19,285	27,172
	Std.	10,295	13,099	11,629	8,189	7,511	9,751	5,924	8,431	8,068	11,441
	Min.	20,600	9,500	9,500	8,000	15,800	8,000	6,000	3,000	3,000	3,000
	Max.	56,700	52,300	56,700	38,700	52,200	52,200	37,100	29,300	37,100	56,700
Segment length (feet)	Avg.	2,080	2,450	2,249	1,851	2,381	2,139	2,083	2,308	2,210	2,197
	Std.	982	1,378	1,183	734	980	910	1,358	1,237	1,285	1,125
	Min.	634	710	634	676	992	676	554	360	360	360
	Max.	4,567	5,280	5,280	3,971	5,280	5,280	7,978	5,280	7,978	7,978
Driveway density (drv/mi)	Avg.	15	34	24	41	58	50	55	57	56	44
	Std.	18	23	22	22	23	24	28	22	25	27
	Min.	0	6	0	4	12	4	2	0	0	0
	Max.	70	90	90	96	110	110	108	116	116	116
Unsig. pub. st. approach density (app/mi)	Avg.	7	11	9	13	10	11	13	13	13	11
	Std.	6	5	6	7	6	7	5	8	7	7
	Min.	0	0	0	0	0	0	4	0	0	0
	Max.	19	24	24	31	22	31	23	24	24	31
Cross section width (feet)	Avg.	66	80	73	56	64	60	49	49	49	60
	Std.	7	12	12	2	4	5	8	12	10	13
	Min.	62	55	55	52	58	52	40	22	22	22
	Max.	92	100	100	60	84	84	60	68	68	100
Speed limit (mph)	Avg.	41	39	40	36	39	38	35	34	34	37
	Std.	2	5	4	2	3	3	3	3	3	4
	Min.	35	25	25	35	35	35	30	30	30	25
	Max.	45	45	45	40	45	45	40	40	40	45

ment types, a description of the calibrated safety model, and a sensitivity analysis of selected model variables.

Evaluation Methodology Framework

The generalized linear modeling (GLIM) technique, described by McCullagh and Nelder (*16*), was used to determine the model coefficients and the statistical quality of fit to the safety data. The GLIM technique uses maximum-likelihood principles to model the distribution of residual errors. This distribution is typically neither normal nor of constant variance, as is assumed when using traditional least-squares regression. As a result, the GLIM regression technique is able to yield unbiased parameter coefficients having the smallest standard error possible. This technique has been applied to accident data by several researchers, including Hauer et al. (*17*), Bowman and Vecellio (*10*), and Bonneson and McCoy (*18*). In fact, Bonneson and McCoy (*18*) have developed procedures for automating the GLIM technique using the SAS statistical analysis software (*19*).

Terminology

The safety, m, of an arterial segment is defined as its mean accident frequency. This quantity can be estimated by taking the average of the m's (i.e., $E(m)$) for a large number of similar segments, each having identical traffic demands. In this context, similar segments have one or more geometric and traffic control characteristics in common. The estimate of m becomes more stable as the segments become more similar (i.e., as the number of characteristics that they must have in common increases).

In the past few years, Hauer et al. (*17*) and others have convincingly argued that the distribution of accident counts for a group of similar sites (e.g., intersections and street segments) can be described by the family of compound Poisson

TABLE 4-12 Safety database summary—number and length of segments

Location:			Omaha		Phoenix		Total	
Treatment Type	Feature		Miles	No. of Seg.	Miles	No. of Seg.	Miles	No. of Seg.
Raised-Curb	Land use	Res	2.5	6	8.5	14	11.0	20
		Off	2.0	5	0.5	3	2.5	8
		Bus	7.6	20	3.5	10	11.1	30
		Ind	0.5	1	0	0	0.5	1
	Thru Lanes	4	11.6	30	4.1	8	15.7	38
		5	0	0	0.2	1	0.2	1
		6	1.0	2	8.2	18	9.2	20
		Total:	12.6	32	12.5	27	25.1	59
TWLTL	Land use	Res	5.8	15	7.1	14	12.9	29
		Off	0	0	2.5	6	2.5	6
		Bus	5.0	16	6.5	15	11.5	31
		Ind	0	0	0.6	2	0.6	2
	Thru Lanes	4	10.8	31	2.0	5	12.8	36
		5	0	0	14.2	31	14.2	31
		6	0	0	0.5	1	0.5	1
		Total:	10.8	31	16.7	37	27.5	68
Undivided	Land use	Res	7.7	17	12.2	23	19.9	40
		Off	0.3	1	1.0	8	1.3	9
		Bus	2.7	9	2.1	4	4.8	13
	Thru Lanes	2	0	0	2.5	4	2.5	4
		4	10.7	27	12.8	31	23.5	58
	Parking	No	6.6	18	13.7	26	20.3	44
		Yes	4.1	9	1.6	9	5.7	18
		Total:	10.7	27	15.3	35	26.0	62

distributions. In this context, there are two different sources of variability underlying the count distribution. One source of variability stems from the differences in the m's among the similar sites. The other source stems from the randomness in accident frequency at any given site, which is traditionally described as Poisson.

Despite being similar, each segment in the group has its own regional characteristics and driver population, which gives it its own unique mean accident frequency, m_i. Thus, the distribution of m's within the group of similar sites can be described by a probability density function with mean $E(m)$ and variance $V(m)$. Hauer et al. (*17*) have shown this distribution to be adequately described by the gamma density function.

Abbess et al. (*20*) have shown that if accident occurrence at a particular segment is Poisson distributed, the distribution of accidents around the $E(m)$ of a group of segments can be described by the negative binomial distribution. The variance of this distribution is as follows:

$$V(x) = E(m) + \frac{E(m)^2}{k} \tag{5}$$

where x is the observed accident count of a given segment with an expected accident count of $E(m)$. Recognizing that the variance of the Poisson distribution is $E(m)$, it is apparent that the variance of the negative binomial distribution exceeds that of the Poisson by the amount $E(m)^2/k$. Hauer et al. (*17*) have shown that this latter quantity is equivalent to the variance of the mean accident frequency for the group of similar segments, $V(m)$. Hauer also has shown that the parameter k can be estimated by fitting Equation 5 to $V(x)$ and $E(m)$ estimates for the group of similar segments. The $V(x)$ is estimated as the squared difference between the accident count and the corresponding $E(m)$ for each segment in the group.

The analysis tool used to estimate the model coefficients was the nonlinear regression procedure (NLIN) in the SAS software (*19*). This procedure is general enough to be mod-

ified to accommodate error structures that are not normally distributed. It also can be modified to yield maximum-likelihood model coefficients. With these modifications, the NLIN procedure can be used as a generalized linear modeling tool. In fact, an example application of NLIN to generalized linear modeling is described in the SAS documentation (*19*, Chap. 29). It should be noted that the SAS code described in this documentation was modified (due to some errors in printing) and enhanced to include the negative binomial and gamma distributions.

Link Function

The generalized linear modeling approach relates a linear predictive equation to the expected value of an observation via a link function. This link function equates the linear predictive relationship to a nonlinear, and perhaps bounded, dependent variable. There is one link function that is theoretically related to the error structure of the data, based on its underlying distribution. This link function is sometimes referred to as the "natural" (or canonical) link. As noted by McCullagh and Nelder (*16*); however, the use of the natural link function is not a requirement. The natural link functions for the Poisson and negative binomial distributions are as follows:

$$Poisson: \quad \eta = \ln[E(m)] \tag{6}$$

$$Neg.\,Bin.: \quad \eta = \ln\left[\frac{E(m)}{k + E(m)}\right] \tag{7}$$

where the linear predictive equation is

$$\eta = b_0 + b_1x_1 + b_2x_2 + \ldots + b_nx_n \tag{8}$$

To obtain a model form that directly predicts the desired expected value, it was necessary to take the inverse of the link function (i.e., $E(m) = f^{-1}(\eta)$), equate it to the right side of Equation 8, and solve for $E(m)$. For the Poisson link function, the resulting model form follows:

$$E(m) = e^{(\ln(n) + b_0 + b_1x_1 + b_2x_2 + \ldots + b_nx_n)} \tag{9}$$

where n is termed the "offset" variable with an implied coefficient of 1.0. For accident data analysis, the offset variable is equivalent to the number of years underlying the observed count (in this study, $n = 3$ years for all observations).

A similar calculation to obtain the linear predictive model form for the negative binomial link function does not yield as simple a form as the aforementioned form. In fact, it is not algebraicly possible to obtain the model in its intended form using the natural link for the negative binomial structure. Because of this loss of generality, and recognizing that it is not a requirement to use the natural link, the Poisson link was used for all analyses in this study.

Quality of Fit

Several statistics are available for assessing model fit and the significance of model coefficients. One measure of model fit provided by NLIN is the generalized Pearson χ^2 statistic. This statistic is calculated as follows:

$$\chi^2 = \sum \frac{\left[x - \hat{E}(m)\right]^2}{\hat{V}(x)} \tag{10}$$

where $\hat{V}(x)$ is estimated from Equation 5 by substituting $\hat{E}(m)$ for $E(m)$. This statistic is available from NLIN as the "weighted sum of squares" for the residual. McCullagh and Nelder (*16*) indicate that this statistic follows the χ^2 distribution with $n - p - 1$ degrees of freedom, where n is the number of observations and p is the number of model parameters. This statistic is asymptotic to the χ^2 distribution for larger sample sizes and exact for normally distributed error structures. As noted by McCullagh and Nelder, this statistic is not well defined in terms of minimum sample size when applied to non-normal distributions; therefore, it probably should not be used as an absolute measure of model significance.

Another, more subjective, measure of model fit can be obtained from a graphical plot of the prediction ratio versus the estimate of the expected accident frequency (i.e., $\hat{E}(m)$). In this context, the prediction ratio is defined as the normalized residual (i.e., the difference between the predicted and observed accident frequencies divided by the standard deviation, $\sqrt{V}(x)$). This type of plot yields a visual assessment of the predictive capability of the model over the full range of $\hat{E}(m)$. A well-fitting model would have the prediction ratios symmetrically centered around zero over the range of $\hat{E}(m)$.

The significance of the parameter coefficients (with respect to the hypothesis that they equal zero) is also helpful in assessing the relevance of model factors. In this regard, NLIN provides the standard error and 95 percent confidence interval for each coefficient. Because the Pearson χ^2 statistic (i.e., Equation 10) has some limitations, the significance of the individual parameter coefficients may represent a more realistic measure of model fit.

A third measure of fit is the dispersion parameter σ_d. This parameter was noted by McCullagh and Nelder (*16*) to be a useful statistic for assessing the amount of variation in the observed data. This statistic can be calculated by dividing Equation 10 by the quantity $n - p$. It is also available from NLIN as the "weighted mean square" for the residual. A dispersion parameter near 1.0 indicates that the assumed error structure is approximately equivalent to that found in the data. For example, if a Poisson error structure is assumed (i.e., $V(x) = E(m)$) and the dispersion parameter is 1.68, this would indicate that the data have greater dispersion than is explained by the Poisson distribution. In this situation, the negative binomial distribution might be considered because it has a larger variance than the Poisson (see Equation 5).

Finally, the coefficient of determination R^2 can be used to assess the quality of model fit. This statistic is commonly used for normally distributed residuals; hence, it loses some of its meaning when applied to non-normal residuals. Nevertheless, Kvalseth (*21*) has investigated the use of R^2 to evaluate model forms calibrated with data having non-normal error structures and concluded that R^2 can still be a useful tool if computed with the following equations:

$$R^2 = 1 - \frac{SSE}{SST} \tag{11}$$

with

$$SSE = \sum_i^n \left(y_{p,i} - y_{o,i}\right)^2 \tag{12}$$

$$SST = \sum_i^n \left(y_{o,i} - y_m\right)^2 \tag{13}$$

where:

$y_{o,i}$ = observed dependent value for a given set of independent variables, i;
$y_{p,i}$ = predicted dependent value for the same set of independent variables, i; and
y_m = mean of all n observed dependent values.

When applied to accident prediction models, the quantity obtained from Equation 11 is not a true R^2 value because the residuals are not necessarily independent, normally distributed variates with constant variance. Nevertheless, it can be loosely compared to traditional R^2 values with similar interpretation.

Analysis Procedure

Coefficient estimation for the proposed model was a multistep process. First, the data were analyzed using a Poisson error structure. Second, NLIN was used to fit Equation 5 to the squared residuals from the first analysis. This second step yielded an estimate of the k parameter and a measure of its statistical significance.

The need for a third analysis step was based on an assessment of the dispersion parameter and the k parameter significance. If the dispersion parameter was more than 1.0 and the k parameter was statistically significant, a third analysis step was conducted using the negative binomial error structure with k from the second step as an initial estimate of the shape parameter.

Finally, during the fourth step, the k parameter was increased (or decreased) incrementally and the analysis was repeated in an iterative manner until the dispersion parameter σ_d converged to 1.0. This procedure is consistent with that described by Hauer et al. (*17*).

Database Review and Analysis

Preliminary Review of Accident Rates

As a practical first step in the analysis of the safety data, the accident rates were computed for each segment. Although the preceding discussion asserts that the relationship between accident frequency and segment length or traffic demand is not linear, it is close enough so that computed rates can still provide some insight into accident cause and effect.

Accident rates for the three left-turn treatment types are shown in Table 4-13. These rates have units of annual midsignal accidents per million vehicle-miles (a/mvm). As noted previously, only accidents occurring on the arterial segment were included in the database; accidents on cross street approaches were not included.

The accident rates reported in Table 4-13 should be interpreted with caution for two reasons. First, as previously discussed, the relationship between accident frequency and exposure often is not linear. Second, differences in rates within and among the various categories may be partly explained by differences in other, unspecified elements. For example, the undivided/office rate is more than four times that of the undivided/residential rate; however, much of this difference can be explained by the fact that parking was most frequently found on segments with office land uses. As a consequence, a portion of the difference between land uses associated with the undivided treatment may be explained by differences in parking activity.

Bearing in mind the aforementioned cautions, several trends can be observed from the rates shown in Table 4-13. For example, it appears that the raised-curb median treatment has the lowest rate (2.1 a/mvm), the TWLTL has a slightly higher rate (3.3 a/mvm), and the undivided treatment has the highest rate (3.8 a/mvm). This trend is consistent with the findings of most research on the relative safety of the three treatment types.

The accident rates for the three treatment types were compared with those reported by Harwood (*5*) and Bowman and Vecellio (*10,22*). In the case of Harwood's rates, some interpolation is necessary because his unsignalized intersection rates include all approaches, whereas those used in the safety database include only the two approaches on the arterial segment. A compromise rate was computed from Harwood's rates to facilitate comparison with those shown in Table 4-13. The compromise rate was computed as the sum of the midblock and one-half of the unsignalized intersection rates (which he did report separately). These rates are compared in Table 4-14.

In general, the rates from the safety database are in the range of those found by Harwood (*5*) and Bowman and Vecellio (*10,22*). Specifically, the raised-curb median rates in the safety database are lower than those reported by Harwood and higher than those reported by Bowman and Vecel-

TABLE 4-13 Accident frequency and rates categorized by location, land use, and lanes

	Location:		Omaha			Phoenix			Total		
Treatment Type	Feature		Acc	Inj[2]	Rate[1]	Acc	Inj[2]	Rate[1]	Acc	Inj[2]	Rate[1]
Raised-Curb	Land use	Res	121	53	**1.7**	431	197	**1.3**	552	250	**1.4**
		Off	282	105	**2.3**	45	10	**2.6**	327	115	**2.4**
		Bus	629	265	**2.5**	279	121	**2.4**	908	386	**2.5**
		Ind	18	1	**0.8**	0	0	-	18	1	**0.8**
	Thru Lanes	4	871	365	**2.2**	164	78	**2.1**	1035	443	**2.2**
		5	0	0	-	12	2	**2.0**	12	2	**2.0**
		6	179	59	**2.8**	579	248	**1.7**	758	307	**1.8**
		Total:	1050	424	**2.3**	755	328	**1.9**	1805	752	**2.1**
TWLTL	Land use	Res	321	136	**3.2**	643	369	**2.2**	964	505	**2.7**
		Off	0	0	-	282	122	**3.2**	282	122	**3.2**
		Bus	530	214	**4.3**	969	466	**3.8**	1499	680	**4.1**
		Ind	0	0	-	17	3	**1.1**	17	3	**1.1**
	Thru Lanes	4	851	350	**3.8**	200	124	**3.8**	1051	474	**3.8**
		5	0	0	-	1684	824	**2.9**	1684	824	**2.9**
		6	0	0	-	27	12	**1.9**	27	12	**1.9**
		Total:	851	350	**3.8**	1911	960	**3.0**	2762	1310	**3.3**
Undivided	Land use	Res	654	303	**3.0**	439	212	**1.6**	1093	515	**2.2**
		Off	12	2	**6.7**	46	13	**10.3**	58	15	**9.9**
		Bus	421	176	**5.5**	252	158	**4.9**	673	334	**5.3**
	Thru Lanes	2	0	0	-	50	15	**1.5**	50	15	**1.5**
		4	1087	481	**4.0**	687	368	**4.3**	1774	849	**4.1**
	Parking	No	652	302	**3.5**	513	239	**1.7**	1165	541	**2.4**
		Yes	435	179	**4.9**	224	144	**10.5**	659	323	**7.7**
		Total:	1087	481	**4.0**	737	383	**3.7**	1824	864	**3.8**

Notes:
1 - Segment accident rate in annual mid-signal accidents per million vehicle-miles (excludes all accidents on cross street intersection approaches).
2 - Number of injuries and fatalities for all accidents.

lio. The TWLTL and undivided rates for the safety database tend to be a little higher than those found by either of the other researchers but not by a significant amount, given the variability in the associated data. Certainly, there is no clear trend indicating that one treatment type is safer than the others among all three sources. This disparity likely stems from some combination of (1) the nonlinear relationship between accident frequency and exposure measures and (2) differences (other than traffic demand and length) among the segments included in the database.

Analysis of Database Element Effects

A two-stage statistical analysis approach was followed in developing the safety model. The first stage involved the use of analysis of variance (ANOVA) techniques to determine which database elements had a significant effect on accident frequency. The second analysis stage involved the calibration of the safety model using the GLIM approach described previously.

The analysis tool used for the ANOVA was the general linear modeling (GLM) procedure provided in the SAS software (*19*). This procedure is well suited to the analysis of the effects of quantitative (e.g., driveway density) and categorical (e.g., land use) factors on annual accident frequency. It is also well suited to datasets that are incomplete (i.e., not all elements of the effects of all categories have observations) and unbalanced (i.e., sample size is different among categories).

During the first analysis stage, the accident data were transformed using the Box-Cox (*23*) transformation algorithm before being submitted to GLM. This algorithm was

TABLE 4-14 Accident rate comparison (segment accident rate in annual midsignal accidents per million vehicle miles; excludes all accidents on cross street intersection approaches)

Treatment Type	Land Use	Safety Database	Harwood (5)[1]			Bowman (10, 22)[2]	
			Midblock	Unsig. Int.	Combined	CBD	Suburban
Raised-Curb	Residential	1.4	0.9	1.6	1.7	2.1	1.9
	Business	2.5	2.4	3.6	4.2		
TWLTL	Residential	2.7	1.3	0.8	1.7	1.6	3.1
	Business	4.1	2.6	2.0	3.6		
Undivided	Residential	2.2	0.9	2.0	1.9	2.0	2.1
	Business	5.3	2.8	3.7	4.6		

Notes:

1 - Rates shown for Harwood represent four through lanes, under 30 driveways per mile for raised-curb median, 30-60 driveways per mile for TWLTL and Undivided, 5-10% trucks, and under 5 unsignalized intersections per mile. The combined rate equals the midblock rate plus 1/2 of the unsignalized intersection rate.

2 - Rates shown for Bowman were obtained from Table 16 of Reference 10 or Table 6 of Reference 22.

used to find the transform function that most effectively stabilized the variance of the residual error. Stabilization permitted the use of the GLM ANOVA technique, which is based on least squares principles, by forcing the transformed residuals to be normally distributed and of constant variance.

As a result of this first-stage analysis, several database elements were found to have a significant effect on midsignal accident frequency: left-turn treatment type, daily traffic demand, segment length, land use, parallel parking, driveway density, public street approach density, and the percentage of PDO accidents. The analysis of land use indicated that there were two groups with similar accident trends: business/office and residential/industrial. Hence, each pair was represented by one variable in the safety model.

The existence of parallel parking was found to have a very significant effect on accident frequency; more accidents were associated with street segments that have parallel parking. Segments were identified as having parking if a large portion (e.g., 75 percent or more) of their street frontage was allocated to parallel parking; those with less parking were identified as having no parking. This dichotomous discriptor of parking (i.e., parking: yes or no) was found to be acceptable for the segments included in the study because only 3 of the 189 segments in the safety database fell into the gray area between 0 and 75 percent parking. Because each of these three segments had only about 10 percent parking, it was reasoned that they could be coded as having no parking with no loss in model accuracy.

None of the segments with the TWLTL or raised-curb median treatment was found to have parallel parking. This omission in the database was not intentional; every attempt was made to include a range of factors for each treatment type. However, TWLTL and raised-curb segments with parking were fairly rare in the two cities included in the database. Because most of the reports found in the literature review did not address the effect of parking, the omission of TWLTL and raised-curb segments with parking was not believed to be a significant deficiency in the database at the time of its construction.

Both driveway and unsignalized public street approach densities were found to have a significant effect on accident frequency on segments whose land use was categorized as business or office. In general, there were more accidents on streets with higher driveway or street densities. On the other hand, driveway and street density was not significantly correlated with the accident frequency of segments with residential or industrial land uses. The separate effects of driveway density and street density were not found to be significantly different from one another and, as a result, they were combined in the safety model. The relationship between combined driveway and public street approach density is shown in Figure 4-2.

The data in Figure 4-2 were derived from the safety database by aggregation of segments with similar driveway-plus-street densities. This aggregation was performed to minimize the variability in accident rates in the individual segments, thereby permitting the general trend to be seen more clearly. For this aggregation, the individual segments were ranked in order of their combined driveway-plus-street density and segregated into sequential groups of seven or eight segments. The average driveway-plus-street densities and associated accident rates for these groups are shown in Figure 4-2.

Data for the undivided cross section treatment are not presented in Figure 4-2 because the effect of parking was so significant that the data were quite varied among groups. This variability made interpretation of driveway density trends difficult to visualize. It also should be noted that the data in Figure 4-2 suggest that the effect of driveway and street density is larger for the TWLTL treatment than for the raised-curb treatment (i.e., TWLTL has a steeper slope); however, the differences in slope were not found to be statistically significant.

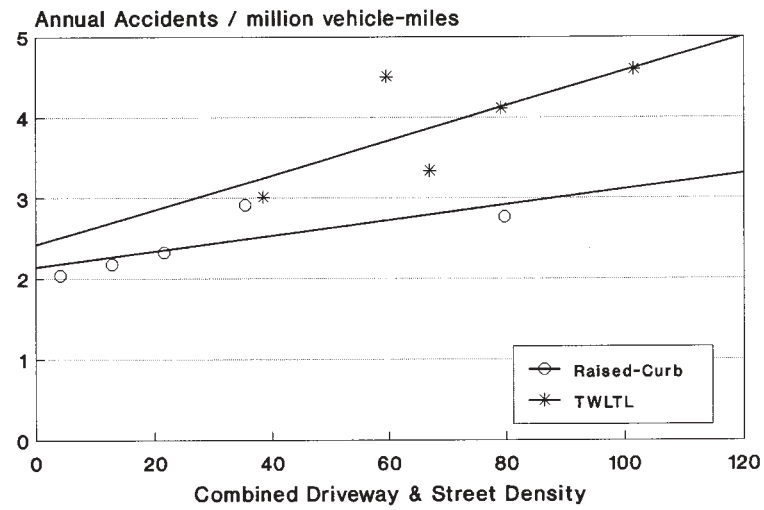

Figure 4-2. Effect of combined driveway and public street approach density on accident frequency in business and office areas.

The PDO accident percentage represents the ratio of PDO accidents to all reported accidents. As such, it is a direct measure of the accident cost reporting threshold and the degree of driver compliance with accident reporting requirements in a given area. Areas with higher reporting thresholds or a larger number of unreported accidents are associated with lower PDO percentages.

The PDO percentage was found to be correlated with accident frequency. Specifically, streets in cities with a higher PDO percentage were found to have more accidents than similar streets in cities with a lower PDO percentage. This finding does not mean that more accidents occur in areas with higher PDO percentages; rather, it means that more accidents are being *reported* in these areas. The inclusion of PDO percentage in an accident prediction model facilitates the comparison of the relative safety of arterial streets in different cities and states through the use of a common normalizing factor.

Model Calibration

The findings from the first stage analysis were used to develop the safety model. The log link function was used to relate the vector of linear terms to the expected accident frequency. The specific terms included in this vector were categorized as exposure measures and explanatory factors. The exposure measures included average daily traffic and street segment length. The explanatory factors included driveway density, street density, midblock left-turn treatment, and land use category. The resulting model has the following form:

$$\ln(A) = B_1 \ln(ADT) + B_2 \ln(Len) + (linear\ terms) \quad (14)$$

The formulation above indicates that the natural log of the exposure measures was used rather than their original units. This transformation was performed because several researchers (e.g., *10,17,18*) have shown that the relationship between accident frequency and exposure is nonlinear.

Equation 14 can be rewritten into a more useful form as follows:

$$A = ADT^{B_1} Len^{B_2} e^{(linear\ terms)} \quad (15)$$

with

$$linear\ terms = C_0 + C_1x_1 + C_2x_2 + \ldots + C_nx_n \quad (16)$$

where:

A = annual accident frequency at midsignal (i.e., midblock and unsignalized) locations;
ADT = average daily traffic demand;
Len = street segment length;
x_i = selected traffic and geometric characteristics; and
B_i, C_i = regression coefficients.

The model building process started with individual models for each treatment type. However, after these initial calibrations, it was noted that several regression coefficients were similar in magnitude among treatment types. This similarity allowed the models to be combined, although indicator variables were still used to maintain a distinction between nonsimilar coefficients. This combination technique was considered desirable because it allowed the maximum number of observations to be used to calibrate the similar coefficients (this characteristic generally yields a lower standard error in the coefficient estimate). The resulting form of the combined model follows:

$$A = ADT^{(B_0 + B_1 I_U I_{rli})} Len^{B_2} e^{(linear\ terms)} \quad (17)$$

with

$$linear\ terms = C_o + C_{type} + C_1(DD + SD)I_{b/o} + C_2 PDO + C_3 I_{Park} I_U \quad (18)$$

$$C_{type} = C_a I_R I_{b/o} + C_b I_T I_{b/o} + C_c I_R I_{r/i} + C_d I_T I_{r/i} + C_e I_U I_{r/i} \quad (19)$$

where:

A = annual midsignal accident frequency for the subject segment, in accident/segment/year;
ADT = average daily traffic demand, in vpd;
Len = street segment length, in feet;
DD = driveway density (two-way total), in driveways/mile;
SD = unsignalized public street approach density (two-way total), in approaches/mile;
PDO = property damage accidents as a percent of total accidents;
I_R = indicator variable for the raised-curb median treatment (1.0 if raised-curb; 0.0 otherwise);
I_T = indicator variable for the TWLTL treatment (1.0 if TWLTL; 0.0 otherwise);
I_U = indicator variable for the undivided treatment (1.0 if undivided; 0.0 otherwise);
$I_{r/i}$ = indicator variable for residential or industrial land uses (1.0 if residential or industrial; 0.0 otherwise);
$I_{b/o}$ = indicator variable for business or office land uses (1.0 if business or office; 0.0 otherwise);
I_{Park} = indicator variable for parallel parking along the roadside (1.0 if allowed; 0.0 otherwise);
C_{type} = intercept C_0 modifier to account for the effects of left-turn treatment and land use; and
B_i,C_i = regression coefficients.

The statistics relating to the calibrated accident prediction model are shown in Table 4-15. The calibrated coefficient values would be used with Equations 17, 18, and 19 to predict the annual accident frequency for a given street segment. A k parameter of 4.5 was found to yield the desired dispersion parameter of 1.0. The Pearson χ^2 statistic for the model is 179.2, and the degrees of freedom are 176 ($= n - p - 1 = 189 - 12 - 1$). Because this statistic is less than $\chi^2_{0.05,\ 176} = 208.0$, the hypothesis that the model fits the data cannot be rejected. The R^2 for the model is 0.69. Because this value is quite large for accident data, it was reasoned that the model yields a very good fit to the data.

With a few exceptions, the coefficients in this model are significant at a 95 percent level of confidence. Three variables are not statistically significant; these are identified by underlines in Table 4-15. Despite their statistical insignificance, these three variables were kept in the model because they represented best estimates of the coefficient values and because they related to two of the three left-turn treatments.

The variance of the predicted accident frequency can be estimated using Equation 5, where the predicted accident frequency, A ($= \hat{E}(m)$), is substituted for $E(m)$ and k is set equal to 1.5 ($= 4.5/3$). The k parameter is divided by 3.0 (corresponding to the 3 years of accident data used) to obtain the variance of the predicted *annual* accident frequency.

The fit of the model to the data also can be assessed using the prediction ratios plotted against the predicted accident frequency. The prediction ratio, PR_i ,for street segment i represents its residual error standardized (i.e., divided by the square root of its predicted variance (i.e., Equation 5). This ratio can be computed as follows:

$$PR_i = \frac{y_{p,i} - y_{o,i}}{\sqrt{V(x)}} \quad (20)$$

The prediction ratios for the three left-turn treatment types are presented in Figures 4-3, 4-4, and 4-5.

As Figures 4-3, 4-4, and 4-5 indicate, the fit of the calibrated model to the data is quite good. The standardized residuals are centered around zero, indicating no bias in the predicted quantity. In fact, the average error was computed to be less than ±0.02 accidents per year for all of the left-turn treatment types. The figures also indicate that the errors are distributed normally about zero for the entire range of predicted values. This trend was the desired result; it is a consequence of the specification of the negative binomial error structure in the SAS NLIN procedure. The standard deviation of the standardized residuals is between 0.93 and 1.02 for the three left-turn treatment types. Again, this trend was the desired result; it is a consequence of adjusting the k parameter until the dispersion parameter equals 1.0.

The regression model can be rewritten to yield the following treatment-specific forms:

$$A_R = ADT^{0.910} Len^{0.852} \times e^{(-15.162 - 0.296 I_{b/o} - 0.596 I_{r/i} + 0.00478(DD+SD) I_{b/o} + 0.0255 PDO)} \quad (21)$$

$$A_T = ADT^{0.910} Len^{0.852} \times e^{(-15.162 + 0.018 I_{b/o} + 0.093 I_{r/i} + 0.00478(DD+SD) I_{b/o} + 0.0255 PDO)} \quad (22)$$

$$A_U = ADT^{(0.910 + 1.021 I_{r/i})} Len^{0.852} \times e^{(-15.162 - 10.504 I_{r/i} + 0.570 I_{Park} + 0.00478(DD+SD) I_{b/o} + 0.0255 PDO)} \quad (23)$$

Sensitivity Analysis

The predictive capability of the calibrated safety model is demonstrated in Figures 4-6 and 4-7 for a range of daily traffic demands. Figure 4-6 demonstrates the relationship between accident frequency and demand for the three treatment types in areas designated as business or office. Figure 4-7 demonstrates the relationship between the treatments in areas designated as residential or industrial. Typical values were selected for each model variable based on the information in Tables 4-11 and 4-12. The selected values are shown in the appropriate figures. The length of each plotted line represents the range of data available to calibrate that line. It should be noted that the model only predicts accident fre-

TABLE 4-15 Calibrated safety model statistics

Model Statistics	Value
R^2:	0.69
Dispersion Parameter:	1.0
Pearson χ^2:	179.2 ($\chi^2_{0.05,\,176} = 208$)
k Parameter:	4.5
Observations:	189 street segments

Range of Model Variables

Variable	Variable Name	Units	Minimum	Maximum
ADT	Average daily traffic demand	vpd	3,000	56,700
Len	Length of street segment	feet	360	7,978
DD	Driveway density	drives/mile	0	116
SD	Unsignalized public street approach density	approaches/mile	0	31
PDO	Percentage property damage accidents	%	64	72

Calibrated Coefficient Values

Variable	Definition	Value	Std. Dev.	t-statistic
B_0	Effect of ADT	0.910	0.101	9.0
B_1	ADT modifier for undivided in res./ind.	1.021	0.358	2.9
B_2	Effect of street segment length	0.852	0.086	9.9
C_0	Intercept for undivided in bus./office	-15.162	1.356	-11.2
C_a	C_0 modifier for raised-curb in bus./office	-0.296	0.215	<u>-1.4</u>
C_b	C_0 modifier for TWLTL in bus./office	0.018	0.189	<u>0.1</u>
C_c	C_0 modifier for raised-curb in res./ind.	-0.596	0.279	-2.1
C_d	C_0 modifier for TWLTL in res./ind.	-0.093	0.262	<u>-0.4</u>
C_e	C_0 modifier for undivided in res./ind.	-10.504	3.594	-2.9
C_1	Effect of driveway and street density	0.00478	0.00229	2.1
C_2	Effect of under-reporting	0.0255	0.0104	2.5
C_3	Effect of parallel parking on undivided	0.570	0.194	2.9

Note:
The variability of the <u>underlined</u> coefficients is sufficiently high that it is difficult to be certain of their true value. In fact, the range of possible values includes zero (i.e., no effect).

quency for TWLTL and raised-curb segments that do not have parallel parking.

Both figures indicate that annual accidents increase with daily traffic demand. In general, segments with the undivided treatment, particularly those with parallel parking, tend to have the most accidents. Segments with a TWLTL treatment have fewer accidents, and segments with the raised-curb median treatment have the fewest accidents. The trends illustrated in Figure 4-6 suggest that the safety difference between the undivided-without-parking and TWLTL (also without parking) treatments may be negligible in business or office areas over the range of traffic demands. Because none of the TWLTL and raised-curb segments had parking, it was not possible to estimate the effect of parking on accident frequency for these left-turn treatment types. The overlap among curves in Figure 4-7 suggests that the safety difference between the undivided-without-parking and the raised-curb or TWLTL (both without parking) treatments may be negligible in residential or industrial areas when the average daily traffic demand is less than 20,000 to 25,000 vpd.

Conclusions

The following conclusions have been formulated based on the findings from safety model development. First, the analysis of the accident data indicates that average daily traffic demand, driveway density, unsignalized public street approach density, left-turn treatment type, and adjacent land use are significantly correlated with accident frequency. In general, accidents are more frequent on street segments with

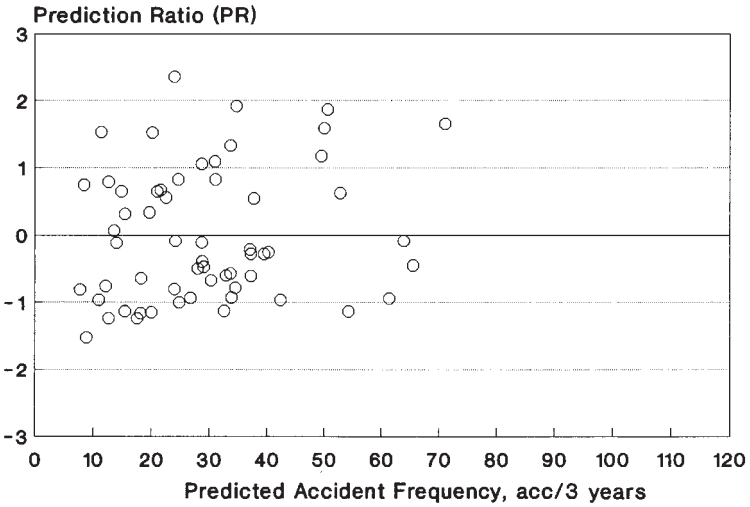

Figure 4-3. Prediction ratio versus accident frequency for the raised-curb median treatment.

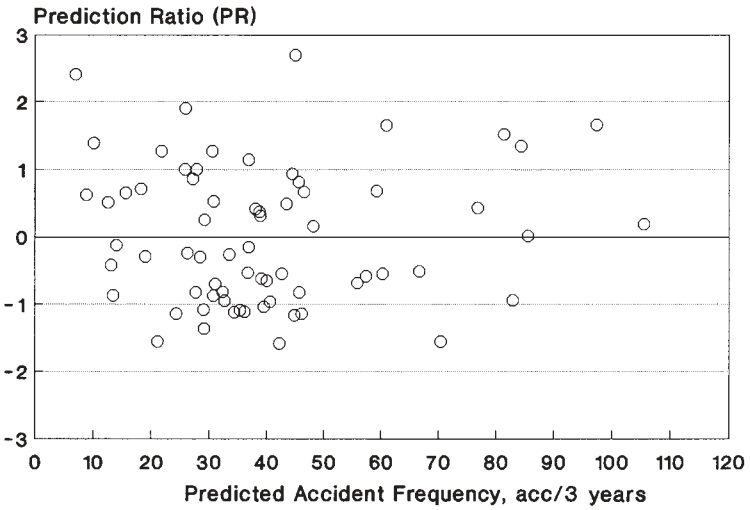

Figure 4-4. Prediction ratio versus accident frequency for the TWLTL treatment.

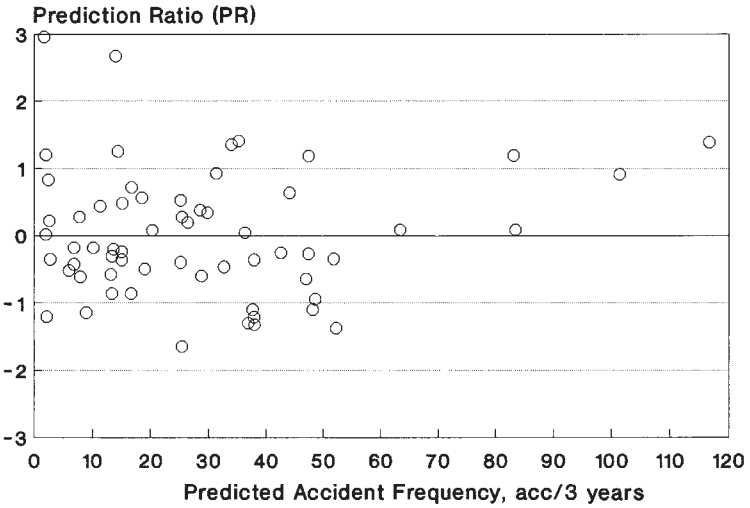

Figure 4-5. Prediction ratio versus accident frequency for the undivided cross section.

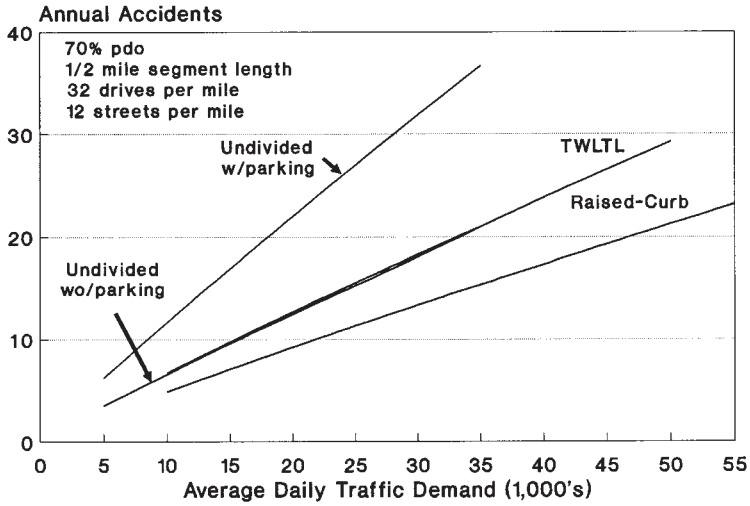

Figure 4-6. Effect of traffic demand on accident frequency in business and office areas.

higher traffic demands, driveway densities, or public street densities. Accidents are also more frequent when the land use is business or office as opposed to residential or industrial.

Second, the analysis indicates that the undivided cross section has a significantly higher accident frequency than the TWLTL and raised-curb median treatments when parallel parking is allowed on the undivided street. When there is no parking allowed on either street, the difference between the undivided and TWLTL treatments is generally small and is negligible for average daily traffic demands of less than 25,000 vpd. In general, the raised-curb median treatment appears to be associated with fewer accidents than the undivided cross section and TWLTL, especially for average daily traffic demands exceeding 20,000 vpd.

Third, regression methods based on maximum-likelihood techniques and a negative binomial distribution of the residuals are necessary to accurately calibrate accident prediction models. The use of these methods has revealed that the relationship between accident frequency and exposure (e.g., average daily traffic demand or segment length) is nonlinear. This finding indicates that the use of accident rates (and models that predict accident rates) may not yield accurate estimates of accident frequency, especially if the range in the database is exceeded.

Fourth, a new variable was introduced for accident model calibration. This variable represents the ratio of PDO accidents to all reported accidents for an urban area. As such, it is a direct measure of the accident cost reporting threshold

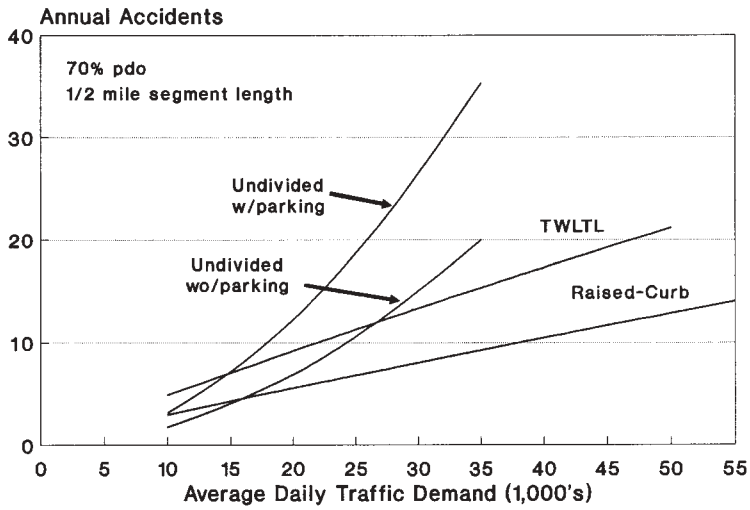

Figure 4-7. Effect of traffic demand on accident frequency in residential and industrial areas.

and the degree of driver compliance with accident reporting requirements in a given area. Areas with higher reporting thresholds or a larger number of unreported accidents are associated with lower PDO percentages. The inclusion of PDO percentages in an accident prediction model facilitates the comparison of the relative safety of arterial streets in different cities and states through the use of a common normalizing factor.

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CHAPTER 5

ACCESS IMPACTS OF MIDBLOCK LEFT-TURN TREATMENTS

This chapter describes the development of a model for predicting the access impacts of alternative midblock left-turn treatments. In this context, a treatment's impact is defined by its enhancement or degradation of accessibility to properties adjacent to the arterial. The degree of impact is based on the perceptions of people most directly affected by the treatment—the owners and managers of businesses on the adjacent property. The following sections describe a review of the literature on access impacts, the development of an access impact database from a survey of business owners, and the formulation of an access impact model.

LITERATURE REVIEW

Harwood (*1,2*) conducted two major studies of the effects of alternative midblock left-turn treatments, focusing primarily on the operational and safety effects. He also recognized the need to consider the effects of various left-turn treatments on access to adjacent properties.

In a 1991 study, Long and Helms (*3*) examined the impact of median changes on two arterials in Fort Lauderdale, Florida. The existing left-turn access was maintained on one arterial, whereas left-turn access was reduced by approximately 50 percent on the other arterial. Results of a survey of adjacent property owners and business managers indicated that the business community was not severely affected by the modification or restriction of left-turn access.

Koepke and Levinson (*4*) conducted case studies in the Atlanta, Georgia, and Denver, Colorado, areas as part of a recent study on access impacts. These studies indicated that restricting left-turn access along highly traveled urban arterials resulted in no long-term, overall negative impact to adjacent land uses.

In general, the literature is replete with before-and-after studies of various changes in left-turn access and its perceived impact on adjacent land uses. However, no methods for quantifying the impact of midblock left-turn treatments on adjacent property access were found.

DEVELOPMENT OF AN ACCESS IMPACT DATABASE

Database Composition

Quantifying the access impacts of alternative midblock left-turn treatments requires a more subjective approach than does quantifying the operational and safety effects of these treatments. The nature and extent of access impact depends on a range of tangible and intangible factors. The nature of the impact depends on two factors: (1) whether the treatment provides a storage area for the arterial left-turn movement and (2) whether the treatment increases or decreases access to the adjacent property. The extent of the impact depends on whether the land use of the adjacent property is auto-related or non-auto-related. In the latter case, the impact from a change in left-turn treatment often has the most significant effect on auto-related businesses (e.g., service stations). Intangible factors relate to the impact of any change in storage or access on adjacent land uses (e.g., business sales and property values) and the quality of arterial traffic flow (e.g., congestion and safety).

For the purpose of this study, a business was determined to be auto-related if at least one-third of its customers come from pass-by traffic. This definition is similar to that used by Stover and Koepke (*5*) in a recent textbook on this subject. Auto-related businesses include fast-food restaurants, service stations, convenience stores, liquor stores, and small retail stores of a similar type.

A land use in which access is provided by all possible driveway turn movements (right and left turns both in and out of the property) is considered to have full access. If the property is served by both right-turn maneuvers but only one left-turn maneuver, it is considered to have partial left-turn access. If it is served by only right-turn maneuvers, it is considered to have no direct left-turn access.

The business land use is typically found along urban and suburban arterials. This land use can be subdivided into the retail, service, and commercial business categories. Other land uses also can be found (e.g., office, residential, and industrial) but at a lower frequency than business.

The purpose of the access impact model is to provide a quantitative method of evaluating the effects of various left-turn treatments on adjacent land uses. Because the predominant land use is business, the model was developed for business-related access impacts. The access impact measures considered for this model include the following:

- Traffic operation
- Traffic safety
- Ease/circuity of access maneuver
- Effect on business operation (sales)

- Effect on property and land values
- Customer convenience.

It should be noted that the traffic operation and traffic safety impact measures used in the development of the access impact model are not the same as those described in other chapters of this report. The effects of these measures, as used in the access impact model, are based on the perceptions of property owners and managers whose businesses are served by arterials that have undergone a change in midblock left-turn treatment.

The field study consisted of two visits to each site for the purpose of collecting background and access-impact information. A study site was defined to be an urban or suburban arterial street that had recently undergone a conversion from one midblock left-turn treatment to another. During the first visit, data were collected from the local highway agency and from an on-site inspection of the study site. The data obtained from the highway agency included the study site's geometric design plans, traffic volumes, and accident history. The data obtained during the on-site inspection included the number, type, and location of land uses and whether the land use was auto- or non-auto-related.

During the second site visit, a survey was conducted to determine the effect of the recent left-turn treatment change on business operations (e.g., sales and property values), access, and arterial traffic conditions. The survey questionnaire was distributed to the representative (i.e., owner or manager) of each business adjacent to the arterial. The questionnaire included questions relating to each of the aforementioned access impact measures and, specifically, to any changes in these measures resulting from the recent change in left-turn treatment.

Study Site Descriptions

Contacts made during the development of case studies for a previous project by Koepke and Levinson (*4*) were reestablished and additional candidate study sites were investigated to ascertain the adaptability of available data to the access impact model's data requirements. Based on this evaluation, 17 candidate study sites were identified in 10 states:

1. Oakland Park (S.R. 816), Commercial, and Sunrise Boulevards—Fort Lauderdale, Florida
2. Merritt Island Parkway (S.R. 520)—Merritt Island, Florida
3. Roosevelt Road (S.R. 38), Ogden Avenue (S.R. 34), and Harlem Avenue (S.R. 43)—DuPage County (Chicago), Illinois
4. Port Washington Road—Mequon, Wisconsin
5. Blue Mounds Road (U.S. 18)—Milwaukee, Wisconsin
6. Arapahoe and Parker Roads—Denver, Colorado
7. Jimmy Carter Boulevard and Memorial Drive—Gwinnett County (Atlanta), Georgia
8. 48th and 56th Streets—Lincoln, Nebraska
9. Robert Street (Trunk Highway 52)—West St. Paul, Minnesota
10. Rice Street (Trunk Highway 49)—St. Paul, Minnesota.

The primary site selection criterion was that each site had to have undergone some type of change in its midblock left-turn treatment within the past few years. The change had to have taken place recently because it was preferred that the surveyed business representatives have experience with the before-and-after left-turn treatments. In this manner, their experience with both treatments would facilitate the development of quantitative measures of access impact resulting from a treatment change. A secondary criterion was the availability of before-and-after data for each site from the local highway agency.

The evaluation of the candidate study sites led to the selection of four sites, which are described briefly in the following sections. A more detailed description is provided later in this chapter.

Oakland Park Boulevard—Fort Lauderdale, Florida

Oakland Park Boulevard (S.R. 816) is a six-lane divided arterial that has an average daily traffic (ADT) demand of 54,000 vehicles per day (vpd). Land use along the arterial is primarily strip commercial developments ranging in frontage from fewer than 50 ft to up to a full block. During 1985 and 1986, a 2.25-mi section of the arterial was reconstructed, which significantly reduced the frequency of median openings.

Merritt Island Parkway—Merritt Island, Florida

A 2-mi section of Merritt Island Parkway (S.R. 520) was widened in 1993 from a four-lane arterial with a two-way left-turn lane (TWLTL) and 8-ft shoulders to a six-lane arterial with a raised-curb median. The raised-curb median varies from 12 ft with single-lane left-turn bays to 30 ft with dual-lane left-turn bays. Land use along the arterial varies among retail, commercial, and office development. Street access varies from full to partial left-turn access. The 1990 ADT was 46,300 vpd.

Roosevelt Road—Wheaton/Glen Ellyn, Illinois

Roosevelt Road (S.R. 38) is a major arterial in the west suburban area of Chicago. It serves more than 50,000 vpd and ranks near the top of a list of locations in Illinois with a high frequency of accidents. A 7.5-mi section of this arterial was widened during 1987 to 1991. It originally had a four-lane undivided cross section but was widened to include four lanes and a TWLTL. Land use along the corridor ranges from residential to retail.

Port Washington Road—Mequon, Wisconsin

Port Washington Road is a county highway in the Milwaukee area that was widened in 1992 from a four-lane undivided arterial to a six-lane arterial with a 28-ft raised-curb median. The raised-curb median accommodates dual-lane left-turn bays at its intersection with S.R. 167. The arterial also accommodates a mixture of full and partial left-turn access at unsignalized intersections along its length. Land use along the arterial is a combination of strip and cluster commercial.

Data Collection Approach

As mentioned previously, the field study consisted of two visits to each site for the purpose of collecting background and access-impact information. The purposes of the first visit were as follows:

- Meet with local highway agency personnel.
- Obtain any before-and-after transportation data (i.e., street design plans, traffic volumes, and accident data).
- Survey the study site to determine the number, type, and location of land uses.
- Determine, for business land uses, whether the businesses are auto- or non-auto-related.

During the second site visit, a questionnaire was distributed to owners and managers of businesses adjacent to the arterial. The owners and managers were informed about the study and its national significance and how their participation (anonymous, if desired) would ensure that their concerns would be incorporated into the results of the study. The owners and managers were asked to take about 10 to 15 min to fill out the survey questionnaire, which would be picked up the following day.

The questionnaire included questions on the impact of recent changes in the design or operation of the arterial fronting the business property. These questions were directed toward changes in traffic safety, traffic congestion, left-turn operation, driving convenience, property value, business operation, customer attitudes, sales volume, and profit margins. The owners and managers were asked to rank, on a scale from 1 to 10 (10 being highest), the factors (e.g., price, quality, and accessibility) that influence a customer's decision to shop at the respondent's business. To ensure that the respondent had the proper perspective for filling out the questionnaire, a question was included regarding whether the business was in operation before the arterial's left-turn treatment was modified. The survey questionnaire is presented in Figure 5-1.

A total of 305 questionnaires were distributed at the four field study sites. Of these, 165 were returned for a response rate of 54 percent. The response rate for the individual study sites ranged from 37 to 69 percent.

Database Summary

Oakland Park Boulevard—Fort Lauderdale, Florida (RM330/6 TO RM660/6)

Site Description. Oakland Park Boulevard (S.R. 816), a six-lane divided arterial that serves 54,000 vpd, was originally designed and built in the 1950s and improved during 1985 and 1986. The improved section is east of I-95 and bordered primarily by adjacent strip commercial developments. The arterial connects I-95 to the Fort Lauderdale beach area. The posted speed limit is 45 mph.

The section of Oakland Park Boulevard that was improved is 2.25 mi in length. It originally contained four signalized intersections and 33 full access median openings. After reconstruction, the four signalized intersections remained, but 17 of the unsignalized median openings were eliminated. The remaining 16 median openings are about 660 ft apart and were reconfigured to allow only two turning maneuvers, the U-turn and the left-turn movement from one direction of travel. Three new openings that permitted only the U-turn maneuver were added. Only one arterial left-turn movement is allowed at each median opening; the direction of this movement alternates among the median openings. This approach allows partial left-turn access to alternating sides of the boulevard at slightly less than 0.25-mi intervals. Figure 5-2 illustrates the before and after median designs at this site.

The improvements to this arterial focused on closing every other median opening, although island channelization also was used to prevent left turns onto the arterial. The median closures increased the spacing between median openings from about 330 ft to about 660 ft. Henceforth, the two variations of raised-curb median treatment used at the Oakland Park study site will be referred to as RM330/6 and RM660/6, respectively. Notations such as these are used in this chapter to describe the arterial's left-turn treatment type, the spacing of its median openings, and its number of through lanes.

Study Section. The 1.3-mi study section begins at the signalized intersection with Andrews Avenue and ends at the signalized intersection with Northeast 18th Avenue. Two additional traffic signals exist along with 10 median openings, two of which are for U-turns only. The study section consists of 153 active businesses and seven vacant buildings. Thirty-one (20 percent) of the businesses have been classified as auto-related and 122 (80 percent) have been classified as non-auto-related.

Of the 153 businesses contacted, 117 indicated that they would fill out the questionnaire. Of these 117 businesses, 5 returned blank questionnaires, 43 returned completed questionnaires, and 69 did not respond. The number of completed questionnaires equates to a relatively low response rate of only 37 percent. Of the 43 respondents, 25 (58 percent) indicated that their businesses were at the same location before the arterial was improved, whereas 18 (42 percent) located

NCHRP Project 3-49
Capacity and Operational Effects of Midblock Left-Turn Lanes
Survey Questions

Name:_____
Address:_____

1. Were you at this location before _____ or after _____ the street was reconstructed?

2. If you were at this location before the street was reconstructed, has the number of customers changed? If so, by what percentage? _____%

3. If you located here after the street was reconstructed, did the changes in left-turn access affect your decision?
Yes _____ No _____

4. What percentage of your customers come to your business as the principal reason for their trip, as opposed to stopping on their way to another destination? _____%

5. Which is the busiest: Month of the year? _____ Day of the week? _____ Time of day? _____

6. Approximately how many customers do you have on an average day? Weekday_____ Saturday_____ Sunday_____

7. In general, has business in the area increased _____, decreased _____, or stayed the same _____ over the last several years?

8. Has the value of your property increased _____, decreased _____, or stayed the same _____? If it has changed, by what percentage? _____%

9. Traffic volumes are increasing. How do you feel about the following issues since the street was reconstructed?

	Better	Worse	The Same
a. Traffic Congestion	_____	_____	_____
b. Traffic Operation	_____	_____	_____
c. Traffic Safety	_____	_____	_____
d. Property Access	_____	_____	_____
e. Business Opportunities	_____	_____	_____
f. Customer Convenience	_____	_____	_____
g. Customer Satisfaction	_____	_____	_____
h. Delivery Convenience	_____	_____	_____

10. On a scale of 1 to 10, how would you rank the following factors as influencing a customer's decision to purchase from you? (10 is highest, 1 is lowest)
Price _____ Quality _____ Service _____ Hours of Operation _____ Accessibility _____
Other (please name) _____

Figure 5-1. Survey questionnaire.

their businesses on Oakland Park Boulevard after the improvement.

Table 5-1 summarizes the distribution of business categories and access types provided at this study site. As this table indicates, the number of completed surveys returned by auto-related businesses is low. Moreover, relative to the number of auto-related businesses identified on the study segment, this business category is underrepresented. In recognition of these deficiencies, it was determined that a valid assessment of access impact on auto-related businesses could not be obtained for this site. In fact, this sample size problem was found at all sites. As a result, the responses for both the auto- and non-auto-related categories were combined for each site to broaden the examination of access impact to all types of businesses.

Public Opinion Survey. Table 5-2 summarizes the results of the survey of business representatives in the Oakland Park Boulevard study segment. Responses to Question 3 shown in this table were obtained only from businesses that were established after the arterial was reconstructed. In contrast, responses to Questions 2, 7, 8, 9, and 10 were obtained only from businesses established before construction. Some survey respondents did not answer all questions, thereby accounting for differences in the number of responses and the number of questionnaires that were returned.

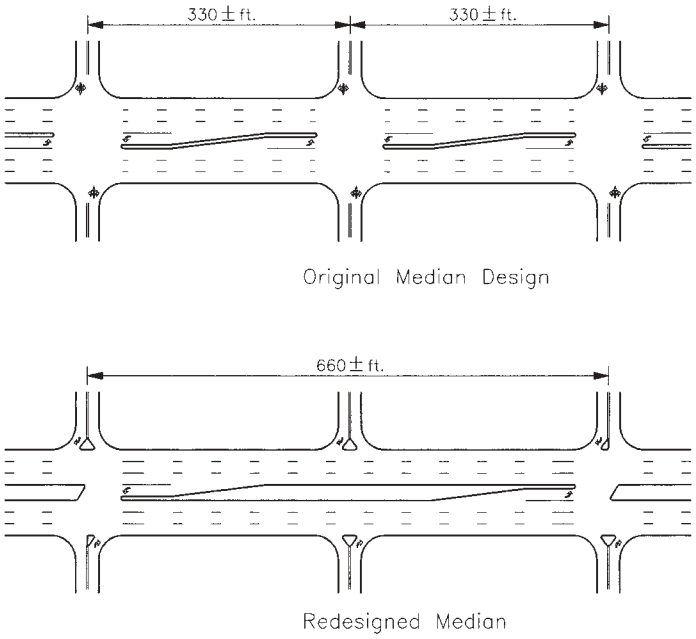

Figure 5-2. Median design on Oakland Park Boulevard.

Evaluation. The largest number of respondents (45 percent) believe that business activity in the area has increased during the past several years. This finding contrasts with the more popular (44 percent) belief that activity at the respondents' businesses did not change as a result of the arterial improvement project. This suggests that businesses whose access was affected by the project may have lost the opportunity to increase their activity in a manner consistent with other businesses in the area. A majority (56 percent) of the business representatives also indicated that property values were unchanged by the project despite the fact that median openings were reduced by about 50 percent and left-turn maneuvers at the openings that remained were limited to one direction. Responses to Question 10 indicate that access ranks the lowest among the factors that influence a customer's decision to patronize a specific business.

With respect to traffic conditions, the majority of respondents indicated that there was no change in traffic congestion, operation, or safety. This finding was somewhat surprising because the reduction in median openings was specifically intended to improve traffic conditions. A majority also believed that property access, business opportunity, and customer convenience had not changed. This finding also was surprising because closing 50 percent of the median openings and the restriction of left-turn maneuvers at the openings that remained should have had some negative effect on property access.

In contrast to their survey responses, business representatives verbally indicated that traffic conditions along the reconstructed arterial were a major problem, even though they acknowledged that traffic appeared to operate more safely. A recurring comment made by these representatives related to frequent illegal left turns being made from driveways and side streets. These turns are made contrary to the direction provided for by the intersection's median and island channelization.

Conclusions. A majority of respondents perceived the closure of about 50 percent of the median openings as not having changed traffic conditions or business activity. The perceived lack of improvement in traffic conditions is troubling because it is contrary to the intent of the improvement project. The majority opinion that business was unaffected is considered a positive factor, given that property access was significantly reduced by this project.

TABLE 5-1 Business category and left-turn access type—Oakland Park Boulevard

Auto-Related - 4 Businesses (9%)	
Full Access	1 (2 %)
Partial Left-Turn Access	3 (7 %)
Non-Auto-Related - 39 Businesses (91%)	
Full Access	0 (0 %)
Partial Left-Turn Access	39 (91 %)

TABLE 5-2 Survey summary—Oakland Park Boulevard

Question	Responses	Before	After	
1. Business established before or after construction.	43	58 %	42 %	
	Responses	Yes	No	
3. Access effect on location decision.	18	67 %	33 %	
	Responses	Increase	Decrease	No Change
7. Change in area's business activity after construction.	24	45 %	17 %	38 %
2. Change in your business activity due to construction.	18	28 %	28 %	44 %
8. Change in property value after construction.	16	25 %	19 %	56 %
9. Issues after reconstruction.	Responses	Better	Worse	No Change
a. Traffic Congestion	25	16 %	32 %	52 %
b. Traffic Operation	25	20 %	16 %	64 %
c. Traffic Safety	25	20 %	32 %	48 %
d. Property Access	25	16 %	28 %	56 %
e. Business Opportunity	25	16 %	24 %	60 %
f. Customer Convenience	25	12 %	28 %	60 %
g. Customer Satisfaction	25	16 %	16 %	68 %
h. Delivery Convenience	25	8 %	40 %	52 %
10. Ranking of factors influencing customer's decision.	Responses	Factor Rank		
		High (8-10)	Medium (4-7)	Low (1-3)
a. Price	24	62 %	38 %	0 %
b. Quality	24	87 %	13 %	0 %
c. Service	24	96 %	4 %	0 %
d. Hours Open	24	42 %	50 %	8 %
e. Accessibility	24	46 %	33 %	21 %

Merritt Island Parkway—Merritt Island, Florida (TWLTL/4 to RM660/6)

Site Description. About 2 mi of Merritt Island Parkway (S.R. 520) were widened during 1993 from four through lanes with a TWLTL to six lanes with a raised-curb median. Median openings that allow for left turns and U-turns are provided at 660-ft intervals. The median varies in width throughout the improvement to accommodate either single- or dual-lane left-turn bays. ADT volumes within the widened area ranged between 30,100 and 49,100 vpd in 1991, between 26,000 and 44,900 vpd in 1992, and between 23,800 and 46,300 vpd in 1993.

Study Section. The 1-mi section of Merritt Island Parkway that was selected for field study begins at the signalized intersection with Courtney Parkway and ends at the signalized intersection with Sykes Creek Parkway. In addition to these intersections, the study section has three signalized intersections and eight unsignalized intersections.

One of the unsignalized intersections in the study section permits all traffic movements, one is restricted to right turns only, and the remaining six permit all right turns but only left turns from the major street, not from the adjacent properties. The study section also includes several minor driveways that allow for right turns only. Land use consists of 59 retail and commercial establishments. Of the 59 questionnaires distributed at this site, 38 were returned (a 64 percent response). Of the 38 respondents, 34 businesses were in place before the arterial was improved and 4 were established after the widening.

Table 5-3 summarizes the distribution of business categories and access types provided at this study site. As noted in the discussion of Table 5-1, the number of surveys from auto-related businesses was determined to be too low to support a valid assessment of the access impacts to this business category. As a result, the responses for both auto- and non-auto related businesses were combined to broaden the examination of access impact to all types of businesses.

Public Opinion Survey. Table 5-4 summarizes the survey results from the Merritt Island Parkway study site. As with the Oakland Park Boulevard site, only responses from businesses that were established before construction were used to determine the effect of a change in left-turn treatment on business activity and traffic conditions.

TABLE 5-3 Business category and left-turn access type—Merritt Island Parkway

Auto-Related - 11 Businesses (29%)	
Full Access	2 (5%)
Partial Left-Turn Access	9 (24%)
Non-Auto-Related - 27 Businesses (71%)	
Full Access	4 (10%)
Partial Left-Turn Access	23 (61%)

Evaluation. The largest number of respondents (47 percent) indicated their belief that business activity in the area has decreased during the past several years. An even larger number of respondents (63 percent) believe that activity at their businesses decreased as a result of the reconstruction project. This finding suggests that some of the business loss may be attributable to a general decline in business in the area and some may be attributable to the change in median treatment. It should be noted that a majority of respondents (52 percent) believe that property values increased as a result of the project.

Regarding traffic conditions, a majority of the respondents indicated that traffic congestion, operations, and safety were improved as a result of the reconstruction project. These improvements are most likely due to the increase in capacity resulting from the addition of two through lanes rather than the conversion of a TWLTL to a raised-curb median treatment. In fact, a large majority of survey respondents indicated that the improved traffic conditions came at the expense of a reduction in property access, business opportunities, customer convenience, and delivery convenience. Access, however, was found to be much less important than price and quality of merchandise in terms of the factors that influence a customer's decision to shop at a specific store.

Many comments from the business representatives had to do with drainage problems along the arterial. An effort was made to focus the respondents on the changes in access, but this was not always successful. As a result, the respondents

TABLE 5-4 Survey summary—Merritt Island Parkway

Question	Responses	Before	After	
1. Business established before or after construction.	38	89 %	11 %	
	Responses	Yes	No	
3. Access effect on location decision.	4	0 %	100 %	
	Responses	Increase	Decrease	No Change
7. Change in area's business activity after construction.	34	35 %	47 %	18 %
2. Change in your business activity due to construction.	30	17 %	63 %	20 %
8. Change in property value after construction.	23	52 %	9 %	39 %
9. Issues after reconstruction.	Responses	Better	Worse	No Change
a. Traffic Congestion	33	67 %	15 %	18 %
b. Traffic Operation	33	55 %	27 %	18 %
c. Traffic Safety	34	50 %	26 %	24 %
d. Property Access	34	12 %	77 %	11 %
e. Business Opportunity	33	18 %	58 %	24 %
f. Customer Convenience	34	9 %	76 %	15 %
g. Customer Satisfaction	34	12 %	53 %	35 %
h. Delivery Convenience	34	6 %	54 %	40 %
10. Ranking of factors influencing customer's decision.	Responses	Factor Rank		
		High (8-10)	Medium (4-7)	Low (1-3)
a. Price	34	71 %	29 %	0 %
b. Quality	34	91 %	9 %	0 %
c. Service	34	97 %	3 %	0 %
d. Hours Open	34	44 %	41 %	15 %
e. Accessibility	33	34 %	39 %	27 %

who indicated that traffic conditions had deteriorated are believed to have incorrectly associated "traffic conditions" with "street drainage."

Conclusions. Although adding through traffic lanes and either eliminating or separating left-turn conflicts improved traffic safety and operation, the street improvements were perceived as having detrimental effects on adjacent businesses. A large majority of respondents perceived a decrease in business activity and property access after the arterial was widened.

Roosevelt Road—Wheaton/Glen Ellyn, Illinois (Undivided/4 to TWLTL/4)

Site Description. Roosevelt Road (S.R. 38) is a major arterial in the west suburban area of Chicago and serves between 25,000 and 48,500 vpd. ADT demand is expected to approach 52,000 by the year 2000. Between 1987 and 1991, 7.5 mi of the arterial were widened. It originally had a four-lane undivided cross section but was widened to include four through lanes and a TWLTL. The TWLTL transitions into exclusive left-turn bays at the signalized intersections.

Roosevelt Road has many "high-accident" locations and carries a red designation in the Illinois Department of Transportation's accident classification system. About 45 percent of the accidents occur at major intersections or at heavily used commercial entrances between major intersections. The vast majority of these accidents (77 percent) are rear-end (47 percent) or other turn-related types (30 percent). Adjacent land uses consist of some residential but primarily retail and commercial businesses. For the most part, property frontages are short and there is a proliferation of private driveways.

Study Section. A 1.3-mi section of Roosevelt Road was selected for the field survey. The section, between the signalized intersections at Blanchard Street and at Park Boulevard, contains five signalized intersections. Signal spacing varies from about 0.25 to 0.5 mi. The section also contains almost 100 private entrances and six local street intersections that operate under stop sign control. The study section has 87 developments that include 71 businesses and 16 residences. Each development has at least one driveway and, in some cases, the larger businesses have two driveways. All driveways have full access.

Of the 71 businesses contacted, all accepted the questionnaire. Forty-nine questionnaires were completed and returned. This corresponds to a return rate of almost 69 percent. Forty-two of the responding businesses were in operation before construction; seven located on Roosevelt Road after construction. Table 5-5 indicates the various development types on this arterial.

Public Opinion Survey. Table 5-6 summarizes the survey results from businesses established after construction. Because the area is stable with respect to development and because street improvements were completed only 3 years before the survey, 86 percent of the businesses in the study section were established before the arterial was widened. Questionnaires were only distributed to business establishments to maintain consistency with the other surveys; residential areas were not included.

Evaluation. Fifty percent of respondents believe that business activity in their areas has increased during the past several years. This finding contrasts with 57 percent of respondents who reported that activity at their businesses did not change as a result of the project. This suggests that businesses whose access was affected by the project may have lost the opportunity to increase their activity in a manner consistent with other businesses in the area. Fifty percent of the business representatives did indicate that property values increased as a result of the project.

With respect to traffic conditions, a large majority of respondents believe that the new TWLTL improved traffic operations and safety and that it reduced congestion, relative to the original undivided cross section. Many respondents believe that property access improved. Almost all representatives who were interviewed said traffic operations are much improved. The main point of dissatisfaction indicated is the loss of land, and sometimes parking, resulting from the associated street widening. It was surprising to find that many of the business representatives considered property access to be worse (19 percent) or unchanged (33 percent) as a result of this project; unfortunately, no quantifiable reason for this trend could be determined.

The ranking of factors that influence customers revealed that site accessibility was the least influential factor at this study section.The price and quality of merchandise, the service given to customers, and the hours of operation were all

TABLE 5-5 Business category and land use—Roosevelt Road

Auto-Related - 10 Businesses (21%)	
Retail	10 (100%)
Service	0 (0%)
Non-Auto-Related - 39 Businesses (79%)	
Retail	28 (72%)
Service	9 (23%)
Other	2 (5%)

TABLE 5-6 Survey summary—Roosevelt Road

Question	Responses	Before	After	
1. Business established before or after construction.	49	86 %	14 %	
	Responses	Yes	No	
3. Access effect on location decision.	7	0 %	100 %	
	Responses	Increase	Decrease	No Change
7. Change in area's business activity after construction.	40	50 %	15 %	35 %
2. Change in your business activity due to construction.	42	36 %	7 %	57 %
8. Change in property value after construction.	30	50 %	7 %	43 %
9. Issues after reconstruction.	Responses	Better	Worse	No Change
a. Traffic Congestion	42	81 %	2 %	17 %
b. Traffic Operation	42	74 %	2 %	24 %
c. Traffic Safety	42	76 %	7 %	17 %
d. Property Access	42	48 %	19 %	33 %
e. Business Opportunity	41	39 %	7 %	54 %
f. Customer Convenience	42	57 %	10 %	33 %
g. Customer Satisfaction	41	39 %	7 %	54 %
h. Delivery Convenience	42	35 %	10 %	55 %
10. Ranking of factors influencing customer's decision.	Responses	Factor Rank		
		High (8-10)	Medium (4-7)	Low (1-3)
a. Price	38	52 %	45 %	3 %
b. Quality	40	90 %	10 %	0 %
c. Service	41	91 %	7 %	2 %
d. Hours Open	38	36 %	53 %	11 %
e. Accessibility	38	35 %	47 %	18 %

ranked higher (i.e., more important) than the ease and convenience of site access. This finding is consistent with those found at the other study sites.

Conclusions. For the most part, survey respondents perceived traffic conditions to be much improved after the addition of the TWLTL. However, the results were mixed when it came to impacts to business. Business activity, opportunity, and delivery convenience are believed to be unchanged, whereas property value, property access, and customer convenience generally are believed to be improved. Most surprising was the 19 percent of respondents who believe that property access decreased as a result of the conversion from an undivided cross section to a TWLTL.

Port Washington Road—Mequon, Wisconsin (Undivided/4 to RM330/6)

Site Description. Port Washington Road is a county highway (County Road W) that was widened in 1992 from a four-lane undivided arterial to a six-lane arterial with a 28-ft raised-curb median. The wide median accommodates dual-lane left-turn bays at the road's intersection with S.R. 167. It also accommodates a mixture of unsignalized intersections, some permitting all movements and some permitting left turns from the arterial but not left turns onto the arterial. The 1991 average daily traffic volume on the arterial was about 13,500 vpd. By the year 2011, this volume is expected to increase to 23,300 vpd. Land use along the arterial is a combination of strip and cluster commercial and office developments. Discussions with area business representatives indicated that accidents were more frequent on the arterial before it was widened.

Study Section. The study section, which is about 0.5 mi in length, contains two signalized intersections, six full-access unsignalized intersections, one unsignalized intersection with partial left-turn access, and nine driveways with no direct left-turn access. The frequent median openings have resulted in the median having an alternating left-turn bay design.

Before the arterial was widened, all access points allowed for full access. The north end of the study section is at the signalized intersection with S.R. 167 and the south end is at the end of the channelized cross section. Land use is retail, non-

TABLE 5-7 Business category and land use—Port Washington Road

Auto-Related - 3 Businesses (9%)	
Retail	2 (6%)
Other Commercial	1 (3%)
Non- Auto-Related - 32 Businesses (91%)	
Retail	23 (66%)
Other Commercial	9 (25%)

retail commercial, and office. The distance between traffic signals is about 1,150 ft. The offices are primarily freestanding buildings with individual parking areas and private driveways. The retail developments are either freestanding (i.e., a gas station with its own driveways) or clustered in a shopping center configuration. All businesses in these shopping centers share a common parking area, and site access is consolidated to select locations. Table 5-7 presents the development type distribution along the study section.

Public Opinion Survey. A representative at each business along the study section was asked to complete the survey. Of the representatives who did not accept the questionnaire, several were employed by large retail developments whose corporate policy is not to respond to surveys. Of the 63 business representatives who accepted the questionnaire, 35 (56 percent) of them returned it completed. Twenty-seven (77 percent) of the survey respondents were in business before the widening and eight (23 percent) started in business after construction. The results of the survey are summarized in Table 5-8.

Evaluation. A majority of respondents (62 percent) indicated that their business activity increased as a result of the improvement project. This high positive response was somewhat surprising because some property access was lost because of the construction of a raised-curb median. However, the increase in activity is consistent with the perception

TABLE 5-8 Survey summary—Port Washington Road

Question	Responses	Before	After	
1. Business established before or after construction.	35	77 %	23 %	
	Responses	Yes	No	
3. Access effect on location decision.	8	100 %	0 %	
	Responses	Increase	Decrease	No Change
7. Change in area's business activity after construction.	26	73 %	12 %	15 %
2. Change in your business activity due to construction.	24	62 %	17 %	21 %
8. Change in property value after construction.	18	72 %	0 %	28 %
9. Issues after reconstruction.	Responses	Better	Worse	No Change
a. Traffic Congestion	25	80 %	16 %	4 %
b. Traffic Operation	25	72 %	20 %	8 %
c. Traffic Safety	26	50 %	31 %	19 %
d. Property Access	24	66 %	17 %	17 %
e. Business Opportunity	23	56 %	9 %	35 %
f. Customer Convenience	25	68 %	20 %	12 %
g. Customer Satisfaction	23	53 %	17 %	30 %
h. Delivery Convenience	24	58 %	13 %	29 %
10. Ranking of factors influencing customer's decision.	Responses	Factor Rank		
		High (8-10)	Medium (4-7)	Low (1-3)
a. Price	27	59 %	37 %	4 %
b. Quality	27	93 %	7 %	0 %
c. Service	27	89 %	11 %	0 %
d. Hours Open	27	15 %	81 %	4 %
e. Accessibility	27	22 %	63 %	15 %

of a majority of respondents (73 percent) that business activity throughout the area has increased during the past several years. Therefore, it is likely that a portion of the increased activity is a result of this areawide activity increase. Moreover, a portion of the increase may be due to the extra traffic capacity provided by the two additional through lanes included in the improvement project.

Regarding traffic conditions, a majority of respondents indicated that traffic operations and safety were improved. These improvements are likely caused as much by the extra traffic capacity provided by the additional through lanes as by the conversion in midblock treatment type. Many respondents commented on the confusion caused by unfamiliar traffic movements associated with the new raised-curb median treatment and its restricted access median openings. It is believed that the close proximity of many alternating left-turn bays, along with a mixture of local and nonlocal drivers, could be the cause of driver confusion and concern. In fact, this concern may account for the relatively low percentage of respondents (50 percent) who believe that traffic safety was improved by the raised-curb median treatment.

Conclusions. The introduction of a raised-curb median was well received by the business representatives located along this arterial. However, based on the comments offered by these representatives, it appears that the signing and marking of restricted access median openings need close scrutiny to minimize driver confusion. The consolidation of access and selective restriction of some left-turn movements can significantly improve traffic operations, particularly when adequate and convenient secondary access routes are available.

Summary of Survey Findings

The responses to several questions were grouped into categories to facilitate (1) a more general assessment of access impact and (2) the examination of the effects of a change in midblock left-turn treatment on traffic conditions, business conditions, and customer influence factors. Responses to Question 9 regarding traffic congestion, operation, and safety were grouped to characterize the effect of treatment changes on traffic conditions. Similarly, responses to Question 9 regarding property access, Question 2 regarding business activity, and Question 8 regarding property values were grouped to characterize the effect of treatment change on business conditions. Finally, responses to Question 10 regarding factors that influence a customer's decision to patronize a business (i.e., accessibility, service, and quality) were grouped to provide perspective on the relative importance of access to the customer. The results of this aggregation are shown in Table 5-9.

TABLE 5-9 Summary of selected survey responses (percentages shown include responses from both auto-related and non-auto-related businesses)

	Impact of a change in midblock left-turn treatment type (all values in percent)											
Location:	Oakland Park Blvd. (RM330/6 to RM660/6)[1]			Merritt Island Pkwy. (TWLTL/4 to RM660/6)			Roosevelt Road (Undivided/4 to TWLTL/4)			Port Washington Road (Undivided/4 to RM330/6)		
Response:	Better	Worse	N.C.[2]	Better	Worse	N.C.	Better	Worse	N.C.	Better	Worse	N.C.
Traffic congestion (9a)	16	32	52	67	15	18	81	2	17	80	16	4
Traffic operation (9b)	20	16	64	55	27	18	74	2	24	72	20	8
Traffic safety (9c)	20	32	48	50	26	24	76	7	17	50	31	19
Overall Traffic:	**18**	**27**	**55**	**57**	**23**	**20**	**77**	**4**	**19**	**67**	**22**	**11**
Property access (9d)	16	28	56	12	77	11	48	19	33	66	17	17
Business activity (2)	28	28	44	17	63	20	36	7	57	62	17	21
Property value (8)	25	19	56	52	9	39	50	7	43	72	0	28
Overall Business:	**22**	**25**	**53**	**23**	**55**	**22**	**44**	**11**	**45**	**67**	**12**	**21**
	Rank of factors influencing a customer's decision to patronize a business (10)											
Response:	High (8-10)	Med. (4-7)	Low (1-3)	High (8-10)	Med. (4-7)	Low (1-3)	High (8-10)	Med. (4-7)	Low (1-3)	High (8-10)	Med. (4-7)	Low (1-3)
Quality	87	13	0	91	9	0	90	10	0	93	7	0
Service	96	4	0	97	3	0	91	7	2	89	11	0
Accessibility	46	33	21	34	39	27	35	47	18	22	63	15

Notes:

1 - Cross section of arterial before and after reconstruction project. Convention: xxxxx/# where, "xxxxx" is the midblock left-turn treatment type and "#" is the number of through lanes. Midblock left-turn treatment types: RM330 - raised-curb median with 330-ft median openings (alternating bays); RM660 - raised-curb median with 660-ft median openings; TWLTL - two-way left-turn lane; Undivided - undivided cross section.

2 - N.C. - no change.

Survey respondents at three of the four study sites indicated that the reconstruction project improved traffic conditions. At two sites, Roosevelt Road and Port Washington Road, this improvement is likely due to the conversion from an undivided cross section. Much of the improvement at the third site, Merritt Island Parkway, is likely due to the extra capacity provided by two additional through lanes. Respondents at the Oakland Park Boulevard site reported that there were no changes in traffic or business conditions. These findings are surprising because the changes at this site were intended to improve arterial traffic operations and safety by eliminating half of the median openings and one left-turn movement at the remaining openings.

Respondents' views of the effects of a change in midblock left-turn treatment on business conditions varied. Many respondents at the two sites that converted from undivided cross sections indicated that business conditions had improved as a result of the treatment change. In contrast, a majority of respondents at each of the two sites that converted to the RM660 (i.e, raised-curb median with openings every 660 ft) reported that the change either degraded business conditions or had no net effect. As noted previously, the lack of an effect on business conditions at the latter site, Oakland Park Boulevard, was somewhat surprising in light of the fact that property access was significantly reduced as a result of the treatment change.

In terms of the factors that influence a customer's decision to patronize a business, the data in Table 5-9 indicate that the respondents believe that service is the most important factor, followed closely by product quality. Access is ranked much lower in importance than either service or quality. This finding indicates that businesses, particularly those that are non-auto-related, may be able to overcome some reduction of access if they offer good, reliable service. This generalization may not be true for auto-related businesses that, for obvious reasons, tend to place a higher premium on access.

It should be noted that questionnaire responses are subjective for each respondent and, although a total of 165 questionnaires were returned, they represent four unique changes in midblock left-turn treatment. Moreover, site-specific factors (e.g., surrounding street network and local economy) also may be indirectly represented in the survey findings. As a result, these results should be interpreted with caution; they are not intended to definitively quantify the relative merits of one midblock left-turn treatment over another.

DEVELOPMENT OF AN ACCESS IMPACT MODEL

Model Development

The purpose of the access impact model is to provide a quantitative method of evaluating the effect of a change in midblock left-turn treatment on adjacent land uses. The model predicts an "access impact index" for a given midblock left-turn treatment at a specific location. A similar index can be computed for one or more alternative midblock treatments to facilitate an assessment of relative access impact.

The access impact index represents the sum of the weighted utility indices for each business property along the subject arterial section. The weighted utility index for a property represents the combined utility indices for a range of impact measures, as weighted by the relative importance of each measure. In areas where properties differ in their demand for access, the weighted utility index for each property can be further weighted by some measure of its access need (i.e., number of driveways, arterial frontage length, or square footage).

The utility index represents the relative impact of a change in left-turn treatment and property access on a business property. This impact is measured in terms of traffic conditions, property access, and business operations. It should be noted that the traffic conditions impact measure used in the development of the access impact model is not the same as that described in other chapters of this report. The values used in the access impact model are based on the *perceptions* of the owners or managers of business properties adjacent to the subject arterial. As a result, these perceptions are biased toward the effect of traffic conditions on customer attitudes.

The form of the access impact model is as follows:

$$AI = \frac{\sum_{i=1}^{N_p} U_{i,(k,L)} m_i}{\sum_{i=1}^{N_p} m_i} \tag{1}$$

where:

AI = access impact index for the subject arterial with a specified midblock left-turn treatment;
$U_{i,(k,L)}$ = weighted utility index of property i based on a change in left-turn storage L and access k;
m_i = "mass" of property i (i.e., number of driveways, frontage length, or square footage); and
N_p = number of individual properties along both sides of subject arterial.

When the individual properties are deemed to be similar, such as properties along an arterial fronted by strip retail stores, the mass factor for each property can be eliminated or set to 1.0. This form of the access impact model follows:

$$AI = \frac{1}{N_p} \sum_{i=1}^{N_p} U_{i,(k,L)} \tag{2}$$

The development of the weighted utility indices $U_{i,(k,L)}$ was somewhat challenging because of the differences in impor-

tance assigned by the various property owners. For example, retailers usually are concerned with sales and income, whereas commercial business owners are concerned with access for employees and customers. Residential land owners are more concerned with their convenience, property values, and safety. All these concerns are important, but subjective, to the property owner and vary by location and type of land use, thereby making them difficult to quantify.

The approach taken to overcome the aforementioned differences in concerns was to quantify a utility index u for each access impact measure and a corresponding weight. Residential owners were not included in the development of the utility index because it was believed that the impact of a change in their access was much less than the impact on business property owners or managers. The utility index is computed as follows:

$$U_{i,(k,L)} = \left(\frac{1}{30} \sum_{j=1}^{3} u_{k,L,j} w_{k,L,j} \right)_i \tag{3}$$

where:

$U_{i,(k,L)}$ = weighted utility index of property i based on a change in left-turn storage L and access k;
$u_{k,L,j}$ = utility index for impact measure j relative to a change in left-turn storage L and access k; and
$w_{k,L,j}$ = weight of impact measure j relative to a change in left-turn storage L and access k.

The utility index $u_{k,L,j}$ represents the percentage of business representatives who believe that the change in left-turn treatment resulted in "better" or "no change" in one of three impact measures: traffic conditions, property access, and business operations. Traffic conditions were quantified using the survey responses to questions about changes in traffic congestion, operations, and safety resulting from a change in left-turn treatment (i.e., Question 9a, 9b, and 9c). Property access indices were quantified using the responses to Question 9d. Finally, the business operations indices were assessed using the response to Questions 2 and 8. Because the number of auto-related businesses in the survey was relatively small, it was decided to combine the responses of representatives from these businesses with those from non-auto-related businesses.

The change in midblock left-turn treatment, as it affected an individual property i, was categorized as a change in "left-turn storage" and a change in "property access." In this instance, left-turn storage refers to the provision of a TWLTL or left-turn bay in a raised-curb median on the arterial to explicitly serve traffic accessing the subject property. The type of left-turn storage provided at a respondent's property in the before and after cases was determined from the address provided on the survey questionnaire and the geometric design data collected during the site visit. Table 5-10 contains the utility indices computed for this study.

The relative weight factors $w_{k,L,j}$ were established using survey responses, interviews with business representatives, and the study team's experience. These factors have been established as having a range from 10 (most important) to 1 (least important). Table 5-11 indicates the weight factors for each impact measure and change in midblock left-turn treatment condition.

Equation 3 was used to compute the weighted utility index $U_{i,(L,k)}$ for all possible combinations of a change in left-turn storage L and property access k. The resulting weighted utility indices are presented in Table 5-12. This computation simplifies the application of the access impact model by combining Tables 5-10 and 5-11; however, if alternative indices or weights are wanted, Equation 3 must be used with the alternative values for each property i to which they apply.

The access impact model was developed so that the access impact index AI could theoretically vary from 0.0 to 1.0. The index for an existing arterial with a specific left-turn treat-

TABLE 5-10 Utility index ($u_{k,L,j}$)

Left-turn Storage (L)	Impact Measure	Change in Property Access (k)		
		No Change (k=1)	Increased (k=2)	Decreased (k=3)
No Change (L=1)	Traffic Condition	0.80	--	0.70
	Property Access	0.67	--	0.40
	Business Operation	0.79	--	0.59
Increased (L=2)	Traffic Condition	0.90	1.00	0.83
	Property Access	0.85	1.00	0.75
	Business Operation	0.92	1.00	0.90
Decreased (L=3)	Traffic Condition	--	--	0.79
	Property Access	--	--	0.33
	Business Operation	--	--	0.51

Notes:
"--" - no data available.

TABLE 5-11 Relative weight factors ($w_{k,L,j}$)

Left-turn Storage (L)	Impact Measure	Change in Property Access (k)		
		No Change (k=1)	Increased (k=2)	Decreased (k=3)
No Change (L=1)	Traffic Condition	3	-	7
	Property Access	5	-	3
	Business Operation	4	-	3
Increased (L=2)	Traffic Condition	6	7	10
	Property Access	6	10	3
	Business Operation	5	10	3
Decreased (L=3)	Traffic Condition	-	-	8
	Property Access	-	-	2
	Business Operation	-	-	3

ment is 0.30. This represents the "no change" condition for both the left-turn storage and property access factors. In application, the impact index is computed for alternative midblock left-turn treatments and compared with the base index of 0.30. Treatments yielding higher index values are likely to have a more positive impact on adjacent business properties.

The calibrated access impact model provides a method for predicting which alternative midblock left-turn treatment is best in terms of its impact on adjacent land uses, from a business owner's or manager's perspective. The utility indices used in this model were obtained from a survey of businesses located on four arterials in three states. Therefore, the relative weight factors are subjective and may require some adjustment for application to other arterial sites.

Example Application

The following example illustrates the use of the access impact model. The arterial associated with this example has a four-lane undivided cross section. Two alternative midblock left-turn treatments are being considered: (1) a TWLTL and (2) a raised-curb median with 660-ft spacing between median openings. A total of 120 businesses are located along the arterial (i.e., $N_p = 120$).

The access impact index for the existing (or base) arterial is computed using the "no change" in access and left-turn storage categories (i.e., $k = 1$ and $L = 1$). The land use along the arterial is essentially strip commercial with numerous small businesses and retail shops. Therefore, for simplicity, the "mass" factor m_i for each property i is assumed to be 1.0. This simplification permits the use of Equation 2 to compute the access impact index AI as follows:

$$AI = \frac{1}{N_p}\sum_{i=1}^{N_p} U_{i,(k,L)}$$

$$= \frac{1}{120}(120 \times 0.30)$$

$$= 0.30 \qquad (4)$$

Alternative 1—Two-Way Left-Turn Lane

The first alternative treatment considered for this arterial is a TWLTL. This treatment does not change property access relative to the existing undivided cross section (i.e., $k = 1$); however, it does increase the left-turn storage provided along the arterial (i.e., $L = 2$). The resulting impact index is computed as follows:

$$AI = \frac{1}{120}(120 \times 0.50)$$

$$= 0.50 \qquad (5)$$

TABLE 5-12 Weighted utility index ($U_{i,(k,L)}$)

Left-turn Storage (L)	Change in Property Access (k)		
	No Change (k=1)	Increased (k=2)	Decreased (k=3)
No Change (L=1)	0.30	-	0.26
Increased (L=2)	0.50	0.90	0.44
Decreased (L=3)	-	-	0.28

Alternative 2—Raised-Curb Median

The second treatment considered for this arterial is a raised-curb median. The 660-ft spacing of the median openings associated with this treatment will reduce access to about 70 businesses (50 businesses will have no change in access). The left-turn storage of these same 70 businesses will be unchanged (i.e., the existing treatment provides no left-turn storage). The left-turn storage of 50 businesses will be increased. These changes are summarized as follows:

50 businesses: unchanged access ($k = 1$) and increased storage ($L = 2$) so $U_{50,(1,2)} = 0.50$
70 businesses: decreased access ($k = 3$) and unchanged storage ($L = 1$) so $U_{70,(3,1)} = 0.26$

$$AI = \frac{1}{120}(50 \times 0.50 + 70 \times 0.26)$$
$$= 0.36 \qquad (6)$$

Assessment

From the standpoint of businesses adjacent to the proposed improvement, the addition of a TWLTL has a positive effect on access. It appears that the impact index increases about 67 percent over the existing left-turn treatment. The raised-curb median alternative also appears to have a positive effect on access, but less than that of the TWLTL. The raised-curb median alternative increases the impact index only about 20 percent. An examination of the weighted utility indices indicates that the increase in left-turn storage associated with the raised-curb median offsets the loss in access due to the restriction of left-turn movements at many locations.

Conclusions

The access impact model is provided as a tool for quantitatively analyzing the access impacts of a change in midblock left-turn treatment. It is recommended that the impact index values contained in this report be used as default values that should be typical of most arterial sites. These values should be updated whenever the characteristics of the sites used to derive these default values are not consistent with the site being analyzed.

Based on the survey of business representatives, it is concluded that these persons believe that the conversion from an undivided cross section to either a raised-curb median (with openings every 330 ft) or a TWLTL *will* improve arterial traffic conditions and business conditions (i.e., property values, access, and sales). In contrast, business representatives believe that the conversion from either a raised-curb median (with openings every 330 ft) or a TWLTL to a raised-curb median with openings every 660 ft *will not* improve business conditions. Therefore, from a business representative's perspective, the undivided cross section should be avoided and median openings should be provided as frequently as possible if a raised-curb median treatment is provided.

In terms of the factors that influence a customer's decision to patronize a business, the survey indicated that business representatives believe that customers rank property access much lower in importance than either service or quality. This finding indicates that the typical business may be able to overcome some reduction of access if it offers good, reliable service. This generalization may not be true for auto-related businesses that, for obvious reasons, tend to place a higher premium on access.

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CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

The following conclusions were reached as a result of this research. The conclusions are categorized as those relating to the operational effects, safety effects, and access impacts associated with the three midblock left-turn treatments considered in this research: the raised-curb median, the two-way left-turn lane (TWLTL), and the undivided cross section.

Operational Effects

The major conclusions of the research into the operational effects of the three midblock left-turn treatments follow:

1. A midblock left-turn treatment can affect traffic flow in a variety of direct and indirect ways. Direct impacts stem from the provision or lack of an arterial left-turn storage area. Indirect impacts relate to the interaction between the arterial turn and through movements. Left and right turns from the arterial frequently tend to slow down and delay through vehicles when turn bays of adequate length are not provided. Although individual delays typically are small compared with those incurred by nonpriority movements, the aggregate delay to the inherently large number of through drivers can be significant. Therefore, left-turn (and right-turn) treatments that provide for the deceleration and storage of arterial turn vehicles should be provided whenever possible.
2. The performance of an unsignalized access point often is degraded by the close proximity of another intersection. Traffic slowing for or queued at a downstream intersection may reduce the capacity of the access point if the deceleration or queueing occurs in the vicinity of the access point. The nature and magnitude of the effect of a downstream intersection depends on a wide variety of factors related to the intersection, including its arterial traffic volume, the distance between it and the subject access point, and the type of traffic control on it (i.e., signalized or unsignalized). As a result, it is very difficult to define a reasonable minimum intersection (or access point) spacing that would be suitable for most conditions.
3. Traffic platoons created by upstream signalized intersections can affect the operation of an access point. As they pass through the intersection, these platoons will block a nonpriority movement, but will leave large gaps afterward. These large gaps have a compensating effect in that upstream signals can increase the capacity of the nonpriority movements.
4. The operations model developed for this research was shown to be an effective tool that can accurately predict delays to arterial traffic movements. The component models included in the operations model make it sensitive to the impact of midblock left-turn treatments on arterial traffic movements. The model verification process indicated that the operations model can replicate driver behavior on urban arterials with closely spaced access points and signalized intersections. The delays predicted by this model can be used to evaluate the operational performance of alternative left-turn treatments.
5. The application of the operations model to a wide range of traffic demand and geometric conditions indicated that the raised-curb median and the TWLTL yield similar delays to arterial drivers (although the raised-curb median yields slightly higher delays than the TWLTL at the highest left-turn and through volume levels). The undivided cross section yields significantly higher delays than the raised-curb median and TWLTL. The difference in delay increases exponentially with an increase in left-turn or through volume.
6. Analysis of the operations model indicates that any of the treatment types can function without creating congestion within the major-street movements at average daily traffic demands of 40,000 vpd or less. Demands exceeding 40,000 vpd are possible for both four- and six-lane streets; however, congested conditions are likely to occur. When demand exceeds 40,000 vpd, congestion on a four-lane street is more likely at the signalized intersections than at the major-street left-turn movement of an access point. In contrast, congestion on a six-lane street is more likely at the major-street left-turn movement of an access point than at the signalized intersections.

Safety Effects

The major conclusions of the research into the safety effects of the three midblock left-turn treatments are as follows:

1. The analysis of the accident data indicated that average daily traffic demand, driveway density, unsignalized public street approach density, left-turn treatment type, and adjacent land use are significantly correlated with accident frequency. In general, accidents are more frequent on street segments with higher traffic demands, driveway densities, and public street densities. Accidents also are more frequent when the land use is business or office as opposed to residential or industrial.
2. The safety model analysis indicated that the undivided cross section has a significantly higher accident frequency than the TWLTL or raised-curb median treatments when parallel parking is allowed on the undivided street. If there is no parking allowed on either street, the difference between the undivided and TWLTL treatments is generally small and is negligible for average daily traffic demands of less than 25,000 vpd. In general, the raised-curb median treatment appears to be associated with fewer accidents than the undivided cross section and TWLTL, especially for average daily traffic demands exceeding 20,000 vpd.
3. Regression methods based on maximum likelihood techniques and a negative binomial distribution of the residuals are necessary to accurately calibrate accident prediction models. The use of these methods revealed that the relationship between accident frequency and exposure (e.g., average daily traffic demand or segment length) is nonlinear. This finding indicates that the use of accident rates (and models that predict accident rates) may not yield accurate estimates of accident frequency, especially if the range in the database is exceeded.
4. A new variable was introduced for accident model calibration. This variable represents the ratio of property-damage-only (PDO) accidents to all reported accidents for an urban area. As such, it is a direct measure of the accident cost reporting threshold and the degree of driver compliance with accident reporting requirements in a given area. The inclusion of PDO percentages in an accident prediction model facilitates comparison of the relative safety of arterial streets in different cities and states through the use of a common normalizing factor.
5. The safety model developed for this research was shown to be an effective tool for estimating the annual accident frequency for urban and suburban arterials. The accident frequencies predicted by this model can be used to evaluate the safety of alternative midblock left-turn treatments.

Access Impacts

Following are the major conclusions of the research into the access impacts of the three midblock left-turn treatments:

1. Thirty-three reconstruction projects that involved a change in midblock left-turn treatment were identified in this research. The majority (22) of these projects involved a left-turn treatment conversion that resulted in no change in the level of access provided to adjacent properties. Two-thirds of these "no change" projects involved a conversion from an undivided cross section to a TWLTL. Nine of the 33 projects resulted in a decrease in property access; all involved a conversion from an undivided cross section, TWLTL, or flush median to a raised-curb median. Only two of the 33 projects resulted in more property access; they involved a conversion from the raised-curb median to the TWLTL.
2. Business owners and managers believe that the conversion from an undivided cross section to either a raised-curb median (with openings every 330 ft) or a TWLTL *will* improve arterial traffic conditions and business conditions (i.e., property values, access, and sales). In contrast, business representatives believe that the conversion from either a raised-curb median (with openings every 330 ft) or a TWLTL to a raised-curb median with openings every 660 ft *will not* improve business conditions. Therefore, from the business representative's perspective, the undivided cross section should be avoided and median openings should be provided as frequently as possible if a raised-curb median treatment is provided.
3. In terms of the factors that influence a customer's decision to patronize a business, the survey indicated that the business representatives believe that customers rank property access much lower in importance than either service or quality. This finding indicates that the typical business may be able to overcome some reduction of access if it offers good, reliable service. This generalization may not be true for auto-related businesses that, for obvious reasons, tend to place a higher premium on access.
4. The access impact model developed for the research represents a new type of tool for quantifying the impact of a change in left-turn treatment on adjacent property access and business opportunity. The access impact index predicted by this model (as reported here) can be used to evaluate the access impacts of alternative midblock left-turn treatments.

RECOMMENDATIONS

Based on the findings and conclusions of this research, the following recommendations are made:

1. The operational effects, safety effects, and access impacts of a midblock left-turn treatment should be fully evaluated when considering the treatment's application to a specific arterial street.

2. The guidelines provided in this report should be used for a preliminary evaluation of the operational and safety effects of alternative midblock left-turn treatments. In situations in which the assumed road user costs are not applicable, the individual model equations can be used. When the guidelines do not indicate that the conversion from one treatment to another is cost-effective, a site-specific examination is recommended.
3. Because the guidelines do not incorporate access impacts, it is recommended that the access impact model be used to make a preliminary evaluation of the impact of a proposed left-turn treatment. Due to the diverse nature of access impacts, an evaluation of these impacts (relative to a proposed left-turn treatment) generally will need to be conducted on a site-specific basis.

FUTURE RESEARCH

During the conduct of this research, several topics for future research were identified. These topics, as they relate to the operational effects, safety effects, and access impacts of midblock left-turn treatments, are listed in the following sections.

Operational Effects

1. The guidelines developed for this research should be extended to other midblock left-turn treatment types (e.g., flush median with paint-delineated left-turn bays, continuous parallel left-turn lanes, and reversible-lane/TWLTL combinations).
2. A detailed sensitivity analysis should be conducted to determine the effect of various traffic conditions and geometric configurations on the operational effects of midblock left-turn treatments. Factors found to have a significant effect on operations should be incorporated into the guidelines. The analysis should consider the following issues: (1) segment length, (2) staggered access point orientation, (3) variation in nonpriority movement demands among the arterial access points, and (4) frequency (or spacing) of median openings for the raised-curb median treatment.
3. Several traffic flow problems that were included in the operations model were observed during the field studies but were beyond the scope of the data collection effort. Future research should be conducted on these flow problems to quantify the nature and extent of their impact on traffic operations. Specifically, research related to access point capacity should be conducted on the following topics: (1) the effect of upstream signals (including cycle length, coordination, and distance), (2) the effect of spillback from a downstream intersection (signalized or unsignalized), and (3) the effect of and propensity for two-stage entry or crossing maneuvers. Future research also should be conducted to quantify the effect of left and right turns into an access point on the saturation flow rate of a downstream signalized intersection approach.
4. Research is needed to determine the true effect of median closures on traffic flow patterns and road user costs. To be useful, this research would need to identify median closure effect on the following: (1) U-turn volume at downstream intersections and median openings, (2) right-turn volumes at the subject access point, and (3) the types and frequency of use of routes taken by displaced left-turn drivers and the travel time associated with using these routes. This research also should address the impact of displaced left-turn drivers on the delay to existing drivers at downstream intersections.
5. Research is needed to advance the software implementation of the operations model so that it can be distributed as software to engineers for site-specific analyses. Alternatively, the model should be incorporated into the existing TRANSYT-7F computer model as a user-requested analysis extension.

Safety Effects

1. The guidelines need to be updated to incorporate the effects of parallel parking on the safety of arterials with a raised-curb median or TWLTL treatment. The safety model database needs to be expanded to include data from street segments with parallel parking and either a raised-curb median or TWLTL treatment. The safety model should then be recalibrated using these data so that it includes the effects of parking in its prediction of annual accident frequency for each left-turn treatment type.

Access Impacts

1. Additional research is needed to expand the database used to calibrate the access impact model. This expanded database should include (1) study sites that have undergone the combinations of change in left-turn storage and access that are not currently represented in the utility index matrix, (2) residential and industrial land uses, and (3) increased representation of auto-related businesses. Using this database, the access impact model could be enhanced to include a wider range of land uses and to explicitly account for the auto-related and non-auto-related business categories.
2. Further research is needed to determine how a midblock left-turn treatment's impact on access can be incorporated into the guidelines. The method of incorporation may include some type of cost conversion based on reduced property values or a loss in business activity (i.e., sales).

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Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S. DOT	U.S. Department of Transportation