NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP Report 420

Impacts of Access Management Techniques

Transportation Research Board
National Research Council

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Report 420

Impacts of Access Management Techniques

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Staff
Transportation Research Board

This report classifies access management techniques and presents methods for estimating the safety and operational effects of the different techniques. For some techniques, quantitative assessment was not practical and case studies are presented to demonstrate good and poor practice. This report will be very useful to those developing access guidelines and policy and those analyzing specific access situations.

Access management provides two major benefits to the transportation system: (1) the preservation of highway capacity and (2) improved safety. The FHWA Report, *Access Management for Streets and Highways*, was published in 1982, and, although much of its contents are still applicable, many subsequent studies and reports have identified new access management techniques and offered guidance on their application. Transportation agencies and real estate developers seek better methods of analyzing, selecting, and predicting the impacts of access management techniques. Much of the existing information is either out of date or too limited to reflect the state of the art in access management.

Under NCHRP Project 3-52, Urbitran Associates and their subcontractors listed and classified more than 100 access management techniques. A comprehensive literature search was performed and the results were synthesized. The techniques were evaluated on the basis of how widely they can be applied to the road network and the likelihood that their benefits could be expressed quantitatively. Twelve techniques were selected for further study and were consolidated into eight categories (i.e., traffic signal spacing, unsignalized access spacing, corner clearance criteria, median alternatives, left-turn lanes, U-turns as alternatives to direct left turns, access separation at interchanges, and frontage roads).

This report describes the research approach used and then discusses each of the selected techniques. In most cases, the literature review and subsequent study supported methods for quantitatively estimating the safety and operational impacts of the access management techniques. When this was not possible, case studies were used to illustrate good practice.

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State, local, and other agencies were very helpful by providing information on their access management practices and procedures. In particular, the support of the following state departments of transportation in providing accident information is acknowledged: Delaware, Illinois, Michigan, Oregon, New Jersey, Texas, Virginia, and Wisconsin.

The insights, guidance, and suggestions of the NCHRP Project 3-52 panel are greatly appreciated. Panel members included Mr. Arthur Eisdorfer (Chair), New Jersey Department of Transportation (DOT); Mr. Gary Coburn, Ohio DOT; Mr. Ronald Giguere, Federal Highway Administration; Mr. Del Huntington, Oregon DOT; Ms. Denise Kors, British Columbia Ministry of Transportation and Highways; Mr. Kenneth Lazar, Illinois DOT; Dr. William McShane, Polytechnic University; and Mr. Michael Tako, Florida DOT. The support and assistance of Mr. B. Ray Derr of the National Cooperative Highway Research Program are gratefully acknowledged.

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ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
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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
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NCHRP Report 420 Impacts of Access Management Techniques TRB

IMPACTS OF ACCESS MANAGEMENT TECHNIQUES

SUMMARY

This report discusses methods for predicting and analyzing the safety and traffic operational effects of selected access management techniques. It classifies access management techniques; identifies the more significant techniques; and suggests safety, operations, and economic impact measures. It quantifies the effects and benefits of priority techniques and sets forth salient planning and policy implications. Chapters concerning access management techniques conclude with sections containing application guidelines. These sections should be consulted for procedures to quantify the effects of access management.

The research effort focused on techniques whose effects can be measured. Where effects could not be quantified, case studies identified good and poor practice.

ACCESS MANAGEMENT TECHNIQUES

More than 100 individual access management techniques were identified. These, in turn, were grouped according to policy and roadway design features as shown in Table 1. This system links techniques to the type of improvements normally applied along highways and access driveways. It is simple to use and understand.

A series of "priority" techniques was identified for detailed analysis. These techniques (1) apply to much of the roadway system; (2) can improve safety, speeds, and emissions; and (3) are generally amenable to measurement. These priority techniques are listed in Table 2.

TRAFFIC SIGNAL SPACING (TECHNIQUE 1A - CHAPTER 3)

The spacing of traffic signals, in terms of their frequency and uniformity, governs the performance of urban and suburban highways. It is one of the most important access management techniques. This is why Colorado, Florida, and New Jersey require long signal spacings (e.g., ½ mi) or minimum through band widths (e.g., 50 percent) along principal arterial roads.

Safety

Several studies have reported that accident rates (per million vehicle miles of travel [VMT]) rise as traffic signal density increases. An increase from two to four traffic sig-

TABLE 1 Recommended classification system for access management techniques

I.	**Policy—Management**	
	a.	Access Codes/Spacing
	b.	Zoning/Subdivision Regulations
	c.	Purchase of Access Rights
	d.	Establishment of Setbacks from Interchanges and Intersections
II.	**Design - Operations (By Roadway Features)**	
	a.	Interchanges
	b.	Frontage Roads
	c.	Medians—Left Turns
	d.	Right Turns
	e.	Access/Driveway Location (Mainly Retrofit—Consolidation, Reorientation, Relocation)
	f.	Traffic Controls
	g.	Access/Driveway Design

nals per mile resulted in roughly a 40 percent increase in accidents along highways in Georgia and roughly a 150 percent increase along US 41 in Lee County, Florida. However, the safety effects may be obscured in part by differing traffic volumes on intersecting roadways and by the use of VMT for computing rates, rather than the accidents per million entering vehicles.

Travel Times

Each traffic signal per mile added to a roadway reduces speed about 2 to 3 mph. Using two traffic signals per mile as a base results in the following percentage increases in travel times as signal density increases (see Table 3). For example, travel time on a segment with four signals per mile would be about 16 percent greater than on a segment with two signals per mile.

TABLE 2 Priority techniques analyzed

1a	Establish Traffic Signal Spacing Criteria
1b	Establish Spacing for Unsignalized Access
1c	Establish Corner Clearance Criteria
1d	Establish Access Separation Distances at Interchanges
2a	Install Physical (Restrictive) Continuous Median on Undivided Highway
2b	Replace Continuous Two-Way Left-Turn Lane with Restrictive Median
3a	Install Left-Turn Deceleration Lanes
3c	Install Continuous Two-Way Left-Turn Lane
3d	Install U-Turns as Alternative to Direct Left-Turns
3e	Install Jug-Handle and Eliminate Left Turns
6a	Install Frontage Road to Provide Access to Individual Parcels
6b	Locate/Relocate the Intersection of a Parallel Frontage Road and Cross Road Further from the Arterial Cross Road Intersection

TABLE 3 Percentage increases in travel times as signal density increases

Signals Per Mile	Percent Increase in Travel Times (Compared with 2 Signals Per Mile)
2.0	0
3.0	9
4.0	16
5.0	23
6.0	29
7.0	34
8.0	39

UNSIGNALIZED ACCESS SPACING (TECHNIQUE 1B - CHAPTER 4)

Access points introduce conflicts and friction into the traffic stream. As stated in the 1994 AASHTO *A Policy on Geometric Design of Highways and Streets,* "Driveways are, in effect, at-grade intersections . . . . The number of accidents is disproportionately higher at driveways than at other intersections; thus, their design and location merit special consideration."

It is increasingly recognized that spacing standards for unsignalized access points should complement those for signalized access. Potentially high-volume unsignalized access points should be placed where they conform to traffic signal progression requirements. On strategic and primary arterials, there is a basic decision of whether access should be provided entirely from other roads.

Safety

Many studies over the past 40 years have shown that accident rates rise with greater frequency of driveways and intersections. Each additional driveway increases accident potential. This finding was confirmed by a comprehensive safety analysis of accident information obtained from Delaware, Illinois, Michigan, New Jersey, Oregon, Texas, Virginia, and Wisconsin.

Roughly 240 roadway segments, involving more than 37,500 accidents, were analyzed in detail. Accident rates were derived for various spacings and median types. The accident rate indexes shown in Table 4 were derived using 10 access points per mile as a base. (Access density is a measure of the total number of access points in both travel directions.) For example, a segment with 60 access points per mile would be expected to have an accident rate 3 times higher than a segment with 10 access points per mile. In general, each additional access point per mile increases the accident rate by about 4 percent.

Representative accident rates by access frequency, median type, and traffic signal density are summarized in Table 5 for urban and suburban areas.

Tables 6 and 7 show how accident rates rise as the total access points per mile (both signalized and unsignalized) increases in urban and rural areas, respectively, as a function of the median treatment. In urban areas, undivided highways had 9.0 accidents per million vehicle miles as compared with 6.9 for two-way left-turn lanes (TWLTLs) and 5.6 for nontraversable medians. In rural areas, undivided highways had 3 accidents per

TABLE 4 Accident rate indexes

Total Access Points Per Mile (Both Directions)	Accident Rate Index
10	1.0
20	1.4
30	1.8
40	2.1
50	2.5
60	3.0
70	3.5

TABLE 5 Representative accident rates (accidents per million VMT) by access density—urban and suburban areas

Unsignalized Access Points Per Mile	Signalized Access Points Per Mile			
	≤ 2	2.01-4.00	4.01-6.00	> 6
≤ 20	2.6	3.9	4.8	6.0
20.01-40	3.0	5.6	6.9	8.1
40.01-60	3.4	6.9	8.2	9.1
>60	3.8	8.2	8.7	9.5
All	3.1	6.5	7.5	8.9

TABLE 6 Representative accident rates (accidents per million VMT) by type of median—urban and suburban areas

Total Access Points Per Mile (1)	Median Type		
	Undivided	Two-Way Left-Turn Lane	Non Traversable Median
≤ 20	3.8	3.4	2.9
20.01-40	7.3	5.9	5.1
40.01-60	9.4	7.9	6.8
>60	10.6	9.2	8.2
All	9.0	6.9	5.6

(1) Includes both signalized and unsignalized access points.

TABLE 7 Representative accident rates (accidents per million VMT) by type of median—rural areas

Total Access Points Per Mile (1)	Median Type		
	Undivided	Two-Way Left-Turn Lane	Non Traversable Median
≤ 15	2.5	1.0	0.9
15.01-30	3.6	1.3	1.2
> 30	4.6	1.7	1.5
All	3.0	1.4	1.2

(1) Includes both signalized and unsignalized access points.

TABLE 8 Percentage of through vehicles affected at a single driveway as right-turn volume increases

Right-Turn Volume Entering Driveway (Vehicles Per Hour)	Percent of Through Vehicles Affected
Less than or equal to 30	2.4
31 to 60	7.5
61 to 90	12.2
Over 90	21.8

million vehicle miles as compared with 1.4 for TWLTLs and 1.2 for nontraversable medians.

In urban and suburban areas, each access point (or driveway) added would increase the annual accident rate by 0.11 to 0.18 on undivided highways and by 0.09 to 0.13 on highways with TWLTLs or nontraversable medians. In rural areas, each point (or driveway) added would increase the annual accident rate by 0.07 on undivided highways and 0.02 on highways with TWLTLs or nontraversable medians.

Travel Times

Travel times along unsignalized multi-lane divided highways can be estimated using procedures set forth in the 1994 *Highway Capacity Manual* (HCM). Speeds are estimated to be reduced by 0.25 mph for every access point up to a 10-mph reduction for 40 access points per mile. The HCM procedure is keyed to access points on one side of a highway, but access points on the opposite side of a highway may be included where they have a significant effect on traffic flow.

Curb-Lane Effects

Detailed analyses were made to estimate curb-lane effects on through traffic resulting from cars turning right into driveways at 22 unsignalized locations in Connecticut, Illinois, New Jersey, and New York.

Affected Vehicles

The percentage of through vehicles in the right (curb) lane that would be affected at a single driveway increases as right-turn volumes increase as shown in Table 8. The percentage of right-lane through vehicles that would be affected at least once per ¼ mi was as shown in Table 9.

TABLE 9 Percentage of right-lane through vehicles affected at least once per ¼ mi

Right-Turn Volume Per Driveway (vph)	Unsignalized Access Spacing (Feet)				
	100	200	300	400	500
Less than or equal to 30	27.3	14.7	10.0	7.6	6.2
31-60	64.2	40.1	29.0	22.6	18.5
61-90	82.1	57.6	43.6	34.9	29.1
Over 90	96.1	80.2	66.1	55.5	47.7

Influence Distances

The influence distances were calculated adding driver perception-reaction distances and car lengths to the effect lengths. The percentages of right-lane through vehicles that would be influenced to or beyond an upstream driveway in a ¼-mi section were estimated for various right-turn volumes, driveway spacings, and posted speeds. The likely percentages of affected vehicles that would extend to or beyond at least one driveway (upstream) per ¼ mi (i.e., "spillback") for a 45-mph speed are shown in Table 10. This information may be used to identify the cumulative effect of decisions concerning driveway locations and unsignalized access spacing.

Right-Turn Lanes

Right-turn deceleration lanes should be provided wherever it is desired to keep the proportion of right-lane through vehicles affected to a specified minimum. For arterial right-lane volumes of 250 to 800 vph, the percentage of through vehicles affected was about 0.18 times the right-turn volume. This results in the following effects (see Table 11) that may provide a basis for decisions regarding provision of right-turn deceleration lanes.

Criteria of 2 percent and 5 percent impacted suggest minimum right-turn volumes of 10 vph and 30 vph, respectively. This range may be applicable in certain rural settings. Criteria of 15 percent and 20 percent affected suggest a minimum of 85 vph and 110 vph, respectively. This range may be applicable in certain urban areas. The length of the deceleration lane is a function of the effect length and storage requirements.

Access Separation

The three factors that influence the desired access separation distances are safety, operations, and roadway access classification. Direct property access along strategic and principal arterials should be discouraged. However, where access must be provided, adequate spacing should be established to maintain safety and preserve movement.

"Spillback" is defined as a right-lane through vehicle being influenced to or beyond the driveway upstream of the analysis driveway. Spillback occurs when the influence length is greater than the driveway spacing minus the driveway width. The spillback rate represents the percentage of right-lane through vehicles that experience this occurrence.

The spillback rate should be kept to a level consistent with an arterial's function and desired safety and operations. Table 12 provides access separation distances for spill-

TABLE 10 Likely percentage of affected vehicles that would extend to or beyond at least one driveway

Right-Turn Volume Per Driveway (vph)	Unsignalized Access Spacing (Feet)				
	100	200	300	400	500
Less than or equal to 30	27.3	14.6	7.8	2.6	0.9
31-60	64.2	40.0	23.0	8.0	2.9
61-90	82.1	57.5	35.3	12.9	4.7
Over 90	96.1	80.1	55.5	22.1	8.3

TABLE 11 Percentage of right-lane through vehicles affected by right-turn volume

Percent Right-Lane Through Vehicles Affected	Right-Turn in Volume (vph)
0	0
2	10
5	30
10	60
15	85
20	110

back rates of 5, 10, 15, and 20 percent. For the lower speeds of 30 and 35 mph, the access separation distance shown is based on the safety implications of driveway density. For roadways with a primary function of mobility, there should not be more than 20 to 30 connections per mile (both directions).

CORNER CLEARANCES (TECHNIQUE 1C - CHAPTER 5)

Corner clearances represent the minimum distances that should be required between intersections and driveways along arterial and collector streets. As stated in the AASHTO *A Policy on Geometric Design of Highways and Streets:* "Driveways should not be situated within the functional boundary of at-grade intersections. This boundary would include the longitudinal limits of auxiliary lanes."

Corner clearance criteria assembled from various state, county, and city agencies showed a wide range of values. Setback distance criteria ranged from 16 to 325 ft.

Eight case studies of corner clearances were reviewed to illustrate current practices, problems, and opportunities. These case studies indicated that (1) definition of corner clearance distances varied among locations; (2) distances ranged from 2 to 250 ft; (3) queuing or spillback across driveways was perceived as the most pervasive problem, making it difficult to turn left into or out of a driveway; (4) roadway widening to increase capacity sometimes reduces corner clearances; (5) placing driveways too close to intersections correlates with higher accident frequencies—sometimes as many as one-half of all accidents involved are driveway-related; (6) corner clearances are lim-

TABLE 12 Access separation distances (ft) on the basis of spillback rate*

Posted Speed (mph)	SPILLBACK RATE**			
	5%	10%	15%	20%
30	335	265 [(a)]	210 [(b)]	175 [(c)]
35	355	265 [(a)]	210 [(b)]	175 [(c)]
40	400	340	305	285
45	450	380	340	315
50	520	425	380	345
55	590	480	420	380

(a) Based on 20 driveways per mile.
(b) Based on 25 driveways per mile.
(c) Based on 30 driveways per mile.

* Based on an average of 30 to 60 right turns per driveway.
** Spillback occurs when a right-lane through vehicle is influenced to or beyond a driveway upstream of the analysis driveway.
The spillback rate represents the percentage of right-lane through vehicles experiencing this occurrence.

ited by the property frontage available; (7) improving or retrofitting minimum corner driveway distances is not always practical, especially in built-up areas.

Adequate corner clearances can best be achieved when they are established before land subdivision and site development approval. Corrective actions include (1) requiring property access from secondary roads, (2) locating driveways at the farthest edge of the property line away from the intersection, (3) consolidating driveways with adjacent properties, and (4) installing a raised median barrier on approaches to intersections to prevent left-turn movements.

MEDIAN ALTERNATIVES (TECHNIQUES 2A, 2B, AND 3C - CHAPTER 6)

The basic choices for designing the roadway median are whether to install a continuous TWLTL or a nontraversable median on an undivided roadway, or to replace a TWLTL with a nontraversable median. These treatments improve traffic safety and operations by removing left turns from through travel lanes. Two-way left-turn lanes provide better access and maximize operational flexibility. Medians physically separate opposing traffic, limit access, clearly define conflicts, and provide better pedestrian refuge; their design requires adequate provision for left- and U-turns to avoid concentrating movements at signalized intersections.

An extensive review of safety and operational experience and models provided guidelines for impact assessment.

Safety

The safety benefits reported in studies conducted since 1970 were as follows:

- Highway facilities with TWLTLs had accident rates that were, overall, roughly 38 percent less than those experienced on undivided facilities (13 studies).
- Highway facilities with nontraversable medians had an overall accident rate of 3.3 per million VMT compared with about 5.6 per million VMT on undivided facilities (10 studies).
- Highway facilities with nontraversable medians had an overall accident rate of 5.2 per million VMT compared with 7.3 per million VMT on facilities with TWLTLs (11 studies).
- The estimated total accidents per mile per year—on the basis of an average of seven accident prediction models—were as shown in Table 13.

Operations

Several operations studies have indicated that removing left-turning vehicles from the through traffic lanes reduces delays whenever the number of through travel lanes

TABLE 13 Estimated total accidents per mile per year on the basis of an average of seven accident prediction models

ADT	Accidents Per Mile Per Year		
	Undivided Highway	Two-Way Left-Turn Lane	Non-traversable Median
10,000	48	39	32
20,000	126	60	55
30,000	190	92	78
40,000	253	112	85

is not reduced. Some 11 operations models developed over the past 15 years confirmed these findings.

Economic Effects

The economic effects of various median alternatives depend on the extent that access is improved, restricted, or denied. The effects on specific establishments also depend on the type of activity involved and on background economic conditions.

Where direct left turns are prohibited, some motorists will change their driving or shopping patterns to continue patronizing specific establishments. Some repetitive pass-by traffic will use well designed or conveniently located U-turn facilities. Effects also will be reduced where direct left-turn access is available. In some cases, retail sales may increase as overall mobility improves.

The maximum effects resulting from median closures can be estimated by multiplying the number of left turns entering an establishment by the proportion of these turns that represents pass-by traffic. Typical proportions of this pass-by traffic are as follows:

- Service Station-Convenience Market—55%
- Small Retail (< 50,000 sq. ft.)—55%
- Fast Food Restaurant with Drive-Through Window—45%
- Shopping Center (250,000–500,000 sq. ft.)—30%
- Shopping Center (Over 500,000 sq. ft.)—20%

Selecting a Median

Selecting a median alternative depends upon factors related to policy, land use, and traffic. These factors include (1) the access management policy for and access class of the roadway under consideration; (2) the types and intensities of the adjacent land use; (3) the supporting street system and the opportunities for rerouting left turns; (4) existing driveway spacings; (5) existing geometric design and traffic control features (e.g., proximity of traffic signals and provisions for left turns); (6) traffic volumes, speeds, and accidents; and (7) costs associated with roadway widening and reconstruction. This report contains a procedure for evaluating and selecting median treatments that was developed for NCHRP Project 3-49.

LEFT-TURN LANES (TECHNIQUE 3A - CHAPTER 7)

The treatment of left turns is a major access management concern. Left turns at driveways and street intersections may be accommodated, prohibited, diverted, or separated depending on specific circumstances.

Safety

A synthesis of safety experience indicates that the removal of left turns from through traffic lanes reduced accident rates by roughly 50 percent (the range was 18 to 77 percent).

Operations

Left turns in shared lanes may block through vehicles. The proportion of through vehicles blocked on approaches to signalized intersections is a function of the number of left turns per traffic signal cycle as shown in Table 14.

TABLE 14 Proportion of through vehicles blocked on approaches to signalized intersections in relation to the number of left turns per traffic signal cycle

Left Turns Per Cycle	Proportion of Through Vehicles Blocked
1	0.25
2	0.40
3	0.60

The capacity of a shared lane might be 40 to 60 percent of that for a through lane under typical urban and suburban conditions. Thus, provision of left-turn lanes along a four-lane arterial would increase the number of effective travel lanes from about 1.5 to 2.0 lanes in each direction—a 33 percent gain in capacity.

Application of the 1994 *Highway Capacity Manual* gives the following illustrative capacities for 2- and 4-lane roads at signalized intersections (see Table 15).

U-TURNS AS ALTERNATIVES TO DIRECT LEFT TURNS (TECHNIQUE 3D - CHAPTER 8)

U-turns reduce conflicts and improve safety. They make it possible to prohibit left-turns from driveway connections onto multi-lane highways and to eliminate traffic signals that would not fit into time-space (progression) patterns along arterial roads. When incorporated into intersection designs, they enable direct left-turns to be rerouted and signal phasing to be simplified.

Safety

U-turns result in a 20 percent accident rate reduction by eliminating direct left-turns from driveways and a 35 percent reduction when the U-turns are signalized. Roadways with wide medians and "directional" U-turn crossovers have roughly one-half of the accident rates of roads with TWLTLs.

Operations

U-turns, coupled with two-phase traffic signal control, result in roughly a 15 to 20 percent gain in capacity over conventional intersections with dual left-turn lanes and multi-phase traffic signal control.

A right turn from a driveway followed by a U-turn can result in less travel time along heavily traveled roads than a direct left-turn exit when there is as much as ½ mi of additional travel.

Indirect U-turns may require a median width of 40 to 60 ft at intersections, depending on the types of vehicles involved. Narrower cross sections may be sufficient when there are few large trucks.

TABLE 15 Capacities of 2- and 4-lane roads at signalized intersections

Condition	Capacity - Vehicles Per Hour Per Approach	
	Two-Lane Road	Four-Lane Road
No Left Turns	840	1,600
Shared Lane (50 to 150 Left Turns/Hour)	425-650	900 - 1,000
Exclusive Left-Turn Lanes	750-960	1,100 - 1,460

ACCESS SEPARATION AT INTERCHANGES (TECHNIQUE 1D - CHAPTER 9)

Freeway interchanges have become focal points of activity and have stimulated much roadside development in their environs. Although access is controlled within the freeway interchange area, there generally is little access control along the interchanging arterial roadways.

Separation distances reported by state agencies ranged from 100 to 700 ft in urban areas and 300 to 1,000 ft in rural areas. Case studies reported separation distances of 120 to 1,050 ft. These distances are usually less than the access spacing needed to ensure good traffic signal progression and to provide adequate weaving and storage for left turns.

Desired access separation distances for free-flowing right turns from exit ramps should include the following components:

- Perception-Reaction Distance (100–150 ft)
- Lane Transition (150–250 ft)
- Left-Turn Storage (50 ft per left-turn per cycle)
- Weaving Distance (800 ft, 2-lane arterials; 1200 ft, 4-lane arterials; 1600 ft, 6-lane arterials)
- Distance to Centerline of Cross Street (40–50 ft)

FRONTAGE ROADS (TECHNIQUES 6A AND 6B - CHAPTER 10)

Frontage roads reduce the frequency and severity of conflicts along the main travel lanes and permit direct access to abutting property. Along freeways and expressways, they can be integrated with interchange and ramping systems to alleviate congestion and to improve access. Frontage roads along arterials should be carefully designed to avoid increasing conflicts at intersections. Reverse frontage or “backage” roads with developments along each side may be desirable in developing areas. In all cases, arterial frontage roads must be carefully designed and located to protect arterial and crossroad operations.

POLICY CONSIDERATIONS (CHAPTER 11)

Access management requires both retrofit and policy actions. Access separation distances should be established as part of statewide access management programs, corridor retrofit plans, and community zoning ordinances. Advance purchase of right-of-way and/or access rights is desirable.

The basic policy issues are as follows:

- Comprehensive access management codes should indicate where access is allowed or denied for various classes of roads, specify allowable spacings for signalized and unsignalized connections, and set forth permit procedures and requirements. Codes may define or limit the application of specific techniques and establish procedures for an administering agency to use in removing access.
- A network of supporting local and collector streets that should provide sufficient direct access to adjacent developments. These secondary streets should connect to arterial streets at appropriate and well-spaced locations. Such streets make it possible to minimize direct property access on major arterials.
- Access should be provided from strategic and primary arterials only when reasonable access cannot be provided from other roadways. In such cases, access should be limited to right turns wherever possible.
- Left-turn and cross egress should be well separated and placed at locations that fit into overall signal coordination patterns with high efficiency.

Sound land use and development planning is essential to permit effective arterial traffic flow and to allow attractive property access. Access spacing standards (including corner clearance requirements) should be established in advance of actual development. Zoning, subdivision, and access spacing requirements should be consistent.

Better coordination of land use, interchange geometry, and arterial streets is essential to avoid “double loading” arterials and to minimize weaving movements and traffic congestion. Strategically placed frontage roads may be integral to this effort. Equally important is developing a suitable supporting street system.

Raised medians are more effective than painted channelization from an access management perspective. Median width and opening policies are essential design elements. Wide medians that allow indirect U-turns in lieu of direct left turns should be considered for new arterials where space permits, because the medians improve safety and simplify intersection operations and signal timing and coordination.

Any access control or management plan must be done systemwide to avoid transferring problems to upstream or downstream intersections.

Several research needs emerged. These include (1) enhancing the safety database, (2) assessing the effects of median closures—including upstream and downstream effects, and (3) obtaining more information on driver selection of roadside businesses on the basis of accessibility considerations.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

RESEARCH PROBLEM STATEMENT

Streets and highways constitute a valuable resource and a major public investment. It is essential to operate them safely and efficiently by managing the access to and from abutting properties. Owners have a right of reasonable access to the general system of streets and highways. Roadway users have the right to freedom of movement, safety, and efficient expenditure of public funds. The need to balance these competing rights is especially acute where significant changes in land development have occurred or are envisioned to occur. The safe and efficient operation of the highway system calls for effectively managing the access to adjacent developments.

Access management provides (or manages) access to land development while simultaneously preserving the flow of traffic on the surrounding road network in terms of safety, capacity, and speed (*1*). Access management benefits the transportation system by preserving capacity, maintaining mobility, and improving safety. These benefits have been recognized at all levels of government. Three states—Colorado, Florida, and New Jersey—have implemented comprehensive statewide access codes. Some states, including Michigan, Minnesota, Montana, Ohio, and Oregon, are reviewing their statewide practices and/or developing access codes. Other states are upgrading their access design criteria. Several counties and cities (e.g., Lee County, Florida, and Lakewood County, Colorado) have patterned their codes on the statewide codes. A growing number of cities, counties, and planning regions are managing property access by closing, consolidating, or improving driveways.

Over the years, many techniques have evolved for improving highway access. An initial "Evaluation of Techniques for the Control of Direct Access to Arterial Highways" was prepared by the Midwest Research Institute in 1975 (*2*). The 1982 FHWA report, "Access Management for Streets and Highways" (*3*) contained access management guidelines that incorporated and updated the various techniques set forth in the 1975 study. *NCHRP Report 348*, published in 1992, contained policy, planning, and design guidelines for developing access management and programs; a follow-up study described the selected case studies (*1*). A 1993, FHWA-sponsored study, "Guidelines for Providing Access to Transportation Systems," shows how specific techniques might be analyzed (*4*).

These documents contain important information on the various access management methods and techniques. However, much of the information they contain is too dated or limited for analyzing and quantifying the effects of access management techniques.

Transportation agencies and private developers continue to seek better methods to evaluate the benefits and effects of various access management techniques. Three reasons, in particular, underscore the need for better methods of application and analysis of the many access management techniques cited in earlier documents:

- The emergence of comprehensive access management codes provides a context for access management decisions and controls and the applications of specific techniques.
- New analytical tools and techniques (*5*) provide updated parameters and procedures for assessing effects.
- Travel time, safety, and economic benefits generally reflect information collected in past decades. A new data base that reflects recent research and conditions, insofar as possible, is needed.

RESEARCH OBJECTIVE AND SCOPE

The research objective—as defined in the project statement—"is to develop methods of predicting and analyzing the traffic-operation and safety impacts of selected access-management techniques for different land use, roadway variables, and traffic volumes. The methods to be developed are for use by state departments of transportation, city and county traffic departments, transportation-planning agencies, and private developers."

The research involved a two-phase approach to achieve these objectives and to produce practical guidelines for the application, analysis, and selection of various access management techniques. The first phase identified the various techniques available; showed how they can be classified in terms of functional objectives, roadway elements, and their likely effects; and suggested priority techniques for further analysis. Likely effects were extracted on the basis of a literature review, the research team's experience, and selected agency review; and the need for further data collection was identified. First-phase efforts concluded with the design of

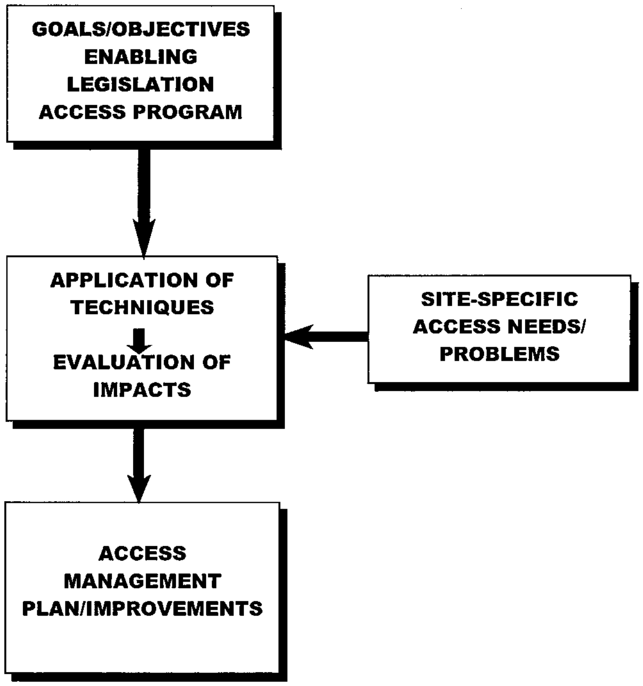


Figure 1. Study context.

data collection plans that addressed the data voids for the more important techniques.

The second phase involved the compilation, collection, and analysis of additional data from both primary and secondary sources. Methods for predicting the safety, operations, and economic effects associated with the more important techniques were developed. Technical memoranda were prepared regarding these techniques, and this final user-oriented report was prepared in order to establish procedures for an administering agency to use in controlling access.

The various techniques—their effects and benefits are important in developing site-specific access solutions and in developing broader corridor or areawide access management plans. Figure 1 shows the study context and shows how the specific access management techniques relate to access management programs and to overall access management plans.

A comprehensive access management code determines when access is provided or denied for various access classes of roads, specifies the allowable spacings for signalized and unsignalized connections, and sets forth access permit procedures and requirements. The code may define or limit the applicability of specific techniques and establish procedures for an administering agency to use in controlling access.

Many access management techniques deal with a single location or site (e.g., closing a median at a driveway). Some techniques may transfer problems to other locations downstream or upstream of the location under consideration. In such cases, broader analyses of effects and benefits will be required. The research focused on roadway and traffic improvement techniques. Although good pedestrian, bicycle, and transit access are essential to developments and should be provided where appropriate and incorporated into site plans, they were beyond the scope of this research.

RESEARCH APPROACH

The work program involved the eight study tasks shown in Figure 2. The first study phase (Tasks 1 through 5) related techniques to effects, identified voids in available research, and prepared study designs for needed data collection. The second study phase involved collecting and analyzing the field data, developing impact analysis parameters and techniques, and producing this project report. A brief description of each work task in Phase I follows:

- **Task 1—Review Techniques and Recommend Classification Scheme.** Access management techniques that are in use or described in the literature were identified. More than 100 access management techniques were identified. A classification system for organizing the techniques was developed on the basis of practicality and usefulness.
- **Task 2—Identify and Stratify Effects by Relevant Variables.** This task defined and grouped the various access management effects, identified the relevant variables (e.g., roadway cross section and development type), and related access management techniques to

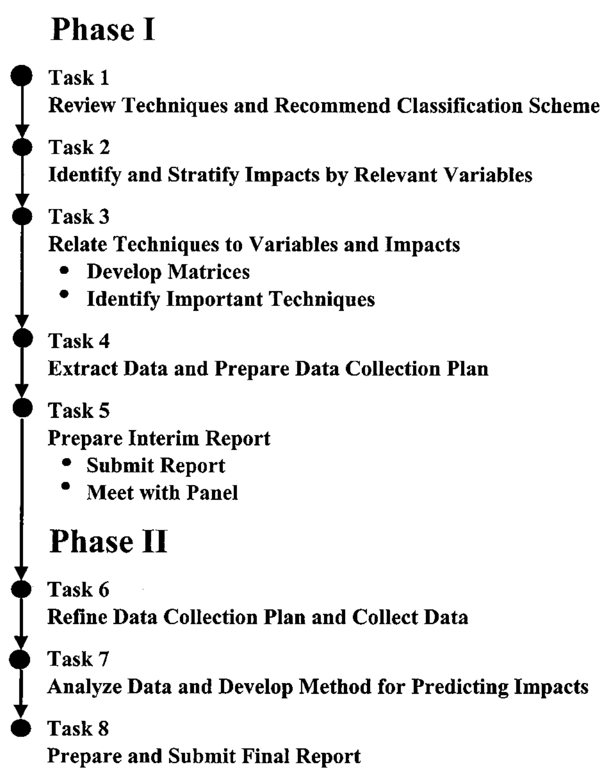


Figure 2. Study tasks.

these variables and effects. In addition, performance measures were identified for quantifying the relationship between each technique and the relevant variables and effects.

- **Task 3—Relate Techniques to Variables and Effects.** This task selected candidate access management techniques and their associated effects for further study and analysis. The list of more than 100 techniques developed in Task 1 was analyzed to identify those techniques considered to be more important on the basis of potential application and effectiveness. Approximately 25 techniques were found to be more important because they were applicable to a significant portion of the roadway system and they were shown to be effective in improving safety, reducing emissions, and/or improving traffic operations.
- **Task 4—Extract Data and Prepare Data Collection Plan.** Each of the 25 priority techniques was assessed in terms of data availability, ability to measure effects, and suitability for analysis.
- **Task 5—Present Interim Report.** The study approach for each technique, including experimental design, was presented in an interim report for review and refinement.

Phase I concluded with the decision to focus Phase II on the following priority techniques:

- Establishing spacing for unsignalized access;
- Establishing criteria for median treatments, including
  —Installing a physical median on an undivided highway,
  —Replacing a TWLTL with a physical median, and
  —Installing a continuous TWLTL where none exists;
- Establishing access separation distances at interchanges;
- Establishing corner clearance criteria; and
- Providing U-turns as an alternative to direct left turns from a driveway.

As work progressed, two more techniques were added to the Phase II effort to provide a more complete assessment of access management effects:

- Installing left-turn lanes and
- Providing frontage roads.

Phase II (Tasks 6, 7, and 8) was initiated by conducting a survey of state transportation and other agencies to obtain information on current practices and policies for the priority techniques, as well as on existing data on the effects of access management techniques.

This phase involved the development of methods to identify the effects of the priority access management techniques. Efforts were focused on selected effects that are important and measurable. The results of the literature search and the agency survey performed of state and other agencies were used wherever possible. Compilation of data from secondary sources and selective field data collection were performed to help quantify effects. A major goal was to assess how traffic performance and safety changed with different traffic conditions, roadway geometry, and environmental factors. However, as the research progressed, it became apparent that the effects of several techniques could not be quantified. Accordingly, the research approach for these techniques focused on identifying desirable and undesirable practices and on suggesting concepts that might be applied. Case studies were developed to help identify good and poor practices.

The products of Phase II included technical memoranda and this report.

REPORT ORGANIZATION

The chapters that follow describe access management techniques and define their effects:

- Chapter 2 presents the results of the Phase I effort pertaining to classifying techniques and identifying effects.
- Chapters 3 through 10 present the research findings and application guidelines for specific access management techniques.
- Chapter 11 presents conclusions and recommendations.

CHAPTER 2

ACCESS MANAGEMENT TECHNIQUES AND IMPACTS

TYPES OF ACCESS MANAGEMENT TECHNIQUES

Access management techniques and classification systems have evolved over a 25-year period. The early classification systems, developed by Stover and Glennon, were based on techniques relating to highways and driveways (*6,7*). This system was expanded in 1993 to include management elements (*8*). The 1982 FHWA report on access management, in contrast, classified techniques by functional objective (*9*). *NCHRP Report 348* in 1992 described various policy and design approaches, but did not develop a specific classification system (*10*).

In developing a classification system, it is important to consider both the strategic and tactical decisions involved in developing access to abutting properties. As shown in Figure 3, the "strategic analysis" involves the basic site access decisions that relate to the location and number of access points. The "tactical analysis" deals with the specific design of access roadways and treatments to help ensure safe and efficient operations.

Figure 4 shows how strategy and tactics relate within an access management context. Thus, the "strategy" covers access codes and design standards that, in turn, influence the provision and spacing of access. The "tactics" encompass the specific design and operational techniques. Both sets of decisions influence the choice of techniques for any specific situation. Thus, a classification system must clearly differentiate between policy (strategic) and design/operation (tactical) treatments. This differentiation becomes even more important as the number of states, counties, and other jurisdictions with access management codes increases. The classification system should also apply to treatments for both new developments and retrofit situations.

CLASSIFICATION OF TECHNIQUES

Classification systems reviewed include those developed in previous access-related documents prepared by Stover and Glennon (1970, 1975), Flora (1982), Bellomo (1993), and Koepke/Levinson (1992); each is described briefly below. This section also identifies the recommended classification system developed as part of this project effort.

Prior Classification Systems

Systems previously used for classification are as follows:

- **Stover and Glennon.** The initial classification system used by Stover (1970) and Glennon (1975) classified some 70 techniques according to the following:
  —Highway design and operation,
  —Driveway location, and
  —Driveway design and operation.
- **FHWA-Flora.** The 1982 system by Flora grouped some 65 techniques according to functional objective as follows:
  —Limit number of conflict points,
  —Separate basic conflict areas,
  —Limit deceleration requirements, and
  —Remove turning vehicles from the through lanes.
- **FHWA-Bellomo.** The 1993 scheme by Bellomo grouped techniques as follows:
  —Management elements,
  —Facility design elements,
  —Access driveway/design elements, and
  —Traffic control elements.
- ***NCHRP Report 348.*** This 1992 NCHRP report focused on concepts rather than specific techniques per se. The report described concepts in the following categories:
  —Interchanges,
  —Frontage roads,
  —Medians,
  —Left turns,
  —Right turns, and
  —Driveway arrangements.

Recommended Classification System Developed for this Project

Several additional systems were developed for purposes of this project. Each classification scheme was analyzed in terms of the following basic factors:

- Clarity—the scheme must be clear to users. It should clearly differentiate policy versus design so that practi-

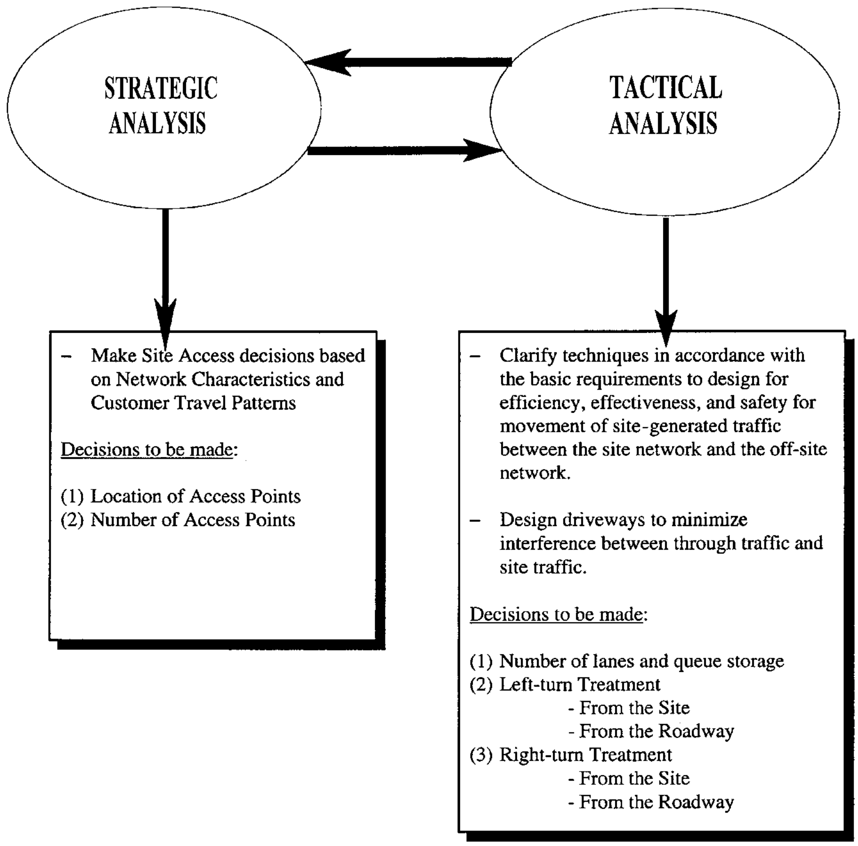


Figure 3. Strategic and tactical decisions in access management.

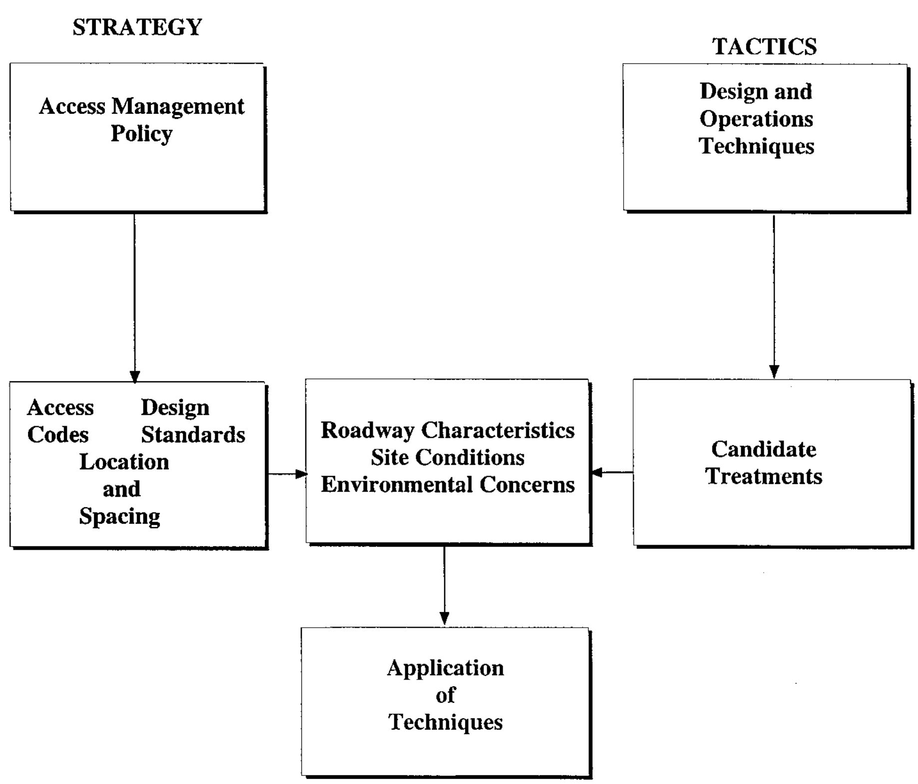


Figure 4. Suggested context for classification/application of improvement techniques.

tioners will focus on those areas within the realm of their responsibility.

- User-Friendly—the scheme must be easy to understand.
- Practicality—the scheme must be easy to apply.
- Manageability—the scheme must contain a reasonable number of classes.
- Comprehensiveness—the scheme must be able to include the various techniques that have been or might be identified.

On the basis of the assessment of the various classification systems according to the five factors, a preferred one was identified. The recommended classification system is shown in Table 16. Appendix A groups individual access management techniques according to this system.

This system covers both policy and design techniques, with each forming a major classification group. The system links techniques to the type of improvements normally applied along highways and access driveways. This system is relatively simple to use and understand, covers virtually all improvements, and provides a reasonable distribution of the various techniques among the various categories. It incorporates medians and left turns into one group and further subdivides driveway location techniques by consolidation, reorientation, and relocation.

TECHNIQUES SELECTED FOR FURTHER ANALYSIS

About twenty-five candidate techniques were identified as important and promising. This short list included techniques that cover much of the roadway system, are effective in improving safety and/or reducing delay and emissions, and may be amenable to analysis. These techniques are frequently encountered in key access management decisions.

TABLE 16 Recommended classification system for access management techniques

I.	**Policy—Management**	
	a.	Access Codes/Spacing
	b.	Zoning/Subdivision Regulations
	c.	Purchase of Access Rights
	d.	Establishment of Setbacks from Interchanges and Intersections
II.	**Design - Operations (By Roadway Features)**	
	a.	Interchanges
	b.	Frontage Roads
	c.	Medians—Left Turns
	d.	Right Turns
	e.	Access/Driveway Location (Mainly Retrofit—Consolidation, Reorientation, Relocation)
	f.	Traffic Controls
	g.	Access/Driveway Design

Policy techniques, such as establishing an access management code, modernizing zoning requirements, and acquiring rights-of-way, are extremely important and provide a basic framework for other techniques. However, because of their broad nature, they do not lend themselves to measurement or quantification. Therefore, they were screened from further analysis. However, "design-related" policy techniques that relate to access spacing were included. Other techniques relate to physical design and/or traffic operations.

The priority access management techniques are as follows:

- 1a Establish Traffic Signal Spacing Criteria,
- 1b Establish Spacing for Unsignalized Access,
- 1c Establish Corner Clearance Criteria,
- 1d Establish Access Separation Distances at Interchanges,
- 2a Install Nontraversable Median on Undivided Highway,
- 2b Replace TWLTL with Nontraversable Median,
- 2c Close Existing Median Openings,
- 2d Replace Full Median Opening with Median Designed for Left Turns from the Major Roadway,
- 3a Install Left-Turn Deceleration Lanes Where None Exists,
- 3b Install Left-Turn Acceleration Lane,
- 3c Install Continuous TWLTL on Undivided Highway,
- 3d Install U Turns as an Alternative to Direct Left Turns,
- 3e Install Jug Handle and Eliminate Left Turns Along Highways,
- 4a Install Right-Turn Acceleration/Deceleration Lane,
- 4b Install Continuous Right-Turn Lane,
- 5a Consolidate Driveways,
- 5b Channelize Driveways to Discourage or Prohibit Left Turns on Undivided Highways,
- 5c Install Barrier to Prevent Uncontrolled Access Along Property Frontage,
- 5d Coordinate Driveways on Opposite Sides of Street,
- 6a Install Frontage Road to Provide Access to Individual Parcels, and
- 6b Locate/Relocate the Intersection of a Parallel Frontage Road and a Cross Road Further from the Arterial-Cross Road Intersection.

Table 17 provides a generalized assessment of each technique in terms of its perceived importance to access management, the availability of secondary sources, the technique's amenability to analysis, and its priority for inclusion in the Phase II efforts.

The high-priority techniques identified by the research team in conjunction with the project panel for subsequent analysis were as follows:

TABLE 17 Summary of significant access management techniques

Technique	Importance in Access Management	Previous Sources	Amenable to Analysis	Analysis in Phase II
A - Policy Techniques				
1 Establish Comprehensive Access Code	High	-	No	No
2 Institutionalize Advance Purchase of Right-of-Way	High	-	No	No
3 Require Internal Circulation/Site Plan Review	High	-	No	No
B - Design Techniques				
1a Establish Traffic Signal Spacing Criteria	High	Some	Yes	Yes
1b Establish Spacing for Unsignalized Access	High	Few	Yes	Yes
1c Establish Corner Clearance Criteria	High	Few for Upstream	Yes	Yes
1d Establish Access Separation Distances at Interchanges	High	-	Yes	Yes
2a Install Nontraversable Median on Undivided Highway	High	Many	Yes	Yes
2b Replace Two-Way Left-Turn Lane With Nontraversable Median	High	Many	Yes	Yes
2c Close Existing Median Openings	High	Some	No	No
2d Replace Full Median Opening With Median Designed for Left-Turns from the Major Roadway	High	Few	Yes	No
3a Install Left-Turn Deceleration Lanes where None Exists	High	Some	Yes	Yes
3b Install Left-Turn Acceleration Lane	Low	Few	Yes	No
3c Install Continuous Two-Way Left-Turn Lane on Undivided Highway	Medium	Many	Yes	Yes
3d Install U-Turns as an Alternative to Direct Left Turns	Medium-High	Few	Yes (Oper.)	Yes
3e Install Jug Handle and Eliminate Left Turns Along Highways	Medium	Few	Yes (Oper.)	Yes
4a Install Right-Turn Acceleration/Deceleration Lane	Medium	-	Yes (Oper.)	No
4b Install Continuous Right-Turn Lane	Low	-	Yes	No
5a Consolidate Driveways	Medium	-	Yes	No
5b Channelize Driveways to Discourage or Prohibit Left Turns on Undivided Highways	High	-	Yes	No
5c Install Barrier to Prevent Uncontrolled Access Along Property Frontage	Medium	-	Yes	No
5d Coordinate Driveways on Opposite Sides of Street	Low-Medium	-	Site-specific	No
6a Install Frontage Road to Provide Access to Individual Parcels	Medium	-	Yes	Yes
6b Locate/Relocate the Intersection of a Parallel Frontage Road and a Crossroad Further From the Arterial - Crossroad Intersection	Medium	-	Yes	Yes

TABLE 18 Format for stratification of impacts by roadway, environmental, and traffic variables

(Technique)

Variable	Key Impacts					
	Traffic Operations			Safety	Economic	Environmental
	Travel Time	Capacity	Other Traffic			
1. Environment (Urban, Suburban, Rural)						
2. Development Type (Commercial, Other)						
3. Cross Section (2 Lanes, 4 Lanes Undivided; 4+ Lanes Non-Restrictive Median; 4+ Lanes with Restrictive Median)						
4. Median Openings Per Mile (Low, Medium, High)						
5. Signals Per Mile (Low, Medium, High)						
6. Connections Per Mile (Low, Medium, High)						
7. Traffic Volumes i.e., ADT/Lane (Low, Medium, High)						
8. Driveway Volumes (Low, Medium, High)						
9. Posted Speeds (Low, Medium, High)						

- Spacing for Unsignalized Access (Technique 1b);
- Establish Corner Clearance Criteria (Technique 1c);
- Establish Access Separation Distances at Interchanges (Technique 1d);
- Integrated Median Techniques, including (Techniques 2a, 2b, 3c)
  —Install Nontraversable Median on Undivided Highway,
  —Replace TWLTL With Nontraversable Median, and
  —Install Continuous TWLTL on Undivided Highway;
- Install U-Turns as an Alternative to Direct Left Turns (Technique 3d).

In addition, several other techniques were included to provide a more complete picture. These were Technique 1a (Establish Traffic Signal Spacing Criteria), Technique 3a (Install Left-Turn Deceleration Lanes Where None Exists), and Techniques 6a and 6b pertaining to frontage roads. Technique 3e (Install Jug Handle and Eliminate Left Turns Along Highways) was included in Technique 3d (Install U-Turns as an Alternative to Direct Left Turns).

These priority techniques cover key aspects of access management. They include access spacing and median treatments and encompass both the transverse and longitudinal roadway elements.

IMPACTS OF ACCESS MANAGEMENT TECHNIQUES

Potential impacts were identified and grouped into four broad categories: traffic operations, traffic safety, environmental, and economic (including transportation service and

TABLE 19 Stratification of impacts by roadway, environmental, and traffic variables for Technique 2a

<table>
<tr><th rowspan="3">Variable</th><th colspan="6">Key Impacts</th></tr>
<tr><th colspan="3">Traffic Operations</th><th rowspan="2">Safety (P)</th><th rowspan="2">Economic (S)</th><th rowspan="2">Environ-mental (S)</th></tr>
<tr><th>Travel Time (P)</th><th>Capacity (P)</th><th>Vehicle Miles Traveled (D)</th></tr>
<tr><td>Development Type</td><td></td><td></td><td></td><td></td><td>√</td><td></td></tr>
<tr><td>Cross Section</td><td>√</td><td>√</td><td></td><td>√</td><td></td><td></td></tr>
<tr><td>Signals Per Mile</td><td>√</td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Connections Per Mile</td><td>√</td><td>√</td><td></td><td>√</td><td>√</td><td></td></tr>
<tr><td>Traffic Volumes i.e., ADT/Lane</td><td>√</td><td>√</td><td>√</td><td>√</td><td>√</td><td>√</td></tr>
<tr><td>Driveway Volumes</td><td>√</td><td>√</td><td>√</td><td>√</td><td>√</td><td>√</td></tr>
<tr><td>Speeds</td><td>√</td><td>√</td><td></td><td>√</td><td></td><td>√</td></tr>
<tr><td>Extent of Rerouting Required</td><td>√</td><td></td><td>√</td><td></td><td>√</td><td>√</td></tr>
<tr><td>Performance Measure(s)</td><td>Average travel rate.</td><td>Vph (by direction) at signalized intersections. v/c ratio from HCM analyses.</td><td>Derived from traffic volumes.</td><td>Accidents, acc. rates by types.</td><td>Changes in number of establish-ments and changes in retail sales.</td><td></td></tr>
</table>

land use). In reviewing these groups, it became apparent that many impacts are interrelated. This was particularly true for environmental impacts (e.g. emissions) that largely depend on the volume and speed of travel. Therefore, subsequent analysis of specific techniques focused mainly on traffic operational and safety impacts. However, economic impacts were considered for some techniques where those impacts are key considerations.

Relevant roadway and traffic variables included area type (e.g. urban and rural), development type (e.g., residential and commercial), roadway cross section, highway volumes, driveway volumes, traffic signal frequency, median opening frequency, driveway connections per mile, and speed. Ranges in these variables were identified. The relevant impacts and their associated variables were explicitly identified for each of these techniques (*11*). Table 18 shows the general "template" that was used in relating each technique to impacts and variables. Table 19 shows an example of the completed template for the installation of nontraversable median barriers. It identifies the primary, secondary, and derived impacts and the key performance measures. Similar analytical frameworks were derived for other techniques and are contained in the interim report (*11*). They also provided an initial context for the Phase II effort.

The Phase II impact analyses reflected the following objectives:

- Concentration on selected impacts that are important and measurable,
- Use of available literature and research whenever possible, drawing on and synthesizing several decades of research for several techniques,
- Collection of selective field data to help quantify impacts (The field investigations focused on analyzing the impacts of right turns into driveways on arterial traffic performance), and
- Performance of case studies to help identify benefits and, in certain cases, the disadvantages of particular techniques (Case studies of desirable and undesirable practice were obtained for corner clearances and for access spacing at interchanges).

CHAPTER 3

TRAFFIC SIGNAL SPACING (TECHNIQUE 1A)

INTRODUCTION

The spacing of traffic signals—in terms of frequency and uniformity—governs the performance of urban and suburban highways. Signals account for most of the delay that motorists experience: they constrain capacity during peak travel periods with attendant queuing and spillback; they delay vehicles during both peak and off-peak periods wherever they are randomly located, ineffectively coordinated, or improperly timed; and closely and/or irregularly spaced signals can reduce arterial travel speeds thereby resulting in an excessive number of stops even under moderate traffic volume conditions (Figure 5). They can also increase accidents.

Establishing traffic signal spacing criteria for arterial roadways is one of the most important and basic access management techniques. This is why New Jersey requires a minimum through band of 50 percent of the signal cycle and why Colorado and Florida require ½-mi signal spacing along principal arterial roads. It is also why Colorado requires a minimum bandwidth where any signal location deviates from the uniform ½-mi interval. (The through bandwidth measures how large a platoon of vehicles can pass through a series of signals without stopping for a red traffic light. It may be expressed in terms of the number of seconds per cycle or the percent of cycle length that the traffic could flow within a platoon.)

This chapter presents the safety and travel time impacts associated with traffic signal spacing. It summarizes reported accident experience, shows how time-space patterns and through bandwidths are impacted by signal spacing and location, and identifies the basic planning considerations. It quantifies the impacts of traffic signal densities and traffic volumes on travel speeds. Finally, it presents guidelines for application and gives examples of their use.

SAFETY

Several studies have evaluated the impacts of traffic signal spacing on safety. Studies conducted on Oregon state highways in the 1950s found the number of accidents increased as the number of driveways, intersections, and traffic signals per mile increased (*12*). The results of multiple linear regression indicated that the number of signalized intersections per mile was perhaps the largest contributor to accidents. Studies by Cribbins in the 1960s also found that the total accident and injury accident rates increased as the number of intersections per mile increased (*13*). The relative importance of variables as predictors used to estimate accidents per mile was as follows:

Type of Intersection	*Relative Importance*
Total number of intersections per mile without left-turn storage	3.29
Number of signalized intersections per mile without left-turn storage	3.00
Total number of intersections per mile with left-turn storage	2.71
Number of signalized intersections per mile with left-turn storage	2.70

Studies by Squires and Parsonson in Georgia in 1989 found that accident rates generally increased as the number of signals per mile increased (*14*). The relative increases in accident rates were about 40 percent when traffic signal density increased from two to four signals per mile. However, the rates displayed some scatter and varied by roadway width and type of median.

The effects of traffic signal densities on accident rates in Lee County, Florida (1993) are shown in Figure 6 (*15*). A doubling of signals from two to four per mile increased the accident rate by roughly 2.5 times.

The safety impacts of increased traffic signal spacing are obscured in part by the traffic volumes on intersecting roadways and the common use of vehicle-miles of travel (VMT) for comparing accident rates rather than the accidents per million entering vehicles or the product of conflicting volumes.

TIME-SPACE ANALYSIS AND IMPACTS

Time-space analysis clearly indicates the desirability of long and uniform signal spacings in achieving efficient traffic signal progression at desired travel speeds. The effects of signal cycle length and spacing on progressive speeds in both directions of travel have been well established. Speeds increase directly as signal spacing increases and inversely with cycle length. The longer the spacing between signals,

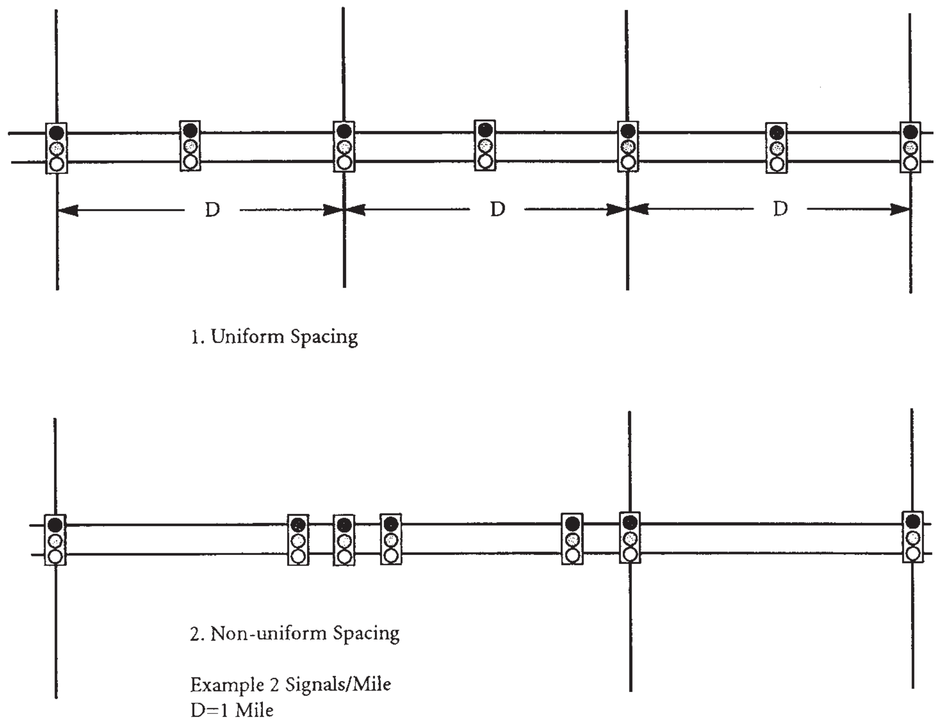
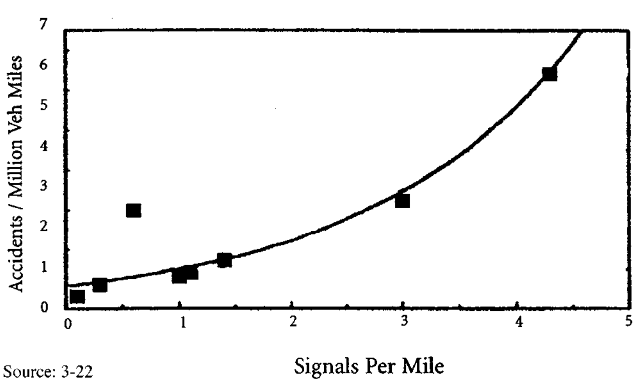


Figure 5. Technique 1a. Establish traffic signal spacing criteria.

the faster the speeds for any given cycle length. Similarly, for any given block spacing, the shorter the cycle length, the higher the speeds.

Signal Coordination Concepts

Successive signals along a roadway may turn green at the same time (a "simultaneous" system) or their green times may alternate (an "alternating" system). In a simultaneous system, all signals along a given street operate with the same cycle length and display the green indication at the same time. In an alternating system, each successive signal or group of signals shows opposite (or alternating) indications to that of the next signal or group. Either system may allow full "through bands" at a desired travel speed in both directions of travel. However, when signals are too closely or too irregularly spaced, multiple alternate patterns are typically provided. These result in a loss of through band efficiency and/or cross street green time. Signals also may be set to favor one direction of travel—but this usually reduces the through band in the other direction of travel.



Source: 3-22

Figure 6. Signal spacing and crashes: US 41 – Lee County, Florida.

Basic Relationships

The formulas for determining speeds in relation to cycle lengths and signal spacing have been long established. They are based on the dynamics of vehicle motion and assume progressive flow in each travel direction. The formula for coordinated simultaneous and alternating traffic signal patterns is as follows:

$$V = \frac{0.681S}{C} \text{ for simultaneous signals} \quad (1)$$

and

$$V = \frac{1.362S}{C} \text{ for alternating signals} \quad (2)$$

where:

S = signal spacing in feet
C = cycle length in seconds
V = speed in mph

In metric units these formulas become

$$V' = \frac{3.6m}{C} \text{ for simultaneous signals} \tag{3}$$

$$V' = \frac{7.2m}{C} \text{ for alternating signals} \tag{4}$$

where:

m = spacing in meters
C = cycle length in seconds
V' = speed in km/h

Thus, the optimum spacing of signals depends upon the cycle length and travel speed. Long cycle lengths combined with high speeds require long distances between signals. Shorter cycle lengths and lower speeds allow closer spacing between signals. Table 20 shows the optimum signal spacing as a function of speed and cycle length assuming an alternating pattern of successive signals. Figure 7 shows the speed-cycle-length relationships for ½-, ⅓-, and ¼-mi signal spacings (i.e., two, three, and four uniformly spaced signals per mile, respectively). (A simultaneous pattern of successive signals would result in half of the speeds.) Table 21 shows the travel speeds for ½-mi signal spacing at various cycle lengths.

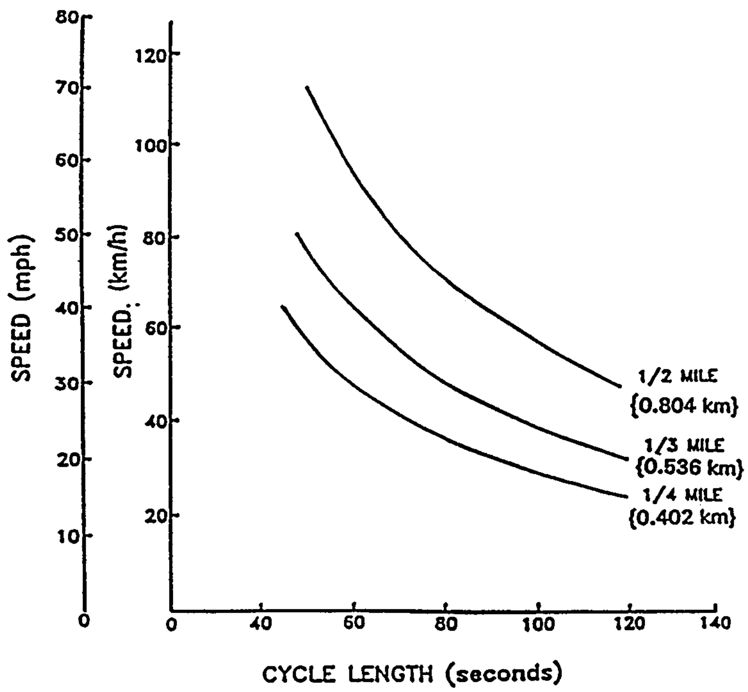
The speed "impacts" of the various spacings can be summarized as follows:

- Spacings that are less than ¼ mi (about 400 m)—i.e., more than four signals per mile—result in progressive speeds that are too low for urban conditions (except perhaps for central business districts).
- Signals spaced at about ¼ mi (about 400 m) can provide progressive speeds from 26 to 30 mph at cycle lengths from 60 to 70 sec. These speeds and cycle lengths are acceptable in cities where traffic volumes are spread over several streets, where two-phase signal operations dominate, and posted speeds are 35 mph or less.
- Longer signal spacings are necessary along many suburban highways where both traffic volumes and speeds increase. Longer cycle lengths are commonly used to increase capacity and provide protected phases for left turns. Cycle lengths of 80 to 120 sec are common, especially during peak periods and require ½-mile signal spacings (about 800 m)—i.e., two signals per mile—to maintain progressive speeds of up to 45 mph.
- Cycle lengths that exceed 120 sec result in progressive speeds less than 25 mph even with ½-mi spacings between signals and, therefore, should be avoided. Moreover, when green times exceed 50 sec, there is

TABLE 20 Optimum signal spacing as a function of speed and cycle length (alternating signals)

Cycle Length (seconds)	Speed, mph						
	25	30	35	40	45	50	55
	Spacing in Feet						
60	1,100	1,320	1,540	1,760	1,980	2,200	2,420
70	1,280	1,540	1,800	2,060	2,310	2,590	2,830
80	1,470	1,760	2,060	2,350	2,640	2,940	3,230
90	1,650	1,980	2,310	2,640	2,970	3,300	3,630
100	1,840	2,200	2,570	2,940	3,300	3,670	4,040
110	2,020	2,420	2,830	3,230	3,630	4,040	4,440
120	2,200	2,640	3,080	3,520	3,960	4,400	4,840
Cycle Length (seconds)	**Speed, km/h**						
	40	**48**	**56**	**64**	**72**	**80**	**88**
	Spacing in Meters						
60	330	400	470	530	600	670	730
70	390	470	540	620	700	780	860
80	440	530	620	710	800	890	980
90	500	600	700	800	900	1,000	1,100
100	560	670	780	890	1,000	1,110	1,220
110	610	730	860	980	1,100	1,220	1,340
120	670	800	930	1,070	1,200	1,330	1,470

Notes: Results rounded.
Source: Computed.



Source: 3-2, 3-3, & 3-5

Figure 7. Relationship between speed, cycle length, and signal spacing (156, 157, 158)

about a 10 percent decline in saturation flows because some drivers become less attentive and do not start moving immediately after the preceding vehicle (*16*).

The "progression efficiency" is measured by the through bandwidth as a proportion or percent of the total signal cycle.

TABLE 21 Progressive speeds for various cycle lengths with ½-mi (uniform) traffic signal spacing

Cycle Length (seconds)	Speed	
	mph	km/h
60	30[a]/60	48[a]/96
65	28[a]/56	45[a]/90
70	26[a]/52	42[a]/84
75	24[a]/48	39[a]/78
80	45	72
90	40	64
100	36	58
110	33	53
120	30	48
a - Progressive speed with simultaneous pattern		

Source: Computed.

It increases slightly as the cycle length increases because there are fewer phase changes and less lost time. Longer cycles also allow greater efficiencies when a fixed time per cycle is allocated to left-turn phases. However, as noted above, there are drawbacks to cycle lengths that exceed 120 sec.

Uniform or near uniform spacing of signals is essential. Uniform spacing, with signals placed at optimum locations from a time-space perspective, allows through bands that are equal to the artery green time. As signals are placed away from the optimum locations, there is a corresponding reduction in the through bandwidth—the time during which progression is maintained.

An analysis of the delays resulting from reducing the through band is summarized in Table 22 (*17*). These delays were estimated on the basis of a 30-mph progressive speed, an unimpeded arrival by the first vehicle in the platoon, and 2.1-sec arrival and departure headways. Delays result whenever the approach volume exceeds the number of vehicles that can be accommodated in the through band. The volume-to-through band capacity ratio is more significant than the actual *v/c* ratio in influencing delays. For example, a volume of nine vehicles per cycle would result in a 12-sec delay when the capacity is six, while a volume of six vehicles per cycle would result in a 17-sec delay when the through band is three vehicles per cycle. Thus, the data underscore the need for preserving the through band, because its reduction would increase delays even at moderate traffic volumes.

TABLE 22 Illustrative delays when traffic demand exceeds bandwidth capacity

Volume Veh/Cycle/Lane	Volume-to-Capacity Ratio	Capacity of Through Band Veh/Cycle/Lane			
		3	6	9	12
3	.25	0	0	0	0
6	.50	17	0	0	0
9	.75	23	12	0	0
12 [1]	1.00	26	17	9	0

1. Capacity assumed at 12 vehicles per cycle based on 29 sec green per 60 sec cycle. First vehicle arrives unimpeded. 2.1 arrival and departure headways. Base progressive speed 30 mph.

Source: *(17)*

Planning Implications

The planning, design, and operation of traffic signals along arterial streets and roadways must achieve a balance between capacity and progression requirements. The key variables include cycle length, signal spacing, travel speeds, and progression efficiency. Key issues to consider are as follows:

1. Long, uniform spacings of traffic signals are desirable to allow effective progression of traffic in both directions of travel. During off-peak periods, arterial roadways should operate at speeds of 25 to 35 mph in urban environments and 35 to 50 mph in suburban settings. During peak conditions, roadways should operate at speeds of at least 20 mph. Throughput is maximized, and fuel consumption and emissions are minimized at speeds of 35 to 45 mph.
2. The green time per cycle for arterial roadway traffic should be maximized. This requires minimizing the time needed for left turns by prohibiting and redirecting the turns or by providing single or multiple left-turn lanes. Where left-turn phases are provided, cycle lengths may have to be increased to ensure sufficient green time and traffic progression efficiency (through bandwidth divided by the cycle length).
3. Major urban and suburban arterials experience high travel demands, especially during the morning and evening peak periods. Therefore, capacity is critical. This may require longer cycle lengths to minimize the "lost" time that occurs each time the traffic signal indication is changed and to provide special phases for left turns. Cycle lengths during peak periods normally range from 80 to 120 sec as compared with 60 to 80 sec at other times.
4. Cycle lengths that preclude achieving desired speeds for any given block spacing should be avoided. For example, with ½-mi signal spacing along a suburban roadway and 30 mph travel speeds, cycle lengths should not exceed 120 sec.
5. Where signals must be provided at locations that do not "fit" in the time-space pattern, additional arterial green is necessary to ensure adequate through bandwidth. This results in less green time for the intersecting street or driveway.

TRAVEL TIME IMPACTS

Frequent and/or non-uniform spacings of traffic signals constrain traffic flow and cause excessive delay. The relative effects of traffic signal spacing on travel speeds have been found in studies over the past 30 years.

Regression Analysis

A 1967 study of 77 street sections in New York State by Guinn (*18*) found that traffic signal density (signals per mile) and traffic volume per lane were the critical variables affecting traffic flow on arterial streets. Stover et al. (*19*) reported similar findings in 1970: operating costs and total costs decreased as signal spacing increased, and longer spacings were needed as the traffic volume per lane increased.

Several multiple linear regression analyses confirmed these earlier findings. A 1982 study in New Haven, Connecticut (*20*), and a 1992 study in Seminole County, Florida (*21*), found that peak-hour travel speeds decreased as traffic signal density and peak-hour traffic volumes per lane increased.

Linear regression equations were also derived as part of NCHRP Project 7-13, *Quantifying Congestion* (*22*) in 1995 for Class I, Class II, and Class III arterials as defined in the 1994 *Highway Capacity Manual* (*23*).

Class I arterials are typically high-speed, suburban arterials, while Class II and III arterials are intermediate-to-low-speed facilities in downtown or urban areas. Typically, speed limits are 40 to 45 mph on Class I arterials, 30 to 40 mph on Class II arterials, and 25 to 35 mph on Class III arterials. Typically, the number of signals per mile is 1 to 5 for Class I arterials, 4 to 10 for Class II arterials and 6 to 12 for Class III arterials.

The New Haven and NCHRP data suggested a 2- to 2.5-mph drop in speeds for every traffic signal added to 1 mi of street and up to a 0.5-mph drop in speeds for every 1,000 vehicles per lane per day increase in traffic.

Simulation Studies

Several recent simulation studies indicated that average speeds decline in a non-linear manner as the spacing between signals decreases and as the traffic volume per lane increases.

Simulation studies performed as part of the Colorado Access Control Demonstration Project (*24*) indicated that substantial reductions in total travel time and in total delay can be achieved with a ½-mi signalized intersection spacing and "mid-block" right turns only as compared with ¼-mi signalized spacings and full median openings at mid-block locations. These reductions in travel time and delay occurred even though more traffic passed through the signalized intersections.

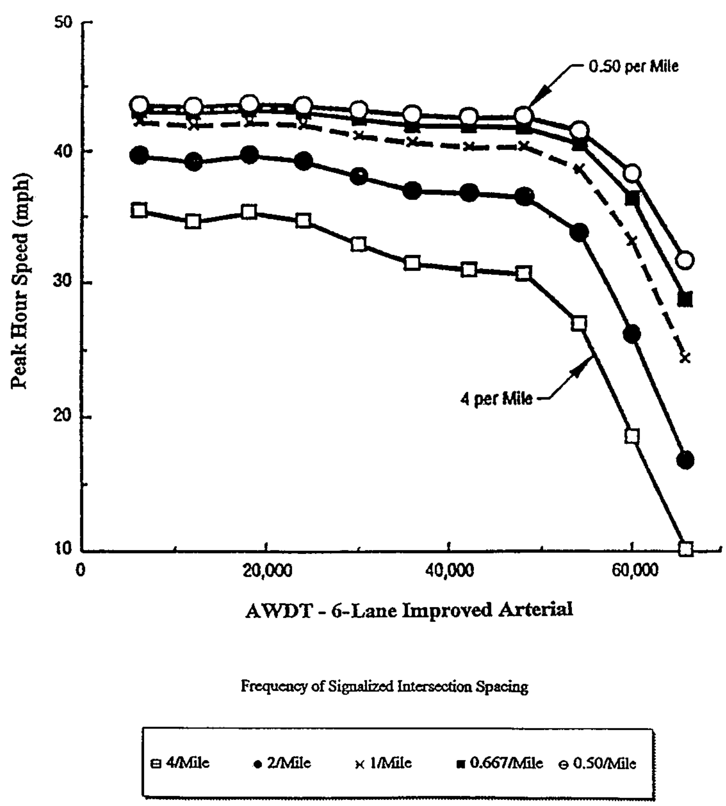
Results of simulations conducted at the University of Texas (*25*, *26*) are shown in Figure 8. Signal spacing is the principal influence on speeds at low volumes. However, the *v/c* or traffic-per-lane ratios become very critical as volumes approach or exceed capacity. Inspection of this figure shows that average speeds on 6-lane arterials drop significantly when AWDTs increase to greater than 50,000 vpd. The figure also shows that the percent reduction in speed increases as signal spacing decreases.

NETSIM was used by Margiotta et al. (*27*) to simulate the effects of traffic signal density and *v/c* ratios on average travel speeds. The simulation results obtained for a 50-mph (83-km/h) free-flow speed, fixed-time signals, and left-turn bays indicated that signal density had the greatest effect on travel speed, with a sharp drop from 0.5 to 3 signals per mile. The simulations show a growing effect of traffic volumes as the *v/c* ratio approaches 1.0.

Suggested Relationships

Curves for estimated peak-hour speeds on arterial streets at different *v/c* ratios and signal densities are shown in Figures 9 and 10 for Class I and Class II and III arterials, respectively. These curves were developed by the Texas Transportation Institute as part of their research in quantifying congestion (*22*). The Class I arterials assume a capacity of 10,000 vehicles per lane per day and the Class II and III curves assume a capacity of 8,000 vehicles per lane per day. The figures reflect traffic volume that ranges from 0.6 to 1.2 times the capacities.

These curves represent a synthesis of the relationships identified in the NCHRP, New Haven, and Margiotta research and, therefore, differ from the individual curves or equations. They provide results that are intuitively correct and that remove some of the anomalies in the individual data sets. They relate to the number of signals per mile in any road



Source: *(25)*

Figure 8. Speed, volume, and signal spacing relationships.

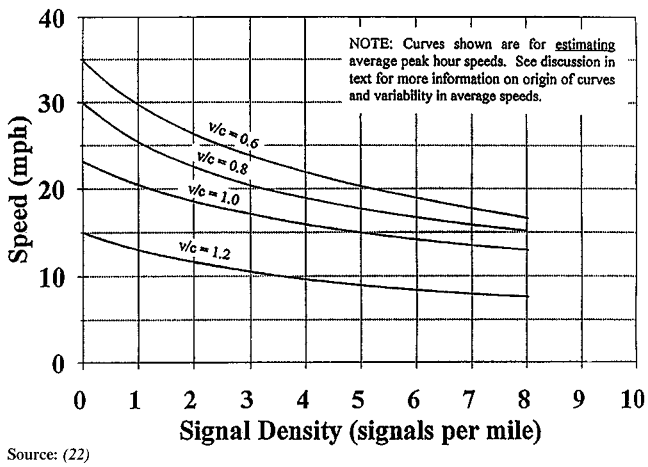


Figure 9. Suggested speed estimation curves for Class I arterials using v/c ratio.

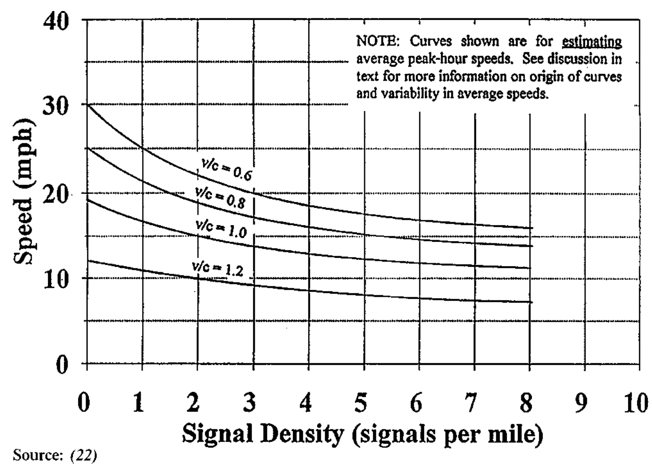


Figure 10. Suggested speed estimation curves for Class II & III arterials using v/c ratio.

section. In application, road sections should have relatively homogenous signal spacing.

The curves indicate the following:

- Traffic signal density has a greater effect than traffic volumes on reducing speeds when the *v/c* ratio is less than 0.8. Signals have their greatest reductive effect when they are introduced into free-flowing or lightly interrupted traffic (from 0 to 3 signals per mile) (0 to 2 signals per kilometer).
- Signal progression can be introduced into the curves by viewing the signal density in terms of "effective" signals per mile. The effective signals per mile may be estimated by the product of 1 minus bandwidth/cycle length and the signals per mile. For example, a 40 percent through band would result in 60 percent of the signal density associated with little or no progression.
- When traffic volumes approach, or exceed capacity, there is a considerable drop in speeds at all signal densities.

A further analysis indicates that the curves shown in Figures 9 and 10 can be represented by the following equation:

$$T = T_0[1+e]^{0.3}\left[1+(v/c)^4\right]^{0.7} \qquad (5)$$

where:

T_o = free-flow travel time in minutes per mile
T = actual travel time in minutes per mile
e = the number of effective traffic signals per mile
v/c = volume-to-capacity ratio

The actual speed in miles per hour is $60/T$.

APPLICATION GUIDELINES

The curves shown in Figures 9 and 10 provide reasonable approximations for estimating travel time impacts for planning and policy purposes. Travel time impedance values based upon Equation 5 and shown in Table 23 also may be used. The effects of signal spacing increase steadily as the number of traffic signals per mile increases—the impedance factor rises from about 1.1 for a 2-mi spacing to greater than 1.9 for a ⅛-mi spacing. The effects of the *v/c* ratio are negligible until the *v/c* ratio exceeds 0.7; the impedance values then rise rapidly. These values can be applied to any assumed free-flow travel time rate (minutes per mile) to determine the combined effects of traffic signal density and traffic volumes. The travel time rates, in turn, can be converted to speeds.

Using two traffic signals per mile as a base, the following percentage increases in travel times as signal density increases are estimated:

Signals Per Mile	*Percent Increase in Travel Times (Two Signals Per Mile as Base)*
3.0	9
4.0	16
5.0	23
6.0	29
7.0	34
8.0	39

Table 24 gives the resulting travel time rates and speeds, assuming a "free-flow" speed of 40 mph. Thus, if there are two effective signals per mile, and a *v/c* ratio of 0.6, the impedance factor is 1.52. When applied to the 1.5-min per-mile free-flow rate, it results in a rate of 2.28 minutes per mile or 26 mph. This approach may be used to assess the impacts of adding traffic signals and/or traffic volumes to a given roadway.

The following application guidelines are suggested relative to the inputs for Equation 5 and Table 23:

TABLE 23 Travel time rate impedance factors resulting from various signal densities and volume-to-capacity ratios

A Effective Signals/Mile	B Volume-to-Capacity Ratio				
	0.0	0.6	0.8	1.0	1.2
0.0	1.00	1.09	1.27	1.62	2.19
0.5	1.13	1.23	1.44	1.83	2.48
1.0	1.23	1.34	1.56	1.99	2.69
2.0	1.39	1.52	1.77	2.25	3.04
3.0	1.52	1.66	1.93	2.46	3.33
4.0	1.62	1.77	2.06	2.62	3.55
5.0	1.71	1.86	2.17	2.77	3.74
6.0	1.79	1.95	2.27	2.90	3.92
7.0	1.87	2.04	2.37	3.03	4.10
8.0	1.93	2.10	2.45	3.13	4.23

Source: Computed from Equation 5.

Notes: (1) Values shown are applied to actual travel time rates.

(2) Factors:

$A = (1 + e)^{0.3}$ where e = effective signals per mile

$B = \left[1 + (v/c)^4\right]^{0.7}$ where v/c = volume-to-capacity ratio

Factors in cells represent products of A and B.

1. The *v/c* ratios may be computed for critical sections of highway. However, the average daily traffic per lane may be used as a surrogate for the peak-hour *v/c* ratio. Suggested values are as follows:

% Green Per Cycle	*Maximum Capacity ADT/Lane/Day*
40	8,000
0	10,000
60	12,000

2. Signal coordination may be treated as follows:
   (a) *No coordination or through band.* The effective signals per mile equals the actual number of signals per mile (i.e., $e = S$).
   (b) *Limited through band.* This condition occurs with irregularly spaced signals or where multiple alternate signal progression patterns exist.

$$e = S\left[1 - \frac{Bandwidth}{Cycle}\right] \qquad (6)$$

where:
e = effective signals per mile
S = actual signals per mile

This factor should be computed whenever the bandwidth exceeds 35 percent.

   (c) *Perfect coordination.* This occurs with regularly spaced signals at ¼-, ⅓-, ½-, or 1-mi intervals and along one-way arterial streets. The progressive speed becomes the free-flow speed. Accordingly, where the through bandwidth exceeds 40 percent, replace the term $T_o\,(1 + e)^{0.3}$ in Equation 5 with $60/P$ where P is the progressive speed in miles per hour.
   (d) *Added signal in one direction.* When a new signal is added in only one direction of travel and fits perfectly into the time-space pattern, it should not be included in any impact analyses.

Examples

Two examples based on Table 23 are set forth in Table 25. (Use of Figures 9 and 10 would yield generally similar results.) A brief description of these examples follows:

TABLE 24 Travel time rates and speeds for 40 mph

Effective Signals/Mile	Volume-to-Capacity Ratio				
	0.0	0.6	0.8	1.0	1.2
	Minutes Per Mile				
0.0	1.50	1.64	1.90	2.43	3.28
0.5	1.70	1.84	2.20	2.74	3.72
1.0	1.84	2.01	2.34	2.98	4.04
2.0	2.08	2.28	2.66	3.37	4.56
3.0	2.28	2.49	2.90	3.69	5.00
4.0	2.43	2.66	3.09	3.93	5.34
5.0	2.56	2.79	3.26	4.16	5.61
6.0	2.68	2.92	3.40	4.35	5.88
7.0	2.80	3.06	3.56	4.55	6.15
8.0	2.90	3.15	3.68	4.70	6.35
Miles Per Hour					
0.0	40	37	32	25	18
0.5	35	33	27	22	16
1.0	33	30	26	20	15
2.0	29	26	23	18	13
3.0	26	24	21	16	12
4.0	25	23	19	15	11
5.0	23	22	18	14	11
6.0	22	21	18	14	10
7.0	21	20	17	13	10
8.0	21	19	16	13	9

Source: Computed.

TABLE 25 Illustrative examples based on Table 8

	CONDITION	
SCENARIO	**Before**	**After**
Signals Per Mile	2	4
Progression	None	None
ADT/Lane Capacity	10,000	10,000
ADT/Lane	6,000	8,000
Free-Flow Speed	40 mph	40 mph
SOLUTION (Table 8)		
Effective Signals Per Mile	2	4
V/C Ratio	0.6	0.8
Free-Flow Rate (Min./Mile)	1.50	1.50
Impedance Factor (Table 3)	1.52	2.06
Actual Travel Time Rate (Min./Mile)	2.28	3.09
Actual Speed	26 mph	19 mph

EXAMPLE 2

	CONDITION	
SCENARIO	**Before**	**After**
Signals Per Mile	4	5
Progression	35	None
ADT/Lane Capacity	10,000	10,000
ADT/Lane	6,000	8,000
Free-Flow Speed	40 mph	40 mph
SOLUTION (Table 8)		
Effective Signals Per Mile	3*	5
V/C Ratio	0.6	0.8
Free-Flow Rate (Min./Mile)	1.50	1.50
Impedance Factor (Table 3)	1.66	2.17
Actual Travel Time Rate (Min./Mile)	2.49	3.26
Actual Speed	24 mph	18 mph

*Effective signals = 4 [1- 0.35] = 2.6 ≈ 3

- **Example 1:** A roadway with two traffic signals per mile has an estimated capacity of 10,000 vehicles per lane per day (vplpd), an actual volume of 6,000 vplpd, and a free-flow speed of 40 mph. Developments along the road would increase the ADT/lane/day to 8,000 and increase the signal density to four signals per mile. The existing signals are not coordinated. The impacts are assessed by directly applying the factors in Table 23 or using Table 24. Because no signal coordination is involved, the effective signals and the actual signals are the same. The example shows a drop in peak-hour speeds from about 26 mph to 19 mph.
- **Example 2:** This example is similar to the first one in terms of free-flow speeds, capacities, and volumes. However, there are four signals per mile initially with a 35 percent through band and five signals per mile "after" with no effective coordination. The impacts are assessed by applying the factors contained in Table 23. However, the before effective signals per mile are reduced from four to three to account for the limited coordination. There is no corresponding adjustment for the "after" condition because the addition of the extra signal precludes coordination. The changes in volumes and speeds reduce the speeds from roughly 24 to 18 mph.

Arterial Simulation

More precise impacts of changes in traffic signal spacing and traffic volumes may be obtained by computer simulation. Simulation models (e.g., PASSER, TRANSYT 7-F, and TRAF NETSIM) may be applied to obtain estimates of system performance.

CHAPTER **4**

UNSIGNALIZED ACCESS SPACING (TECHNIQUE 1B)

INTRODUCTION

Access points, such as driveways, introduce conflicts and friction into the traffic stream. Vehicles entering and leaving the main roadway often slow the through traffic, and the difference in speeds between through and turning traffic increases accident potential. As stated in the 1994 AASHTO *A Policy on Geometric Design of Highways and Streets*, "Driveways are, in effect, at-grade intersections. . . . The number of accidents is disproportionately higher at driveways than at other intersections; thus their design and location merit special consideration."

The consensus is that increasing the spacing between access points improves arterial flow and safety by reducing the number of conflicts per mile, by providing greater distance to anticipate and recover from turning maneuvers, and by providing opportunities for use of turn lanes. It is increasingly recognized that spacing standards for unsignalized access points should complement those for signalized access points and that potentially high-volume unsignalized access points should be located where they conform to traffic signal progression requirements.

Many studies have shown that driveway spacing is one of the key factors that influence accidents. However, relatively few studies have actually related access spacing to driver performance. This chapter summarizes and compares the salient findings of the various research studies. It also presents the results of special safety and operations analysis.

SAFETY EXPERIENCE AND ANALYSIS

The research linking access density and accidents spans many decades. More than 40 years of research efforts have documented the basic relationships between access and safety. The methods of analyses and resulting relationships among individual studies vary, but the patterns are generally similar. Roadways with full control of access have lower accident rates than other roadways. Arterial roadways with many driveways and signals often have double or triple the accident rates of roadways with wide spacings between access points or of those where access is fully controlled. Accident rates generally increase with greater frequencies of intersections and driveways.

Safety Experience

An extensive review was made of the safety research and experience associated with access spacing. The first part of the review summarized the benefits resulting from the full control of access. This was followed by a summary of the early research (1952–1980) and, in turn, the more recent studies (1980–1996).

Full Control of Access

The safety benefits of access control have long been recognized and were a fundamental justification for the development of the freeway systems. Access control reduces the number and variety of events, while increasing the spacing of events (and conflicts) to which drivers must respond. This translates into fewer accidents—roadways with full control of access consistently have lower accident rates than other roadways.

Early Studies (1950–1980)

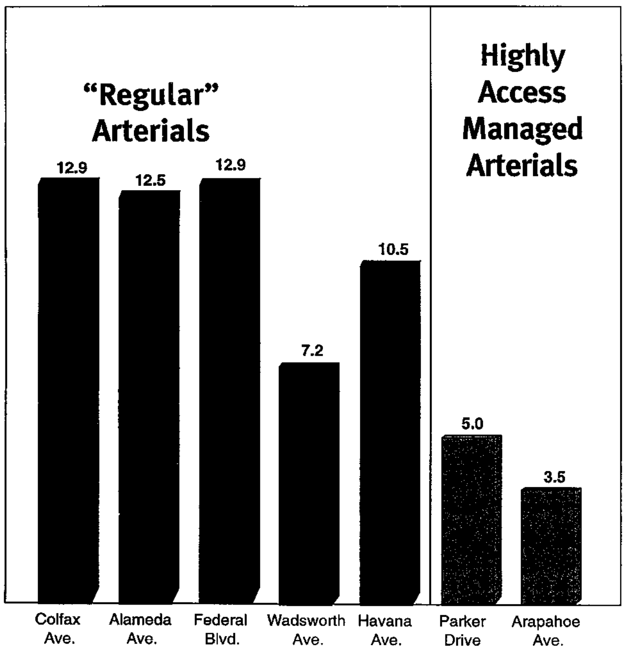
Almost 12 research investigations between 1950 and 1960 attempted to correlate accident rates with the number, frequency, and type of roadside features and access points. (See references *28* through *39* for further information).

Recent Studies

Studies since the mid-1980s have also shown that increasing the frequency of access points adversely affects safety. Most of these studies were conducted to demonstrate the benefits of access management. Some show aggregate relationships while others utilize analytical or regression models. (See references *40* through *52* for further information.)

Arapahoe Avenue and Parker Drive, Denver (1985) (40)

A demonstration project conducted by the Colorado Department of Highways compared the 3-year accident experience on two access-managed highways (Arapahoe Avenue and Parker Drive) with that of five regular arterials. The accident rate comparisons are shown in Figure 11. The



"Regular" represents those arterials sampled without a high degree of access management.
Source: *(40)*

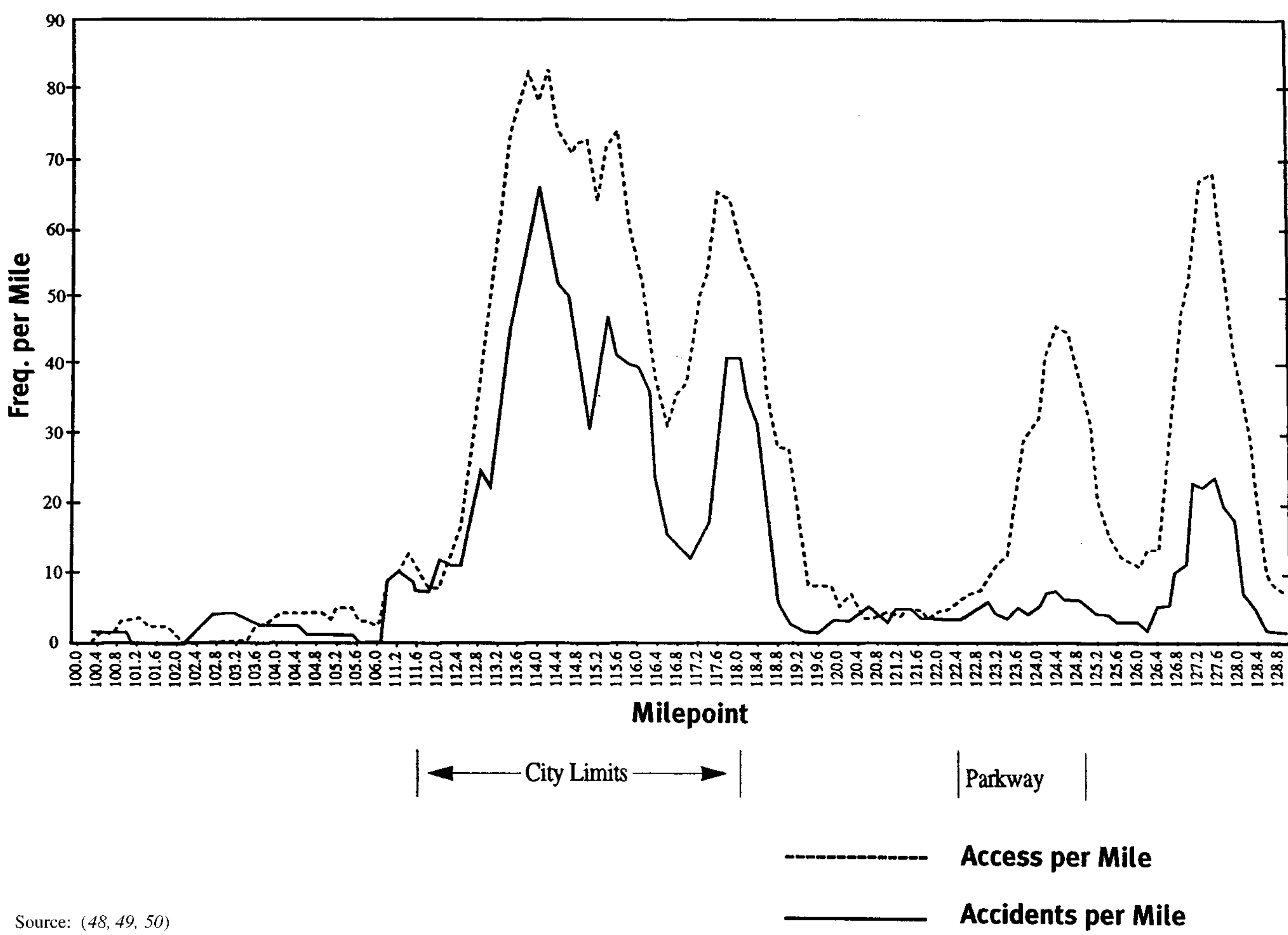
Figure 11. Accidents per million vehicle miles–Denver.

two highly access-managed arterials (with physical medians, full access generally limited to ½-mi intervals, most left-turn access prohibited, and right-turn access provided at ¼-mi intervals) had about 40 percent of the accident rate found along the roads with more frequent access (the range was 27 to 69 percent).

Oregon Coast Highway, Oregon (1995–1996) (48, 49, 50)

A comprehensive accident analysis was conducted for 29 mi of the Oregon Coast Highway (US Route 101) by Portland State University in association with the Oregon Department of Transportation. The study area, located on the Oregon coast in and around Lincoln City, has tourist traffic as well as the usual urban and rural traffic. Seven hundred and fifty accidents were analyzed for the period from 1990 to 1994.

Figure 12 shows how the frequency of accidents relates to access density. This chart shows a consistent relationship between access per mile and accidents per mile, except for the "Parkway" section. The low number of accidents per mile on the Parkway section reflects the presence of a continuous



Source: *(48, 49, 50)*

Figure 12. Relationship between access density and accident frequency – Oregon coast highway.

nontraversable median. As expected, the higher accident frequencies along US 101 were found within the city limits where urban development not only resulted in higher driveway densities, but probably higher driveway volumes as well.

Lee County, Florida (1993–1996) (51)

The effects of connection and traffic signal densities on accident rates in Lee County, Florida, are shown in Figure 13. A doubling of connections from 20 to 40 per mile doubled the accident rate.

Australian Experience (1997) (53)

Studies by ARRB Transport Research indicated the following safety impacts when intersection and/or driveway frequency was increased:

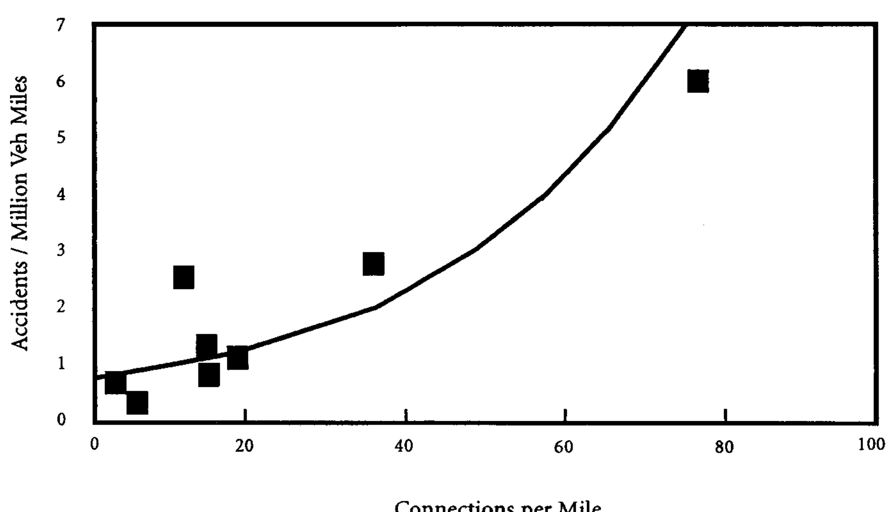
- Divided urban arterial roads with direct property access and frequent minor intersections had a 30 percent higher accident rate than those with few property access points and infrequent minor intersections. This difference increased to 70 percent for undivided roads.
- In rural areas, each minor intersection added about 0.35 accidents per million entering vehicles for a 2-lane road and about 0.25 accidents per million entering vehicles for a 4-lane road.
- Increasing minor intersection density in rural areas from 0 to 1 per kilometer (0 to 1.6 per mile) increased accident rates by about 25 percent on rural roads. An increase in minor intersection density in urban areas from 2 to 6 per kilometer (3.2 to 9.7 per mile) increased accident rates by 20 to 100 percent on 4-lane roads and 50 to 100 percent on 2-lane roads.
- Each additional private driveway per kilometer in both urban and rural areas increased accident rates about 1.5 percent for 2-lane roads and 2.5 percent for 4-lane roads. These translate into 2.4 and 4.0 percent increases per private driveway on a per mile basis. In urban areas, each commercial driveway had about 5 times the effect of a private driveway on accident rates.
- In general, the effects noted above increased with decreasing standards of horizontal alignment and decreased if medians were present.

Synthesis of Findings

The various studies point to one consistent finding. An increase in the number of access points translates into higher accident rates. Thus, the greater the frequency of driveways and streets, the greater the number of accidents.

The specific relationships vary, reflecting differences in road geometry (e.g., lane width and presence or absence of turn lanes and physical medians), operating speeds, and driveway and intersection traffic volumes. Still, in every case, more access means more accidents. This upward trend in accident rates is apparent from Figure 14, which shows reported results for experience in the United States and Canada, graphed on a common scale.

Indexes were prepared that correlated accident rates with access density using the accident rates for 10 access points per mile as a base (total access points per mile on both sides of the road). The indexes were averaged for each access density. Figure 15 presents the composite accident rate indexes. These indexes suggest that doubling of access frequency



Source: *(51)*

Figure 13. Connections and crashes: US 41 – Lee County, Florida

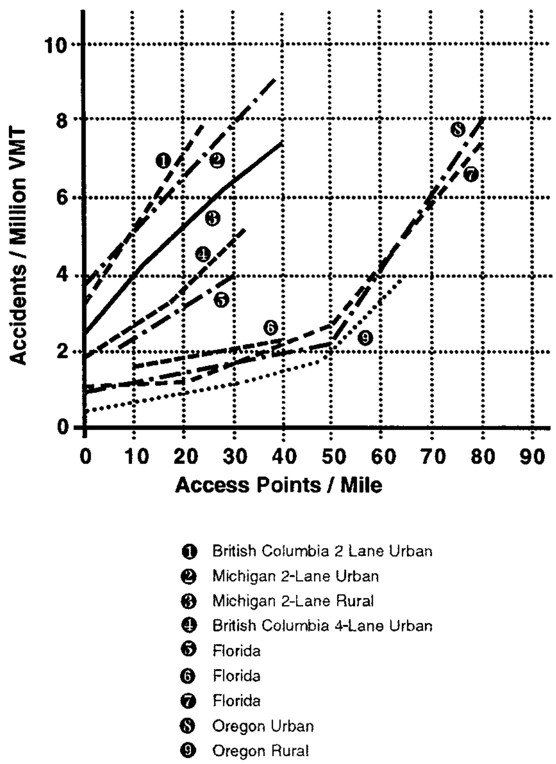
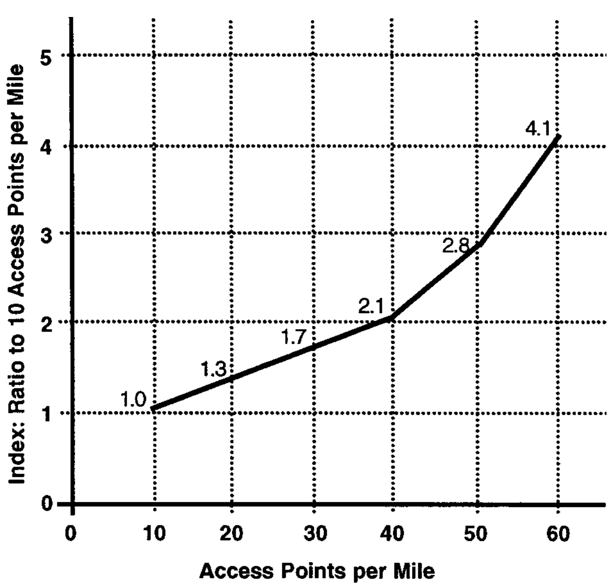


Figure 14. Effect of access spacing on accident rates (composite).

from 10 to 20 per mile increases accident rates by roughly 30 percent. An increase from 20 to 40 driveways per mile would increase accident rates by more than 60 percent. These increases are similar to those reported in Australia (*53*).

The access spacing implications are clear. Increasing the spacing between access points and providing greater separations of conflicts will reduce the number and variety of



Source: Estimated from Various Sources

Figure 15. Composite accident rate indices.

events to which drivers must respond. This translates into fewer accidents, as well as shorter delays.

Safety Analyses

Comprehensive safety analyses were performed for accident information obtained from Delaware, Illinois, Michigan, New Jersey, Oregon, Texas, Virginia, and Wisconsin. Overall, some 386 roadway segments were analyzed. Analyses of the data further established the relationships between access and accidents for various spacings and median types.

Analysis Procedures

The literature review indicated (and subsequent analyses confirmed) that accident rates (accidents per million vehicle miles) generally increased as access density—the number of at-grade intersections, driveways, and median openings per mile—increased. Signalized access density was one of the more influential factors. The type of median treatment (e.g., undivided, TWLTL, or physical median) also influenced accident rates.

Accordingly, the accident database for the 386 road segments was stratified by the number of signalized and unsignalized access points per mile, the area type (i.e., urban/rural) and the median treatment. The segments were further stratified by land use, number of lanes, and ADT range. In urban areas, there were 264 segments covering 254 mi, including 116 segments with medians, 95 segments with TWLTLs, and 53 undivided segments. In rural areas, there were 122 segments covering 168 mi including 57 segments with medians, 14 segments with TWLTLs, and 51 undivided segments. A screening of the database reduced the number of segments to 369, including 252 urban and 117 rural sections.

To provide sufficient samples for stratification purposes, data were grouped by geographical region (i.e., data from states in the same region were aggregated) or combination of geographical regions. For example, the records from Michigan, Illinois, and Wisconsin formed one region and records from New Jersey and Delaware formed a second region. Figure 16 illustrates the data analysis sequence.

Accident rates varied by area type, because urban and suburban areas have significantly different roadway activity and operational characteristics than do rural areas (e.g., a review of the accident data for the rural segments in Michigan indicated a significant number of accidents involved animal crossings). Furthermore, accident frequency/rates would be expected to increase as access density increases, because the opportunity for conflicts is greater and the available space for maneuvering decreases.

Exploratory analyses (e.g., frequency distributions, cross-classifications, and means) were performed for key variables in the database to define the appropriate stratifications. These analyses revealed that (a) area type was significant because accident rates for rural areas were significantly lower than for urban and suburban areas; (b) the average accident rates for

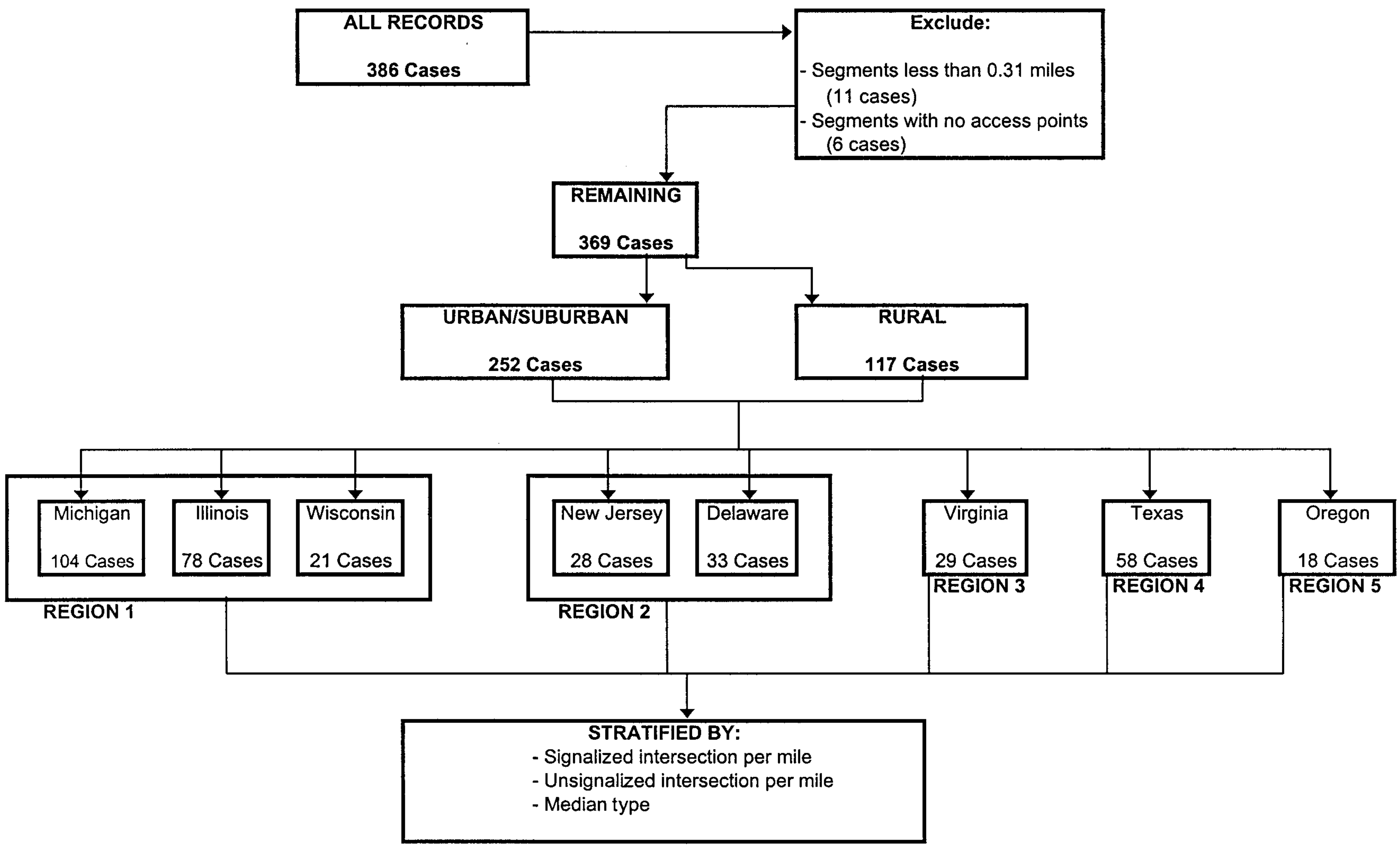


Figure 16. Data analysis sequence.

TABLE 26 Accident reporting threshold

State	Reporting Threshold
Michigan	$ 400
Illinois	$ 500
Texas	$ 500
Oregon	$ 500
Wisconsin	$ 500
Delaware	$1,000
New Jersey	$ 500

urban and suburban areas in Texas, Virginia, and Oregon were almost 50 percent lower than comparable areas from the other states; (c) Virginia segments exhibited twice as high average volumes per lane, compared with segments from the other states (excluding New Jersey); and (d) the average access density and the average volume per lane for the urban and suburban segments in New Jersey were significantly higher than comparable segments from the other states.

At first, it was thought that the low accident rates in Texas, Virginia, and Oregon were attributable to significantly higher accident reporting dollar thresholds. However, as shown by the actual thresholds in Table 26, this was not the case. Further investigation revealed that many accidents in Texas are not reported. In Dallas, for example, only accidents with injury or death are investigated and reported by the police. Other accidents are supposedly reported by the parties involved; however, not all of these accidents involving only property damage (PDO) are reported. An unofficial estimate of the PDO accident reporting rate is about 50 percent.

No reason was identified for the lower rates in Oregon or Virginia. Because the data sent from Virginia were originally collected as part of a research study that concentrated only on very short urban segments with a high number of access points and high traffic volumes, the data from Virginia were excluded from further analysis. Texas and Oregon were analyzed individually and excluded from any aggregate analysis.

The resulting database that was used for further aggregate analysis reflected about 37,500 accidents and included 152 urban and suburban segments and 89 rural segments.

Urban and Suburban Areas

Detailed analyses were conducted for the urban/suburban database for Illinois, Michigan, Wisconsin, New Jersey, and Delaware. The first step was to further screen segments for characteristics or accident rates that did not appear to be consistent with the rest of the data. Next, the accidents were stratified by geometric and access density variables. Finally, statistical analyses were performed for the accident rates in the various strata.

After the potential "outliers" were removed from the database, frequency distributions and cross-classifications were performed to identify potential strata and to explore relationships. Based on this analysis, three strata for total access points (TAP) per mile and unsignalized access points per mile and four strata for signalized access points (SAP) per mile were established. The resulting access density strata are shown in Table 27 along with the number of segments in each stratum. The strata—in increments of 20 access points per mile—avoid cells with few points.

Accident rates by total access density and type of median treatments are shown in Table 28. Means, coefficients of variation, students 't' distribution statistics, and p-values are given. The p-values represent the probabilities of differences between means occurring because of chance; thus, a 0.05

TABLE 27 Access density strata–urban/suburban segments

Access Density/Strata	Stratum 1	Stratum 2	Stratum 3	Stratum 4	Total
Total Access Points per Mile	<=20	20.01-40	40.01-60	>60	
Percent of Segments	9.9	34.2	30.9	25.0	100.0
Number of Segments	15	52	47	38	152.0
Mean Access Points	14.0	31.49	51.02	74.57	43.58
Median Access Points	11.35	31.90	50.30	77.49	46.96
Unsignalized Access Points Per Mile	<=20	20.01-40	40.01-60	>60	
Percent of Segments	11.2	41.4	25.0	22.4	100.0
Number of Segments	17	63	38	34	152.0
Mean Access Points	12.00	29.70	49.75	71.90	40.00
Median Access Points	10.30	30.28	50.03	74.00	42.76
Signalized Access Points Per Mile	<=2	2.01-4	4.01-6	>6	
Percent of Segments	21.7	30.9	27.6	19.7	100.0
Number of Segments	33	47	42	30	152.0
Mean Access Points	0.56	3.33	5.00	7.11	4.00
Median Access Points	0.81	3.28	4.95	8.30	4.20

DUR12C

TABLE 28 Accident rates by access density and median treatment – urban/suburban segments

Access Density [1]	Statistics	Median Treatment: Undivided	Median Treatment: TWLTL	Median Treatment: Non-Travers-able	Total
<=20	Mean	3.82		2.94	3.24
	C.V.	0.28	--------	0.45	0.40
	Cases	5		10	15
20.01-40	Mean	8.27	5.87	5.13	5.90
	C.V.	0.72	0.43	0.60	0.53
	Cases	5	33	14	52
	t-statistic	1.64	NA	2.36	NA
	p-value	0.09	NA	0.02	NA
40.01-60	Mean	9.35	7.43	6.47	7.37
	C.V.	0.43	0.52	0.47	0.49
	Cases	7	23	17	47
	t-statistic	0.35	1.72	1.21	NA
	p-value	0.37	0.05	0.12	NA
>60	Mean	9.55	9.17	5.40	8.59
	C.V.	0.43	0.56	0.75	0.53
	Cases	21	9	8	38
	t-statistic	0.11	0.92	NA	NA
	p-value	0.46	0.19	NA	NA
Total	Mean	8.59	6.88	5.19	
	C.V.	0.52	0.52	0.61	
	Cases	38	65	49	

DUR12A

Notes: Accident Rates=Accidents per Million Vehicle Miles Traveled.
p-values computed top to bottom and are not computed where inconsistencies in the accident rate trends exist.
(1) Access Density reflects both signalized and unsignalized access points per mile.
C.V.= Coefficient of variation.
NA = Not Applicable

p-value is similar to a 5 percent level of significance. The p-values are shown for changes in access frequency (top to bottom). They are for a one-sided, upper-tail test (i.e., to determine if differences are significantly greater).

Table 28 shows an increase in accidents for each type of median treatment as the total access density increases. The accident rate for access densities of more than 60 per mile was more than 2.5 times higher than the accident rate for access densities of fewer than 20 per mile.

Table 28 also shows the accident reductions associated with various median alternatives. Overall, TWLTLs had a 20 percent lower accident rate, and nontraversable medians had a 40 percent reduction than undivided road sections. These patterns were generally consistent for all access density ranges.

The effects of signalized access density on accident rates are shown in Table 29. The p-values are shown top to bottom. Accident rates increased as signalized access density increased. The rate for more than six signals per mile was more than 2.5 times that for signal densities of two or fewer per mile.

TWLTL segments appeared to have lower accident rates than undivided road sections. The one inconsistency may have reflected the low sample size for undivided segments with fewer than two signals per mile. Nontraversable medians had lower accident rates than the other median treatments for all signal spacing frequencies.

Accident rates were also computed for various cross-classifications of signalized and unsignalized access densities as shown in Table 30. The upper tail p-values are shown for changes in unsignalized access frequencies (left to right). The data showed an overall increase in accident frequency as unsignalized access density rises.

Overall accident rates for access densities of more than 60 points per mile were about 2.2 times than for densities of 20 or fewer access points per mile. This pattern was generally consistent at each level of signal density.

Table 30 provides guidance for estimating the effects of increasing unsignalized access density. However, because signal density may be a surrogate for heavy cross-street vol-

TABLE 29 Accident rates by signalized access density and median treatment – urban/suburban segments

Signalized Access Density (1)	Statistics	Median Treatment: Undivided	TWLTL	Non-Travers-able	Total
<=2	Mean	4.01	4.13	2.75	3.53
	C.V.	0.41	0.72	0.52	0.66
	Cases	4	15	14	33
2.01-4	Mean	8.20	7.02	5.66	6.89
	C.V.	0.53	0.46	0.52	0.51
	Cases	12	20	15	47
	t-statistic	2.80	2.76	3.40	NA
	p-value	0.01	0.01	0.00	NA
4.01-6	Mean	9.87	7.42	5.99	7.49
	C.V.	0.55	0.36	0.51	0.51
	Cases	10	17	15	42
	t-statistic	0.79	0.41	0.30	NA
	p-value	0.22	0.34	0.39	NA
>6	Mean	9.45	9.13	8.26	9.11
	C.V.	0.38	0.46	0.40	0.41
	Cases	12	13	5	30
	t-statistic	0.21	1.30	1.22	NA
	p-value	0.42	0.11	0.14	NA
Total	Mean	8.59	6.88	5.19	
	C.V.	0.52	0.52	0.61	
	Cases	38	65	49	

DUR12A

Notes: Accident Rates=Accidents per Million Vehicle Miles Traveled.
p-values are computed top to bottom and are not computed where inconsistencies in the accident rate trend exist.
(1) Access Density reflects signalized access points per mile.
C.V.= Coefficient of variation.
NA = Not Applicable

umes, the values for signal density may not apply where signals are added at lightly traveled crossroads.

Rural Areas

A similar analysis was performed for road segments in rural areas. The accident rates were stratified by total access point density and median treatment, because the number of signalized access points in the database was small. Accidents rates for Michigan were recalculated to remove animal-related and rail-crossing accidents.

After the potential outliers were eliminated from the database, frequency distributions and cross-classifications were performed to identify potential strata and to explore relationships. The number of strata was kept to a minimum to avoid cells with very few points. Accordingly, three strata for TAP were identified as summarized in Table 31.

Accident rates are stratified by total access density and median treatment in Table 32. The upper tail p-values compare various access densities (top to bottom) on the table. P-values were not computed where inconsistencies in the accident rate trend exist.

The increase in access density from fewer than 15 access points to more than 30 access points per mile resulted in a 65 percent increase in the overall accident rate. Again, TWLTLs had about a 40 percent lower accident rate and nontraversable medians had a 60 percent lower accident rate than undivided road sections. This pattern was generally consistent at all access densities.

OPERATIONS EXPERIENCE AND ANALYSIS

This section contains the results of a detailed literature review and special operational studies relating traffic performance (i.e., speeds, delays, and affected vehicles) to drive-

TABLE 30 Accident rates by access density – urban/suburban segments

Signalized Access Density (access points per mile)	Statistics	Unsignalized Access Density (Access Points Per Mile) <=20	20.01-40	40.01-60	>60	Total
<=2	Mean	2.63	4.33	3.01	3.80	3.53
	C.V.	0.49	0.69	0.51	0.68	0.66
	Cases	8	14	9	2	33
	t-statistic	NA	1.85	NA	0.41	NA
	p-value	NA	0.04	NA	0.37	NA
2.01-4	Mean	3.94	5.58	8.30	8.22	6.89
	C.V.	0.28	0.33	0.44	0.53	0.51
	Cases	5	16	12	14	47
	t-statistic	NA	2.45	2.35	0.05	NA
	p-value	NA	0.02	0.02	0.48	NA
4.01-6	Mean	4.83	6.91	8.37	8.54	7.49
	C.V.	0.36	0.52	0.43	0.58	0.51
	Cases	3	19	12	8	42
	t-statistic	NA	1.62	1.10	0.08	NA
	p-value	NA	0.08	0.14	0.47	NA
>6	Mean	8.61	8.06	11.30	9.53	9.11
	C.V.	NA	0.39	0.33	0.48	0.41
	Cases	1	14	5	10	30
	t-statistic	NA	0.98	1.75	0.44	NA
	p-value	NA	0.17	0.07	0.34	NA
Total	Mean	3.76	6.26	7.47	8.42	
	C.V.	0.51	0.51	10.55	0.53	
	Cases	17	63	38	34	

DUR12A

Notes: Accident Rates=Accidents per Million Vehicle Miles Traveled.
p-values are computed left to right and are not computed where inconsistencies in the accident rate trend exist.
Separate Variance t-statistic to account for unequal Cell Variances.
C.V. = Coefficient of Variation.
NA = Not Applicable.

way spacing. Collectively, these investigations underscore the importance of adequate spacing.

Operations Experience

Various operational studies have addressed the travel time impacts associated with access spacing and have also simulated traffic performance. A research synthesis summarized the results of the studies performed in the 1960s and 1970s (see references *54* through *58* and more recently (see references *53* and *59* through *63*).

Recent Studies

The following are the highlights from two of the more recent studies.

British Columbia (1992) (61). A manual developed by the Planning Services Branch for evaluating highway programs

TABLE 31 Access density strata – rural segments

Access Density/Strata	Stratum 1	Stratum 2	Stratum 3	Total
Total Access Points per Mile	<=15	15.01-30	>30	
Percent of Segments	61.8	29.2	9.0	100.0
Number of Segments	55	26	8	89
Mean Total Access Points	8.00	21.00	36.00	13.00
Median Total Access Points	7.89	21.08	36.86	14.35

TABLE 32 Accident rates by access density and median treatment – rural segments

Access Density [1]	Statistics	Median Treatment: Undivided	TWLTL	Non-Traversable	Total
<=15	Mean	2.54	2.06	0.90	1.64
	C.V.	0.63	NA	1.24	0.95
	Cases	24	1	30	55
15.01-30	Mean	2.60	1.26	1.18	1.79
	C.V.	0.62	NA	1.26	0.92
	Cases	11	1	14	26
	t-statistic	0.10	NA	0.64	NA
	p-value	0.46	NA	0.27	NA
>30	Mean	4.65	1.67	1.47	2.71
	C.V.	0.13	0.78	0.85	0.68
	Cases	3	2	3	8
	t-statistic	3.40	NA	0.35	NA
	p-value	0.01	NA	0.38	NA
Total	Mean	2.73	1.67	1.02	
	C.V.	0.59	0.49	1.20	
	Cases	38	4	47	

DATAR3

Notes: Accident Rates = Accidents per Million Vehicle Miles Traveled.
p-values are computed top to bottom and are not computed where inconsistencies in the accident rate trend exist.
(1) Access Density reflects both signalized and unsignalized access points.
Separate Variance t-statistic to account for unequal Cell Variances.
C.V. = Coefficient of variation.
NA = Not Applicable.

incorporated speed adjustments for access density. Accesses included unsignalized intersections, commercial establishments, and driveways. To account for different access volumes at various access points, it was assumed that

- 5 driveways = 1 access
- 1 commercial establishment = 1 access
- 1 unsignalized intersection = 2 accesses

The speed adjustment factors shown in Table 33 were suggested for 2-lane highways.

Reilly-HCM. The 1994 *Highway Capacity Manual (60)* describes the impacts of access frequency on travel speeds for multi-lane rural and suburban arterials. The facilities generally have posted speed limits of between 40 and 55 mph. They usually have four or six lanes, often with physical medians or TWLTLs, although they may also be undivided. Traffic signals may be found along these facilities, but traffic signals spaced at 2.0 mi or less typically create urban arterial conditions. The speed adjustment factors in the 1994 HCM are based on the analysis performed for NCHRP Project 3-33 (*59*). The 1994 HCM states that

> An important influence on free-flow speed is the number of access points along the right side of the roadway. The data base used to establish the procedures in this chapter indicated that the number of access points was the critical ele-

TABLE 33 Speed adjustments for access density: British Columbia

Access Density (Access/km)	Running Speed Adjustment Factor
0	1.00
5	0.96
10	0.93
15	0.90
20	0.86
25	0.83
30	0.79

Source: *(61)*

ment in reducing free-flow speeds along a section of multi-lane highway. Although the amount of activity at each point also contributes to changes in travel speed, it is apparent that drivers adjust their travel speed not only on the basis of entrances and exits at such points but also on the mere existence of access points. As expected, the addition of intersections or driveways along a multi-lane highway will reduce travel speeds. The procedures of this chapter show that for every 10 access points per mile that affect a given direction of travel on a multi-lane highway, travel speed will be reduced by 2.5 mph.

Note that this procedure takes into account only those access points on one side of the roadway and not those on the opposite side of the roadway or openings in the median. If access points on the opposite side of the roadway or median openings for U-turns are expected to have a significant effect on traffic flow in the direction of interest, these intersections, driveways, or openings may be included in the determination of access-point density.

Table 34 provides the suggested adjustment factors. Where data on access frequency are not available, the 1994 HCM suggests the following access densities be used as default values:

- Rural—0–10 driveways per mile
- Low-Density Suburban—11–20 driveways per mile
- High-Density Suburban—21 or more driveways per mile

The adjustment factors make no distinctions between driveways and street intersections, nor do they differentiate between high-volume and low-volume access points.

The research for NCHRP Project 3-33 indicated that each turning movement per hour per mile of highway (for one direction of flow) reduces free-flow speeds by .005 mph, up to a maximum reduction of 10.0 mph. The presence of an access point itself was found to reduce speed by 0.15 mph.

For a right-turn volume of 500 vph and five access points per mile, the speed loss would be 3.25 mph. However, with 40 access points per mile and a right-turn volume of 500 vph, the speed loss would be 8.50 mph.

Implications

The studies found that increasing the number of driveways (i.e., reducing driveway spacing) along a section of highway increased delays and reduced roadway capacities. The methods and results varied from study to study, and there were no "before and after" studies. The field studies by Reilly (*59*) and the simulations by McShane (*62, 63*) gave generally consistent results. For driveway volumes of 600 per mile per hour, the Reilly studies (for uninterrupted flow) showed a speed loss of 1.0 to 1.7 mph per driveway, while the McShane simulations (for signalized arterials) suggested a 1.5- to 2.0-mph loss per driveway. (These comparisons are for up to four driveways per mile.)

Operations Analysis

Field studies were conducted to identify how right turns at a driveway affect other drivers following in the same lane. As a surrogate measure of the number of impacts, the incidents of brake lights being activated or evasive maneuvers by a following through vehicle were counted.

The field investigation and analyses were conducted for 22 sites in Connecticut, Illinois, New Jersey, and New York. Each site represented a major traffic generator along a suburban arterial roadway. The arterials had no deceleration lane, and the driveways were not signalized. Salient characteristics of the study sites are shown in Table 35. These include dates and times of study, median type, number of lanes, and distances from upstream and downstream intersections.

Information was gathered on the number and percentage of through vehicles affected by right turns. The impact lengths of through vehicles affected were determined, and, in turn, influence areas were computed. The results were used to quantify the effects of multiple driveways and to develop inputs for establishing unsignalized access spacing guidelines. The analysis procedure is outlined in Figure 17.

TABLE 34 Access point density adjustment factors

Access Points Per Mile	Reduction in Free-flow Speed (mph)
0	0.0
10	2.5
20	5.0
30	7.5
40 or more	10

Source: Table 7-5 of 1994 HCM.

TABLE 35 Physical characteristics of study locations

Location No.	Study Site Location	Route No.	City	State	Date	Time	Median Type	Thru Lanes	No. of Driveway Movements	Dist. from Upstream Signal (Feet)	Dist. from Downstream Signal (Feet)
1	Edison Square	27 SB	Edison	NJ	07/17/96	7:00 AM - 9:00 AM	TWLTL	4	4	725	600
2	Tops	27 SB	Edison	NJ	07/20/96	2:00 PM - 5:00 PM	UNDIVIDED	4	4	1050	2,250
3	Home Depot	24 EB	East Meadow	NY	09/27/96	4:00 PM - 6:00 PM	NON-TRAVERSABLE	6	3	700	725
4	Frank's Nursery	1 NB	Norwalk	CT	10/05/96	12:10 PM - 1:10 PM	UNDIVIDED	4	4	1000	400
5	Bradlee's/Stop & Shop	1 SB	Norwalk	CT	10/05/96	2:15 PM - 4:15 PM	UNDIVIDED	4	4	490	800
6 *	Jewel Osco	38 WB	Lombard	IL	11/09/96	-	TWLTL	4	3	800	575
7	Sportmart & Marshall's	38 EB	Lombard	IL	11/09/96	2:30 PM - 4:00 PM	TWLTL	4	4	1050	575
8	Golf Plaza II (Walgreens)	83 SB	Mount Prospect	IL	11/10/96	11:30 AM - 1:30 PM	TRAVERSABLE	4	6	525	900
9	Butera Market Place	83 NB	Des Plaines	IL	11/10/96	2:45 PM - 5:00 PM	TRAVERSABLE	4	6	925	525
10	Ancona's Market	102 EB	Georgetown	CT	11/16/96	1:10 PM - 2:10 PM	UNDIVIDED	2	4	N/A	450
11	TOYS 'R' US	1 NB	Norwalk	CT	11/16/96	3:30 PM - 6:00 PM	UNDIVIDED	4	3	420	1,100
12	Kids 'R' Us/Gap/Office Max	18 SB	E. Brunswick	NJ	11/17/96	11:00 AM - 2:00 PM	NON-TRAVERSABLE	6	2	550	775
13	Archway Plaza	100 SB	Greenburgh	NY	11/23/96	12:35 PM - 2:35 PM	UNDIVIDED	4	2	975	675
14	Price Club	Vineyard Rd. WB	Edison	NJ	11/24/96	11:30 AM - 1:30 PM	UNDIVIDED	2	4	N/A	1,000
15 *	Greenville Plaza	100 SB	Greenburgh	NY	11/23/96	-	TWLTL	4	4	550	825
16	Offices of EAB, Canon, etc.	New Hyde Park Rd. SB	Manhasset Hills	NY	12/05/96	7:30 AM - 9:30 AM	UNDIVIDED	4	1	1100	475
17	T.J. Maxx & Noodle Kidoodle	25A EB	Greenvale	NY	12/05/96	11:30 AM - 1:30 PM	UNDIVIDED	4	2	500	750
18	Astoria Federal Office	Marcus Ave. EB	Lake Success	NY	12/10/96	7:00 AM - 9:30 AM	UNDIVIDED	2	4	1550	750
19	L.I.J. Medical Center	Marcus Ave. EB	Lake Success	NY	12/10/96	7:30 AM - 9:30 AM	UNDIVIDED	2	4	1950	375
20	North Shore Atrium	25 EB	Locust Grove	NY	12/11/96	7:15 AM - 9:15 AM	TWLTL / UNDIVIDED	4	4	1100	950
21	King Kullen	25 WB	Huntington Manor	NY	12/15/96	2:00 PM - 4:00 PM	TWLTL	4	4	2150	525
22	Warner Lambert Office	53 SB	Morristown	NJ	12/20/96	7:30 AM - 9:30 AM	UNDIVIDED	2	4	1100	N/A

*** Dropped from analysis because arterial traffic was backing up periodically from downstream signal and blocking the driveway.**
N/A - Not Applicable

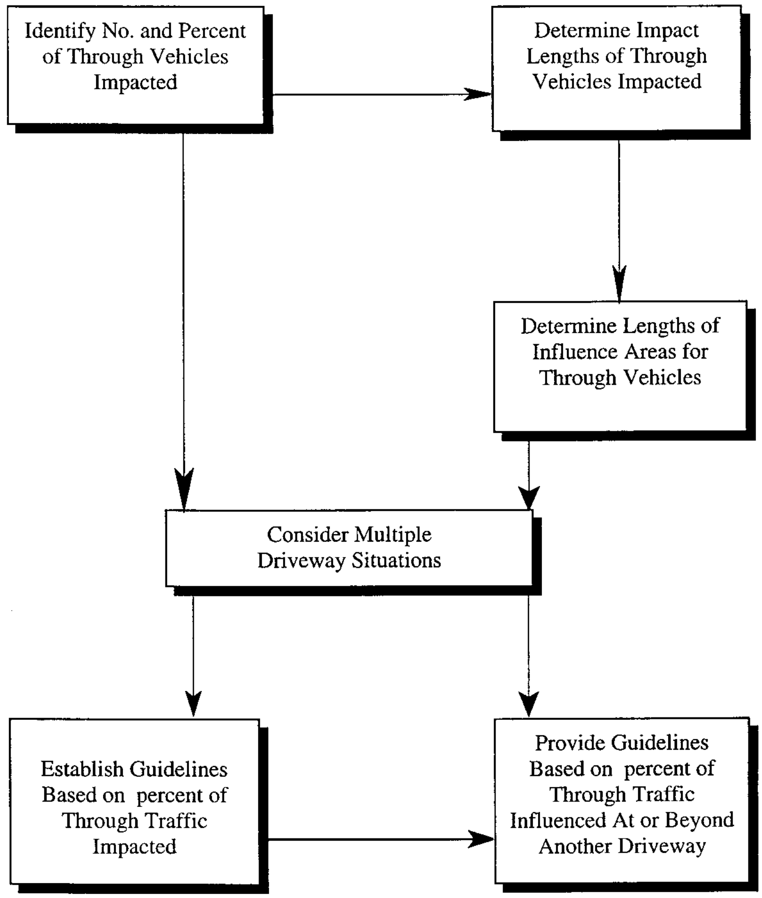


Figure 17. Flow chart for establishing unsignalized access spacing guidelines.

The study identified the following:

1. The number and percentage of through vehicles in the curb lane that are impacted at a single driveway,
2. The percentage of through vehicles in the curb lane that are impacted over a series of driveways,
3. The distances back from a single driveway entrance that cars begin to be affected—the impact length and the spatial distributions of impacted vehicles,
4. The "influence areas" or influence distances before (upstream of) a driveway entrance (This involves adding perception-reaction distance and car length),
5. The variations of influence distances by roadway operating speed,
6. The proportions of affected through vehicles in the curb lane that would extend to or beyond at least one driveway over a section of road at various operating speeds, and
7. The emergent access spacing implications. (A basic premise is that minimizing the number of access points that a driver must monitor simultaneously simplifies the driving task, thus, spacing guidelines could be established to reflect the acceptable frequency with which the influence length for a right-turn-in vehicle would extend to or beyond another driveway).

Through Vehicles Affected by Right Turns

The number and percentage of through vehicles affected by vehicles turning right were obtained from field operations. The effects of right turns were analyzed. The results were extended to assess the percentage of through vehicles in the right lane that would be impacted over a series of driveways.

Single Driveways. Traffic volume and impact characteristics at each study site are shown in Table 36. The right-lane volume ranged from roughly 245 to 820, with an average of roughly 525. The right-turn-in volume ranged from roughly 10 to 245, with an average of roughly 100. The percent of right-lane through vehicles impacted by right turns ranged from roughly 2 to more than 45 percent, with an average of 17 percent.

Figure 18 plots the percent of through vehicles affected as a function of right-turn-in volumes. A good linear relation-

TABLE 36 Volume and impact data for one-hour intervals for each study site

Site No.	Site	Right Lane Volume, V (vph)	Right Turn In Volume, R (vph)	Right Lane Through Volume, V-R (vph)	Right Lane Thru Vehicles Impacted by Right Turn In Actual, I_T (vph)	% of Right Lane Thru Vehicles Impacted by Right Turn In Actual, $I_T/(V-R) = P_1$
1	Edison Square	317	61	256	26	10%
1	Edison Square	395	92	303	52	17%
2	Topps	393	233	160	75	47%
2	Topps	385	245	140	56	40%
2	Topps	464	199	265	66	25%
3	Home Depot	575	48	527	37	7%
3	Home Depot	646	53	593	39	7%
4	Frank's Nursery	676	66	610	42	7%
5	Bradlees	566	153	413	158	38%
5	Bradlees	639	138	501	167	33%
7	Sportsmart & Marshall's	760	132	628	133	21%
7	Sportsmart & Marshall's	719	89	630	96	15%
8	Golf Plaza	418	69	349	67	19%
8	Golf Plaza	456	66	390	49	13%
9	Butera Market Place	472	138	334	89	27%
9	Butera Market Place	452	137	315	76	24%
10	Ancona's Market	244	48	196	25	13%
11	Toys 'R' Us	450	113	337	108	32%
11	Toys 'R' Us	449	80	369	66	18%
12	Kids R Us	719	96	623	81	13%
13	Archway	819	91	728	72	10%
13	Archway	776	125	651	126	19%
14	Price Club	300	51	249	20	8%
14	Price Club	383	63	320	32	10%
17	T.J. Maxx	615	156	459	129	28%
17	T.J. Maxx	585	132	453	92	20%
18	Astoria Federal	364	15	349	12	3%
18	Astoria Federal	696	104	592	72	12%
18	Astoria Federal	557	9	548	9	2%
19	L.I.J. Medical Center	445	57	388	25	6%
19	L.I.J. Medical Center	557	105	452	49	11%
21	King Kullen	688	86	602	60	10%
21	King Kullen	748	86	662	84	13%
22	Warner Lambert	330	78	252	19	8%
22	Warner Lambert	379	66	313	39	12%
	Min	**244**	**9**	**140**	**9**	**2%**
	Max	**819**	**245**	**728**	**167**	**47%**
	Avg	**527**	**99**	**427**	**67**	**17%**

ship exists with a coefficient of determination (R^2) of 0.78. The percentage of through vehicles affected was about 0.18 times the right-turn volume.

A comparison of actual and predicted values is shown in Table 37. In this table, the 38 entries are ranked by increasing right-turn-in volume. The absolute difference between the predicted and actual number of right turns averaged 16. Some 16 site-entries had a difference of fewer than 10 vehicles, and 8 had a difference of between 10 and 20 vehicles.

Multiple Driveways. The percentage of through traffic that would be affected over a ¼-mi road section was derived by extending the preceding analysis. It was estimated that each driveway would have approximately the same right-turn volume. "Through" volumes reflected those vehicles not making right turns at any one driveway.

Four levels of right-turn volumes were derived. Their impacts were based on the following values given on the four "subtotal" rows in Table 37.

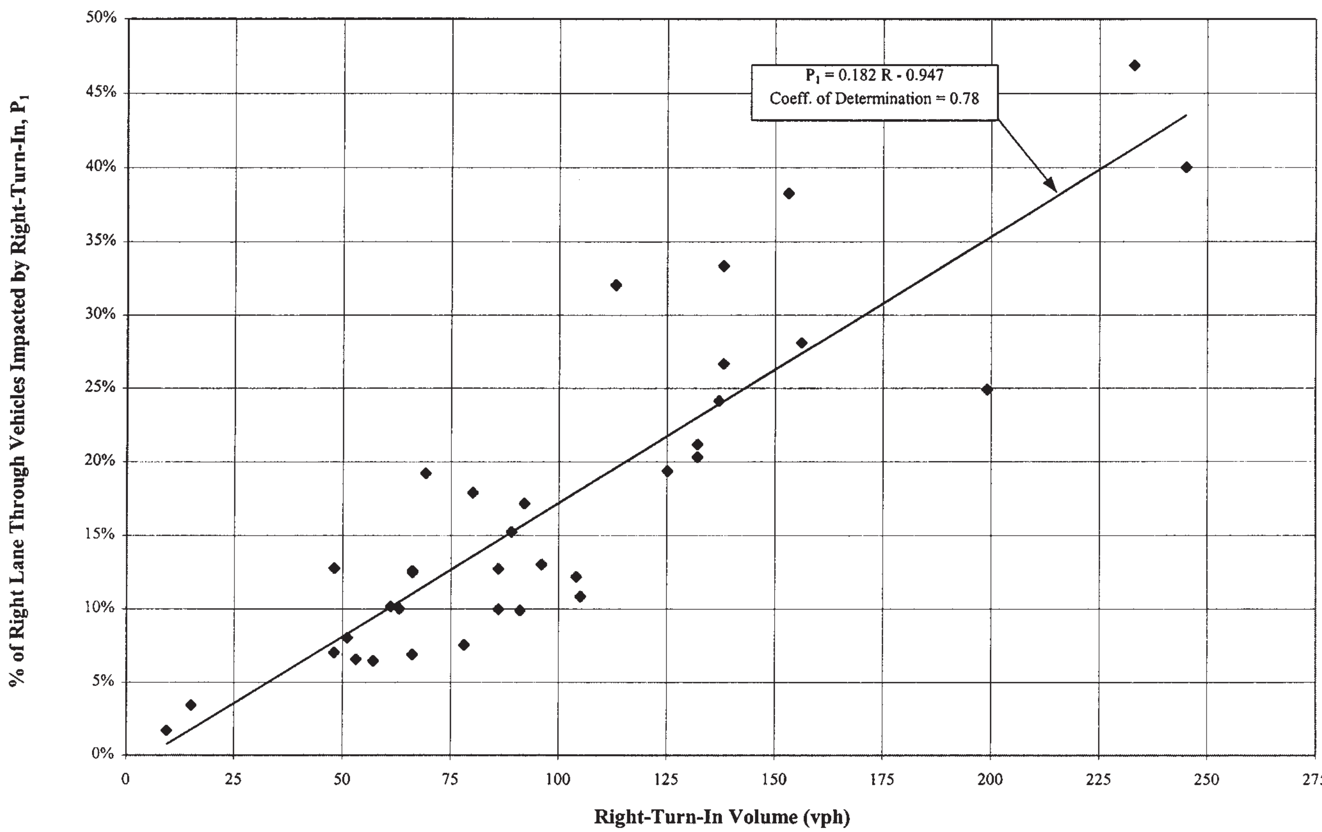


Figure 18. Percent of right-lane through vehicles impacted by right-turn-in vs. right-turn-in volume.

Right-Turn Volume (vph)	*Percent of Through Vehicles in Right Lane Impacted*
<30	2
31–60	7
61–90	12
Over 90	22

The probability of right-lane through vehicles being impacted at least once per ¼ mi was estimated by the formula:

$$p_r = 1 - (1 - P_1)^n \tag{7}$$

where:

n = number of driveways per ¼ mi
P_1 = probability of a through vehicle being impacted at a single driveway

The results of these calculations are given in Figure 19 and Table 38. These values are independent of speeds because they deal only with the percent of right-lane through vehicles affected—not how far back the impact area extends. Thus, if there was a driveway spacing of 100 ft and resultant 13.2 driveways per ¼ mi and a right-turn volume of 30 to 60 vph, about 64 percent of the through vehicles would be affected. If the driveway spacing was increased to 400 ft, 23 percent of the through vehicles would be affected.

Driveway Impact Lengths

The information gathered from the 22 sites was analyzed to identify key patterns of driver behavior. Frequency and cumulative frequency distribution curves were prepared of impact lengths for each site. Figure 20 presents a composite of the cumulative distributions of impact length for all sites. For these plots, the x-axis gives impact length and the y-axis gives the percent of impacted vehicles that are impacted beyond a specified length.

A review of the individual patterns indicated the following impact length characteristics:

- Range (15–425 ft),
- Mean (99–234 ft),
- Median (79–211 ft),
- Mode (90–275 ft), and
- 85th Percentile (116–302 ft).

However, more important than the percentages of affected vehicles is the distribution of impact lengths expressed in terms of the percentages of all right-lane through vehicles, whether affected or not. These values were obtained by multiplying the percentage of right-lane through vehicles affected by right turns for each hour at each study site by its corresponding impact length distribution curve. A similar procedure was used to obtain com-

TABLE 37 Comparison of actual and predicted through vehicles impacted

1	2	3	4	5 = 3 - 4	6	7 = 6/5	8	9 = 8 x 5	10 = 9 - 6	11 = |9 - 6|
Site No.	Site	Right Lane Volume, V (vph)	Right Turn In Volume, R (vph)	Right Lane Through Volume, V-R (vph)	Right Lane Thru Vehicles Impacted by Right Turn In - Actual, I_T (vph)	% of Right Lane Thru Vehicles Impacted by Right Turn In - Actual, $I_T/(V-R)$	% of Right Lane Thru Vehicles Impacted by Right Turn In - Predicted, $\hat{I}_T/(V-R)$	Right Lane Thru Vehicles Impacted by Right Turn In - Predicted, $\hat{I}_T$ (vph)	Difference, Predicted Minus Actual, $\hat{I}_T - I_T$	Absolute Difference, $\|\hat{I}_T - I_T\|$
18	Astoria Federal	557	9	548	9	2%	1%	4	5	5
18	Astoria Federal	364	15	349	12	3%	2%	6	6	6
	Subtotal	**921**	**24**	**897**	**21**	**2%**				
3	Home Depot	575	48	527	37	7%	8%	41	-4	4
10	Ancona's Market	244	48	196	25	13%	8%	15	10	10
14	Price Club	300	51	249	20	8%	8%	21	-1	1
3	Home Depot	646	53	593	39	7%	9%	51	-12	12
19	L.I.J. Medical Center	445	57	388	25	6%	9%	36	-11	11
	Subtotal	**2210**	**257**	**1953**	**146**	**7%**				
1	Edison Square	317	61	256	26	10%	10%	26	0	0
14	Price Club	383	63	320	32	10%	10%	34	-2	2
4	Frank's Nursery	676	66	610	42	7%	11%	67	-25	25
8	Golf Plaza	456	66	390	49	13%	11%	43	6	6
22	Warner Lambert	379	66	313	39	12%	11%	35	4	4
8	Golf Plaza	418	69	349	67	19%	12%	40	27	27
22	Warner Lambert	330	78	252	19	8%	13%	33	-14	14
11	Toys 'R' Us	449	80	369	66	18%	14%	50	16	16
21	King Kullen	688	86	602	60	10%	15%	88	-28	28
21	King Kullen	748	86	662	84	13%	15%	97	-13	13
7	Sportsmart & Marshall's	719	89	630	96	15%	15%	96	0	0
	Subtotal	**5563**	**810**	**4753**	**580**	**12%**				
13	Archway	819	91	728	72	10%	16%	113	-41	41
1	Edison Square	395	92	303	52	17%	16%	48	4	4
12	Kids R Us	719	96	623	81	13%	16%	103	-22	22
18	Astoria Federal	696	104	592	72	12%	18%	106	-34	34
19	L.I.J. Medical Center	557	105	452	49	11%	18%	82	-33	33
11	Toys 'R' Us	450	113	337	108	32%	20%	66	42	42
13	Archway	776	125	651	126	19%	22%	142	-16	16
7	Sportsmart & Marshall's	760	132	628	133	21%	23%	145	-12	12
17	T.J. Maxx	585	132	453	92	20%	23%	104	-12	12
9	Butera Market Place	452	137	315	76	24%	24%	75	1	1
5	Bradlees	639	138	501	167	33%	24%	121	46	46
9	Butera Market Place	472	138	334	89	27%	24%	81	8	8
5	Bradlees	566	153	413	158	38%	27%	111	47	47
17	T.J. Maxx	615	156	459	129	28%	27%	126	3	3
2	Topps	464	199	265	66	25%	35%	93	-27	27
2	Topps	393	233	160	75	47%	41%	66	9	9
2	Topps	385	245	140	56	40%	44%	61	-5	5
	Subtotal	**9743**	**2389**	**7354**	**1601**	**22%**				
	Min	244	9	140	9	2%	1%	4	-41	0
	Max	819	245	728	167	47%	44%	145	47	47
	Avg	527	99	427	67	17%	17%	69	-2	16

Note: Ranked by Right-Turn-In Volume, Column 4

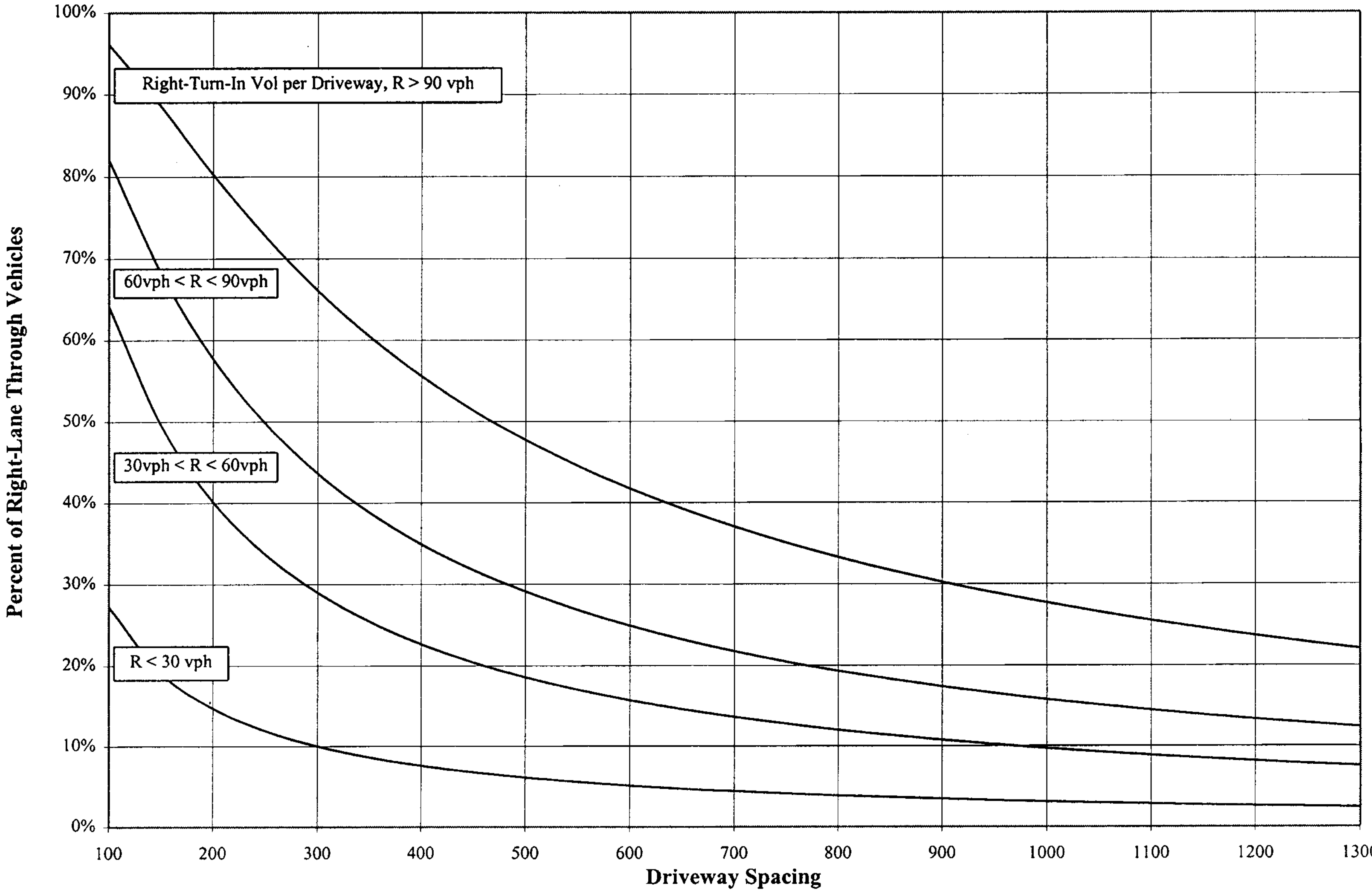


Figure 19. Percent of right-lane through vehicles impacted at least once per ¼ mi.

TABLE 38 Percentage of right-lane through vehicles impacted at least once per ¼ mi

Driveway Spacing (ft)	No. of Driveways per 1/4 Mi., n	% of Right Lane Through Vehicles Impacted at Least Once per 1/4 Mi. = $1 - (1 - P_1)^n$			
		R < 30 vph $P_1 = 2.4\%$	30 vph < R < 60 vph $P_1 = 7.5\%$	60 vph < R < 90 vph $P_1 = 12.2\%$	R > 90 vph $P_1 = 21.8\%$
100	13.2	27.2%	64.1%	82.1%	96.1%
150	8.8	19.1%	49.5%	68.2%	88.5%
200	6.6	14.7%	40.1%	57.6%	80.2%
250	5.3	11.9%	33.7%	49.7%	72.6%
300	4.4	10.0%	29.0%	43.6%	66.1%
350	3.8	8.7%	25.4%	38.8%	60.4%
400	3.3	7.6%	22.6%	34.9%	55.5%
450	2.9	6.8%	20.4%	31.7%	51.3%
500	2.6	6.2%	18.5%	29.1%	47.7%
550	2.4	5.6%	17.0%	26.8%	44.5%
600	2.2	5.2%	15.7%	24.9%	41.7%
650	2.0	4.8%	14.6%	23.2%	39.3%
700	1.9	4.4%	13.6%	21.8%	37.1%
750	1.8	4.1%	12.8%	20.5%	35.1%
800	1.7	3.9%	12.0%	19.3%	33.3%
850	1.6	3.7%	11.4%	18.3%	31.7%
900	1.5	3.5%	10.8%	17.4%	30.2%
950	1.4	3.3%	10.2%	16.5%	28.9%
1000	1.3	3.1%	9.7%	15.8%	27.7%
1050	1.3	3.0%	9.3%	15.1%	26.6%
1100	1.2	2.8%	8.9%	14.5%	25.5%
1150	1.1	2.7%	8.5%	13.9%	24.6%
1200	1.1	2.6%	8.2%	13.3%	23.7%
1250	1.1	2.5%	7.9%	12.8%	22.8%
1300	1.0	2.4%	7.6%	12.4%	22.1%

posite curves. The subtotal rows in Table 37 give group averages for the four classes of right-turn-in volumes previously identified (i.e., less than 30, 31–60, 61–90, and over 90.) These percentage values were applied to the composite cumulative frequency distribution (Figure 20). The results are shown in Figure 21.

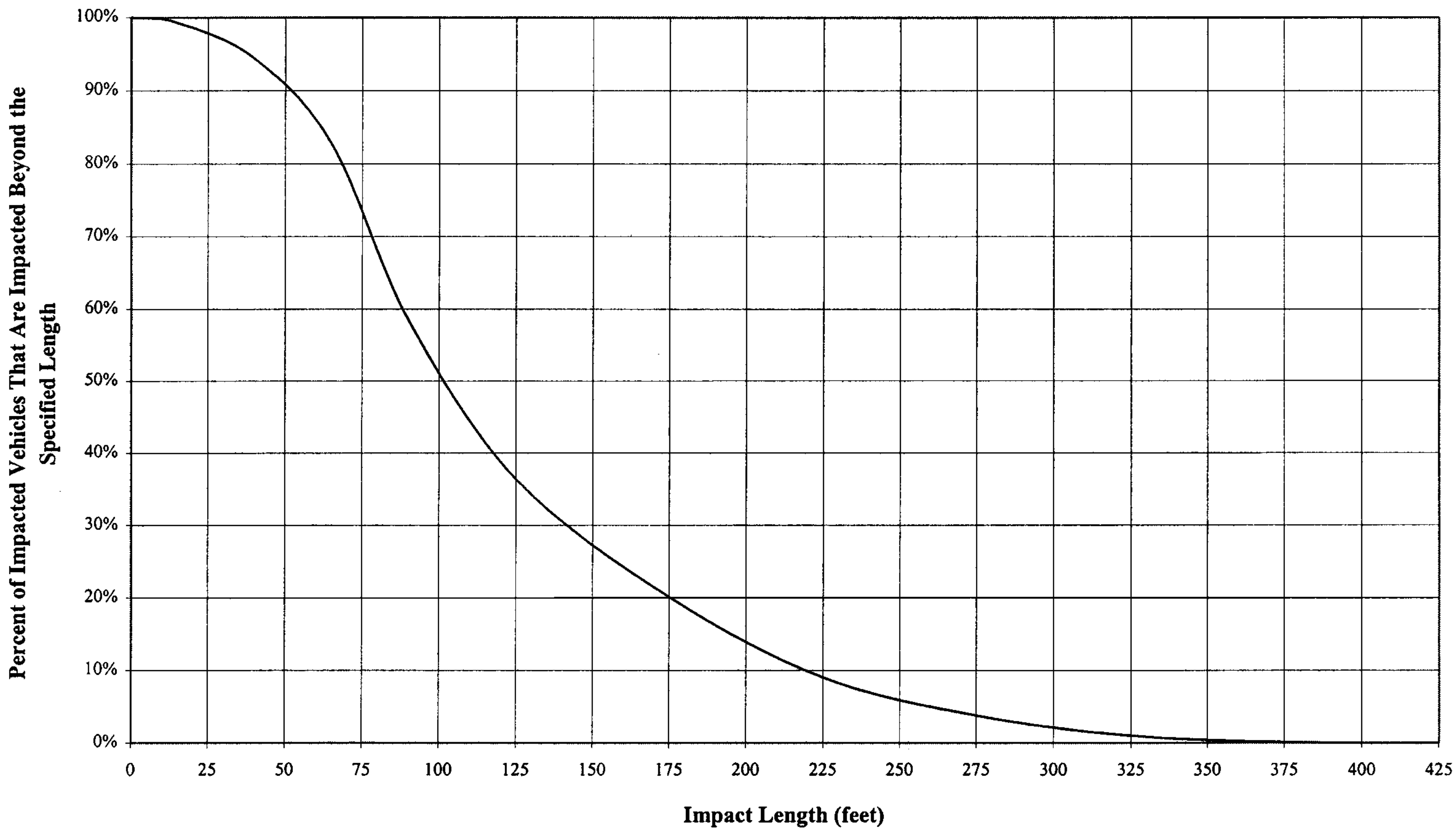
This figure shows the composite curves for the four ranges of right-turn volumes. The curves can be used to estimate the percentage of through vehicles in the right lane that would be affected for various distances from a driveway for each range of right-turn-in volumes. Thus, for a distance of 150 ft upstream of the driveway entrance and a right-turn volume greater than 90 vehicles per hour, roughly 7 percent of the right-lane through vehicles would be affected. At a distance of 100 ft upstream of a driveway, and a right-turn volume of 60 to 90 vehicles per hour, almost 7 percent of the right-lane through traffic would be affected. At a distance of 200 ft, roughly 2 percent would be affected.

Driveway Influence Lengths

The influence lengths or areas associated with various right-turn volumes and driveway frequencies were also established. This involved defining influence length components, determining influence distances for a single driveway at various speeds, and extending the results to a series of driveways.

Influence Distance Concepts. The "influence area" or "influence distance" associated with right turns at a driveway consists of three components. These are as follows:

- The impact length—Determined from field observations.
- Car length—The car length was added because the field observations of impact lengths were taken at the front of each car and the influence length should be measured to the rear of a vehicle. A value of 25 feet was used.



* Arterial posted speeds ranged from 30 to 45 mph for all sites, with an average of 35.6 mph. The relationship between speed and impact length is discussed later.

Figure 20. Cumulative frequency distribution of impact lengths for all impacted vehicles; composite for all sites.

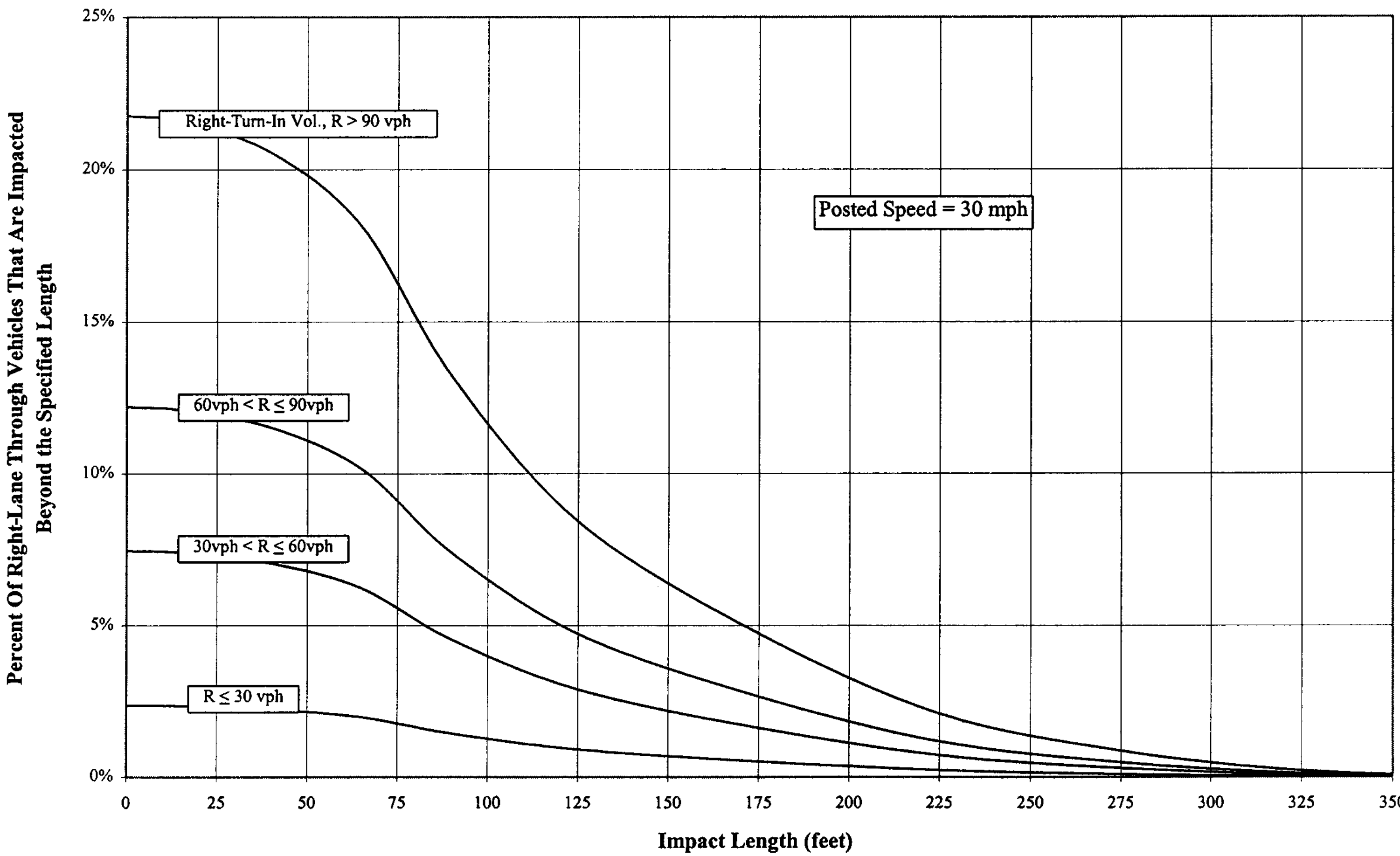


Figure 21. Cumulative frequency distribution of impact lengths for right-lane through vehicles, for aterials classified by right-turn-in volume.

- Perception-reaction distance—A perception reaction time of 2 seconds was used as typical of suburban conditions. (This represents the average of the 1.5 sec and 2.5 sec that AASHTO specified for urban and rural conditions, respectively.)

  The equation for perception and reaction distance is:

  $$d = 1.47\, S\, t$$

  Where:

  d = the perception-reaction distance in feet
  1.47 = the conversion factor from miles per hour to feet per second
  S = the speed in miles per hour
  t = the reaction time in seconds

  A vehicle is considered to be influenced at or beyond another driveway if the influence length is greater than or equal to the driveway spacing, minus the driveway width. Figure 22 shows the situation where the vehicle is not influenced at or beyond another driveway (influence length < driveway spacing – driveway width).

  Influence distances were computed for the study sites. They were based on an average running speed of 30 mph. (Running speed is the travel distance divided by running time—the duration during which a vehicle is in motion.) The resulting influence area (in feet) is:

  Influence Area = Impact Length + Car Length + PIEV distance
  Influence Length = Impact Length + 25 + 30(2)(1.468)
  = Impact Length + 113

Figure 23 shows the cumulative frequency distribution of influence lengths for the four right-turn-in volume groups. It is similar to Figure 21, with the curves shifted 113 ft to the right to account for the above calculation. The posted speeds at the study sites ranged from 30 to 45mph, with an average of 35.6 mph. Therefore, to be conservative, this figure was considered for posted speeds of 30 mph.

Single Driveway. The influence distance of a single driveway will increase as speeds increase. This is because driver behavior is keyed to separation in time (as well as space) and because perception-reaction distances increase as speeds rise. The analysis found that the impact length was related to speed and the distance from the upstream traffic signal.

1. **Length Changes.** Accordingly, the following equation was used to obtain impact lengths for various speeds in suburban settings:

$$L = 0.361[(s - 30)^2 + s] + \sqrt{0.050d + 86.073} \qquad (R^2 = 0.778)$$

where:

L is the mean impact length in feet.
s is the running speed in mph. ($s \geq 30$ mph)
d is the distance in feet from the nearest upstream traffic signal.

This equation predicts the mean impact length for different running speeds. It was used to convert the impact length for any percentile from a running speed of 30 mph to other speeds. The running and posted speeds were considered to be comparable for purposes of calculating impact lengths and influence areas.

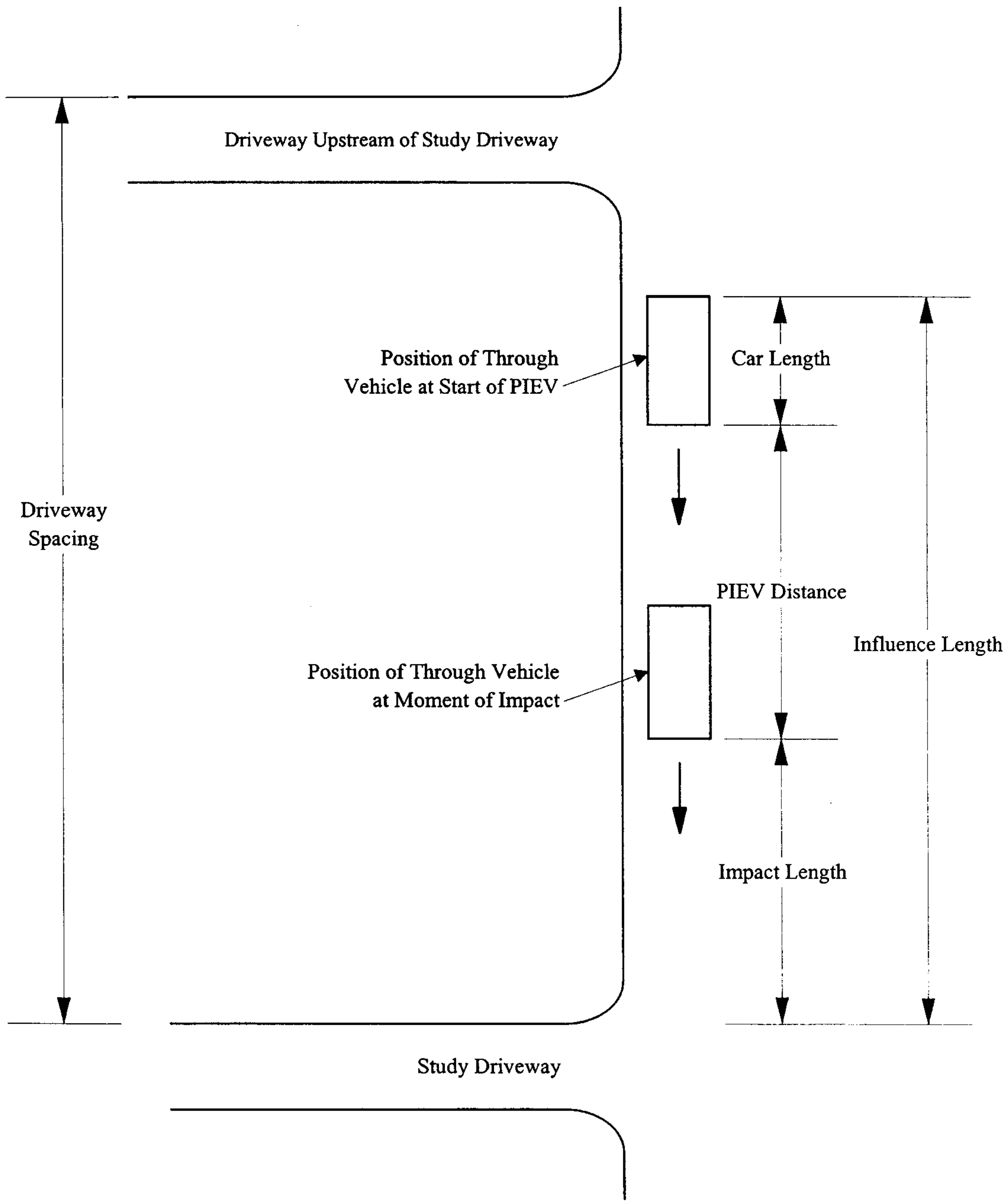
Solving the above equation for "*d*," yields a value of 1,142 ft. Substituting different speed values into the equation while holding "*d*" constant yields their corresponding mean impact lengths. Dividing these numbers by 154 ft, the mean impact length for a posted speed of 30 mph, gives a factor for converting impact lengths at any percentile for a posted speed of 30 mph to impact lengths at the same percentile for any other speed. The results are shown below.

Posted Speed	*Mean Impact Length*	*Factor*	*Speed Ratio*
30	154	1.00	1.00
35	164	1.07	1.17
40	194	1.26	1.33
45	241	1.56	1.50
50	306	1.98	1.67
55	389	2.52	1.83

Thus, to transform the 30-mph impact length curve to that for any other speed, the impact length for each percentile should be multiplied by the factor given above. Alternatively, impact lengths could be estimated based on the ratio of the observed speed to 30 mph. These indexes are also shown.

2. **PIEV Distance and Car Length.** The car length remains constant. The PIEV distances are calculated with a different speed. The resulting values are as follows:

Posted Speed (mph)	*PIEV Distance*
30	88
35	103
40	117
45	132
50	147
55	161



Note: When the Influence Length is less than the Driveway Spacing minus the Driveway Width, the vehicle is not influenced at or beyond another driveway.

Figure 22. Determination of influence length.

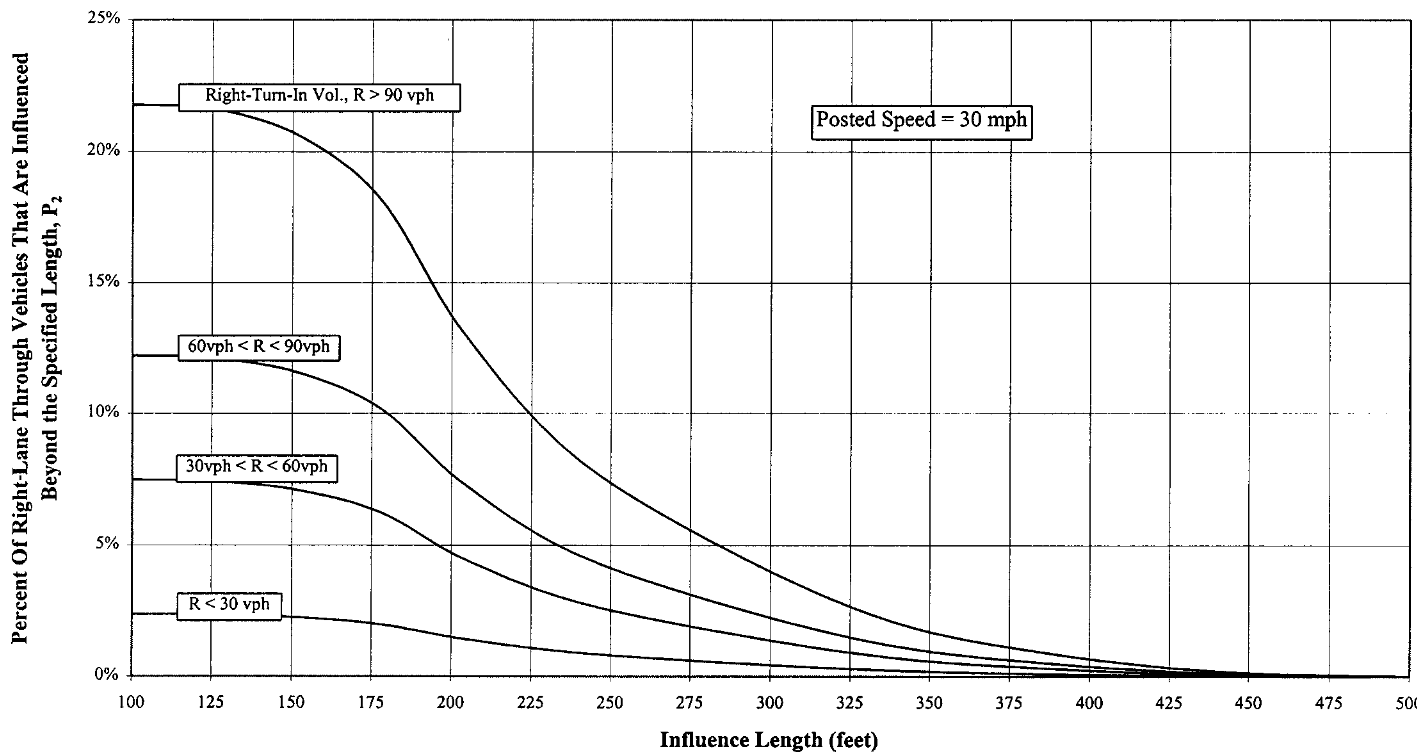
3. **Example.** The following example illustrates these procedures for estimating the influence length to ensure that not more than 10 percent of the through vehicles are influenced beyond a given distance.

   - As shown in Figure 23, for the influence length curve for driveways with right-turn-in volumes greater than 90 vph, the 10 percent value on the y-axis corresponds to 225 ft on the x-axis. This means that 10 percent of the right-lane through vehicles are influenced beyond a distance of 225 ft for a posted speed of 30 mph. This distance may be calculated as follows:

Influence Length = Impact Length + PIEV Distance + Car Length
= 112 + 30 (2) (1.468) + 25
= 225 ft

   - For 35 mph, the influence length is estimated as follows:

Influence Length = Impact Length + PIEV Distance + Car Length
= 112(1.07) + 35 (2)(1.468) + 25
= 248 ft



* Influence length is equal to the impact length (Figure 21) plus the PIEV distance plus the car length.

Figure 23. Cumulative distribution of influence lengths for right-lane through vehicles, for aterials classified by right-turn-in volume.

4. **Influence Areas for a Single Driveway.** Influence length curves for posted speeds of 35 to 55 mi per hour were prepared and are included in Appendix B.

Multiple Driveways. The influence length curves were expanded to assess the effects of multiple driveways. The results are shown in Table 39 for a posted speed of 30 mph. Tables for speeds ranging from 35 mph to 55 mph are included in Appendix B. These exhibits provide useful guides for assessing the effect of a series of driveways on through vehicle performance.

A description of Table 39 follows. The first two columns in this table show driveway spacing in increments of 25 ft and the corresponding number of driveways per 14 mi (defined as n). For each of the four right-turn-in volume per driveway categories, P_2 is the probability of a vehicle being influenced at or beyond another driveway for a single driveway condition. Figure 23 is used to get the P_2 values for any given driveway spacing. Thus, for a 225-ft influence length (30 mph) with more than 90 entering right turns per driveway, 10 percent of the through vehicles would be influenced beyond this distance (Figure 23). When 30 ft are deducted for the driveway width and the figure is reentered at an influence length of 195 ft, the corresponding value for a single driveway is 14.7 percent. This is the value that is entered as P_2 in Table 24 (for $R > 90$ vph and a driveway spacing of 225 ft).

The percentage of through vehicles in the right lane influenced at or beyond another driveway at least once per ¼ mi is derived as follows. The probability of being influenced at or beyond another driveway is P_2. The probability of the complement, not being influenced at or beyond another driveway, is $1 - P_2$. The probability of not being influenced at or beyond another driveway for n driveways is $(1 - P)^n$. If n is the number of driveways per ¼ mi, $(1 - P)^n$ is the probability of not being influenced across another driveway for a ¼-mi segment. The complement of this, the probability of being influenced at least once per ¼ mi, is then $1 - (1 - P)^n$.

The tables provide a means of assessing impacts where a driveway is added, closed, or consolidated. This involves comparing the impact percentages for speed and right-turn volumes for the before and after conditions. This information may be used to identify the cumulative effect of decisions concerning driveway location and unsignalized access spacing.

Comparisons of Results

Table 40 identifies a range of unsignalized driveway spacings that are based upon "spillback," a measure of operational performance. "Spillback," is defined as a right-lane through vehicle being influenced at or beyond the driveway upstream of the analysis driveway. Spillback occurs when the influence length is greater than the driveway spacing minus the driveway width. The spillback rate represents the percentage of right-lane through vehicles that experience this occurrence.

Table 40 presents a comparison of access spacings computed for spillback rates that range from 2 to 25 percent. The more liberal the standard (i.e., the greater the percentage of vehicles influenced at or beyond another driveway), the shorter the required driveway spacing. Similarly, the lower the assumed right-turn volumes (average per driveway), the shorter the allowable spacing. The table also shows New Jersey and Colorado standards for comparative purposes.

TABLE 39 Percentage of right-lane through vehicles influenced at or beyond another driveway: posted speed = 30 mph

Driveway Spacing (ft)	No. of Driveways per 1/4 Mi., n	Right-Turn-In Volume per Driveway, R (vph)							
		R < 30		30 < R < 60		60 < R < 90		R > 90	
		Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$
100	13.2	2.4%	27.3%	7.5%	64.2%	12.2%	82.1%	21.8%	96.1%
125	10.6	2.4%	22.5%	7.5%	56.0%	12.2%	74.7%	21.8%	92.5%
150	8.8	2.4%	19.0%	7.5%	49.4%	12.2%	68.1%	21.7%	88.4%
175	7.5	2.3%	16.1%	7.2%	43.1%	11.8%	61.1%	21.0%	83.1%
200	6.6	2.1%	13.0%	6.6%	36.1%	10.7%	52.6%	19.1%	75.3%
225	5.9	1.6%	9.1%	5.0%	26.2%	8.2%	39.6%	14.7%	60.7%
250	5.3	1.2%	6.0%	3.6%	17.8%	5.9%	27.6%	10.6%	44.7%
275	4.8	0.9%	4.0%	2.7%	12.2%	4.4%	19.3%	7.8%	32.3%
300	4.4	0.6%	2.8%	2.0%	8.6%	3.3%	13.8%	5.9%	23.5%
325	4.1	0.5%	1.9%	1.5%	5.9%	2.4%	9.4%	4.3%	16.3%
350	3.8	0.3%	1.2%	1.0%	3.7%	1.6%	6.0%	2.9%	10.5%
375	3.5	0.2%	0.7%	0.6%	2.2%	1.0%	3.6%	1.9%	6.4%
400	3.3	0.1%	0.4%	0.4%	1.4%	0.7%	2.2%	1.2%	3.9%
425	3.1	0.1%	0.3%	0.3%	0.8%	0.4%	1.3%	0.7%	2.3%
450	2.9	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

TABLE 40 Unsignalized access spacing comparisons

Posted Speed Limit (mph)	Existing Spacing Standards (ft)		Possible Spacing Guideline Based Upon Maximum Allowable Spillback Rate of Right Lane Through Vehicles Influenced by Right-Turn-In At or Beyond Another Driveway at Least Once per 1/4 Mi.											
			2 % Allowed				5 % Allowed				10 % Allowed			
			Right Turn In Vol. per D'way, R (vph)				Right Turn In Vol. per D'way, R (vph)				Right Turn In Vol. per D'way, R (vph)			
	New Jersey DOT	Colorado DOT	R < 30	30<R<60	60<R<90	R > 90	R < 30	30<R<60	60<R<90	R > 90	R < 30	30<R<60	60<R<90	R > 90
30	125	200	320	380	405	430	260	335	360	385	220	290	320	355
35	150	250	345	405	435	460	280	355	385	415	235	310	345	380
40	185	325	380	460	490	520	305	400	430	465	250	340	380	420
45	230	400	430	530	565	610	340	450	495	540	270	380	430	485
50	275	475	490	620	665	725	380	520	575	630	285	425	490	560
55	330	550	550	725	780	855	420	590	665	740	290	480	550	645

Posted Speed Limit (mph)	Existing Spacing Standards (ft)		Possible Spacing Guideline Based Upon Maximum Allowable Spillback Rate of Right Lane Through Vehicles Influenced by Right-Turn-In At or Beyond Another Driveway at Least Once per 1/4 Mi.											
			15 % Allowed				20 % Allowed				25 % Allowed			
			Right Turn In Vol. per D'way, R (vph)				Right Turn In Vol. per D'way, R (vph)				Right Turn In Vol. per D'way, R (vph)			
	New Jersey DOT	Colorado DOT	R < 30	30<R<60	60<R<90	R > 90	R < 30	30<R<60	60<R<90	R > 90	R < 30	30<R<60	60<R<90	R > 90
30	125	200	185	260	295	320	140	245	270	310	110	230	255	295
35	150	250	190	280	315	355	140	260	290	330	110	245	275	315
40	185	325	195	305	345	390	140	285	320	370	110	265	300	345
45	230	400	195	340	385	445	140	315	355	410	110	290	335	385
50	275	475	195	380	435	510	140	345	400	470	110	315	370	435
55	330	550	195	420	490	580	140	380	445	530	110	330	415	495

APPLICATION GUIDELINES

This section contains guidelines for assessing the safety, travel time, and operations impact of unsignalized access spacing. It also suggests guidelines for identifying the need for right-turn deceleration lanes and for establishing access separation distances. Finally, it identifies the emergent planning and policy implications. The various guidelines reflect, build upon, and apply the information contained in previous sections of this report.

Safety Impacts

The generalized effects of access spacing on accidents can be estimated by applying the accident rate indexes shown in Table 41. The composite indexes show the relative increase in accidents that can be expected as the total driveway density in both directions increases. These indexes suggest that doubling the access frequency from 10 to 20 per mile would increase accident rates by 40 percent. A road with 60 access points per mile would have triple the accident rate (200 percent increase) as compared with a spacing of 10 access points per mile. Each additional access point increases the accident rate by about 4 percent.

Figures 24 and 25 present accident rates by median type and total access density (both directions) for urban-suburban and rural roadways, respectively. These are shown for the midpoints of the unsignalized access spacing groups and reflect adjustments to eliminate apparent inconsistencies in the reported data. In urban and suburban areas, each access point (or driveway) added would increase the annual accident rate by 0.11 to 0.18 on undivided highways and by 0.09 to 0.13 on highways with TWLTLs or nontraversable medians. In rural areas, each access point (or driveway) added would increase the annual accident rate by 0.07 on undivided highways and 0.02 on highways with TWLTLs or nontraversable medians.

Representative accident rates by signalized and unsignalized access density are shown in Figure 26 for urban and suburban areas. These rates contain adjustments to account for apparent inconsistencies.

Each unsignalized driveway may add about 0.02 to the accident rate at low signal densities and from 0.06 to 0.11 at higher signal densities.

The rates in Figure 26 may be used to estimate the changes associated with increasing unsignalized access density at any given signal density (driveways to single-family residences should be excluded). However, the figure should not be used to estimate the effects of adding signals. This is because in deriving the rates, signal density served as a surrogate for cross-street traffic.

States may underestimate accidents along sections of roadway with both heavy ADTs and driveway traffic because there is a greater proportion of nonreportable accidents. Therefore, care should be exercised when these rates are applied along heavily traveled roadways in large metropolitan areas. In such cases, basic accident rates should be obtained; the values in the table should be used to assess the differential cumulative impact of adding driveways.

TABLE 41 Suggested accident indices for unsignalized access spacing

Access Points Per Mile*	Literature Synthesis (Fig. 4-5)	Safety Analysis	Suggested Value
10	1.0	1.0	1.0
20	1.3	1.4	1.4
30	1.7	1.8	1.8
40	2.1	2.1	2.1
50	2.8	2.3	2.5
60	4.1	2.5	3.0
70	-	2.9	3.5

* Total for both directions.

The following procedure may be used to estimate the cumulative impacts of changing unsignalized access spacing along a section of road:

Given: Actual Accident Rate = A
Existing Driveways Per Mile = D_1
Existing Signals/Mile = S_1
Proposed Driveways Per Mile = d_2
Obtain: Estimated existing and future rates (R_1 and R_2) from Figure 26.
Apply: The ratio of R_2/R_1 to the actual rate A.

The following example will help to illustrate the application of this procedure.

The actual accident rate on a roadway with three signals per mile and 18 driveways per mile is 7.0 accidents per million VMT. An additional 12 driveways are planned, resulting in a total of 30 driveways per mile.

The projected accident rate is calculated as follows using Figure 26 to estimate R_1 and R_2.

$$\text{Projected Rate} = \text{Actual Rate} \times \frac{R_2}{R_1} = 7.0 \times \frac{5.6}{4.5}$$
$$= 8.7 \text{ acc./VMT}$$

Travel Times

The travel times along unsignalized multi-lane divided highways with no traffic signals can be estimated by the procedures developed by Reilly (*59*) and set forth in the 1994 *Highway Capacity Manual*. Speeds are estimated to be reduced 2.5 mph for every 10 access points up to a 10 mph reduction for 40 access points per mile. The procedure takes into account those access points on one side of a roadway. However, if access points on the opposite side of the roadway have a significant effect on traffic flow, they may be included in determining access point density.

More detailed analysis by Reilly showed a speed reduction of 0.15 mph per access point and 0.005 mph per right-turning movement per mile of road (see Table 42). Thus, for 40 access points per mile and 400 right turns per mile, the speed reduction would be 8.0 mph. When the right-turn volume increases to 600, the speed reduction becomes 9 mph. The 1994 HCM value in both cases is 10 mph.

Operations Impacts

Operations impact procedures and estimates are set forth for the following:

- Travel times;
- Vehicles "impacted" by a single driveway and by multiple driveways; and
- Influence area lengths, including spillback implications across upstream driveways.

Through Vehicles Impacted

The percentage of through vehicles in the right lane that would be impacted at a single driveway is approximately 18 percent of the right-turn volume. This typically applies wherever the total right-lane volume ranges from 250 to 800 vph. The approximate percentages of right-lane through vehicles impacted for various right-turn volumes are as follows:

Right-Turn Volume	*Percent of Right-Lane Through Vehicles Impacted at a Single Driveway*
≤ 30	2
31–60	7
61–90	12
> 90	22

Table 43 extends these results to a series of driveways along a ¼-mi section of road. The right-turn volumes should represent the averages for the study section. Thus, for a 200-ft driveway spacing and right-turn volume of 40 vph per driveway, about 40 percent of the vehicles in the curb lane *not* turning right would be impacted at least once. Similarly, for a 400-ft spacing and 80 right turns per hour per driveway, about 35 percent of the right-lane through vehicles would be impacted.

Table 44 summarizes the cumulative distributions of impact distances.

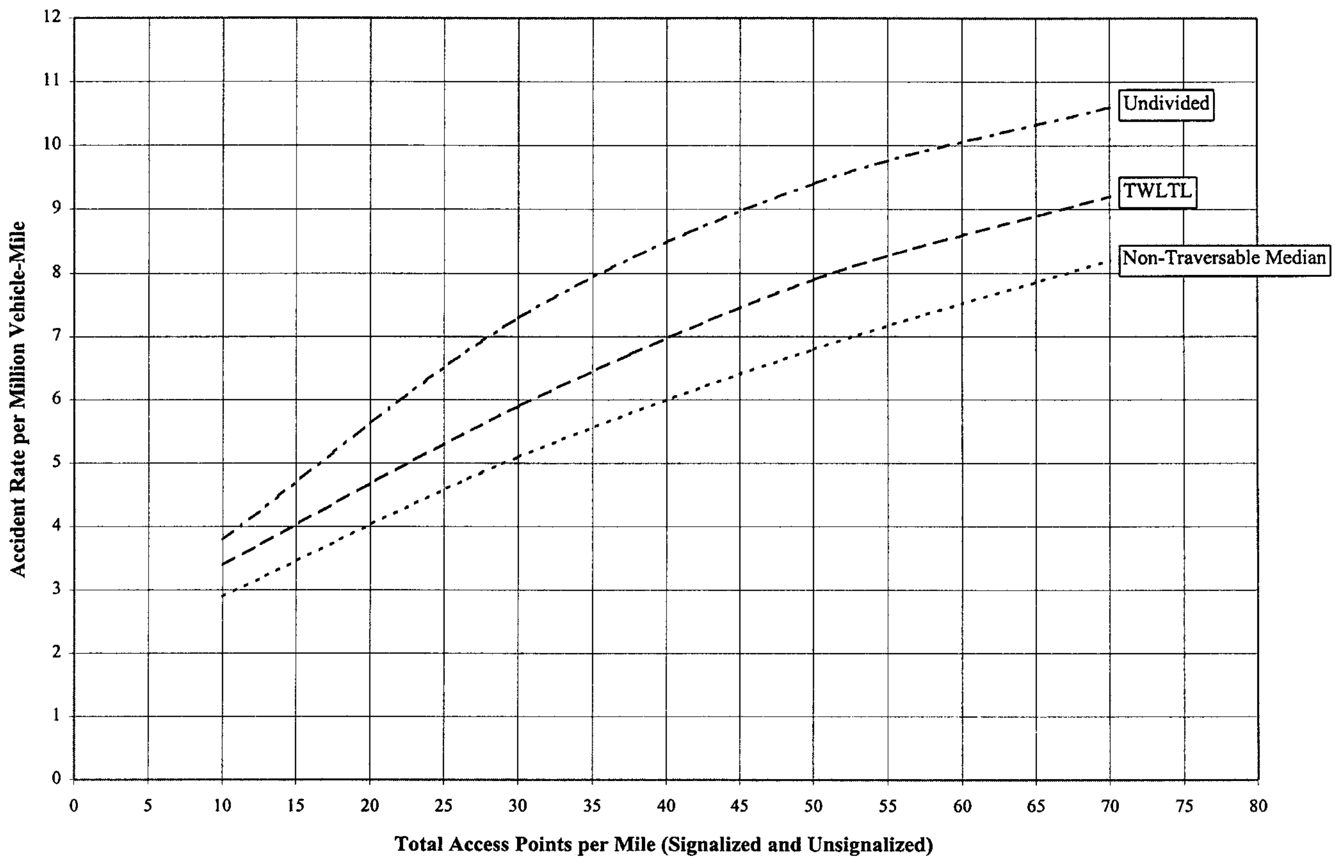


Figure 24. Estimated accident rates by type of median–urban & suburban areas.

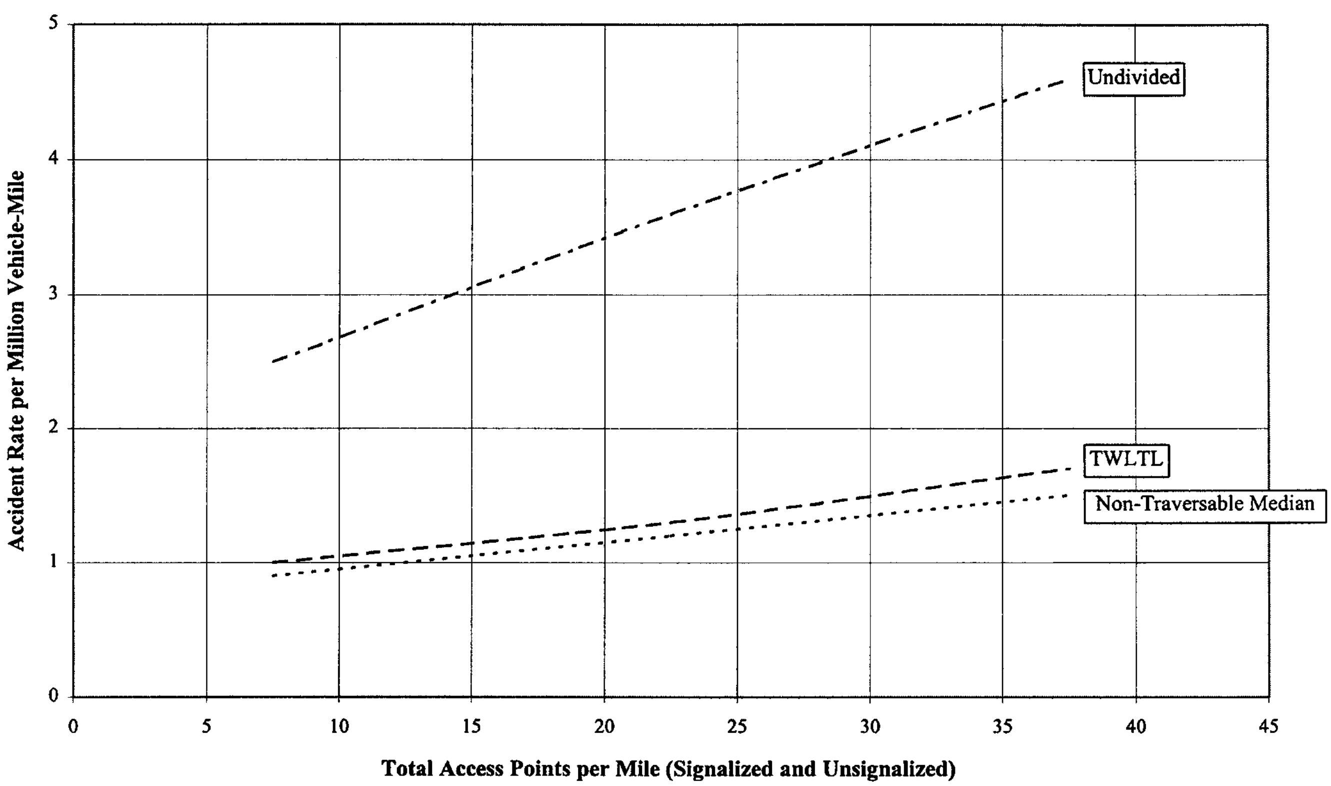


Figure 25. Estimated accident rates by type of median–rural areas.

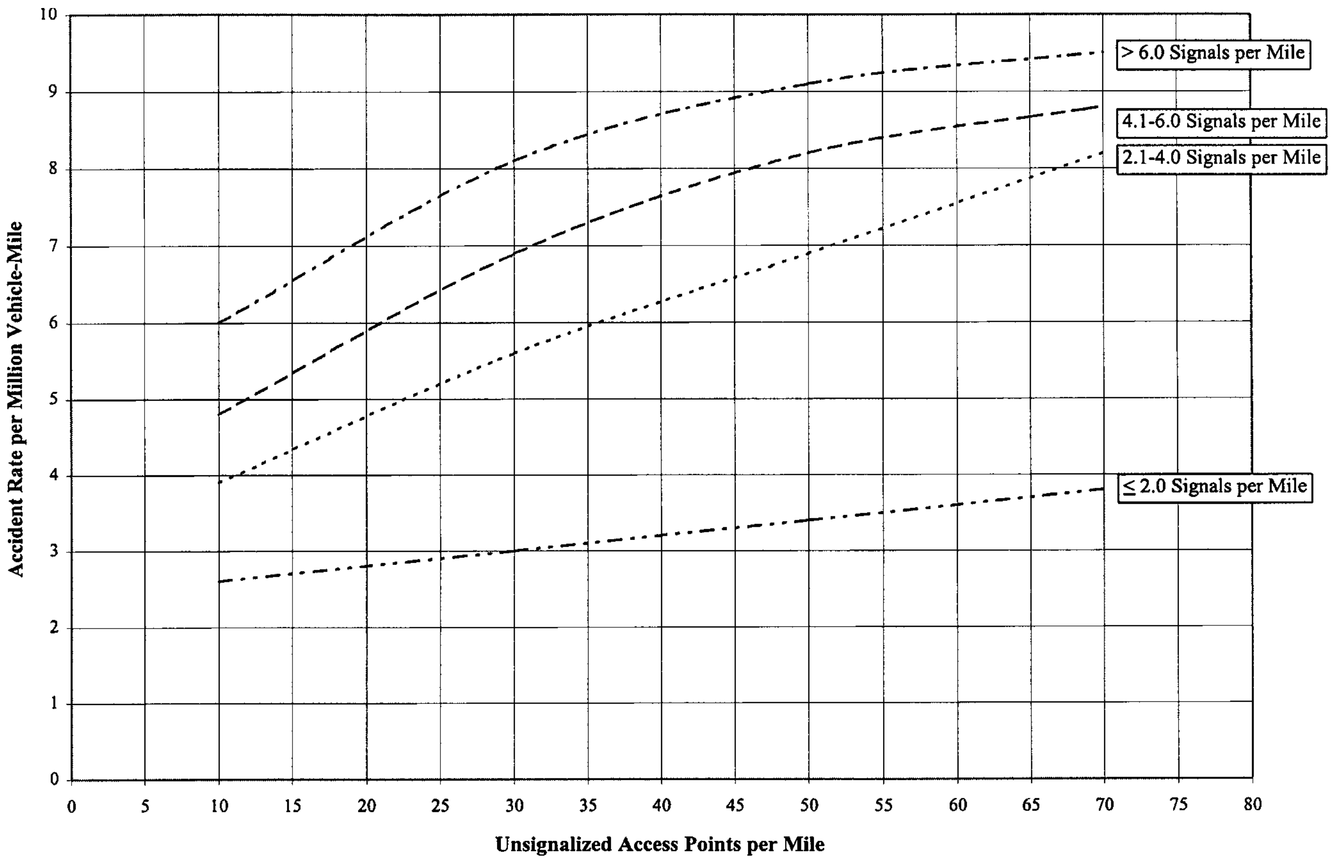


Figure 26. Estimated accident rates by access density–urban & suburban areas.

Influence Distances

The influence distances add driver perception-reaction distances and car lengths to the impact lengths. The exhibits in Appendix B to this report give the percentage of right-lane through vehicles that would be influenced at or beyond a single driveway for various posted speeds and right-turn lane volumes and driveway spacings. Table 45 presents the results for a 45-mph posted speed.

The use of this (and related tables in the appendix) is straightforward:

1. Select the table with the appropriate posted speed. In this case, the speed is 45 mph.
2. Estimate (or obtain) the number of right turns entering a driveway during the peak hour.
3. Where there are several driveways along a section of road, estimate the average right-turn volume per hour per driveway and the number of driveways per ¼ mi.
4. Look up the appropriate entry in the table to obtain the likely percent of through vehicles influenced by driveway traffic, both at individual driveways and in the section of road.

This information may be used to identify the cumulative impact of making decisions concerning driveway location and unsignalized access spacing.

Two examples using Table 45 follow:

1. There are six driveways in the ¼-mi section to be analyzed. This corresponds closely to a 225-ft spacing. The average driveway right-turn entry volume is 40 vph. At a spacing of 225 ft and a right-turn volume of 40 vph, 7.3 percent of the right-lane through traffic would be influenced at or beyond a single driveway. However, 36.0 percent would be influenced at or beyond at least one of the driveways in the ¼-mi section.
2. If the driveways were consolidated so there were three driveways per ¼ mi, the impacts would be as follows: three driveways per ¼ mi results in close to a 450-ft spacing. The average right-turn-in volume per driveway would be 80 vph. At this spacing, 2.9 percent of the right-lane through vehicles would be influenced at or beyond an individual driveway and 8.2 percent of the vehicles would be influenced at or beyond at least one of the driveways in the ¼-mi section. Thus, driveway consolidation has reduced the impact.

Illustrative Applications

Illustrative applications, reflecting the results of the safety and operations analysis, were developed for deceleration lanes and for unsignalized access spacing.

TABLE 42 Speed reductions for uninterrupted multi-lane arterials

Access Points/ Mile	Speed Loss Per Access Point (mph)	Right-Turn Volume Per Mile (Per Hour): 100	200	300	400	500	600	900	HCM @ .25 Speed Loss Per Access
		Speed Loss (mph): 0.5 [b]	1.0	1.5	2.0	2.5	3.0	4.5	
		Combined Speed Loss (mph)							
1	0.15 [a]	0.65 [c]	1.15	1.65	2.15	2.65	3.15	4.65	1.25
5	0.75	1.25	1.75	2.25	2.75	3.25	3.75	5.25	1.25
10	1.50	2.00	2.50	3.00	3.50	4.00	4.50	6.00	2.50
15	2.25	2.75	3.25	3.75	4.25	4.75	5.25	6.75	3.75
20	3.00	3.50	4.00	4.50	5.00	5.50	6.00	7.50	5.00
30	4.50	5.00	5.50	6.00	6.50	7.00	7.50	9.00	7.50
40	6.00	6.50	7.00	7.50	8.00	8.50	9.00	10.50	10.00

Note: Numbers within box represent sum of marginal totals (i.e. (c) = (a) + (b)).
Source: *(59)*

Deceleration Lanes

Right-turn deceleration lanes are desirable to remove turning vehicles from through travel lanes, thereby reducing speed differentials and minimizing effects and delays to through vehicles. The percentage of through vehicles in the right lane that must slow down or change lanes provides one possible basis for establishing right-turn lanes. Safety must be a major consideration in decisions to provide a right-turn lane.

For arterial right-lane volumes of 250 to 800 vph, the percentage of through vehicles impacted was about 0.18 times the right-turn volume. This results in the following impacts; these may provide a basis for decisions regarding provision of right-turn lanes:

% Right-Lane Through Vehicles Impacted	*Right-Turn-In Volume vph (Approximate)*
0	0
2	10
5	30
10	60
15	85
20	110

A criterion of 2 percent impacted suggests a minimum right-turn volume of 10 vph. This may be applicable in certain rural settings. A criterion of 10 percent impacted suggests a minimum of approximately 60 vph. A criterion of 20 percent suggests a right-turn volume of 110 vph. The latter criteria may be applicable in certain urban areas. The

TABLE 43 Percentage of right-lane through vehicles impacted at least once per ¼ mi

Driveway Spacing (Feet)	Driveways Per Quarter Mile	Percent of Right-Lane Through Vehicles Impacted at Least Once Per Quarter Mile — Right Turn Volume (VPH): ≤ 30	31-60	61-90	Over 90
100	13.2	27.2	64.1	82.1	96.1
200	6.6	14.7	40.1	57.6	80.2
300	4.4	10.0	29.0	43.6	66.1
400	3.3	7.6	22.6	34.9	55.5
500	2.6	6.2	18.5	29.1	47.7
600	2.2	5.2	15.7	24.9	41.7
900	1.5	3.5	10.8	17.4	30.2
1,200	1.1	2.6	8.2	13.3	23.7
1,320	1.0	2.4	7.5	12.2	21.8

TABLE 44 Cumulative distribution of impact distances

Distance Upstream from Driveway (Feet)	Right-Turns Volume (VPH) ≤ 30	31-60	61-90	Over 90
0	2.4	7.5	12.2	21.8
50	2.0	6.9	11.1	19.8
100	1.4	4.0	6.5	11.7
150	0.7	2.2	3.5	6.4
200	0.4	1.2	1.8	3.3
250	0.1	0.5	0.8	1.4
300	0.0	0.2	0.3	0.5
350	0.0	0.0	0.0	0.0

length of the deceleration is a function of the impact length and storage requirements.

Access Separation Distances

Both operational and safety considerations should govern unsignalized access spacing. A third consideration is the access classification of the roadways involved.

Direct property access along strategic and principal arterials should be discouraged. However, where access must be provided, adequate spacing should be established to maintain safety and preserve movement.

Figure 27 compares the access separation distances for a range of spillback rates with the standards for Colorado and New Jersey. Except for posted speeds of less than 40 mph, the two methods produce values that fall between the New

TABLE 45 Percentage of right-lane through vehicles influenced at or beyond another driveway: posted speed = 45 mph

		Right-Turn-In Volume per Driveway, R (vph)							
		$R < 30$		$30 < R < 60$		$60 < R < 90$		$R > 90$	
Driveway Spacing (ft)	No. of Driveways per 1/4 Mi., n	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$
100	13.2	2.4%	27.3%	7.5%	64.2%	12.2%	82.1%	21.8%	96.1%
125	10.6	2.4%	22.5%	7.5%	56.0%	12.2%	74.7%	21.8%	92.5%
150	8.8	2.4%	19.1%	7.5%	49.6%	12.2%	68.2%	21.8%	88.5%
175	7.5	2.4%	16.6%	7.5%	44.4%	12.2%	62.6%	21.8%	84.4%
200	6.6	2.4%	14.6%	7.5%	40.0%	12.2%	57.5%	21.7%	80.1%
225	5.9	2.3%	12.9%	7.3%	36.0%	11.9%	52.6%	21.3%	75.5%
250	5.3	2.2%	11.3%	7.0%	32.0%	11.5%	47.5%	20.5%	70.2%
275	4.8	2.1%	9.7%	6.6%	27.9%	10.8%	42.1%	19.2%	64.1%
300	4.4	1.8%	7.8%	5.8%	23.0%	9.4%	35.3%	16.8%	55.5%
325	4.1	1.5%	5.8%	4.6%	17.5%	7.5%	27.3%	13.5%	44.4%
350	3.8	1.2%	4.4%	3.8%	13.5%	6.1%	21.2%	11.0%	35.4%
375	3.5	1.0%	3.4%	3.1%	10.3%	5.0%	16.5%	8.9%	28.0%
400	3.3	0.8%	2.6%	2.5%	8.0%	4.1%	12.9%	7.3%	22.1%
425	3.1	0.7%	2.1%	2.1%	6.5%	3.5%	10.4%	6.2%	18.0%
450	2.9	0.6%	1.6%	1.8%	5.1%	2.9%	8.2%	5.2%	14.4%
475	2.8	0.5%	1.3%	1.4%	3.9%	2.3%	6.3%	4.2%	11.1%
500	2.6	0.4%	0.9%	1.1%	2.9%	1.8%	4.7%	3.2%	8.3%
525	2.5	0.3%	0.7%	0.8%	2.1%	1.4%	3.4%	2.5%	6.1%
550	2.4	0.2%	0.5%	0.6%	1.5%	1.0%	2.5%	1.8%	4.4%
575	2.3	0.2%	0.4%	0.5%	1.1%	0.8%	1.8%	1.4%	3.2%
600	2.2	0.1%	0.3%	0.4%	0.8%	0.6%	1.3%	1.1%	2.3%
625	2.1	0.1%	0.2%	0.3%	0.6%	0.4%	0.9%	0.8%	1.6%
650	2.0	0.1%	0.1%	0.2%	0.4%	0.3%	0.6%	0.5%	1.1%
675	2.0	0.0%	0.1%	0.1%	0.2%	0.2%	0.4%	0.3%	0.7%
700	1.9	0.0%	0.0%	0.1%	0.1%	0.1%	0.2%	0.2%	0.4%
725	1.8	0.0%	0.0%	0.0%	0.1%	0.1%	0.1%	0.1%	0.2%
750	1.8	0.0%	0.0%	0.0%	0.0%	0.0%	0.1%	0.1%	0.1%

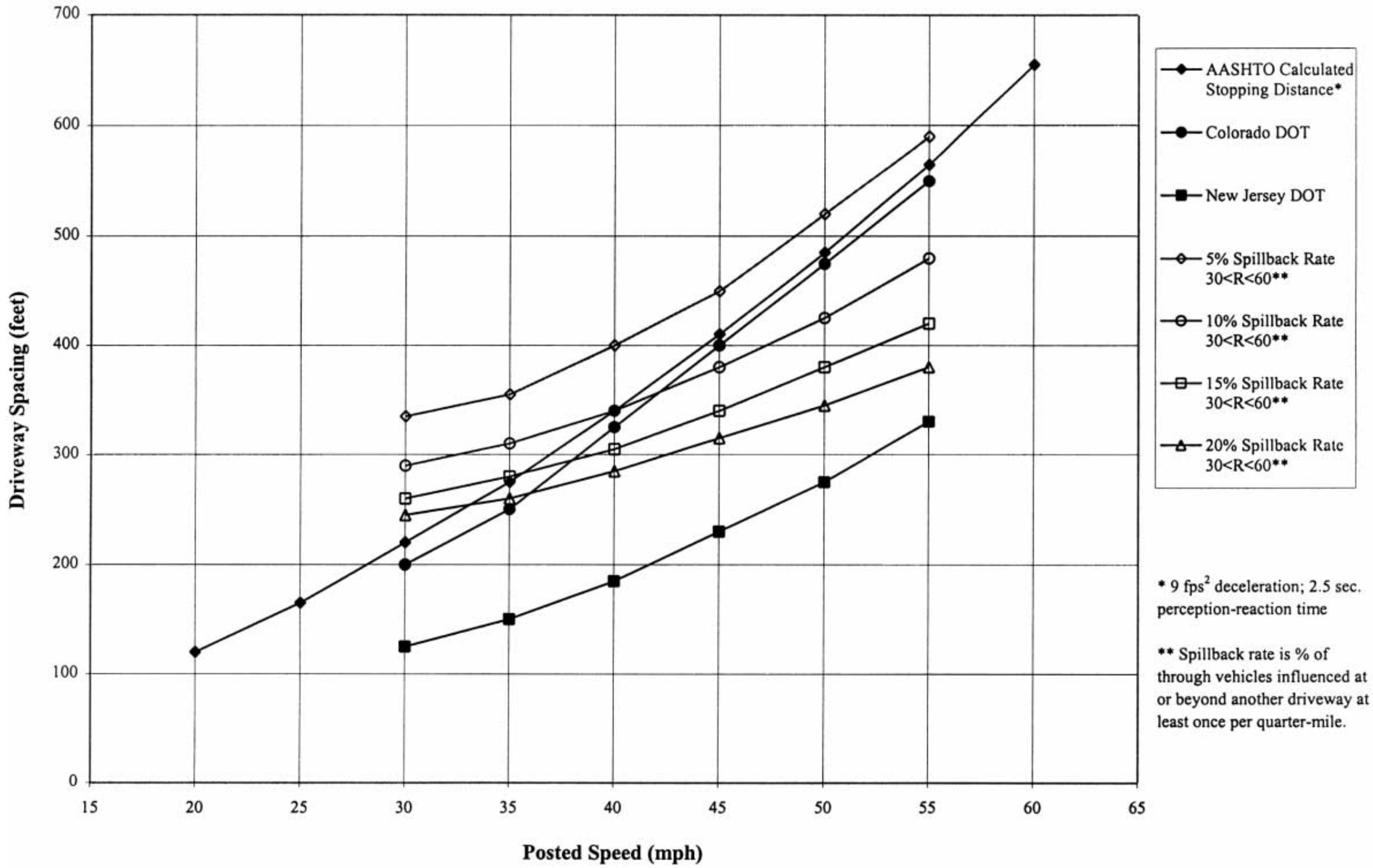


Figure 27. Comparison of access separation criteria.

TABLE 46 Access separation distances based on 10-percent and 20-percent spillback

Posted Speed (mph)	SPILLBACK RATE**			
	5%	10%	15%	20%
30	335	265 (a)	210 (b)	175 (c)
35	355	265 (a)	210 (b)	175 (c)
40	400	340	305	285
45	450	380	340	315
50	520	425	380	345
55	590	480	420	380

(a) Based on 20 driveways per mile.
(b) Based on 25 driveways per mile.
(c) Based on 30 driveways per mile.

* Based on an average of 30 to 60 right turns per driveway.
** Spillback occurs when a right-lane through vehicle is influenced to or beyond a driveway upstream of the analysis driveway.
The spillback rate represents the percentage of right-lane through vehicles experiencing this occurrence.

Jersey and Colorado (AASHTO safe stopping sight distance values) criteria.

Access separation distances, based on an average driveway volume of 30 to 60 vph, are shown in Table 46 for spillback rates of 5, 10, 15, and 20 percent. For the lower speeds of 30 and 35 mph, the access separation distance shown is based on the safety implications of driveway density. For roadways with a primary function of mobility, there should not be more than 20 to 30 connections per mile (both directions).

As shown in Table 46, for a posted speed of 40 mph, the access spacing would range from 285 ft to 400 ft, depending on which spillback rate was selected. For a posted speed of 50 mph, the access spacing would range from 345 ft to 520 ft, depending on the spillback rate.

CHAPTER 5

CORNER CLEARANCE CRITERIA (TECHNIQUE 1C)

INTRODUCTION

Corner clearances represent the minimum distances that are (or should be) required between intersections and driveways along arterial roads. As stated in the AASHTO *A Policy on Geometric Design of Highways and Streets:* "Driveways should not be situated within the functional boundary of at-grade intersections. This boundary would include the longitudinal limits of auxiliary lanes."

Inadequate corner clearances can result in traffic-operation, safety, and capacity problems. These problems can be caused by blocked driveway ingress and egress movements, conflicting and confusing turns at intersections, insufficient weaving distances, and backups from far-side driveways into intersections.

Specific operational and safety problems include

- Through traffic is blocked by vehicles waiting to turn into a driveway.
- Right or left turns into or out of a driveway (both on artery and crossroad) are blocked.
- Driveway traffic is unable to enter left-turn lanes.
- Driveway exit movements are impacted by stopped vehicles in left-turn lanes.
- Traffic entering an arterial road from the intersecting street or road has insufficient distance.
- The weaving maneuvers for vehicles turning onto an artery and then immediately turning left into a driveway are too short.
- Confusion and conflicts resulting from dual interpretation of right-turn signals.

This chapter summarizes relevant literature pertaining to corner clearances and the key findings of eight case studies and identifies planning and operational actions to improve operations and safety.

STUDIES AND ANALYSES

A few studies have explored the operational and safety aspects of corner clearances. McCoy and Heimann (*64, 65*) assessed the impacts of corner clearances on saturation flows. Long and Cheng-Tin (*66*) derived formulas for estimating required corner clearance distances, and Kaub (*67*) derived a model for estimating corner clearance distances.

Studies were conducted by McCoy and Heimann (*64, 65*) at two locations in Lincoln, Nebraska, to evaluate the effects of driveway traffic on saturation flow rates at signalized intersections. The studies were based on more than 400 pairs of departure and prevailing headways in the curb lane. Some 148 headway pairs were for queues of passenger cars that passed straight through the intersection. The authors found that driveway traffic can reduce the saturation flow rate on signalized intersection approaches. The amount of the reduction was found to depend on the corner clearance of the driveway and the proportions of curb-lane volume that enter and exit the driveway. The authors suggested that additional studies should be performed for a wider range of driveway and approach conditions.

Long and Cheng-Tin (*66*) developed an analytical model for determining desired corner clearances. The model was patterned after the 1985 *Highway Capacity Manual* in establishing an initial minimum corner clearance (IMCC) and then adjusting for prevailing conditions. The model, expressed analytically, is

$$MCC_i = IMCC_i \times \Pi f_i \qquad (8)$$

where:

MCC_i = corner clearance for traffic conditions
$IMCC_i$ = initial minimum corner clearance
Πf_i = product of individual adjustment factors for facility type, median type, driveway channelization, driveway width, driveway volumes (daily and peak hours), coincidence of driveway and arterial peak period volumes, driveway corner turning speed, and curb-lane widths.

The individual adjustment factors generally ranged from about 0.80 to 1.50; composite factors would affect the unadjusted values by as much as 20 percent.

Separate minimum corner clearance distances were derived for saturated and unsaturated conditions. The larger of these distances would govern the minimum corner clearances.

The minimum initial corner clearance for saturated conditions represents the distance at which there would be no

TABLE 47 Initial minimum corner clearance distances for undersaturated conditions

Speed (MPH)	IMCC $_{uf}$ (feet)
30	325
35	425
40	525
45	630
50	750
55	875
60	1005

Sources: *(66)* and *(67)*

increase in saturation flow rate and hence no loss in intersection capacity. It was approximated by the space per vehicle multiplied by the effective green time per cycle and then divided by the saturation headway. A saturation of 2 sec and a space per vehicle of 25 ft results in an initial corner clearance of 250 ft and 500 ft for 20 and 40 sec of effective green time per cycle, respectively.

For undersaturated conditions, the model assumes that safety and traffic operational needs govern corner clearances. Stover's (*68*) minimum functional distances for desirable conditions were assumed to represent the initial corner clearance for undersaturated conditions. These values are listed in Table 47.

Several points are worth noting regarding this analysis. First, there is little basis for assessing the various adjustment factors, the validity of the basic models, and the practicality of the results. Second, it does not consider queuing that would decrease as the green per cycle increases. Third, it focuses on establishing spacing guidelines for corner clearance—not assessing effects.

Kaub (*67*) developed an access spacing model based upon driver perception-response times and vehicle dynamics. His "Access Management Accident" (AMA) models reflect driver perception-reaction times, acceleration rates, and braking rates for both through and turning vehicles. When a driveway is located on the "far" side of an intersection, allowance is made for accelerating right-turn vehicles at the street intersection. Table 48 gives minimum access distances to protect right-turn vehicles in deceleration. The illustrative values are generally higher than AASHTO safe stopping sight distances.

POLICIES AND PRACTICES

Current corner clearance criteria were assembled for selected cities, counties, and states. These criteria, summarized in Table 49, show a wide range of practices. Corner clearances range from as little as 16 ft (urban area in Iowa) to more than 300 ft (Colorado). Many fall within the 100- to 200-ft range.

Case Study Overview

Case studies of corner clearances were prepared to illustrate current practices, problems, and opportunities. The case study examples were generally selected by public agencies and included sites in Florida, Michigan, New Jersey, Nevada, and New York. Available roadway geometry, traffic volumes, and accident histories were reviewed. Corner clearance distances were scaled from maps and plans. Several examples had features that eliminated them from more detailed evaluations: some had unusual intersection geometry that would limit their applicability elsewhere; others were located along collector rather than arterial roads; and sometimes, insufficient background information was available.

TABLE 48 Access management accident (AMA) model example minimum access distance to project right-turn vehicles in deceleration

Operating Speed (mph)	Design Speed (mph)	Radius (feet)				AASHTO Stopping Sight Distance
		25	50	75	Average	
25	30	130 [(1)]	136 [(1)]	147 [(1)]	137	125
30	35	254	237	232	241	150
35	40	324	301	293	306	200
40	45	402	374	361	379	225-250
45	50	488	454	438	460	275-325
50	55	582	543	522	549	325-400
55	60	684	639	615	646	450-550

Notes: Using 8.5 and 3.2 ft/sec 2 for deceleration of lead and following vehicles, respectively.
Curves speeds and distances controlled as in Case 3 above.
(1) Values cited seem inconsistent.

Source: *(68)*

TABLE 49 Summary of corner clearance criteria

Government Unit	Criteria
Collier County, FL	With Median: 75 to 115 ft. upstream, 100 to 230 ft. downstream Without Median: 100 to 230 ft. upstream, 100 to 230 ft. downstream.
Colorado DOT	325 feet from intersection for 40 mph.
Florida DOT	75 to 115 ft. upstream, 100 to 230 ft. downstream.
Ingham County, MI	125 feet from intersection.
Iowa DOT	16 feet from intersection in urban area.
Maine DOT	Suggested spacing in urban areas: Signalized intersections - 115 to 230 feet Unsignalized intersections - 85 to 115 feet Suggested spacing in rural area is double the above
New York DOT	Approximately 35 to 75 feet from the intersection.
North Dakota DOT	Signalized intersections: Local - 50 ft., Collector-85 to 175 ft., Arterial 115 to 230 ft. Unsignalized intersections: Local - 50 ft., Collector - 75 to 85 ft., Arterial 85 to 115 ft.
New Jersey	50 ft. unsignalized/100 ft. signalized.
Oshtemo Township, MI	75 to 250 feet from intersection.
Palm Beach, FL	75 to 125 feet from intersection.
Pennsylvania DOT	"follows AASHTO criteria."
Texas DOT	"AASHTO green book for corner clearances without medians."
Virginia DOT	50 desirable, 25 feet minimum from intersections.
Washington DOT	Varies depending on classification of road.

Source: Agency Surveys

The case studies focused on typical four-leg and 'T' right-angle intersections and exhibited a broad range of practices. They contained a mix of lot frontages and land uses. Most were in suburban environments.

Salient characteristics of the case studies analyzed are summarized in Table 50. Some key observations follow:

- Corner clearance distances varied widely depending upon specific circumstances; they ranged from 2 to 250 ft. The distances were measured based on the methods used by the governing agency because the definition of corner clearance varied. The measurement from near-edge-to-near-edge was the most common, but centerline-of-intersecting-street to centerline-of-driveway and other methods were found. Ideally, a uniform method of measuring should be adopted.
- Queuing or spillback across driveways was reported to be the most pervasive problem. However, in a few cases, backups into intersections occurred when heavily used driveways were too close to intersections.
- The near-side queuing problem is compounded by several related factors: heavy traffic, multi-phase traffic signals, and the ability to turn left into or out of driveways. The left-turn problem can be alleviated by installing physical medians on multi-lane roads.
- Roadway widening to increase capacity sometimes reduces corner clearance requirements.
- Placing driveways too close to intersections appears to result in a higher rate of accidents. Accident data, where available, indicate a high incidence of driveway-related accidents. At the Okemos Road-Jolly Road intersection in Ingham County, 34 percent of reported/recorded accidents were driveway-related. At the Western Avenue-New Kamer Road intersection in Guilderland (Albany), New York, 50 percent of accidents were driveway-related. Along Pemberton Road south of Broad Street

TABLE 50 **Summary of corner clearance case studies**

Location	Roadways	Corner Clearance*		Comments
		Near-Side	Far-Side	
Plantation (Broward County), Florida	University Drive and Peters Road	25 ft. before 140 ft. after	10 ft. before 150 ft. after	Good Practice - Driveways closest to intersection were closed in conjunction with roadway widening.
Alaiedon/Meridian Townships, (Ingham County), Michigan	Okemos and Jolly Roads	Varies; 25 to 85 ft.	Varies; 102 to 185 ft.	Poor Practice - Traffic backs up across driveways. Thirty-four percent of accidents were driveway-related. Left turns to/from across two-way left-turn lanes. Narrow lot frontages preclude adequate corner clearances.
Delta Township, Michigan	West Saginaw Highway and Elmwood Road	70 ft.	250 ft.	Poor Practice - Driveway on near side extends beyond stop-line at skewed multi-phase signal-controlled intersection. Traffic backs up across this drive. Access to/from left-turn lanes difficult. Corner clearance and subdivision standards are not consistent.
Lewis and Clark County, Nevada	Sahara Avenue and Decatur Boulevard	Varies; 36 to 250 ft.	Varies; 12 to 200 ft.	Poor and Good Practice - Spillback reported across near-side driveways with little clearance that were established before 1992 standards. New standards are less than those for other areas, but were contested by City of Las Vegas. Providing good standards before development reduces need for retrofit.
Aberdeen, New Jersey	State Route 35 and Cliffwood Avenue	125 ft.	18 ft.	Poor Practice - Left turns conflict with driveway traffic with some spillback into intersection. Some queuing across driveway on approach to intersection.
Stafford, New Jersey	State Route 72 and East Road	50 ft.	2 ft.	Poor Practice - Increased traffic volumes caused by rapid developments in corridor resulted in roadway widening and reduced corner clearances. State proposal to close far-side driveway challenged in court.
Guilderland, New York	Western Avenue and New Kamer Road	Varies; 40 to 250 ft.	Varies; 80 to 200 ft.	Good and Poor Practice - Fifty percent of 53 accidents occurring between 1992 and 1995 were driveway-related. Most driveways exceed region's 80-foot spacing and many exceed state's proposed 120-foot spacings; heavy traffic flows result in queues across near-side driveways.
Henrico County, Virginia	Broad Street Road and Pemberton Road	90 ft.	45 ft.	Poor Practice - Of 35 driveway accidents along Pemberton Road (Jan. 1991 - Nov. 1994), 34 involved left-turning vehicles. There were 16 injuries. Traffic backups across driveways contributed to accidents. Property owners resisted moving driveway to edge of property line and/or banning left turns.

* Dimensions are measured based on the methodology used by the governing agency. There is no uniform methodology for measuring corner clearance.

Road in Henrico County, Virginia, 35 accidents occurred at a single driveway during a 4-year period. However, most accident reporting systems do not provide a sufficient level of detail to identify driveway/corner clearance-related accidents.

- The likelihood of accidents and poor intersection operations increases when there are multiple sites with poor corner clearances that add many conflict points at the same intersection (e.g., as at Okemos and Jolly Roads in Ingham County, Michigan).
- Corner clearances are often limited by the dimensions of the properties involved. Even then, it is sometimes possible to relocate driveways near the property line farthest from the intersection. It also may be possible for the corner lot to have access via an easement through the next lot.
- Improving or "retrofitting" minimal corner driveway distances is not always possible or practical; it is often opposed by property owners and may be costly to implement. Efforts to combine, close, or relocate driveways frequently meet with resistance from property owners involved, especially where service stations or retail outlets rely heavily on pass-by traffic.
- The added costs of driveway closure, relocation, or modification or the reconfiguration of site circulation make retrofitting expensive. Implementing improved corner clearances through retrofitting has varied impacts on owners' perception of how easily customers can reach their businesses.
- For existing developments, standards may not be easily met because of typically small corner lot frontages in older communities. Land use and zoning practices throughout the United States historically created small corner lots with narrow frontages. Small corner frontages exist along many arterials, must be acknowledged, and cannot routinely be brought up to "best practice" standards. Other approaches to improving intersection safety, capacity, and overall performance should be considered (e.g., either joint or shared access with adjacent lots or frontage/rear access roads).
- There are several ways to address insufficient corner spacing distances. One is to provide, as in New Jersey, a limitation on maximum vehicular usage (based on type of environment—urban or rural—and total site acreage); another is to provide alternative and joint access, as in Ingham County, Michigan, where the corner clearance distance on a main roadway was increased by using a joint access point for one of the sites.
- One of the most important lessons observed is that adequate corner clearance distances can be achieved with the least impact and cost when they are required before land subdivision and site development approval.

APPLICATION GUIDELINES

The examination of corner clearance should distinguish between near-side and far-side requirements. Near-side corner clearance requirements should consider the spillback or queuing across a driveway at a traffic-signal-controlled intersection. Far-side clearances should provide adequate separation between vehicles turning onto a roadway and those entering or leaving a driveway. Ideally, both requirements should be met. However, too often there are inadequate property frontages, and/or highway-oriented land uses require corner locations with proximate access.

The following principles should guide corner clearance and driveway planning:

- Ideally, no driveways should be permitted off of major highways. This requires safe and convenient alternative access and reasonable internal site circulation.
- Where this is not possible, major highways should have physical (restrictive) medians to preclude left turns. Each corner parcel should have one driveway per roadway that is placed as far from the intersections as possible.
- Along undivided major highways, it is desirable to eliminate left-turn ingress and egress at driveways within the "influence area" of an intersection. This may entail providing short sections of a median divider and/or adopting a driveway design that discourages or prevents left-turn maneuvers.
- Driveways should be located as far from the intersection as possible—either at or within 10 ft of the property line furthest from the intersection.

Actions vary for retrofits and new facilities. Corrective retrofit actions include

- Locating driveways at the farthest edge of the property line from the intersection;
- Consolidating driveways with adjacent properties, thereby increasing corner clearances;
- Closing driveways along the arterial and requiring property access from the secondary road; and
- Installing a raised median barrier on approaches to intersections to preclude left turns into or out of a driveway.

From a planning perspective, two actions should be encouraged; both require a proactive approach to corner clearances:

- Establishing the desirable location of access points before property is subdivided or developed and
- Establishing minimum requirements for property frontages in zoning and subdivision regulations.

CHAPTER **6**

MEDIAN ALTERNATIVES (TECHNIQUES 2A, 2B, & 3C)

INTRODUCTION

The treatment of roadway medians has important bearing on how well roadways operate, their accident experience, and the access they provide to adjacent developments. The basic choices for designing the medians are

- Whether to install a continuous TWLTL,
- Whether to install a nontraversable (physical) median on an undivided roadway, and
- Whether and when to replace a TWLTL with a nontraversable median.

This basic decision process is illustrated in Figure 28. This chapter contains an integrated analysis of the three median techniques. This chapter also

- Reviews and synthesizes the extensive literature that describes the safety impacts of median options,
- Presents available information on the operational features and benefits of the three options,
- Describes the various safety and operational models that have been developed, and
- Suggests guidelines and parameters for assessing impacts.

TWLTLs and medians improve traffic operations and safety by removing left turns from through travel lanes. TWLTLs provide more ubiquitous access and maximize operational flexibility. Medians physically separate opposing traffic, limit access and conflicts, and provide better pedestrian refuge. Median design requires adequate provision for left turns and U-turns to avoid problems associated with concentrating these movements at signalized intersections.

SAFETY EXPERIENCE

Many studies have analyzed the safety benefits of installing TWLTLs or nontraversable medians on undivided highways and replacing TWLTLs with nontraversable (barrier) medians. This section summarizes the individual studies and compares their results for each of the three median options. Its primary focus is on accident rates for various median types and designs drawn from the recent literature. However, it also draws from significant research conducted in the 1950s and 1960s. These older studies have been included in the literature review for completeness, but are omitted from the summary conclusions.

The various studies assess safety in two ways. Some studies (particularly those where TWLTLs or medians were installed on "undivided" highways) report results of before-and-after comparisons for a given facility. Many studies, however, compare accident experience and rates on highways with different cross-sections (i.e., medians versus TWLTLs).

The before-and-after studies assume that there is little (or no) change in roadway geometry or traffic conditions other than the left-turn treatment. This approach has been used for many years by traffic engineers in assessing the benefits of various treatments. However, some researchers have suggested that benefits can be overstated unless sites for treatment are selected randomly.

The comparative approach evaluates accident histories for various sites with given midblock left-turn treatments. The comparisons imply that conditions are essentially similar at the various sites except for the median treatments—a situation that is not generally realized. Therefore, differences between sites are examined statistically through regression-based procedures.

Both types of studies indicate that accident rates are reduced when TWLTLs or medians are introduced on undivided multi-lane highways. Most studies, and the models derived from them, also suggest that safety is improved where physical medians replace TWLTLs.

The following sections summarize the results of various studies by type of treatment.

Two-Way Left-Turn Lanes (TWLTLs)

The first continuous TWLTLs were reported to have been installed in Michigan. Since then, they have been widely applied as a means of improving traffic flow on 2-lane and 4-lane undivided roadways. Their application was especially widespread on roadways in developing suburban areas with intensive commercial developments and frequent driveways. They also have been applied to 6-lane roads. Many perceive

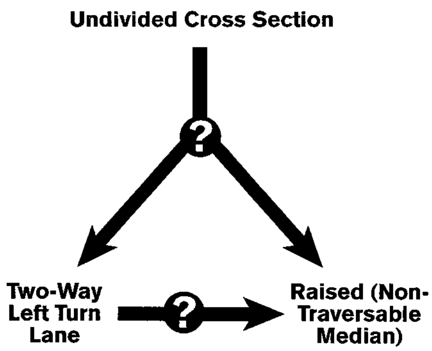


Figure 28. Median decision choices.

TWLTLs as a compromise between no median and a curbed one, especially where right-of-way is limited.

The center lane is used for left turns in both directions of travel. At signalized intersections, there is a transition to conventional left-turn treatments (Figure 29). Pavement markings are sometimes supplemented with overhead signs. Where there are many driveways along an arterial, the TWLTL area obviates the need for continued transitions for left-turning vehicles such as found with conventional left-turn lane designs. The head-on accident collision-potential, occasionally perceived as a problem, has not been realized in most situations.

A literature review found 15 studies that contained information on accident frequencies, types, and rates associated with TWLTLs. Table 51 compares the results of the post-1970 studies in terms of changes in numbers of accidents and in rates. Many of the variations reflect the different ways that data were collected and accidents were reported. Reductions in total accidents were reported in 9 out of 10 cases, with a median reduction of about 33 percent. Reductions in accident rates were reported in 10 out of 12 entries; the median reduction was about 38 percent. The Bowman-Vecellio data for suburban arterials dramatically conflicts with the other findings and may reflect how the data were obtained.

Table 52 summarizes the accident reductions by type of accident. Consistent reductions were reported in rear-end, sideswipe, head-on, and fixed-object accidents. Left-turn accidents generally decreased. Right-angle and other accidents showed no consistent benefits.

Medians

Nontraversable medians are an important means of managing access along multi-lane highways (Figure 30). They may be continuous between street intersections, provide access for left-turning vehicles only ("directional") or permit opening for all traffic ("full").

Medians have several important safety benefits. They physically separate opposing directions of travel, thereby virtually eliminating head-on accident potentials. They control (sometimes eliminate) left turns and other movements across the median. This translates into fewer conflicts, greater safety, and more uniform arterial speeds. However, these benefits may be offset by the increased turning volumes at median openings—especially at nearby signalized intersections where left turns may be transferred (especially where medians are continuous between street intersections). Where left turns from the arterial are permitted, it is essential that the medians provide separate lanes with ample storage. Otherwise, the safety and capacity benefits associated with removing the turns from the through-travel lanes will be lost.

Many studies over the years have shown that divided highways (i.e., highways with a nontraversable raised median) experience lower accident rates than undivided

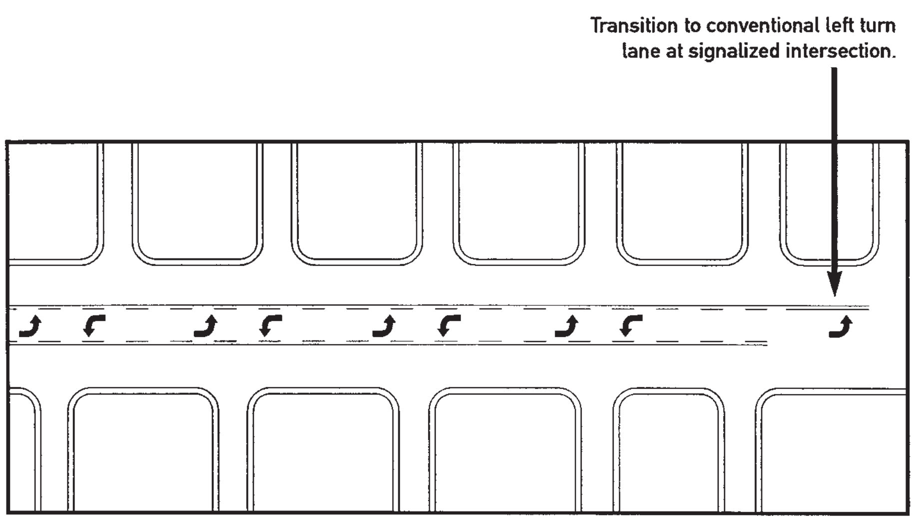


Figure 29. Continuous two-way left-turn lane.

TABLE 51 Accident experience with TWLTLs

Study & Location		YEAR	ACCIDENTS Undivided	ACCIDENTS TWLTL	ACCIDENTS % Diff.	ACCIDENT RATES (Per Million VMT) Undivided	ACCIDENT RATES (Per Million VMT) TWLTL	ACCIDENT RATES (Per Million VMT) % Diff.	REMARKS	SOURCE
1 Busbee		1974	-	-	-38	-	-	-	Before and after study.	*(69)*
2 Southern Section ITE		1975	-	-	-31	-	-	-	Before and after study.	*(70)*
3 Burritt and Coppula (Arizona)		1978	-	-	-36	-	-	-	Seven locations. Before and after study.	*(71)*
4 Walton, Horne, Fung (Texas)		1978	-	-	-33	-	-	-	Before and after study.	*(72)*
5 Parker (Virginia)	(1)	1983	-	-	-	6.79	6.11	-9	14 four-lane undivided sections 17 sections with traversable medians	*(73)*
6 Thakkar (Illinois)		1984	824 222	558 130	-32 -41	90.8 53.3	54.3 28.6	-40 -46	15 five-lane sections. Before and after study. 16 three-lane sections. Before and after study.	*(74)*
7 Harwood and St. John	(1)	1985	-	-	-	3.14 1.79	0.86 0.26	-73 -85	2-lane highways; 7 sites with TWLTL compared to 4 without 4 sites Before/After	*(75)*
8 Harwood (California) Harwood (Michigan)	(1) (1)	1986	-	-	-	2.06 1.79	1.28 1.89	-38 6	Non-intersection accidents/commercial land use	*(76)*
9 Institute of Transportation Engineers		1986	2,479	1,788	-28	-	-	-36	30-road stretches. Before and after study.	*(77)*
10 Kuhlmann (Metro Toronto)		1987	-	-	-	-	-	-21	11-road sections. Before and after study.	*(78)*
11 Box (Illinois)		1989	174	104	-40	-	-	-	4-lane urban arterials. Before and after study.	*(79)*
12 Long (Florida)	(1)	1993	-	-	-	4.44	3.20	-28	4-lane urban arterials	*(80)*
13 Bowman-Vecellio (Arizona, CA, GA)	(1)	1994	2,751 4,487	2,181 15,110	-21 236.7	9.92 4.23	5.56 6.89	-44 63	15-road sections. {CBD arterials and suburban arterials, respectively}	*(81)*

Note: (1) These represent rates for different sections of roadway.

TABLE 52 Accident experience by type of accident with TWLTLs percent difference

Study & Location	YEAR	DATA COMPARED	Rear-End	Sideswipe	Right Angle	Left Turn	Head-on	Fixed Object/ Fixed Parked Vehicle	Other	REMARKS	SOURCE
1 Busbee	1974	Frequency		-90							*(69)*
2 Burritt and Coppula (Arizona)	1978	Frequency	-45	-100	same direction	-20	-67	-65	-30[(1)]		*(71)*
				-52	opposite direction						
3 Walton, Horne, Fung (Texas)	1978	Frequency	-45	-	-	-	-42	-	-		*(72)*
4 Thakkar (Illinois)	1984	Rates	-34 [(2)]	-26	-		-	-	-	5 lanes	
			-40 [(2)]	-45	-		-	-	-	3 lanes	*(74)*
5 Long, Gan and Morrison (Florida) (1)	1993	Midblock Rates	-24	-47	-16	-27	-46		37[(3)]	4 lanes	*(80)*

Notes:

[(1)] Pedestrians

[(2)] Includes left turns

[(3)] Right turns

(1) This study compares different sections or groups of roadways.

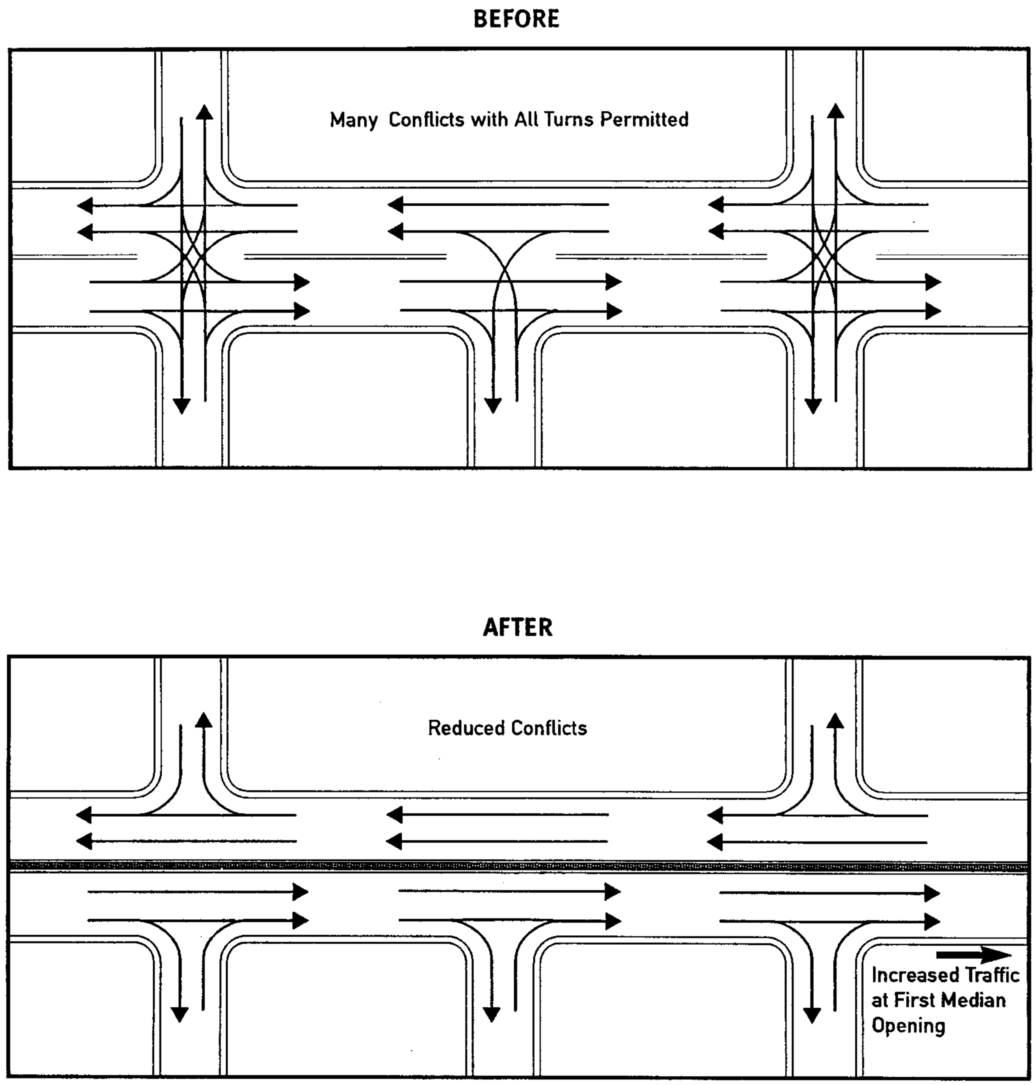


Figure 30. Reduction in conflicts by installation of continuous nontraversable median on a previously undivided highway.

highways. This is because the median allows fewer opportunities for conflicts and erratic movements. Table 53 compares the results of the post-1970 studies. Accident reductions were reported in most cases. Accident rates were reduced in all studies, with a median reduction of about 35 percent.

The accident rates for 10 undivided highways ranged from 1.11 to 11.28 per million VMT. The mean was 5.29; standard deviation was 3.43; and coefficient of variation was 61 percent. The accident rates for highways with medians ranged from 0.94 to 7.43; the mean was 3.34; standard deviation was 2.17, and coefficient of variation was 61 percent. A student 't' test indicates that the means are significant at the .06 level.

Several observations are of interest:

- The low accident rates for the Rhode Island data, both with and without medians are for a fully controlled access facility.
- The median data for Minnesota and Utah include partially controlled access roadways.
- The Bowman-Vecellio data for suburban arterials appear to understate the benefits of introducing medians.

Replacing TWLTLs with Nontraversable Medians

A current access management and safety concern in many communities is where and when nontraversable medians should replace TWLTLs. TWLTLs have improved safety (and traffic flow) by removing left turns from the through traffic lanes. Therefore, they have been widely used to provide access to closely spaced, low-volume commercial driveways along arterial roads. But from an access management perspective, they increase rather than control access opportunities. For this reason, a number of highway

TABLE 53 Synthesis of median safety experience

Study & Location	YEAR	Accidents Undivided	Accidents Median	Accidents % Diff.	Accident Rates (Per Million VMT) Undivided	Accident Rates (Per Million VMT) Median	Accident Rates (Per Million VMT) % Diff.	REMARKS	SOURCE
1 Parker (Virginia)	1983	-	-	-	6.79	4.42	-35	19 - median section 14 - four-lane section	*(73)*
2 Arlington, Texas	1983	-	-	-66	-	-	-	4-lane roads	*(82)*
3 New York State (1)	1984	-	-	-	11.28	7.43	-34	Six-lane roads Statewide study	*(83)*
4 Murthy (Rhode Island)	1992	31	29	-7	1.11	0.94	-15	2-Lane road-controlled access	*(84)*
5 Long, Gan, Morrison (Florida) (1)	1993	-	-	-	4.44	2.09	-53		*(80)*
6 Bowman-Vecellio (1) (Arizona, California, Georgia)	1994	2,751	1,714	-38	9.92	6.42	-35	15 sections CBD	*(81)*
		4,487	7,663	71	4.23	3.79	-10	Suburban	
7 Harwood et al	1995								
California - Urban (1)		-	-	-	3.59	2.58	-28	Statewide study, includes uncontrolled	*(85)*
California - Rural (1)		-	-	-	2.13	1.15	-46	access highways only.	
Minnesota - Rural (1)		-	-	-	7.14	2.37	-67	Statewide study includes highways with	
Utah - Rural (1)		-	-	-	2.27	2.22	-2	partial access control or with no control.	

Note: (1) These studies compare different sections or groups of roadways.

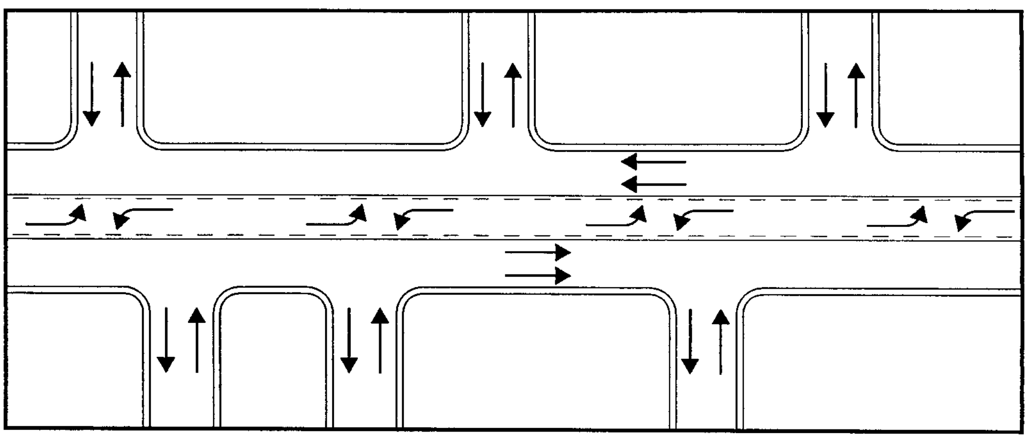
agencies have installed physical (restrictive) medians on 4- and 6-lane highways to better manage highway access (Figure 31). The medians reduce the number and location of conflicts. This results in improved safety, even though there may be some increase in rear-end accidents at median openings. However, rerouted left-turn volumes may increase congestion and accidents at downstream signalized intersections, and the median may have an adverse economic impact on some business establishments.

In the past, the safety benefits of medians versus TWLTLs appeared to produce somewhat conflicting results. However, a growing body of information assembled since the 1980s indicates that 4-lane and 6-lane divided roadways with non-traversable medians (with protected left-turn lanes) have much better safety records (lower average accident rates) than 5-lane and 7-lane roadways where the odd lane is a TWLTL. A few studies have shown benefits based on before-and-after studies of the same roadway; most, however, compare accident rates for the two basic types of roads.

The accident rate comparisons from the various studies are summarized in Table 54. The accident rates for TWLTLs ranged from 3.20 to 11.07 per million VMT, with a mean of 7.25, a standard deviation of 2.64, and a coefficient of variation of 36.0 percent. The accident rates for restrictive medians ranged from 2.09 to 8.15 per million VMT, with a mean of 5.17, a standard deviation of 1.82, and a coefficient of variation of 35.2 percent. A student ‘t’ test between the means shows a highly significant difference (at the .01 level).

The accident rates were reduced in 15 out of 16 entries, with the percentage difference ranging from a 15 percent increase (on CBD streets in Atlanta, Phoenix, and Los Angeles/

Two-Way Left Turn Lane - Uninhibited Access
BEFORE



Restrictive Median - Controlled Access
AFTER

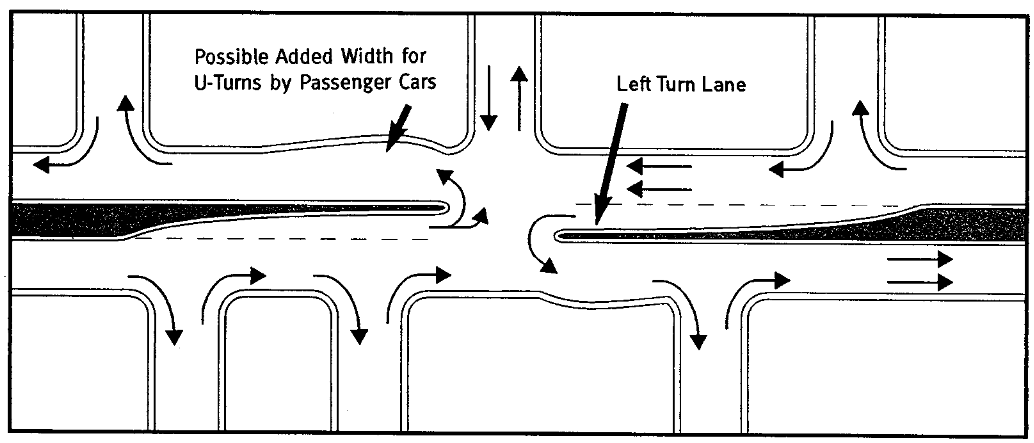


Figure 31. Replacing continuous left-turn lane with restrictive median.

TABLE 54 Synthesis of safety experience comparing TWLTLs with nontraversable medians by percent difference

Study & Location		YEAR	Description	Accidents			Accident Rates (Per Million VMT)			REMARKS	SOURCE
				TWLTL	Median	% Diff.	TWLTL	Median	% Diff.		
1 Bretherton, Womble & Parsonson (Georgia)	(1)	1990	Jimmy Carter Blvd. Atlanta	391	385	-2	8.09	6.47	-20		*(86, 87)*
2 Parsonson (Georgia)	(1)	1996	Memorial Blvd. Atlanta	947	523	-45	11.86	7.87	-34	Based on 1 year before (1988-1989), and 3 years after (1991-1993)	*(88, 89, 90)*
3 Banks, et al (Ontario)		1993	Hespeler Rd. Cambridge	45	33	-27	5.91	3.67	-38	Road widened to 6 lanes (2.8/km) midblock collisions.	*(91)*
4 Hartman and Spalett (Arizona)	(1)	1989	Phoenix Tucson	- -	- -	- -	5.85 5.17	5.70 3.99	-3 -22		*(92)*
5 Bowman and Vecellio (Arizona, California, Georgia)	(1)	1994	Atlanta, Phoenix, Los Angeles/Pasadena (15 road sections)	2,181 15,110	1,714 7,663	-21 -49	5.56 6.89	6.42 3.79	15 -45	CBD streets Suburban arterials	*(81)*
6 Parker, (Virginia)	(1)	1983	17 Traversable section 19 Median sections	-	-	-	6.11	4.42	-28		*(73)*
7 Benac (Michigan)	(1)	1988	Four-lane arterials Six-lane arterials	-	-	-	9.56 11.07	4.07 5.63	-57 -49	Based on 1985-1987 data	*(93)*
8 Squires and Parsonson (Georgia)	(1)	1989	57 Four-lane sections 25 Six-lane sections	-	-	-	8.99 10.82	7.67 8.15	-15 -25	42 with TWLTL, 15 with medians 8 with TWLTL, 17 with medians	*(94)*
9 Parsonson (Georgia)	(1)	1996	State routes	-	-	-	6.23	3.67	-41	4 and 6 through lanes	*(95)*
10 Long, Gan, Morrison (Florida)	(1)	1993	Four-lane arterials Six-lane arterials	-	-	-	3.20 4.28	2.09 3.20	-35 -25		*(80)*
11 Margiotta and Chatterjee	(1)	1995	12 segments - median 13 segments - TWLTL	-	-	-	6.48	5.96	-8		*(96)*

Note: (1) Represents comparison of different road sections.

Pasadena) to a 57 percent reduction reported for 4-lane arterials in Michigan. The median percentage reduction was about 27 percent.

Table 55 shows the reported percentage differences in various kinds of accidents on roads with medians in relation to TWLTLs. Sideswipe, rear-end, right-angle, left-turn, head-on, and pedestrian accidents were consistently reduced. Rear-end accidents were generally reduced, although Tennessee reported an increase in rear-end accidents. These differences reflect the more positive control of pedestrians and motorists provided by medians, the reduction in unsignalized left-turn access, and the provision for left turns at signalized intersections. However, the greater pressure and traffic congestion at signalized locations may increase rear-end accidents.

In summary, the medians appear safer than TWLTLs. Care, however, should be exercised in providing adequate capacity and design at signalized intersections to ensure that the safety benefits do not become offset by congestion-related collisions.

ACCIDENT PREDICTION MODELS

Estimating the benefits from installing different median types in urban and suburban settings requires the ability to predict the reductions in vehicle and pedestrian accidents. This requires assessing the accident history at locations with similar geometric, design, traffic, and environmental conditions from which predictive relationships can be derived.

This section reviews and compares the various accident prediction models that have been developed over the past 25 years.

These models, summarized in Table 56, predict annual accidents per million vehicle miles or per mile of roadway. One model predicts annual midblock accidents at a specific driveway location. Appendix A of the median synthesis report provides a detailed description of the individual models.

Table 57 compares the various safety model results. An examination of this table indicates the following:

- The models show generally consistent results for the relative safety of the three median alternatives, even though they predict somewhat different accident rates for any given set of conditions. The “undivided” treatment has the highest expected accident frequency over the range of traffic volumes. The model results support the 30 to 35 percent accident reduction found in before and after studies (for converting from an undivided cross section to either a TWLTL or nontraversable median).
- The raised median generally has the lowest predicted number of accidents. The main exceptions are the results predicted by the Harwood model that estimates fewer accidents for TWLTLs at all traffic levels.
- The Bowman-Vecellio model consistently predicts fewer accidents on roadways with raised medians than on TWLTLs and fewer on roadways having a TWLTL than on undivided roadways. This accident model suggests that the number of predicted accidents increases in a linear manner from an ADT of 10,000 to 40,000. Whereas, the rate of increase in the predicted accidents for raised medians overall begins to level off from 30,00 to 40,000.
- The average of the various models generally results in fewer accidents on roadways with raised medians than with TWLTLs. This difference is more pronounced when the Harwood data are excluded. Figure 32 shows the resulting patterns and provides a broad guide for application.

That the different models produce different results probably reflects the localized database from which each regression model was developed. The consistency of the Bowman-Vecellio model in predicting total accidents may be explained by the large and geographically diverse database.

The following factors explain some of the differences among the number of accidents predicted by the various models:

- The number of accidents will decrease as the reporting threshold increases. The Bowman-Vecellio model included this variable because their database included data from three cities in three different states. This may explain some of the difference between the Walton-Machemehl (Texas) and Parker (Virginia) models, for example.
- The Bowman-Vecellio model (*81, 101*) considered the number of signalized intersections per mile, but did not find it statistically significant. This presumably was because the number of signals per mile is correlated with other variables in the model (e.g., the number of driveways and the unsignalized intersections per mile and type of adjacent land development). McCoy-Ballard also found signals to be not significant for undivided or TWLTL roadways as did Chatterjee et al. for both nontraversable medians and TWLTLs and Squires-Parsonson for raised medians.

The Bowman-Vecellio model appears to offer the most logical and consistent results. Again, this may be the result of its large, geographically diverse database. It may, therefore, be the most transferable.

The Bonneson-McCoy model for midblock accidents (not shown in Table 57) also appears to give consistent results. Comparison of annual midblock accidents per year for the

TABLE 55 Synthesis of safety experience comparing TWLTLs with nontraversable medians by type of median

Study & Location	YEAR	DATA COMPARED	PERCENT DIFFERENCE								REMARKS	SOURCE
			Rear-End	Side-swipe	Right Angle	Left Turn	Head-on	Fixed Object/ Parked Vehicle	Pedestrian	Other		
Memorial Drive, Atlanta	1990	Accidents	-36	-29	-75	-42 (2)	-80	-100	-67	-38		*(86, 89, 90)*
Hespeler Road, Cambridge	1993	Midblock Accidents	-50	-	-25	-	-	-	-	-33		*(91)*
Atlanta, Phoenix, Los Angeles, Pasadena	1994	Suburban Midblock Rates	-42	-	-45	-54	-47	-	-	-11		*(81)*
Tennessee	1995	Accidents	15	-25	-24 (4)	-32 (3)	-35	-	-	79		*(96)*
Florida	1993	Rates	-23	-38	-30	-46	-58	-	-60	-30 (1)	4-lane urban arterials	*(80)*
			0	-31	-	-44	-50	-	-36	-	6-lane urban arterials	

Notes:
(1) Right turn
(2) All turns
(3) Front angle
(4) Broadside and rear angle

TABLE 56 Summary of safety models for median alternatives

No.	Model	Year	Dependent Variable	Source
1.	Glennon et al	1975	Accidents/Mile/Year	*(97)*
2.	Walton & Machemehl	1979	Accidents/Mile/Year	*(98)*
3.	Parker	1983/91	Accidents/Mile/Year	*(73, 99)*
4.	Harwood et al	1986	Accidents/Million/VMT	*(76)*
			Non-Intersection, Intersection, Overall	
5.	Squires/Parsonson	1989	Accidents/Million/VMT	*(94)*
			Accidents/Mile/Year	
6.	Chatteerjee and Margiotta	1995	Accidents/Mile/Year	*(100)*
7.	Bowman-Vecellio	1994	Accidents/Mile/Year	*(81, 101)*
8.	Bonneson-McCoy	1996	Annual Midblock Accidents	*(102)*

three types of median options are summarized in Table 58 for the following conditions:

- No parallel parking,
- Sixty-five driveways per mile, and
- Business or office land use.

The model found that accident frequency is significantly correlated with average daily traffic demand, driveway density, the density of unsignalized street approaches, median type, and adjacent land use. In general, accidents were more frequent on street segments with higher traffic demands, driveway densities, or street densities. Accidents were also more frequent when land use is business or office instead of residential or industrial. When parallel parking is allowed on an urban arterial street, accident frequency was higher with the undivided cross section than with the other two median treatments. When no parking is allowed, the differences were less distinct. In most situations, however, the raised-curb median tended to yield the lowest accident frequency.

OPERATIONS EXPERIENCE AND ANALYSIS

It is generally recognized that TWLTLs and physical medians reduce delays and improve traffic operations. There is, however, very little "before and after" information on the operational effects of these median designs. Several com-

TABLE 57 Comparison of safety model results

	Expected Accidents / Mile / Year											
ADT:	10,000			20,000			30,000			40,000		
Left-Turn Treatment:	Un-divided	TWLTL	Raised Median	Un-divided	TWLTL	Raised Median	Un-divided	TWLTL	Raised Median	Un-divided	TWLTL	Raised Median
Walton *(98)*	na	37	na	na	58	na	na	78	na	na	98	na
McCoy *(103)*	33	31	na	oor	52	na	oor	oor	na	oor	oor	na
Squires *(94)*	na	ne	37	na	31	56	na	69	75	na	108	94
Parker *(99)*	na	27	18	na	43	32	na	58	45	na	73	59
Chatterjee *(100)*	na	55	46	na	90	81	na	125	116	na	oor	oor
Harwood *(76)*	36	27	36	72	54	72	109	81	108	145	108	144
Bowman *(101)*	63	43	25	126	85	50	190	128	75	253	170	101
Average Freq.	44	37	32	99	59	58	149	90	84	199	111	100
Std. Deviation	16	11	11	38	21	19	57	39	29	76	36	35
Coeff. of Variation (%)	36	30	34	38	36	33	38	43	35	38	32	35

Excluding Harwood Data												
Average Frequency	48	39	32	126	60	55	190	91	78	253	112	85
Std. Deviation	21	11	12	-	23	20	-	33	29	na	41	23
Coeff. of Variation (%)	44	28	38	na	38	36	-	36	37	na	37	27

Source: *NCHRP Report 395.* Transportation Research Board, National Research Council, Washington, D.C. (November 1996).

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.
ne - Model yields negative results.

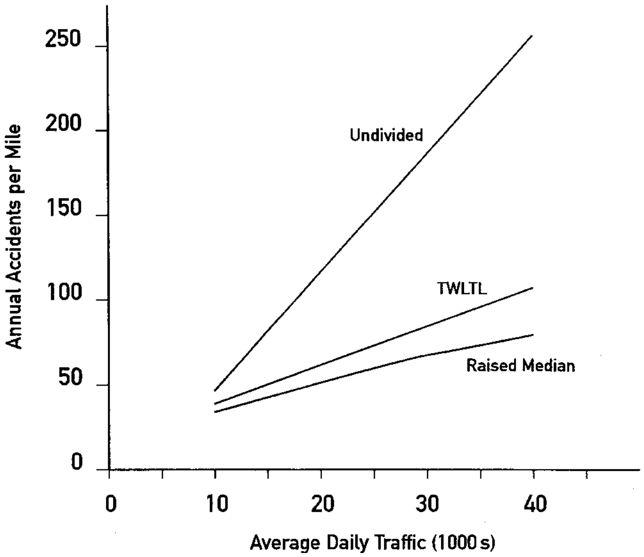


Figure 32. Predicted average annual accident frequency (excludes Harwood data).

puter simulations have attempted to "model" these impacts and to set thresholds for where each treatment should be applied.

Operations Experience

Most operational studies focused on TWLTLs. Many analyzed driver behavior in response to roadway geometry. A few "before and after" comparisons were made, and several studies reported that delays were reduced by median improvements. Operational experience has been limited, making systematic comparisons difficult.

Operations Models

Several models have been developed over the past 15 years to assess the operational effects of various median alternatives. These models have utilized both simulation and analytical techniques to quantify changes in stops and delays (or speeds) for various roadway types, traffic volumes, and driveway frequencies. The principal models are summarized in Table 59. A further description of each is provided in Appendix B of the median synthesis report.

Implications

There have been relatively few actual operational studies (i.e., "before and after" of the effectiveness of various median alternatives. These studies (along with operational models) clearly indicate that removing left-turning vehicles from the through traffic lanes reduces delays whenever the number of through travel lanes is not reduced.

The models of traffic performance at midblock locations where left turns take place have generally utilized simulation techniques such as TRAF-NETSIM or TWLTL-SIM. The most sophisticated model is the one prepared by Bonneson and McCoy as part of their 1996 *NCHRP Report 395,* "Capacity and Operational Effects of Midblock Left-Turn Lanes" (*102*). Their multi-faceted model provides a sound basis for assessing the through and left-turn approach delays associated with various median alternatives.

The various models consistently indicate that TWLTLs and nontraversable medians result in fewer delays than undivided roadways, especially as arterial traffic volumes increase. The models show TWLTLs resulting in lower delays than raised medians—especially in high-volume situations. But the differences are not generally statistically significant and have not been fully documented in actual practice. They appear to be attributed to left-turn bay blockage in the models and/or added travel distances involved—conditions that are amenable to correction by design.

ECONOMIC IMPACTS

Property acquires value because of its location, the keys being accessibility and exposure. Accessibility is generally measured by the ease with which people and vehicles can

TABLE 58 Annual midblock accident frequency

	ADT		
	20,000	**30,000**	**40,000**
Undivided	16	23	28
TWLTL	9	13	17
Median	7	10	13

[(1)] Conditions: no parallel parking, 65 driveways per mile, and abutting commercial development.

Sources: *(102, 104)*

TABLE 59 Summary of operations models for median alternatives

No.	Model	Year	Type	Scope	Dependant Variable	Source
1.	McCoy-Ballard-Wisaya	1982	Simulation	2-Lane TWLTL	Reductions in Stops & Delays	*(105)*
2.	McCoy-Ballard	1983	Simulation	4-Lane TWLTL	Reductions in Stops & Delays	*(106)*
3.	Heikal-Nemeth	1985	Simulation	4-Lane TWLTL	Stops per Vehicle (Inside Lane)	*(107)*
4.	Harwood-St. John	1985	Regression	2-Lane TWLTL	Reduction in Delays to Through Traffic	*(75)*
5.	Harwood	1986	Simulation	Multi-lane Highway	Reduction in Delays to Through Traffic (Per Left-turn Veh.)	*(76)*
6.	Ballard-McCoy	1988	Simulation & Regression	4-Lane Urban Highways TWLTL	Av. Stopped Delay % of Vehicles Stopping	*(108)*
7.	Modur-Machemehl-Lee	1990	Simulation	4-Lane Highways	Approach Delay Per Vehicle	*(109)*
8.	Parker	1991	Regression	All Highways	Midblock Left-turn Delay	*(99)*
9.	Venigalla	1992	Simulation	4-Lane Highways	Left-turn & Through Traffic Delay	*(110)*
10.	1994 HCM	1994	Analytical	Multi-lane (Uninterrupted Flow)	Through & Left-turn Approach Delay Per Vehicle	*(111)*
11.	Bonneson-McCoy	1996	Analytical	Multi-lane Highways	Through & Left-turn Approach Delay Per Vehicle	*(102)*

reach, arrive at, and depart from a site. Exposure is usually measured by the number of people and vehicles that pass by the site. The economic impacts of the various median alternates largely reflect the extent to which access is improved, restricted, or denied.

The installation of a physical median limits direct access to properties and may require the rerouting of left turns and involve longer travel distances. This, in turn, may limit both the exposure to and the effective exposure of a site. However, the increase in capacity associated with the installation of a median may improve the accessibility and increase the exposure, just like adding travel lanes to the roadway.

Measuring and assessing the impacts of restricting left turns has been difficult. The impacts not only depend on the extent that access to adjacent property increases or decreases, but also on the type of activity involved and the background economic conditions. Some activities, such as a regional shopping center or office complex attract their clientele from a large area, and overall access time to markets play a major role. Other activities, such as service stations and drive-in restaurants, rely on intercepting pass-by traffic; in such cases, left-turn restrictions and increased travel distances could adversely affect business. Table 60 gives illustrative examples of business sensitivity to pass-by traffic. Table 61 shows pass-by percentages for a range of retail activities.

The effects of left-turn restrictions also depend on changes in business conditions and traffic volumes, shifts in population and purchasing power, and the development of competitive business sites. Consequently, only a few studies have quantified actual effects. Instead, most studies have focused on *perceptions* of effects and the attitudes of the various groups affected.

The introduction of a raised median limits left-turn access to those locations with median openings. This tends to

TABLE 60 Illustrative examples of business sales sensitivity to pass-by traffic

Proportion of Business Sales Coming from Pass-by Traffic	Sample Business Type (Standard Industrial Code)
Lowest	Mobile Home Dealers (527) Boat Dealers (555) Photographic Studios (722) Dentists (802)
Moderate	Hardware Stores (525) Household Appliance Stores (572) Automotive Repair Shops (753)
High	Grocery Stores (541) Laundry, Cleaning and Garment Services (721)
Highest	Miscellaneous Food Stores (549) Gasoline Service Stations (554)

Source: *(112)*

adversely affect those businesses (i.e., service stations and fast food restaurants) that rely on pass-by traffic and, in a few cases, other businesses. In Texas, traffic serving businesses not located at median openings declined 44 percent. Along Jimmy Carter Boulevard in metropolitan Atlanta, 21 businesses experienced decreased sales volume while 15 had increased their sales. Along Memorial Boulevard in Atlanta, several businesses closed, but the closings were influenced by other factors.

The perceived effects on business appear to be greater than those that actually occurred. Along Oakland Park Boulevard, almost 30 percent believed their business volumes declined, while in other Florida communities, negative effects were perceived by 43 percent. Poorer business conditions resulting from installing medians were perceived by 12 to 55 percent of the respondents in the Bonneson-McCoy survey.

The Texas study and analysis of service stations indicate that left-turn access into businesses decreases in relative terms as traffic volumes increase. Thus, business impacts depend not only on the business type and location, relative to median openings along a road, but on traffic volumes as well.

APPLICATION GUIDELINES

This section contains guidelines for assessing the safety, travel time, and economic effects associated with the various median alternatives. The guidelines build on the information contained in previous sections of this chapter.

Safety

Safety experience suggests that the installation of TWLTLs or nontraversable medians, reduces accident rates by about 30 to 40 percent of those experienced with undi-

TABLE 61 Reported pass-by trips as percent of total (averages)

Land Use	No. of Sites	AM Peak Hour	PM Peak Hour	Source
1. Convenience Stores	na		71	*(150)*
2. Convenience Mart with Gasoline Pumps	15	62	66	*(151)*
3. Convenience Mart	20		60	*(152)*
4. Gasoline Service Station with Convenience Mart	9	61	56	*(151)*
5. Gasoline Service Station	6	58	52	*(151)*
6. High Turnover sit-down restaurant	6		40	*(151)*
7. Fast Food Restaurant	25	45	47	*(151)*
with drive-through window	7		43	*(152)*
8. Supermarkets	5	5	27	*(153)*
			42	*(152)*
9. Discount Stores			22	*(153)*
10. Shopping Centers	67			*(152)*
50,000 sq.ft.			60	
100,000			45	
200,000			36	
300,000			31	
400,000			28	
500,000			27	
250,000			22	
1,000,000			21	

TABLE 62 Estimated total accidents/mile/year average of various safety models

ADT	Undivided	TWLTL	Raised Median
10,000	48	39	32
20,000	126	60	55
30,000	190	92	78
40,000	253	112	85

Source: *(102)*

vided cross sections that do not remove left turns from the through travel lanes. Studies conducted since the 1980s have shown that roads with raised medians are generally safer than those with TWLTLs; accident rates averaged about 5.2 and 7.3 per million VMT, respectively. However, both rates and effects vary from location to location, reflecting facility-specific conditions and accident reporting procedures. Thus, these values should be used only to provide a first broad-gauged assessment.

Accordingly, various safety models have been developed to better refine estimation of impacts. The generalized results of these models—shown earlier in Figure 32—provide a broad guide for estimating accidents. They suggest the annual accident rates per mile shown in Table 62 for the three median options.

More refined estimates can be obtained by applying the Bowman-Vecellio accident prediction model. The model takes the form

$$A = \left(B_0\ ADT^{B_1}\right)\left(Length^{B_2}\right)(\text{Linear Terms}) \tag{9}$$

The coefficients for the model are shown in Table 63.

Midblock accident impacts may be estimated using the various Bonneson-McCoy tables and graphs. Representative results are summarized in Table 64, and more detailed tabulation are contained in Appendix C. These tables give expected annual accident frequencies for ¼-mi road segments. Effects of changes in median type or access frequencies can be obtained by re-entering the appropriate tables and comparing the differences. Effects of changes in segment length can be estimated by applying the following equation:

$$\begin{array}{l}\text{Acc/mile} \\ \text{for length x,} \\ \text{in feet}\end{array} = \begin{array}{l}\text{Acc/mile} \\ \text{for } \tfrac{1}{4} \text{ mile} \\ \text{section}\end{array} \times \frac{\text{Length}^{0.852}}{1320} \tag{10}$$

Thus, for a ½-mi section, the accidents per mile would be 1.8 times the accident values shown in the tables.

The guidelines from *NCHRP Report 395* are most applicable in areas where (1) the percentage of property-damage-only accidents ranges from 60 to 75 percent; (2) traffic signals are spaced 1,000 ft or more apart; (3) the traffic signals

TABLE 63 Bowman-Vecellio vehicle accident prediction model

Variable		Variable Name	Median Type		
			Unividied	TWLTL	Raised Median
Exposure	B_0	Intercept	0.000365	0.000365	0.000365
	B_1	ADT	1	1	1
	B_2	Segment Length, Len	1	1	1
Explanatory	C_0	Intercept	1.88	3.71	7.21
	C_1	Reporting Threshold, Thr	-0.00303	-0.00278	-0.00788
	C_2	Office Land Use, Off	1.06	-0.0723	-0.448
	C_3	Business Land Use, Bus	0.657	0 (1)	0 (1)
	C_4	Area Type, Area	0.457	0 (1)	0 (1)
	C_5	Median Width, Med	0 (1)	0.0354	-0.0276
	C_6	Unsig. Approach Density, Unsig	0 (1)	-0.0606	0 (1)
	C_7	Driveway Density, Drv	0.0132	0.0129	0 (1)
	C_8	Crossover Density, Cross	0 (1)	0 (1)	0.0962
	C_9	Speed Limit, Spd	0 (1)	-0.0339	-0.070
Database		Years of Accident Data		3-5	
		Number of Sections	152	178	150
		Total Section Length (miles)	38.9	55.1	51.9
		Through Lanes		4 and 6 (1)	

Note: (1) Considered, but not found to be statistically significant.
Sources: *(81, 101)*

$$A = B_0\ ADT^{B_1}\ Length^{B_2}\ (\text{Linear Terms})$$

TABLE 64 Annual midblock accidents per ¼-mi section–business or office land use

Driveways/Mile	Undivided (a)	TWLTL	Raised Median
	ADT 22,500		
30	7 - 9	7	5
60	8 - 10	8	6
90	9 - 12	9	6
	ADT 32,500		
30	9 - 13	9	7
60	11 - 14	11	8
90	12 - 17	12	9

(a) Higher value with parallel parking.

Note: Assumes 65 percent of all accidents are property damage only.

Source: *(102)*

are coordinated; (4) there is no parallel parking; (5) the arterial has four or five through lanes; (6) the access points are aligned to form four-leg intersections; and (7) there are no exclusive right-turn bays. Guidelines can be used outside of these ranges, but become less reliable as the amount of deviation increases.

Operations

Traffic operations along most urban and suburban arterial roadways are strongly influenced by conditions at traffic-signal-controlled intersections. Impacts of alternative mid-block median treatments can be estimated from the *NCHRP Report 395* analyses, which are summarized in Table 65 and detailed in Appendix C.

A simple example illustrates the application of these tables for a roadway carrying 32,500 vehicles per day with left turns per 1,320-ft segment accounting for 10 percent of the daily traffic. For a TWLTL with 90 access points per mile, there would be 3,200 annual hours of delay. Conversion to a raised median with 30 driveways per mile would result in 3,100 annual hours of delay. Note that for an undivided cross section with 90 driveways per mile, there would be 8,000 annual hours of delay.

Economic Impacts

The economic impact associated with installing a raised median and limiting certain access points to right turns only will depend upon the following factors:

- The size and type of each abutting land use at the locations where left-turn access will be reduced,
- The reliance of each land use on pass-by traffic,
- The number of vehicles turning left into the activity or land use,
- The average purchase per vehicle (or person), and
- Economic trends for the surrounding areas.

It is reasonable to expect that some pass-by trips that can no longer turn left into a roadside establishment will seek

TABLE 65 Annual delay to major street left-turn and through vehicles

Driveways/Mile	Undivided	TWLTL	Raised Median
	ADT 22,500		
30	2,200	1,300	1,300
60	2,200	1,400	1,400
90	2,200	1,400	1,400
	ADT 32,500		
30	7,100	3,000	3,100
60	7,800	3,200	3,500
90	8,000	3,200	3,400

Note: Assumes 10 percent Left Turns/1320-foot segment.

Source: *(102)*

other ways to reach the same location or a similar use that can be reached without turning left; there would be no reduction in these trips at locations where left-turn access remains. It is also reasonable to expect that "destination-oriented" trips will find alternate routes. For any site where left-turn access is denied, the maximum adverse impacts represent the product of (1) the number of left-turn entrants and (2) the proportion of those turns that represent pass-by (intercept) trips. The loss would represent the average dollars per purchase multiplied by the number of trips involved. The economic impacts over a section of highway would be summed for the individual establishments involved. Thus, the maximum loss would be

$$\sum_{1}^{M} N_i P_i D_i \tag{11}$$

where:

N_i = number turning left at location i per day
P_i = % pass-by at location i
D_i = Dollars/Purchase
M = number of establishments where left-turn entrance is denied

The number of left turns can be observed in the field. The percent pass-by traffic can be estimated based upon the proportion of pass-by traffic reported in various studies. Table 66, Column A, gives the generalized values for the proportion of pass-by traffic for various land uses.

Where the number of left turns is not known, the proportion may be estimated from Column B in Table 66. These values reflect the increasing reticence of drivers to turn left from an arterial highway as opposing traffic volumes increase.

These estimates represent maximum impacts, because repetitive pass-by traffic might change travel patterns, stop on the return trip, or take advantage of well designed or conveniently located U-turn facilities. Impacts also would be less where an alternate left-turn access into a property remains open. There may be no overall impact on the community because this business traffic would be diverted to other area establishments. Moreover, sales at other establishments along a section of road might show an increase as a result of improved accessibility.

The following examples illustrate the application of Table 66:

- Assume that 500 vehicles per day turn left into a community shopping center and 30 percent of these vehicles would represent "pass-by" traffic. Thus, the maximum daily loss in traffic would be about 150 vehicles per day. If the average purchase is $20 per vehicle, then the daily loss would be $3,000.
- Assume left turns will be prohibited into a service station along a road with 10,000 ADT. From Table 66, the pass-by traffic represents 55 percent of the total and 40

TABLE 66 Economic impact model

Land Use	A % Pass-by	B Estimated Left Turns As % of Total Entering Traffic	
		ADT	%
1 Gasoline Service Station Convenience Mart Small Retail < 50,000 sq. ft.	55	5,000	43
		10,000	40
		20,000	30
		30,000 or more	15
2 Fast Food Restaurant with Drive Through Window Supermarkets Shopping Center 50,000 - 100,000 sq. ft.	45		
3 High Turnover sit-down restaurant	40		
4 Shopping Centers 250,000 - 500,000 sq. ft.	30		
5 Shopping Centers Over 500,000 sq. ft.	20		

Source: Estimated from Table 61 shown earlier.

percent of entrants are turning left. Thus, a maximum of 22 percent of the customers would be lost if left turns were prohibited.

- Assume that left turns would be prohibited into a high-turnover restaurant along a roadway carrying 30,000 vehicles per day. The pass-by traffic accounts for 40 percent of the total entrants. About 15 percent of the customers would turn left into the restaurant. The anticipated maximum impact would be a 6 percent loss in customers.

To estimate the maximum daily and annual economic loss, information would be needed on the purchases per vehicle (or customer) at any given establishment—both on a daily and annual basis.

Development Costs

Construction costs for TWLTLs and raised medians were estimated in *NCHRP Report 395* (*102*). The estimated construction costs per mile in 1996 dollars are shown in Table 67. These costs were based upon the urban highway construction costs reported by Cohen and Reno (*159*) and the incremental costs estimated by Harwood and Glennon (*160*) and Parker (*97*). An incremental cost of $24,000 per mile was estimated for providing TWLTLs on undivided highways and a cost of $211,000 per mile for providing raised medians on roads with TWLTLs. These costs were assumed to be in addition to the costs of providing the fifth lane.

The differences in development costs per mile for the various median options are shown in Table 68. This table also shows annual costs based upon a 20-year design life and a debt service (amortization) rate of 4 percent. Actual costs will vary widely from region to region, depending on specific local conditions, and could be as much as twice these values.

Selecting a Median

Selecting a median alternative will depend upon many policy, land use, and traffic factors. These factors include (1) the access management policy and access class for the roadway under consideration; (2) the types and intensities of the adjacent land use; (3) the supporting street system and the opportunities for rerouting left turns; (4) existing driveway spacings; (5) existing geometric design and traffic control features (e.g., proximity of traffic signals and provisions for left turns); (6) traffic volumes, speeds, and accidents; and (7) costs associated with roadway widening and reconstruction.

Table 69 gives a comparative analysis of the three midblock left-turn treatments based upon the research for *NCHRP*

TABLE 67 Estimated development costs per mile associated with alternative midblock left-turn treatments

	Area Type:	Built-Up Urban Areas			Outlying Urban Area		
Cost Item	**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 (thousand of dollars per mile)[1]							
Construction[4]		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 (113).

3 - Incremental cost of Raised-curb over TWLTL was based on the average of values reported by Harwood (114) and Parker (99) (i.e., 211, 000).

4 - Construction costs for TWLTLs and raised medians equal 5 times the unit cost for an undivided roadway plus $24,000 and $235,000, respectively.

Source: *(99, 102, 113, 114)*

TABLE 68 Ranges in reconstruction costs for midblock left-turn treatments

Reconstruction (or Conversion) Combination	Estimated Difference in Construction Costs (Thousands)	Annualized Costs	
		Dollars Per Mile	Dollars Per Quarter-Mile (Rounded)
Undivided to Raised-Curb Median	$1,452 (a)	$106,841	$27,000
Undivided to TWLTL	$1,241 (a)	$ 91,315	$23,000
TWLTL to Raised-Curb Median	$ 980 (b)	$ 72,110	$18,000

Notes: (a) Difference in construction and R.O.W. costs (Table 52).
(b) Cost/Mile to build raised-median.

Debt Service Factor - (20 years at 4%) is .073582

TABLE 69 Comparison of three midblock 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 & through 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.
Safety Effects			
1 Vehicle accident frequency	Raised-Curb	Raised-Curb	TWLTL
2 Pedestrian accident frequency	Raised-Curb	Raised-Curb	n.d.
3 Turning driver misuse/misunderstanding of markings	Raised-Curb	Raised-Curb	Undivided
4 Design variations can minimize conflicts (e.g., islands)	Raised-Curb	Raised-Curb	TWLTL
5 Positive guidance (communication to motorist)	Raised-Curb	Raised-Curb	n.d.
Other Effects			
1 Cost of access (access management tool)	Raised-Curb	Raised-Curb	n.d.
2 Direct access to all properties along the arterial	TWLTL	Undivided	n.d.
Access 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 a review of the literature.

2 - n.d. negligible difference or lack of a consensus of opinion on this factor.

Source: *(102)*

TABLE 70 Illustrative computations of benefit-cost ratio conversion from TWLTL to raised median

Item	A. TWLTL 60 Approaches/Mile	B. Raised Median 30 Approaches/Mile	Difference
1 Annual Accidents (Tables E17, E-18, & E19 in Appendix C)	13	8	5
2 Annual Accident Cost Savings at $15,000/accident	$ 195,000	$ 120,000	$ 75,000
3 Annual Delay Savings (Tables E13, E-14, & E15 in Appendix C)	$ 10,700	$ 10,600	$ 100
4 Annual Delay Savings Cost at $16/hr.	$ 171,200	$ 169,600	$ 1,600
5 (2+4) Total Annual Benefits	$ 366,200	$ 289,600	$ 76,600
6 Annual Development Cost (Table 68)			$ 18,000
7 Benefit /Cost Ratio (Item #5 divided by #6)			4.26

Source: *(102)*

Report 395. It contains a detailed description of the strengths and weaknesses associated with midblock left-turn treatments.

More detailed guidelines for alternative midblock left-turn treatments were derived by Bonneson and McCoy based on benefit-cost comparisons. Illustrative benefit-cost computations are shown in Table 70. Appendix C contains further examples of *NCHRP Report 395* tables. These tables indicate when TWLTLs should be converted to raised medians for business-office and residential land uses.

These tables were based on the following assumptions:

- Annual accidents per ¼ mi were multiplied by $15,000 to obtain annual accident costs for each alternative.
- Annual through and left-turn delays (in hours) were multiplied by $16/hr to obtain annual delay costs.
- The differences in total annual costs (delay costs plus accident costs) between the two options represent the net benefits.

The tables show the “tradeoff” conditions for converting from one median option to another. When the benefit-cost ratio exceeds 2.0, the alternative left-turn treatment is recommended. When the benefit-cost ratio is less than 1.0, no change is recommended. The gray areas on the tables depict conditions where the benefit-cost ratio ranges between 1.0 and 2.0; more detailed site-specific evaluation is needed before considering a change.

The tables assume that there is no change in the number of driveways. However, especially when raised medians are installed, the number of left-turn driveways will be reduced. In these cases, it is necessary to use the appendix tables (E-17, E-18, or E-19) to obtain the annual accidents. These values then can be expressed in monetary terms for each median option, and differences in annual costs can be computed and compared directly with differences in construction costs.

The quantification of delays, accidents, and development costs should be tempered by the practical realities associated with roadway retrofit. Compatibility with adjacent roadway cross-sections, and availability of right-of-way, for example, may influence median selection. Still, the tables provide useful inputs into the median selection process.

CHAPTER 7

LEFT-TURN LANES (TECHNIQUE 3A)

INTRODUCTION

Left turns may pose problems at driveways and street intersections. They may increase conflicts, delays, and accidents and often complicate traffic signal timing. These problems are especially acute at major suburban highway intersections where heavy left-turn movements take place, but occur also where left turns enter or leave driveways serving adjacent land development. The following illustrate these problems:

- More than two-thirds of all driveway-related accidents involve left-turning vehicles (*113*).
- Where there are more than six left turns per traffic signal cycle, virtually all through vehicles in the shared lane may be blocked by the left-turning vehicles (*114*).
- Where left-turn lanes are provided along multi-lane highways, each opposing left-turning vehicle reduces the through vehicle capacity by the number of through lanes it crosses (e.g., 100 left turns/hour across three traffic lanes reduces the through vehicle capacity by 300 vehicles) (*114*).

Thus, the treatment of left turns has an important bearing on the safety and movement along arterial roadways. It is one of the major access management concerns. Left-turn movements at driveways and street intersections may be accommodated, prohibited, diverted, or separated depending on specific circumstances. Table 71 gives examples of each option and shows when each should be considered (*115*).

Left-turn lanes are normally provided by offsetting the center line or by recessing the physical (or painted) median. Examples of single and dual left-turn lanes are shown in Figure 33; a typical shared lane treatment is shown for comparison purposes (*115*).

The left-turn lanes offer the following important benefits:

- They remove the turns from the through travel lanes. This reduces rear-end collisions and increases capacity.
- They improve the visibility of oncoming traffic for vehicles turning left (Figure 34). This helps to reduce right-angle collisions.

SAFETY EXPERIENCE AND ANALYSIS

Many studies, mainly conducted in the 1960s and 1970s, have documented the safety and operational benefits of left-turn lanes. The widespread acceptance and use of left-turn lanes by traffic engineers and designers suggests that this treatment has been cost-effective.

The safety benefits of providing left-turn lanes as reported by the individual studies are presented in Table 72. This table shows that the removal of left turns from the through traffic lanes resulted in accident rate reductions ranging from 18 to 77 percent; the statistical median reduction was more than 50 percent.

Table 73 shows the reported percentage changes in various kinds of accidents when left-turn lanes are introduced. There is a generally consistent reduction in rear-end- and left-turn-related accidents. Right-angle (i.e., crossing-related) accident rates decline at signalized intersections but show a mixed result at unsignalized locations. This may involve greater driver uncertainty on the crossroad. On balance, however, left-turn lanes do improve safety and should be provided wherever practical.

OPERATIONS EXPERIENCE AND ANALYSIS

Operations-related studies have generally focused on assessing the delay reductions and capacity gains resulting from replacing shared lanes with left-turn lanes. Studies have also identified the conditions where left-turn lanes are warranted. Current practice of left-turn treatments at intersections are summarized in *NCHRP Synthesis 225* (*128*).

Through Vehicle Effects

Shared left-turn lanes result in a complex interaction among the left-turning vehicles, the through traffic in the same lane, and the opposing traffic. As shown in Figure 35, left turns may block following through vehicles. The number of through vehicles impeded or delayed will depend on the number of left-turning vehicles and their positions in the queue at a traffic signal.

TABLE 71 Treatment of left turns at intersections and driveways

Option	Condition	Application Considerations
Provide	Shared Lane	Limit to minor roads or places where R/W is not available for left-turn lane
	Left-Turn Lane	Protected or permissive phasing
	Dual Left-Turn Lane	Protected phasing only
Prohibit	Full Time	Requires alternate routes
	Peak Periods Only	Requires alternate routes
Divert	Jug-Handle	Divided highways at minor roads (signalized junctions only)
	Modified Jug-Handle	6-lane divided highways
	Michigan "U"	Divided highways with wide median - Allows two-phase signals
Separate	Directional Design	Very heavy turns in one direction
	Left-Turn Flyover	Very heavy turns in one direction
	Through Lane Flyover	Major congestion points

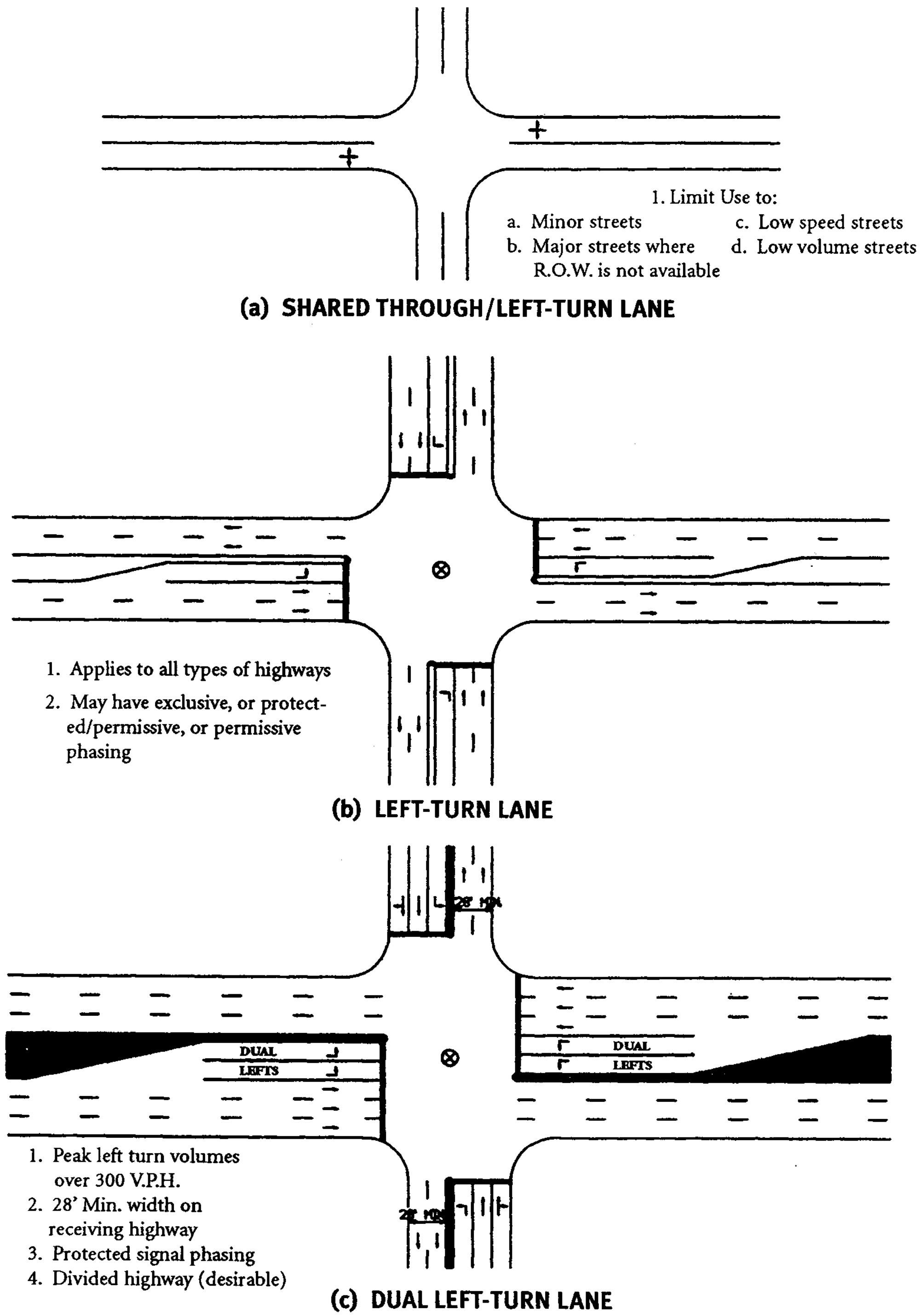
Source: (*115*)

Simulation and probability analysis have suggested the following values for the proportion of through vehicles blocked by left turns (*114*):

Left Turns/ Per Cycle	*Proportion of Through Vehicles Blocked*
0.5	0.25
1	0.40
2	0.60
3	0.70
4	0.75
5	0.80
6	0.84
7	0.86
8	0.88
9	0.89
10	0.90

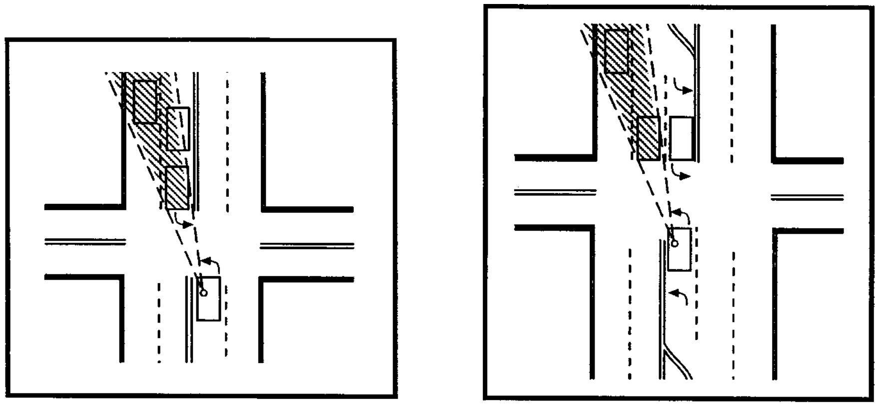
Thus, when there is one left turn per cycle, approximately 40 percent of the through vehicles in the shared lane are blocked; when there are three left turns per cycle, approximately 70 percent of the through vehicles are blocked. A protected left-turn lane, in contrast, generally results in no impedance to the same-direction through traffic.

Results of a simulation analysis of two-lane roads by Craus and Mahalel (*122*) are shown in Table 74. The proportions of through vehicles stopping and decelerating are a function of the number of vehicles in the opposing direction and the percentage of left turns in the same direction of travel. The proportions of stopped and slowed vehicles increase as the left-turn percentages, opposing traffic flows, and same direction volumes increase. For an opposing volume of 800 vph and a through volume of 800 vph, the per-



Source: *(115)*

Figure 33. Examples of left-turn lanes.



4-LANES -- LIMITED VISIBILITY

Adapted from: 116

4-LANES PLUS LEFT-TURN LANE -- IMPROVED VISIBILITY

Figure 34. Improved visibility from providing turn lanes.

TABLE 72 Synthesis of safety experience with left-turn lanes

Study Location	Year	Type	Accidents			Accident Rates Per Million Entering Vehicles			Remarks	Source
			Without	With	% Diff.	Without	With	% Diff.		
1. California (DPW) Wilson, Tamburri-Hammer	1967-8	Before/ After								*(117, 118)*
Unsignalized						1.16	0.58	-50	53 Locations	
Signalized						1.00	0.82	-18		
All Locations						1.08	0.70	-35		
Painted			157	106	-32				40 Locations	
Curbed			61	25	-59					
Raised Bars			95	31	-67					
All			313	162	-48					
2. Indiana (Shaw, Michael)	1968	Compares Locations				1.65	0.59	-65	8 Intersections without lanes; 3 with lanes	*(119)*
3. Ohio (Foody, Richardson)	1973	Compares Locations								
Unsignalized						4.35 [1]	1.04 [1]	-76	239 legs without left-turn lanes,	*(120)*
Signalized						2.47 [1]	1.54 [1]	-38	and 94 legs with left-turn lanes	
4. Israel (Ben-Yakov, Craus)	1980	Before/ After				1.65 [2]	1.03 [2]	-38	25 Intersections	*(121, 122)*
5. Kentucky (Agent)	1983	Before/								
Unsignalized		After				5.7 [3]	1.3 [3]	-77		*(123)*
Signalized						7.9 [3]	3.6 [3]	-54		
6. Indianapolis (Greiwe)	1986	Before/ After	102 [4]	44 [4]	-57				8 Intersections	*(124)*
7. Nebraska (McCoy-Malone)	1989	Compares Locations							3-year comparison	*(125)*
Unsignalized			95	62	-35	1.00	0.49	-51	14 sites with, 14 sites without	
Signalized			145	67	-54	1.28	0.56	-56	15 sites with, 20 sites without	
8. New Jersey, Route 47	1992	Before/ After	109	67	-39				1.8 miles -- 4-lane road converted to 3-lane	*(126)*
9. New Jersey, Route 130	1993	Before/				3.36	2.16	-35	8 miles (southern section)	*(127)*
		After				3.88	1.99	-51	28 miles (northern section)	

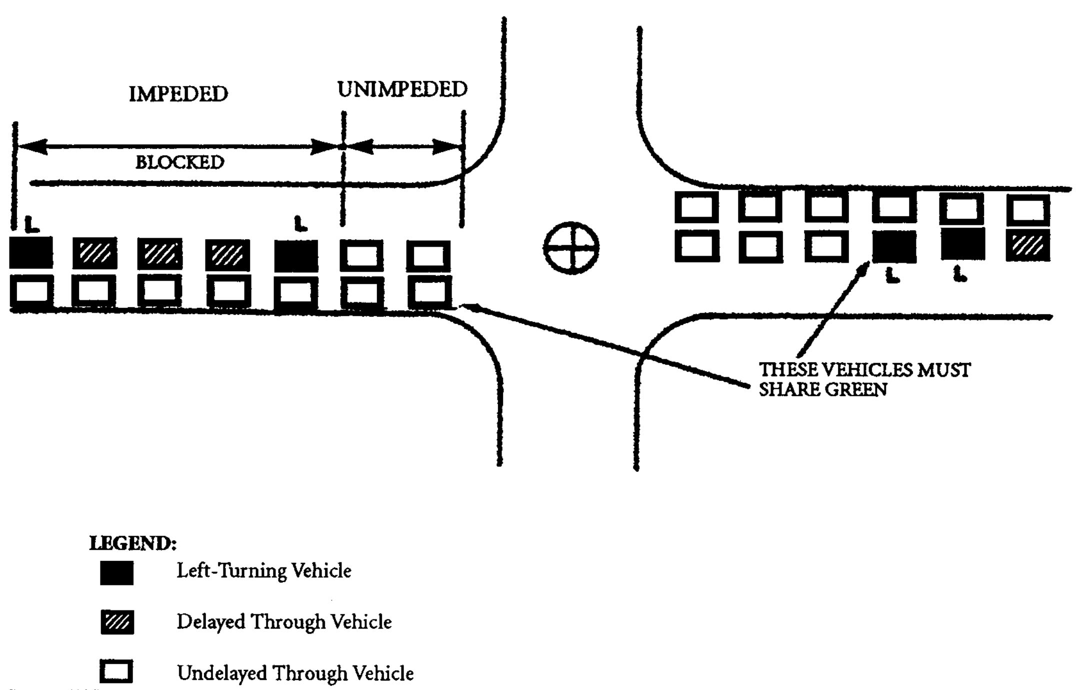
Note:

(1) Per Million Vehicles Per Leg Per Year.
(2) Accidents Per Intersection/Year.
(3) Per Million Left-Turn Vehicles.
(4) Mean Accidents/Intersection/Year.

TABLE 73 Synthesis of accident experience by type of accident

Study Location	Year	Conditions Compared	Percent Change in Accidents: Rear-End	Right-Angle	Left-Turn	Other	Remarks	Source
A. Unsignalized								
1. California (DPW, Wilson, Tamburri, Hammer)	1967-8	Acc/Million Entering Vehicles	-87 (1)	+50	-37	-45		*(129)*
2. Indiana (Shaw, Michael)	1968	Acc/Million Entering Vehicles	-62	-65				*(119)*
3. Ohio (Foody, Richardson)	1973	Acc/Million Vehicles Per Leg			-90			*(120)*
4. Kentucky (Agent)	1983	Acc/Million Left-Turning Vehicles			-77 (1)			*(123)*
5. Nebraska (McCoy, Malone)	1989	Acc/Million Entering Vehicles	-88 (3)	+68 (3)	-86 (3)	-53	Four-Lane Arterials	*(125)*
B. Signalized(4)								
1. California (DPW, Wilson, Tamburri, Hammer)	1967-8	Acc/Million Entering Vehicles	+16 (5)	-9	-56	-29		*(129)*
2. Ohio (Foody, Richardson)	1973	Acc/Million Vehicles Per Leg			-43			*(117)*
3. Kentucky (Agent)	1983	Acc/Million Left-Turning Vehicles			-54 (2)			*(123)*
4. Nebraska (McCoy, Malone)	1989	Acc/Million Entering Vehicles	-59 (3)	-38	-66 (3)	-74 (3)	Four-Lane Arterials	*(125)*

Notes:
(1) Statistically significant at .10 level.
(2) Includes left-turn related, rear-end, and sideswipe accidents.
(3) Statistically significant at .05 level.
(4) Without protected left-turn phases.
(5) Appears inconsistent with other findings.



Source: *(114)*

Figure 35. Intersection operations—shared traffic lane.

TABLE 74 Simulation of left-turn delays—(tow lane road – no left-turn lane), slow and stopped through vehicles as a function of number of vehicles in the opposite direction

Opposing Volume	Through Volume	Percentage of Stopped (Slowed) Vehicles									
		2% Left-Turning Veh.		5% Left-Turning Veh.		10% Left-Turning Veh.		15% Left-Turning Veh.		20% Left-Turning Veh.	
800	800	7.2	(3.3)	18.2	(7.0)	37.9	(11.0)	54.4	(12.2)	68.4	(9.6)
800	500	2.5	(3.9)	6.2	(8.7)	13.4	(14.2)	22.8	(19.5)	29.7	(21.1)
800	200	0.8	(3.7)	2.5	(7.9)	3.5	(14.2)	5.9	(21.9)	8.1	(26.0)
600	800	4.3	(2.1)	11.8	(5.4)	25.7	(8.6)	35.5	(11.4)	42.8	(12.7)
600	500	1.8	(2.3)	4.7	(5.4)	8.5	(9.6)	13.2	(13.8)	18.6	(16.9)
600	200	0.7	(2.2)	1.1	(5.2)	2.1	(9.6)	3.4	(13.4)	4.7	(17.9)
500	800	3.1	(1.8)	9.8	(4.6)	16.6	(7.3)	26.5	(9.5)	31.9	(11.6)
500	500	1.3	(2.0)	2.9	(3.9)	6.5	(8.3)	9.9	(12.0)	13.7	(15.1)
500	200	0.3	(1.7)	1.2	(4.3)	1.9	(8.2)	3.0	(12.2)	2.8	(15.2)
400	800	2.7	(1.4)	6.6	(4.0)	12.7	(5.6)	19.4	(8.6)	25.0	(9.7)
400	500	0.8	(1.4)	2.5	(3.7)	5.0	(5.6)	7.3	(0.2)	10.2	(12.4)
400	200	0.2	(1.3)	0.6	(3.3)	1.2	(6.0)	1.6	(9.1)	2.4	(11.8)
200	800	1.1	(0.6)	2.8	(1.8)	6.7	(2.7)	9.5	(3.8)	12.2	(5.2)
200	500	0.2	(0.7)	0.6	(1.3)	2.3	(2.7)	3.1	(3.9)	3.8	(5.5)
200	200	0.0	(0.6)	0.3	(1.3)	0.5	(2.3)	1.0	(3.4)	1.2	(5.9)

Source: *(122)*

centages of through traffic delayed and stopped were estimated as follows:

Percent of Left Turns	*Percent Delayed*	*Percent Stopped*
2.0	3.3	7.2
5.0	7.0	18.2
10.0	11.0	37.9
15.0	12.2	54.4
20.0	9.6	68.4

For an opposing volume of 200 vph and a through volume of 800 vph, the percentages of through vehicles affected were as follows:

Percent of Left Turns	*Percent Delayed*	*Percent Stopped*
2.0	0.6	1.1
5.0	1.8	2.8
10.0	2.7	6.7
15.0	3.8	9.5
20.0	5.2	12.2

The values shown may be used to estimate the reduction in through vehicle stops and slowing if a left-turn lane were provided.

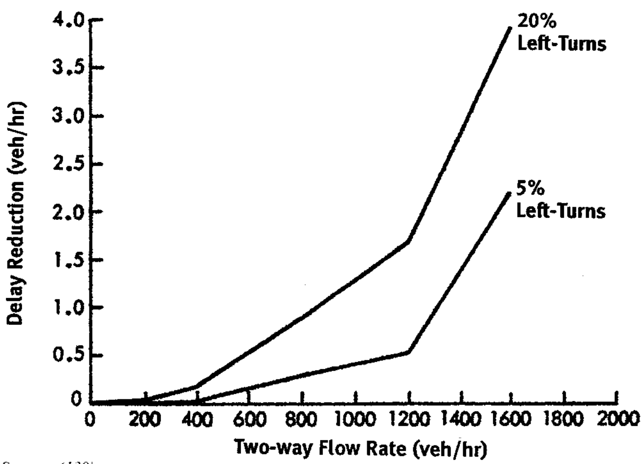
Delay Effects

Studies by Harwood and Hoban (*130*) have quantified the reduction in delay that results from providing left-turn lanes on 2-lane highways. Their findings are shown in Figure 36. There is relatively no delay (and minimal benefits) for two-direction traffic volumes below 400 vph. Delay (and benefits) increases significantly for two-way volumes above 1200 vph.

Capacity Effects

The capacity of a roadway is controlled by its critical intersections. The capacity depends upon the conflicting lane volumes, the green times available for movement, and the composition of the traffic stream. The capacity effects of shared, single, and multiple left-turn lanes can be estimated in various ways: (1) complex procedures in the 1994 *Highway Capacity Manual* (*131*) can be used to estimate capacities, *v/c* ratios, delays, and service levels; (2) simplified formulas can be applied that give reasonable results for most traffic planning purposes; and (3) critical lane conflict volumes can be used.

The capacity of a through lane depends on the effective green time available, although sometimes this green time must be shared with opposing left turns. The capacity of a shared lane is generally less, and under typical urban or suburban conditions might be about 40 to 60 percent of that of a



Source: *(130)*

Figure 36. Delay savings of left-turn lanes on two-lane rural highways.

through lane. Thus, along a four-lane arterial, provisions of left-turn lanes would increase the capacity from about 1.5 to 2.0 lanes in each direction—a 33 percent increase.

A series of formulas derived by Levinson (*114*) can be used to provide more precise estimates of through and shared lane capacity. The formulas reflect the assumption that the capacity of a through lane at an intersection is reduced by opposing left turns or by the blockage effect of left turns in the same direction. The capacity of a shared lane represents the minimum of these computations. The suggested formulas are shown in Table 75.

The effective opposing traffic per lane is computed by dividing the opposing traffic by the following factors, when there are shared left-turn lanes on that approach:

- 1 lane 1
- 2 lanes 1.5
- 3 lanes 2.5

TABLE 75 Shared lane capacity formulas (simplified)

Case	Description	Lane Capacity Per Cycle	Constraints[(1)]
1	Through lane with no opposing (conflicting) left-turns or shared lane on one-way street	(A) $c = \frac{g}{h}$	
2	Through lane with opposing (conflicting) left-turns	(B) $c = \frac{g}{h} - (\ell_2 - s_2)$	$\ell_2 \geq s_2$
3	Single shared lane (on two-lane street)	Minimum of	
	Conflict	(C) $c = \frac{g}{h} - (\ell_2 - \ell_1 - s_2)$	$(\ell_2 - \ell_1 - s_2) \geq 0$
	Blockage	(D) $c_s = \frac{g}{h} - B(o_2 - \ell_2)$	
4	Shared lane on multi-lane street	Minimum of	
	Conflict	(E) $c = \frac{g}{h} - (\ell_2 - s_2)$	$\ell_2 \geq s_2$
	Blockage	(F) $c_s = \frac{g}{h} - Bo_2$	

Notes:

g = effective green time (sec/cycle)
h = headway, adjusted for factors other than left turns (sec/veh)
ℓ_1 = left turns per cycle in given direction
ℓ_2 = opposing left turns per cycle
s_2 = opposing sneakers per cycle (always $\leq \ell_2$)
o_2 = effective opposing traffic per lane per cycle
B = modified blockage (impedance factor)
c = capacity (veh/lane/cycle)
c_s = capacity, veh/lane/cycle (shared lane -- blockage)
(1) When these conditions are not met, apply Formula A.

Source: *(114)*

This is necessary to account for the uneven distribution of traffic among lanes. The values of the "Modified" blockage or impedance factors, B, are as follows:

Left Turns Per Cycle	*B*
0.5	0.30
1	0.48
2	0.72
3	0.84
4	0.90
5	0.96
6 or more	1.00

Each of the approaches to estimating intersection capacity recognizes the reciprocal relationship between through traffic and conflicting left turns. In each case, the number of through vehicles that can be accommodated increases as the conflicting left turns are reduced; this relationship underlies the provision of dual left-turn lanes and the diversion or prohibition of left turns.

APPLICATION GUIDELINES

Planning Considerations

Left turns should be removed from the through travel lanes wherever possible. Therefore, provisions for left turns (i.e., left-turn lanes) have widespread application. Ideally, left-turn lanes (or jughandles) should be provided at driveways and street intersections along major arterial and collector roads wherever the turns are permitted. This is essential to improve safety and preserve capacity.

The 1994 *Highway Capacity Manual* (*131*) indicates that exclusive left-turn lanes at signalized intersections should be installed as follows:

- Where fully protected left-turn phasing is to be provided;
- Where space permits, left-turn lanes should be considered when left-turn volumes exceed 100 vph (Left-turn lanes may be provided for lower volumes as well on the basis of the judged need and state or local practice, or both); and
- Where left-turn volumes exceed 300 vph, a double left-turn lane should be considered.

Further guidelines for when left-turn lanes should be provided are set forth in several documents for both signalized and unsignalized intersections (*115, 128*). These guidelines key the need for left-turn lanes to (1) the number of arterial lanes, (2) design and operating speeds, (3) left-turn volumes, and (4) opposing traffic volumes.

The design of left-turn lanes is straightforward. The lanes should be shadowed (protected) from the through travel lanes and transitions around the lanes for through traffic (where required) should be gradual. The storage lengths should be maximized by keeping entry tapers relatively short. Storage for 1.5 to 2 times the peak-hour vehicles turning left per cycle will minimize the chances of overflow resulting from random arrivals. (The lower value is appropriate where signals are coordinated and arterial traffic moves in platoons.)

Estimating Effects

The estimation of safety and capacity effects is straightforward.

Safety

The provision of left-turn lanes has been found to reduce accidents and accident rates by about 20 to 65 percent. Table 76

TABLE 76 Reported accident reduction factors for left-turn lanes

Treatment	Accident Reduction Factor *	Source
UNSIGNALIZED-		
1. Add Left-Turn Lane (Physical Separation)	65 24 (Fatal + Injury)	132, 133 134
2. Add Left-Turn Lane (Painted Separation)	27 30	132 133
SIGNALIZED-		
3. Add Left-Turn Lane (Physical Separation)	40	132
4. Add Left-Turn Lane (Painted Separation)	15	132

Source: Compiled from reference noted.

Notes: * For all accidents, except where noted.

TABLE 77 Capacity implications of shared and exclusive left-turn lanes

Condition	Two-Lane Road	Four-Lane Road
No Left-Turns	840	1,600
Shared Through/Left-Turn Lane-Left-Turns/Hour:		
50	650	1,000
100	500	960
150	425	900
Exclusive Left-Turn Lane		
Unsignalized	960	1,100
Left-Turn Phase	750-800	1,250-1,460

Note: Computation assumes 60-90 second cycle, 50% green plus clearance time per cycle, 3 seconds lost time, and 1,900 vphpl saturation flow.
Source: *(131)*

gives accident reduction factors reported in the literature that may be used to estimate benefits of left-turn lanes.

Capacity

The capacity gains (and delay reductions) may be estimated by applying the 1994 *Highway Capacity Manual* (*131*) procedures to signalized intersections with various left-turn arrangements. Table 77 illustrates the capacity gains that are estimated from the 1994 HCM procedures. It is based on a volume of about 500 to 700 vph per lane each way, a 50 percent green plus clearance time for the arterial roadway, 3-sec lost time, and a 1,900-vph saturation flow rate. It provides a broad guide as to the benefits of providing left-turn lanes or prohibiting left turns.

CHAPTER **8**

U-TURNS AS ALTERNATIVES TO DIRECT LEFT TURNS (TECHNIQUE 3D)

INTRODUCTION

Increasingly, U-turns are being used as an alternative to direct left turns in order to reduce conflicts and to improve safety along arterial roads. U-turns make it possible to prohibit left turns from driveway connections onto multi-lane highways and to eliminate traffic signals that would not fit into time-space (progression) patterns along arterial roads. When incorporated into intersection designs, U-turn provisions enable direct left turns to be rerouted and signal phasing to be simplified.

Figure 37 illustrates the many conflicting movements where there are closely spaced, full median openings and how the number of conflicts can be substantially reduced by replacing full median openings with "directional" openings that only allow left-turn ingress to abutting developments; the left-turn egress movements would be made by turning right onto the arterial road and then making U-turns downstream. The figure also illustrates the reduction in conflict points that could be achieved if most median openings were closed; the remaining median openings at intersections would have to be redesigned to accommodate the additional turning movements.

CURRENT PRACTICES

Cities and states use various approaches for reducing the number of conflicts along their arterials. California provides dual left turns at intersections with collector streets, with the innermost lane accommodating U-turns. Florida prohibits left-turn exits onto major arterials, and provides midblock U-turn lanes to accommodate these movements. New Jersey uses jughandles along multi-lane divided highways. Michigan uses U-turn channels on highways with wide medians and prohibits all left turns at signalized intersections. However, most states do not have standards and handle U-turn provisions on a case-by-case basis.

The prohibition of direct left turns from existing driveways may transfer the displaced left turns to the nearest traffic-signal-controlled intersection unless intermediate U-turn lanes are provided. The increased left-turn volumes at public road intersections would require longer left-turn phases which could reduce the green time and capacity for the through movements. U-turns provisions are especially important along roadways with relatively few median openings.

Several approaches have evolved for accommodating the diverted left-turn volumes by providing U-turn lanes in advance of, at, or beyond intersections. The U-turns may be made from conventional left-turn lanes or via jughandles from the right (curb) lanes. Illustrative treatments are shown in Figure 38. These approaches are as follows:

- Left-turn lanes can be provided for U-turning vehicles in advance (i.e., upstream) of signalized intersections. This avoids concentrating development-related turning traffic at signalized junctions of major crossroads.
- Dual left-turn lanes can be provided at signalized intersections with the inner lane dedicated to U-turns. Many states now provide these lanes, however, they still require multiphase traffic signal controls.
- Left- and U-turn lanes can be provided downstream of signalized intersection, thereby allowing two-phase traffic signal controls.

These approaches translate into two basic design concepts for providing U-turns along multi-lane divided highways without overloading signalized public road intersections:

1. Figure 39 shows how left turns can be provided in advance of intersections in combination with dual left-turn lanes at intersections. This concept avoids concentrating all development-related turning traffic at signalized junctions of major cross roads. The dual left-turn lanes at the signalized crossroad increase left-turn capacity, but still require multiphase operations.
2. Left turns can be prohibited at signalized intersections. Left-and U-turn lanes can be provided about 660 ft on the far side (i.e., downstream) of intersections. These lanes may be signalized and may accommodate dual left-turn/U-turn movements. This concept is sometimes called the "Michigan U" or directional crossover, because Michigan has provided many such lanes along its divided "boulevard" arterials with wide medians (usually 45 ft or more).

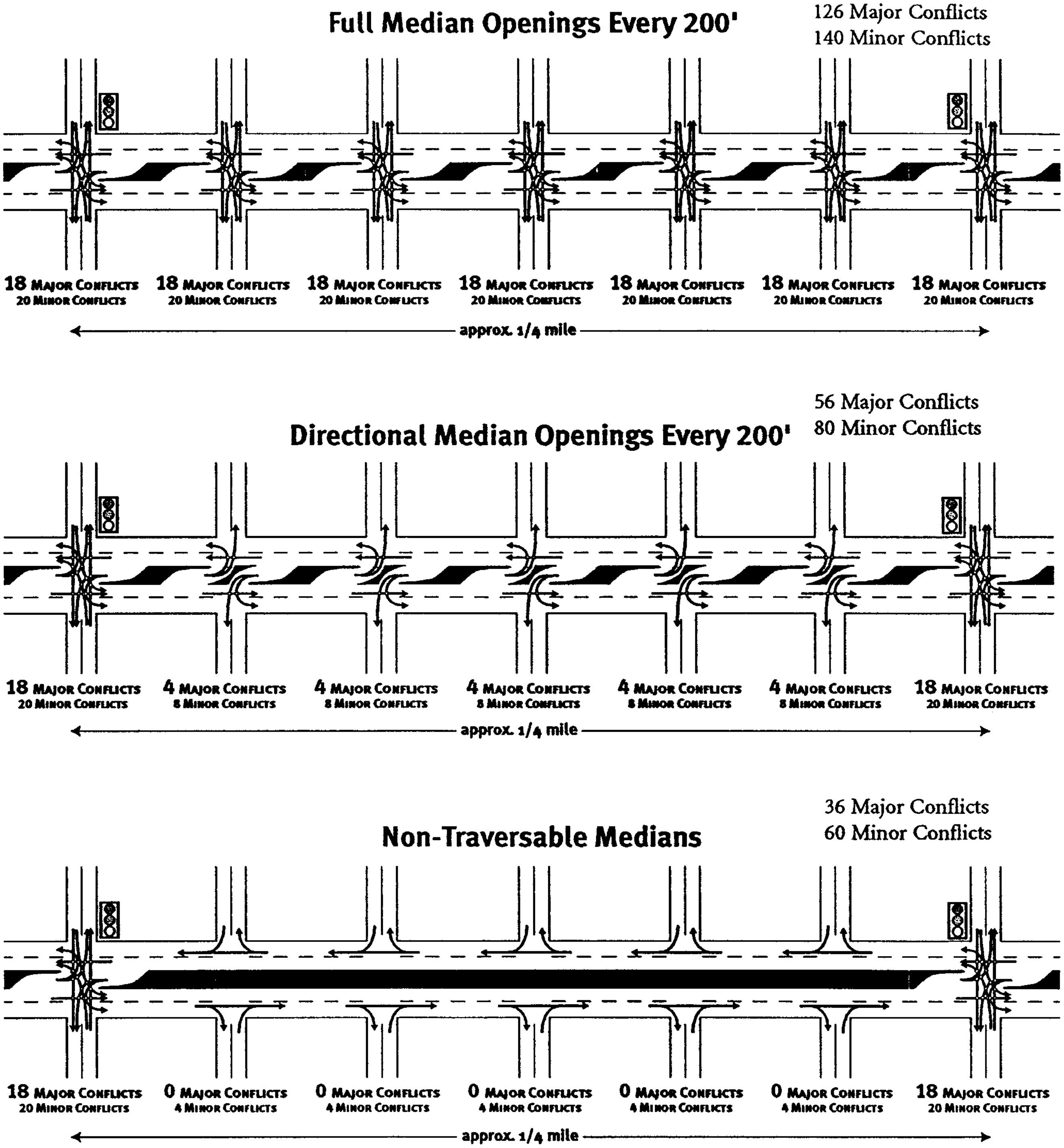


Figure 37. Conflicts at median openings.

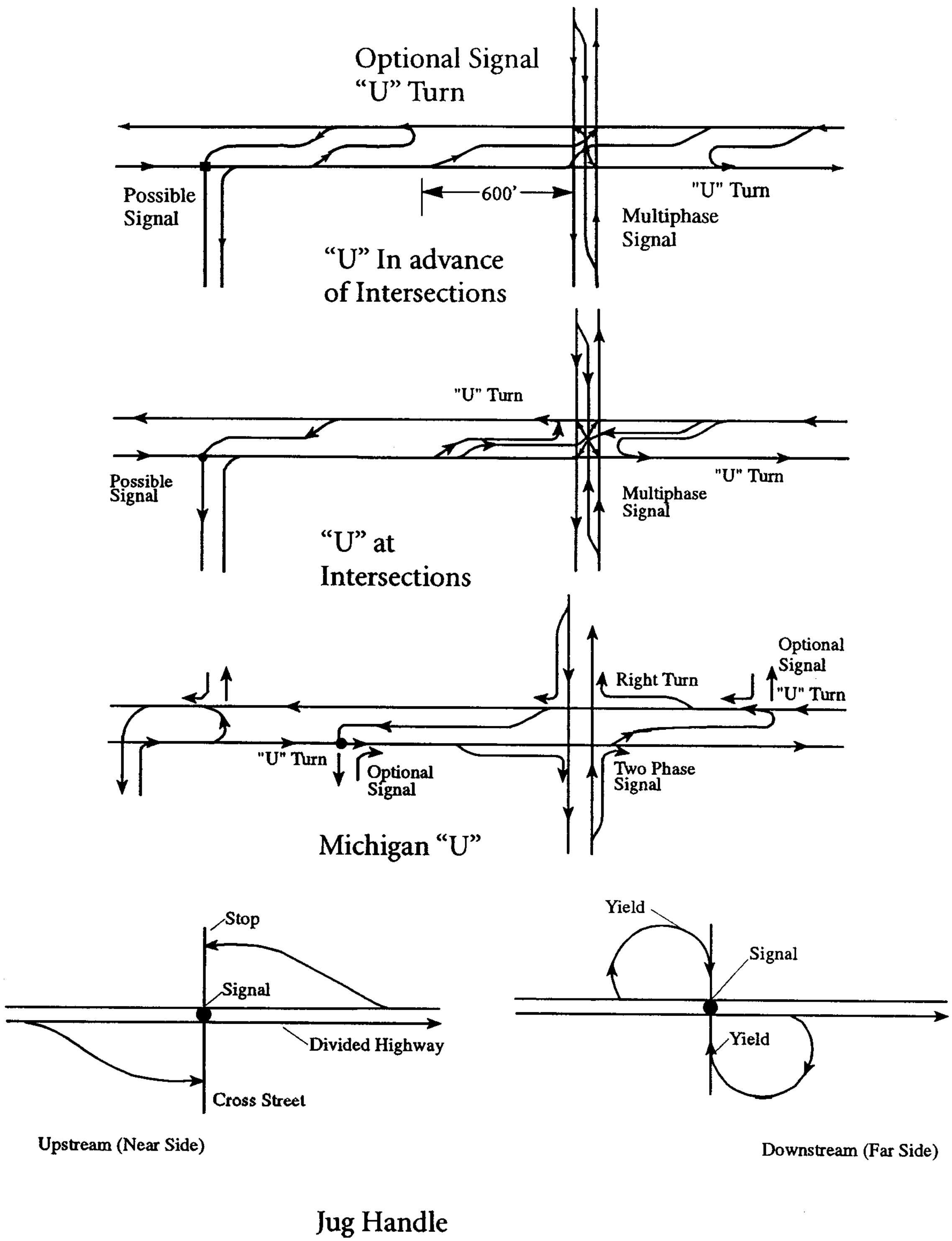


Figure 38. U-turns as an alternative to direct left turns.

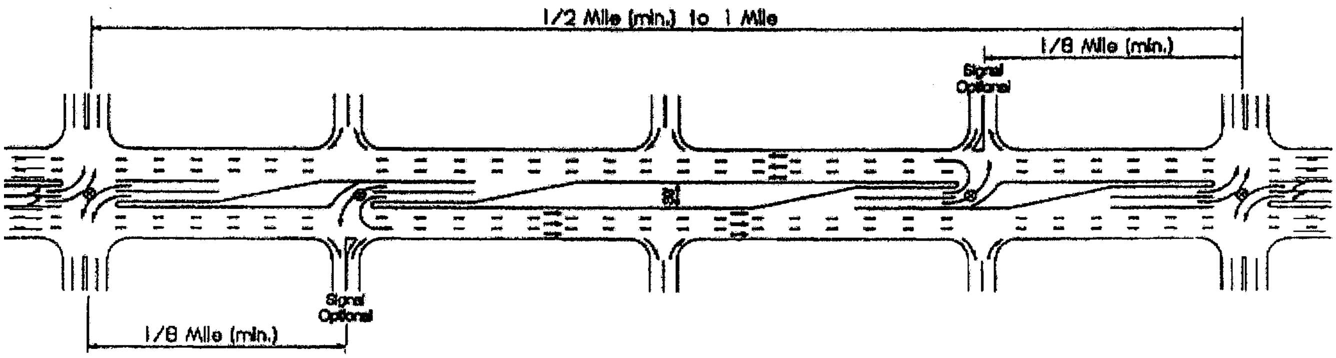


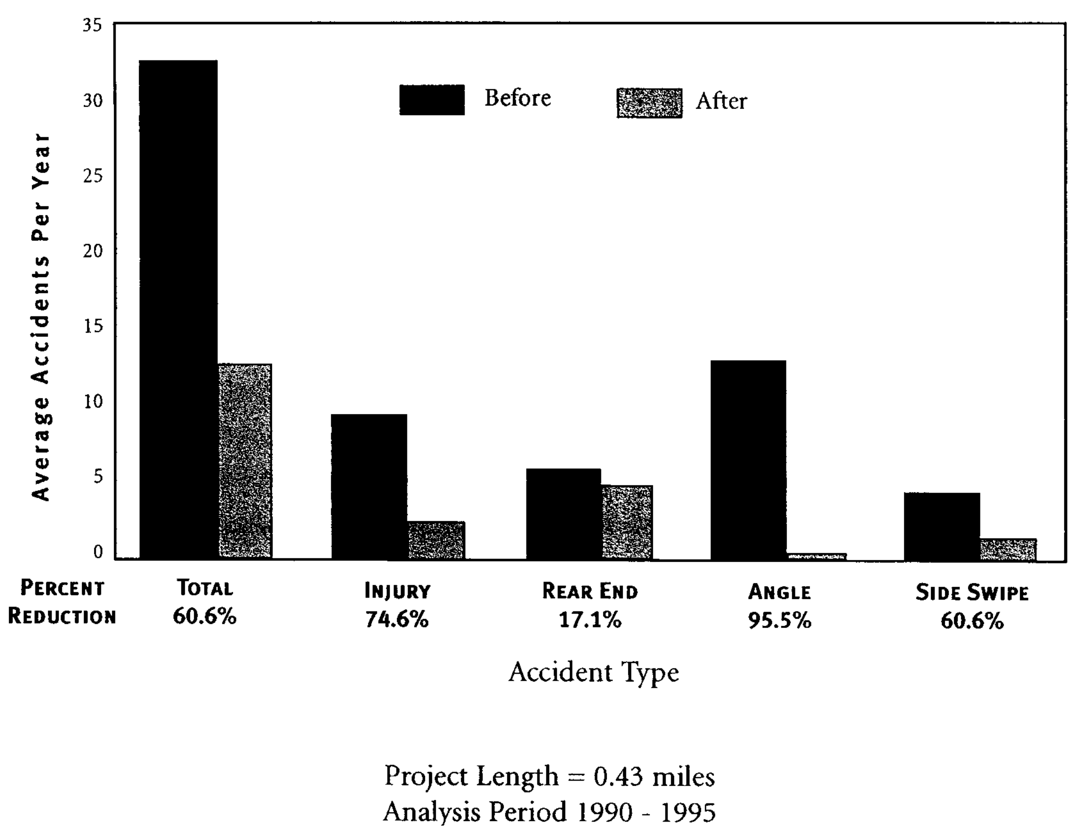
Figure 39. Left-turn/U-turn lanes in advance of intersection.

U-turns as an alternative to direct left-turn exits may also be applied in environments where there are no traffic signals. The effects will be site specific.

SAFETY EXPERIENCE AND ANALYSIS

Several states have reported that closing full-median openings and replacing them with directional U-turns improves safety. Michigan has installed directional U-turn crossovers to accommodate indirect left turns for more than 20 years. Designs have evolved over the years, and several studies have documented their safety and operational benefits. The mean intersection-related rates were 1.388 for directional crossovers as compared with 1.644 for bidirectional crossovers—a 15 percent reduction. The corresponding rates for intersection-related injury accidents were 0.407 and 0.580 respectively—a 30 percent reduction. The study showed substantial reductions in right-angle, rear-end, left-turn, and head-on accidents (*135*).

The results of replacing four bidirectional (full) median openings on 0.43 mi of Grand River Avenue in Detroit, Michigan, with directional openings are shown in Figure 40.



Source: (*136*)

Figure 40. Accident comparisons Grand River Avenue, Detroit.

The average number of accidents per year was reduced from 32 to 13 —about a 61 percent decline. Angle accidents were reduced by 96 percent, sideswipes by 61 percent, and rear-end accidents by 17 percent. Injury accidents decreased by 75 percent (*136*).

The safety effects of directional versus bidirectional crossovers in Michigan were analyzed for some 123 segments of boulevard containing 226 mi of highway (*137*). The segments were separated into those with either bidirectional or directional crossovers, and then further stratified by the number of signals per segment. The results are summarized in Table 78, for those segments where signal density was specified. The percent differences in accidents per 100 million vehicle miles at various signal densities were as follows:

Signals Per Mile	*Completely Bidirectional*	*Completely Directional*	*Percent Difference Difference*
0	420	480	+14
>0–1<	533	339	−36
1–3	1,685	856	−49
> 3	2,658	1,288	−52

These results indicated that on divided highway sections without traffic signals the directional U-turn median crossovers had a 14 percent higher accident rate than those with bidirectional median crossovers. However, as the density of traffic signals increased, divided highways with only directional crossovers had a decreasing relative accident rate as compared with sections of divided highways with bidirec-

TABLE 78 Accident rates by type of crossover and signal density, Michigan

Zero Signals	**Completely Bi-directional**	**Completely Directional**
Segments	11	2
Mileage	28	2.58
ADT	9103	18421
Signal Density	0	0
Driveway Density	12.39	5.81
X-Over Density	7.11	4.65
Accident Rate	420	480
Signals >0 - 1<	**Completely Bi-directional**	**Completely Directional**
Segments	7	3
Mileage	22.64	5.81
ADT	11429	25241
Signal Density	0.44	0.52
Driveway Density	20.45	14.29
X-Over Density	10.6	6.71
Accident Rate	533	339
1 to 3 Signals	**Completely Bi-directional**	**Completely Directional**
Segments	3	19
Mileage	2.20	32.09
ADT	15157	25904
Signal Density	1.36	2.12
Driveway Density	5.91	33.56
X-Over Density	6.82	10.5
Accident Rate	1685	856
Signals >3	**Completely Bi-directional**	**Completely Directional**
Segments	4	25
Mileage	2.32	37.19
ADT	14319	28154
Signal Density	4.74	3.68
Driveway Density	20.69	48.97
X-Over Density	12.93	12.21
Accident Rate	2658	1288

Accident rate per 100 million vehicle miles.
Source: *(137)*

TABLE 79 Estimated capacity gains Michigan "U" vs. dual left-turn lanes

Capacity	Arterial*	
	6 Lanes	8 Lanes
Artery Only 60-40 directional split vs. 45% through 15% left-turn 40% cross traffic	+14%	+18%
Arterial Plus Cross Street 60-40 directional split vs. 45% arterial through 15% arterial left-turn 30% cross street through 10% cross street left turn	+16%	+18%

Note: * Cross street - 4 lanes with right and left-turn lanes on all approaches before, right turn lanes after.

Source: *(138)*

tional crossovers. When there was more than one signal on average per section, the accident rate for directional crossovers was roughly one-half of that for bidirectional crossovers. The study also compared accident rates for boulevard sections with road sections containing TWLTLs. Boulevard sections with directional crossovers had 426 accidents per hundred million VMT as compared with 857 for roads with TWLTLs.

OPERATIONS EXPERIENCE AND ANALYSIS

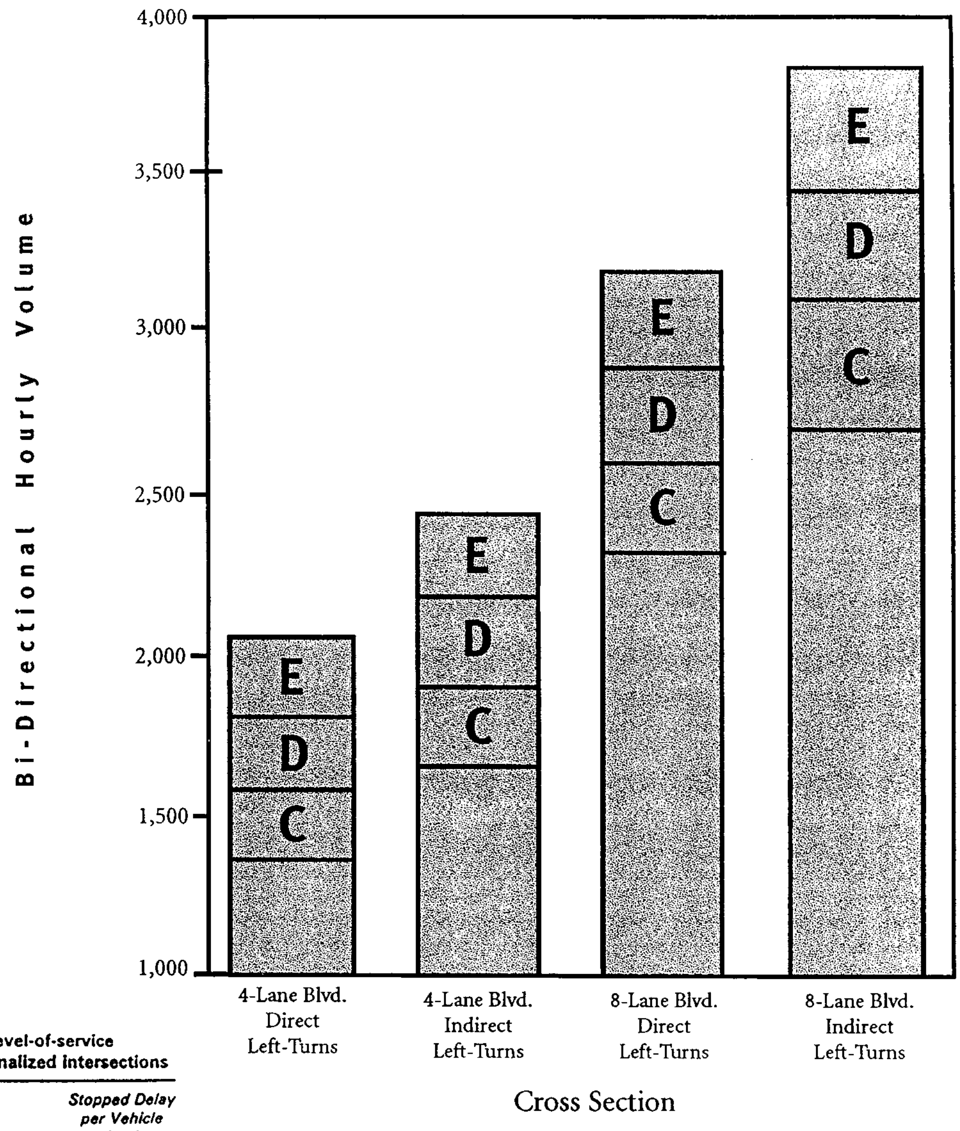
A few studies have analyzed the capacity gains and delay reductions associated with providing U-turns as an alternative to direct left turns. A study by Koepke and Levinson (*138*) found that the directional U-turn design provided about 14 to 18 percent more capacity than the conventional dual left-turn lane designs. Table 79 summarizes the detailed analysis results. Results of simulations of critical lane volumes, taking into account overlapping movements, are shown in Table 80. The simulations showed reductions of about 7 to 17 percent in critical lane volumes, depending on the number of arterial lanes (six or eight) and the traffic mix.

A Michigan study (*136*) cited capacity gains of 20 to 50 percent as a result of prohibiting left turns at intersections and providing two-phase signal operations. Reported level of service comparisons for 4- and 8-lane boulevards, shown in Figure 41, suggested a 20 percent capacity gain. This increase is consistent with that estimated by Koepke and Levinson (*138*).

TABLE 80 Estimated reduction in critical lane volumes Michigan "U" vs. dual left-turn lanes

Condition	Arterial	
	6 Lanes	8 Lanes
Left-Turns on Artery Proportional to Through Traffic Volumes	- 7%	-10%
Heavy Left-Turns on Artery Opposing Heavy Through Traffic	-15%	-17%

Source: *(138)*



Level-of-service criteria for signalized intersections

Level of Service	*Stopped Delay per Vehicle (sec)*
A	≤ 5.0
B	5.1 to 15.0
C	15.1 to 25.0
D	25.1 to 40.0
E	40.1 to 60.0
F	> 60.0

SOURCE: *Highway Capacity Manual*, Special Report 209, Transportation Research Board, Washington, DC, 1985, Table 9-1, p. 9-4.

Source: *(136)*

Figure 41. Divided highways level of service comparison, Michigan.

There has been little documentation of the effects of providing U-turns as an alternative to direct left turns from driveways. Therefore, additional analysis of the operational effects associated with diverting the left turns was performed.

A review of the elements and factors associated with direct and indirect left turns at unsignalized intersections indicates that direct left turns must find gaps in the two-directional traffic stream. In contrast, the right-turn/U-turn maneuver involves obtaining gaps in one direction at a time.

1. Direct Left Turns. The direct left-turn egress movement from an access drive or minor cross street must yield to all other movements. Thus, it is the most likely movement to be delayed. On roadways with wide medians, the direct left-turn exit from an abutting development requires (a) stopping in the driveway, (b) selecting a suitable gap in the traffic stream approaching from the left, (c) accelerating across the traffic lanes and coming to a stop in the median, (d) selecting a suitable gap in the traffic stream

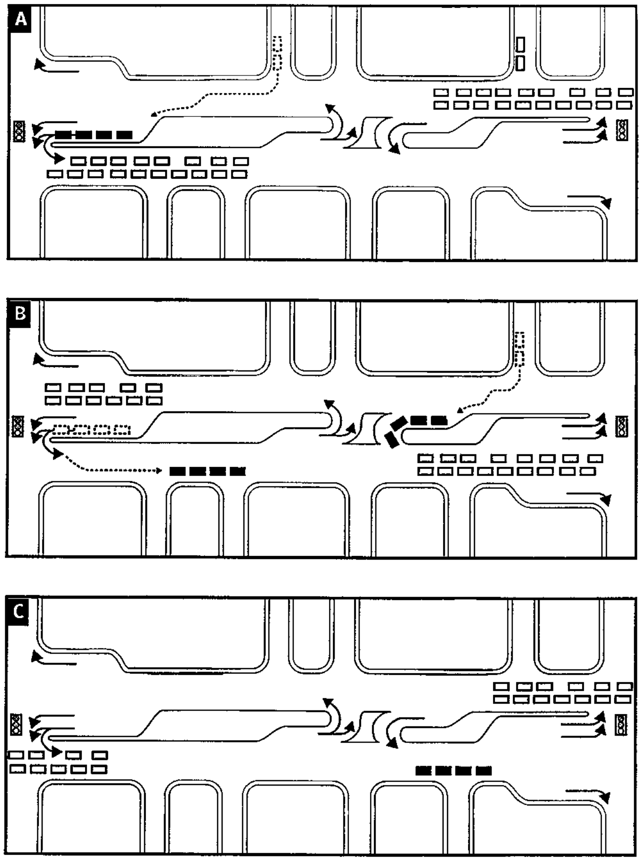


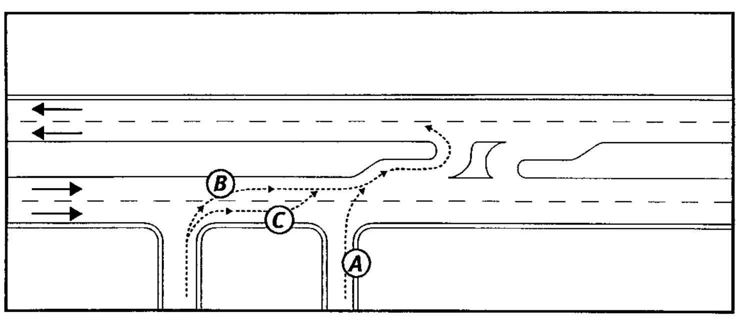
Figure 42. Right-turn/U-turn design maximizes drivewway egress in a signalized system.

approaching from the right, and (e) completing the left-turn movement. However, with narrow medians (i.e., less than 20 ft in width), drivers must search for gaps available in both directions of travel.

The high arterial volumes found in urban and suburban areas—500 to 700 vehicles per hour per lane—result in few usable gaps for exiting left turns. This translates into limited capacity and long waits. Sometimes motorists turn right to avoid the difficulty associated with the left-turn delays. Moreover, the potential capacity for left-turn exits becomes very small when total conflicting volumes exceed 1,500 vph (*154, 155*). Large vehicles and access drives on upgrades further reduce the actual capacity.

2. Indirect Left Turns. The right-turn/U-turn movements only involve obtaining gaps in one direction at a time. The elements in the right turn followed by a U-turn include (a) stopping in the driveway exit; (b) selecting a suitable gap in traffic approaching from the left; and (c) turning right, accelerating, weaving, and then coming to a stop in the left-turn/U-turn lane. Assuming "stop sign" control, vehicles then must wait for a gap in the opposing traffic before completing the U-turn movement. As shown in Figure 42, the operating problems associated with direct left turns are largely eliminated.

The safety and travel time effects of the right-turn/U-turn maneuver from access drives are a function of artery traffic volumes and the separation distances between driveway exits and the U-turn channel. Figure 43 illustrates the gap acceptance and



Weaving Patterns

A Short separation:
Drivers select a suitable simultaneous gap in all traffic lanes and then make a direct entry into the left-turn/u-turn lane

B Long separation, low volume approaching from the left:
Drivers select a simultaneous gap in all traffic lanes, turn right, and make a direct entry maneuver into the left through lane

C Long separation, high volume or low volume and high-speed traffic from the left:
Drivers wait for suitable gap, turn right, accelerate and make a lane change maneuver, then decelerate as they enter the left-turn lane

Figure 43. Right-turn/U-turn maneuver from access drive to U-turn median opening.

weaving patterns for various separation distances and artery traffic volumes. Increasing separation distance gives drivers more maneuvering space, allows longer storage lanes, and improves safety, although travel times could increase.

Travel Time Effects

The travel time effects associated with providing U-turns as an alternative to direct left turns were estimated. An analytical model was developed and calibrated to estimate the travel time savings (or losses) when unsignalized left turns are diverted for various distances. It can apply to both suburban and rural environments where there are no nearby traffic signals. This model is shown in Figure 44. It reflects the number and importance of the conflicts associated with the events involved in each movement. The key findings are as follows:

- A right turn followed by a U-turn will require up to 2 min of travel time, assuming a diversion distance of about 1,320 ft.
- A single-stage left-turn exit (where medians are too narrow to safely store two or more vehicles) will involve the following delays (not including acceleration times):

Volumes (vph)		
Artery Two directions	*Left-Turn Exit*	*Delay per Vehicle (Seconds)*
1,000	50	20
1,000	100	25
2,000	50	200
2,000	100	530

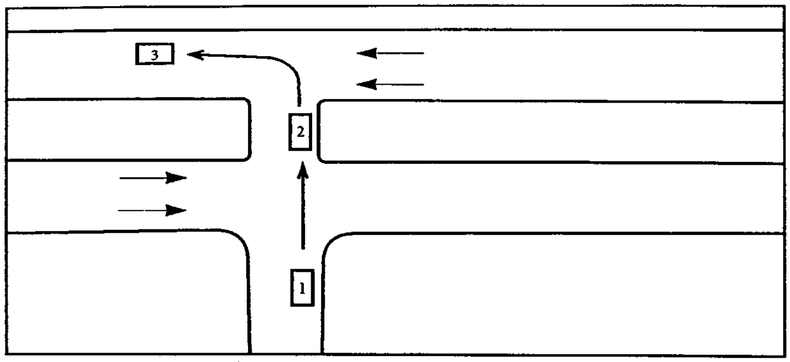
  These values suggest that when arterial traffic exceeds 375 to 500 vphpl on a four-lane facility the computed delays would exceed those associated with the right-turn/U-turn movement. Higher volumes (700–900 vphpl) that are common along many suburban arterials would produce even higher left-turn egress delays in theory. In practice, motorists become impatient when gaps exceed 1 to 2 min and are apt to avoid the direct left-turn egress.
- The two-stage left-turn process, where medians can safely store waiting vehicles, reduces delays to left-turning traffic. Nevertheless, this process still results in long delays to left-turning vehicles when the volumes on the major street are relatively high (i.e., more than 2,000 vph), and the left turns exceed 50 per hour. In these cases, even with substantial circuity (1,320 ft or 402 m from the access drive to the U-turn median opening, or a 0.5 mi of additional travel) the right turn followed by a U-turn involves less time than calculated left-turn egress movements under moderate to high volumes.

APPLICATION GUIDELINES

The following assumptions underly the application of indirect left turns as an alternative to direct left turns:

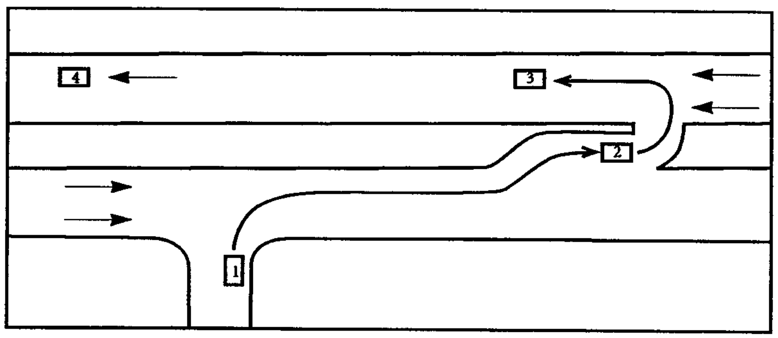
- A U-turn median opening can serve several access drives and eliminate the need for direct left-turn exit movements from driveways.
- A median at least 25 ft (7.6 m) wide is necessary to help ensure that a crossing or left-turning vehicle, stopped in the median perpendicular to the through traffic lane, will not extend beyond the median.
- A vehicle turning left from an access drive and stopping in the median opening must yield to through traffic approaching from the left and vehicles turning left from the through lane. If there is even a moderate volume of left turns from the through lane, the left-turn egress capacity is small. If it is a full median opening, the left turn from an access drive also needs to yield to an opposing left-turning vehicle already stopped in the median opening. These conditions are alleviated when the direct left-turn exits are prohibited. A narrow median opening will allow only one left-turning vehicle at a time to advance into the median opening. A wide median opening allows multiple vehicles to stop in the opening. However, this may create a confusing and conflicting pattern of movements, angle stopping in the median opening, and some drivers' vision obstructed by other vehicles.
- As the intensity of land development increases, the traffic demand to access abutting properties also increases. Left-turn traffic at closely spaced full median openings can "interlock."
- A left-turn lane at a median opening for directional left-turn/U-turn movements can be designed to store several vehicles because storage is parallel to the through traffic lanes.
- Median storage for larger vehicles such as recreational vehicles, school buses, trucks, and a car pulling a trailer cannot be provided unless the median is exceptionally wide. It is usually more practical to provide for U-turns by such vehicles at selected locations using a jughandle design. Alternatively, added width can be provided in the opposing paved travel way at selected locations to accommodate these wide-radius turns.
- In prohibiting direct left-turn exits from driveways, it is desirable to provide U-turn lanes in advance of downstream signalized intersections. Passenger cars can normally make U-turns along divided six-lane arterials. Along divided four-lane arterials, it may be desirable to add width or to use paved shoulders to accommodate U-turns.
- When U-turns are provided as an alternative to left turns, median width at signalized intersections should be adequate to accommodate the vehicles normally making the

Direct Left-Turn Egress Movement



Vehicle Location	Driving Event
1	Waits for suitable gap in traffic approaching from the left
1 ⟶ 2	Crosses to the median
2	Waits for suitable gap in traffic approaching from the right
2 ⟶ 3	Accelerates to average speed on major roadway
3	Average speed is achieved

Right Turn Followed by a U-turn



Vehicle Location	Driving Event
1	Waits for suitable gap in traffic approaching from the left
1 ⟶ 2	Accelerates, cruises at constant speed (if applicable) and enters left-turn bay while decelerating to a stop
2	Waits for suitable gap in oncoming traffic
2 ⟶ 3	Accelerates to speed of traffic
3	Average speed is achieved
3 ⟶ 4	Maintains constant speed
4	The same point at which a driver having made a direct left-turn egress movement would attain the average speed on the major roadway

Figure 44. Analytical framework for providing U-turns as an alternative to direct left turns.

U-turns. Generally, a median width of at least 40 ft (preferably 60 ft) should be available. Midblock median openings may be made with less than 30-ft width.

- It is essential to provide a consistent treatment for left turns along any highway. The differing left-turn options (direct left turns, jughandle, Michigan U) should not be mixed. Driver expectancy must be respected.

The “Michigan U” concept for indirect left turns places the U-turn channels about 660 ft downstream of intersections, eliminates all left turns at the main intersection, and allows two-phase signal controls. However, it requires a median width at intersections of 40 to 60 ft, depending on the type of vehicles involved. Narrower cross sections may be sufficient where there are few large trucks.

The directional U-turn design, such as applied in Michigan, generally requires more median width than the conventional design. Its operational advantages include the following:

- It allows two-phase signal operations with a greater proportion of time allocated to arterial traffic flow. Shorter cycle lengths are possible, allowing more flexibility in signal progression.
- Each direction of travel can be treated as a one-way street, with separately signalized driveways if desired.
- The wider median improves aesthetics and provides storage space for pedestrians.
- Through lane fly-overs or fly-unders can be incorporated within the right-of-way with relatively little or no widening as the need arises.

Safety Effects

The safety effects of U-turns as an alternative to left turns can be estimated from Table 81. This table suggests a reduction of about 20 percent by eliminating direct left turns from driveways (*139*). Roadways with wide medians and directional crossovers had half the accident rates of roads with TWLTLs (*137*).

Statewide accident analyses of stop-sign-controlled, directional versus bidirectional left turns in Michigan (*135*) reported a 15 percent reduction in accident rates. More recent studies (*137*) were performed of directional and bidirectional left turns at signalized and unsignalized locations. On highway sections without signals, the directional U-turn median crossover had a 14 percent higher accident rate than the bidirectional median crossover. However, as the density of traffic signals increased, divided highways with only directional crossovers had a decreasing relative accident rate compared with sections with bidirectional crossovers. Accident rate reductions of 35 to 50 percent were shown where there was more than one traffic signal per section. Reductions of more than 60 percent were reported at individual intersections.

Operations Effects

Operational benefits include shorter travel times, less delay, and increased capacity. Right turns followed by U-turns can provide comparable, if not shorter, travel times than direct left turns from driveways under heavy volume conditions when the diversion distances are generally less than 0.5 mi. Simulation analysis in Michigan reported that indirect left turns at unsignalized locations may experience less delay than direct

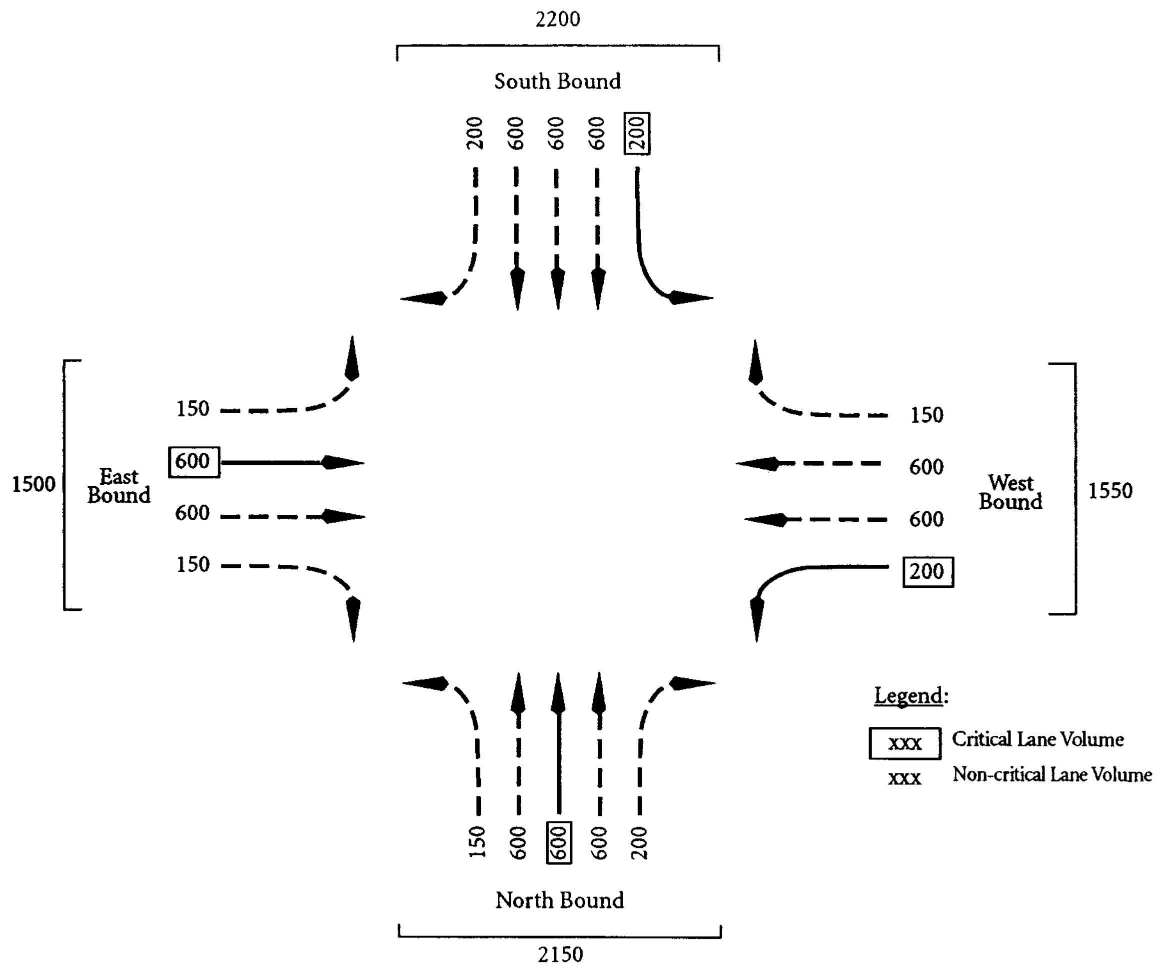
TABLE 81 Accident rate differences—U-Turns as alternate to direct left turns

Location	Change	Difference in Accident Rate	Source
1. US-1, Florida	Driveway Left-Turns Replaced by Right-turn/U-turn	-22%	*(139)*
2. Michigan U	Directional Crossover Versus Bi-Directional Crossover (Unsignalized with Opposing Traffic)	+14%	*(137)*
3. Michigan U	Directional Crossover Versus Bi-Directional Crossover (Signalized with Opposing Traffic)	-35% to 50%	*(137)*
4. Michigan U	Directional Crossover Versus TWLTL	-50%	*(137)*

left turns depending on the arterial volume, the left-turn volume, and the additional travel distance involved. Overall arterial delay would be less when the volume-to-saturation ratio exceeds 0.3. The provision of U-turns on the downstream side of signalized intersections and right-turn lanes on all approaches as well as the prohibition of direct left turns can increase intersection capacity by 14 to 20 percent over intersections where single (or dual) left-turn lanes are provided.

Capacities can be readily computed by conventional HCM methods. Alternatively, critical lane analysis can be used to provide an initial picture of intersection performance. In both cases, the actual traffic entering the intersection (after diversion of left turns) must be considered.

Figures 45 through 48 shows the effects of providing multiple left-turn lanes and redirecting left-turn lanes at intersections based on critical lane analysis. Similar computations



SUMMARY OF CRITICAL VOLUMES

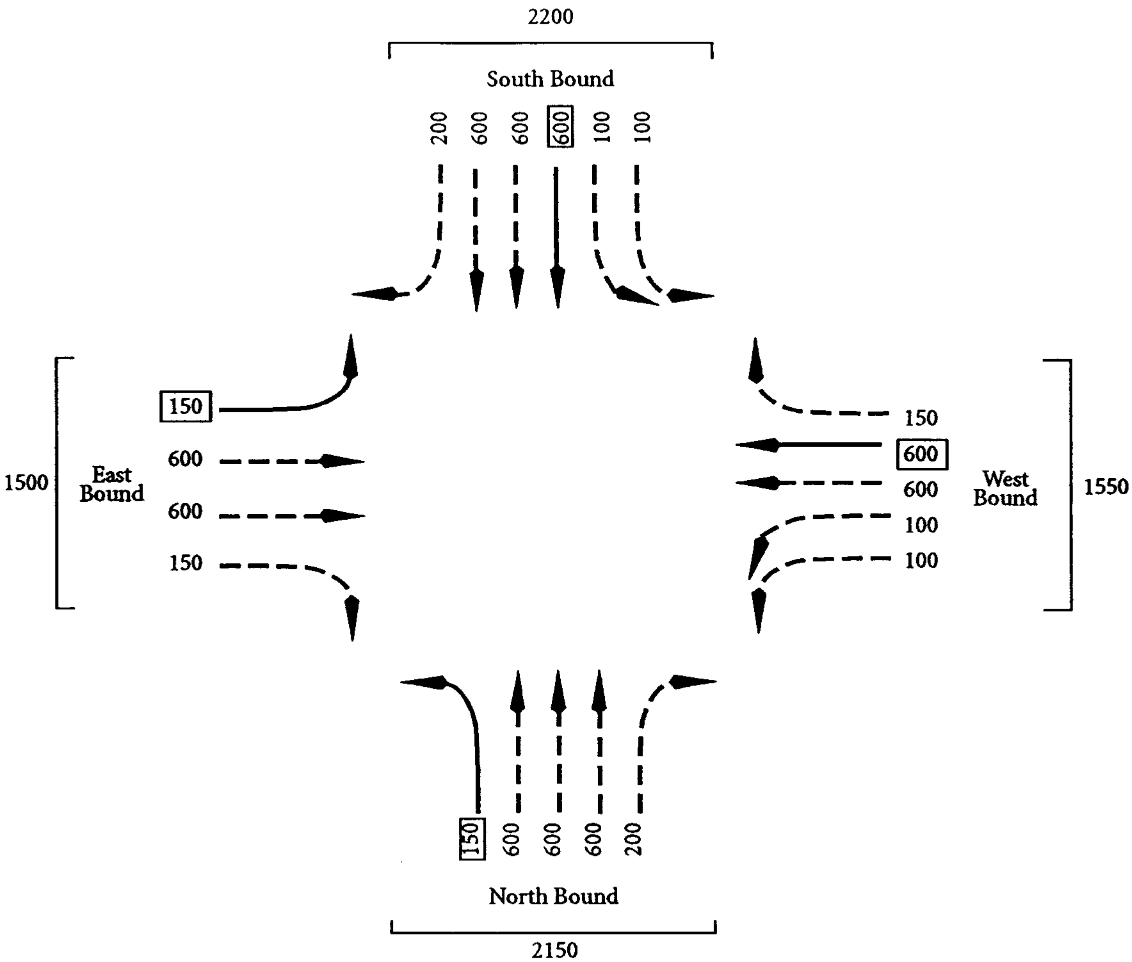
EB LT -	150	NB LT -	150
WB TH -	600	SB TH -	600
	750		750
WB LT -	200	SB LT -	200
EB TH -	600	NB TH -	600
	800		800

or / or

$\frac{800}{\text{E-W Critical}} + \frac{800}{\text{N-S Critical}} = \frac{1600}{\text{Total}}$

Source: *(140)*

Figure 45. Critical volumes with single left-turn lanes.



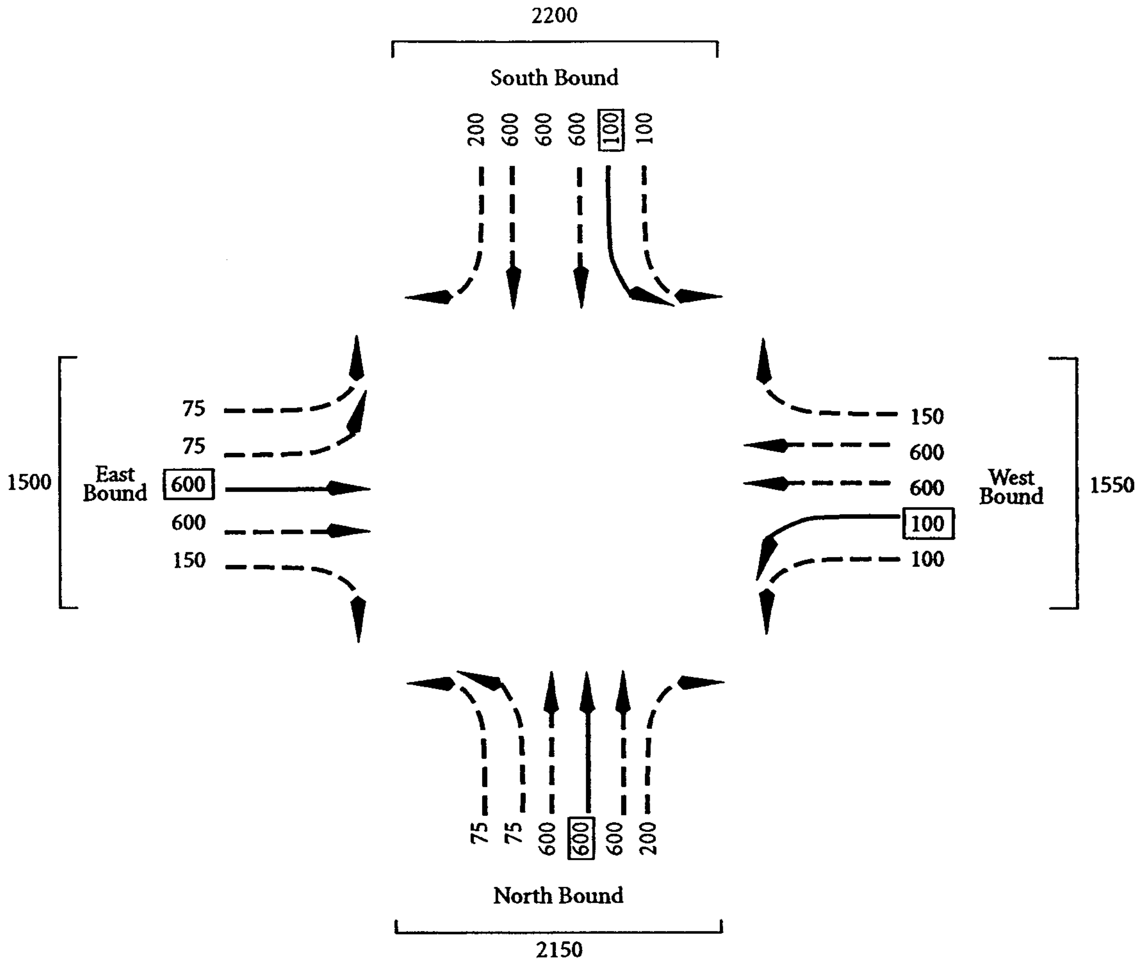
SUMMARY OF CRITICAL VOLUMES

EB LT -	150		NB LT -	150	
WB TH -	600		SB TH -	600	
	750			750	
WB LT -	100	or	SB LT -	100	or
EB TH -	600		NB TH -	600	
	700			700	

$\frac{750}{\text{E-W Critical}} + \frac{750}{\text{N-S Critical}} = \frac{1500}{\text{Total}}$

Source: *(140)*

Figure 46. Critical volumes with dual left turns on high-volume approaches.



SUMMARY OF CRITICAL VOLUMES

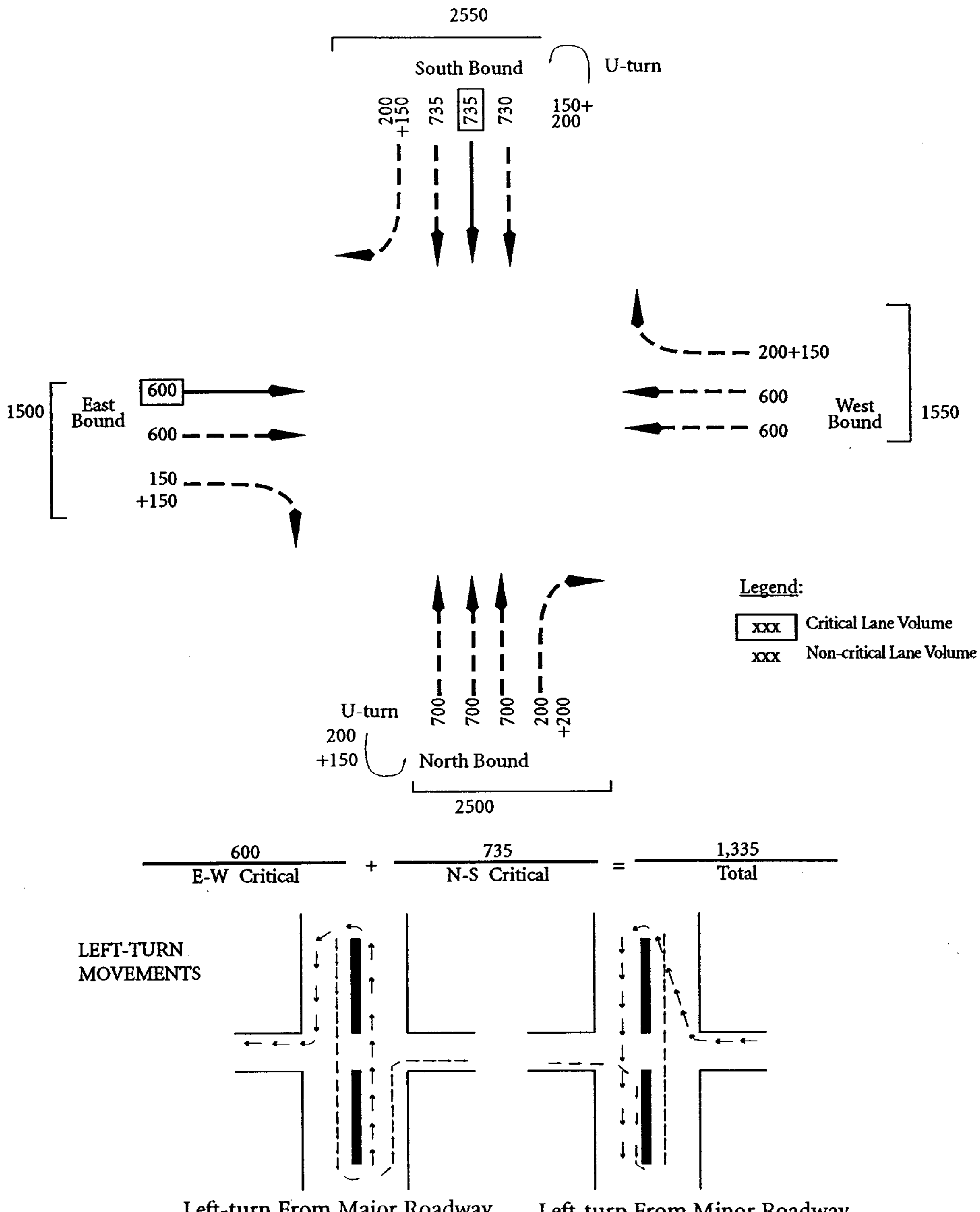
EB LT -	75	NB LT -	75
WB TH -	600	SB TH -	600
	675		675
WB LT -	100	SB LT -	100
EB TH -	600	NB TH -	600
	700		700

or / or

700 E-W Critical + 700 N-S Critical = 1400 Total

Source: (140)

Figure 47. Critical volumes with dual left turns on all approaches.



Source: Estimated

Figure 48. Critical volumes with left turns diverted (i.e., Michigan "U").

can be made for other intersection configurations or traffic volume mixes. The results are as follows:

Figure	*Conditions*	*Critical Lane Volume*	*Percent Reduction*
45	Single Left-Turn Lanes	1,600	–
46	Dual Left Turns on High-Volume Approach	1,500	6
47	Dual Left-Turn Lanes on All Approaches	1,400	12
48	Left Turns Rerouted	1,335	17

The provision of dual left-turn lanes on all approaches reduces critical lane volumes by 12 percent, but still requires multiphase traffic signal controls. The rerouting of left turns (and their prohibition at the main intersection) reduces critical lane volumes by 17 percent and allows two-phase signal controls.

CHAPTER **9**

ACCESS SEPARATION AT INTERCHANGES (TECHNIQUE 1D)

INTRODUCTION

Freeway interchanges provide the means of moving traffic between freeways and arterial streets and have become important focal points of activity in urban, suburban, and even some rural locations. They have become magnets for road traffic, and they have stimulated much roadside development in their environs. Where intersections are too close to the ramp termini of the arterial/freeway interchange, heavy weaving volumes, complex traffic signal operations, frequent accidents, and recurrent congestion have resulted. As a result, land development at interchanges should be sufficiently separated from ramp terminals.

Although access is controlled on the freeway within the interchange area, there is often little, if any, access control along the arterial roads. Existing street intersections along the arterial are often spaced too close to interchanges. In addition, curb cuts and median breaks for large and small traffic generators compound the problem. There is growing recognition that access separation distances and roadway geometry should be improved from an access management perspective. This need was recognized in a recently completed research study, NCHRP Project 3-47—Capacity Analysis of Interchange Ramp Terminals (*141*).

There are also land-use issues (e.g., how an interchange relates to the surrounding community, how new land development conflicts with existing activities, and how improper use of the land will affect its potential). These too affect or may be affected by access separation distances.

POLICIES AND PRACTICES

Access separation policies are contained in various AASHTO publications and in state DOT design policies. The AASHTO booklet, *A Policy on Design Standards - Interstate System, July 1991* (*142*), for example, states that, "control should extend beyond the ramp terminal at least 100 feet in urban areas and 300 feet in rural areas. These distances should usually satisfy congestion concerns. However, in areas where the potential exists which would create traffic problems, it may be appropriate to consider longer lengths of access control." Many states have established more stringent policies than AASHTO that reflect the importance of providing sufficient access control lengths and/or separation distances along crossroads (arterials) at interchanges.

State Policies

Table 82 summarizes the separations reported by some 19 state (or provincial) DOTs. Separation distances in rural areas range from about 300 to 1,000 ft, and those in urban areas range from 100 to 700 ft. The guidelines generally are less than some of the access spacing requirements that are needed to ensure good arterial signal progression and to provide adequate weaving and storage for turning traffic—left turns in particular.

Case Studies

The following lessons are apparent from the case studies of access separation distances summarized in Table 83:

1. The proximity of traffic signals to upstream free-flowing or yield-control ramps results in congestion, with spillback onto ramps.
2. Movements from free-flow ramps into left-turn lanes pose two problems: (a) weaving distances are usually inadequate, and (b) heavy left-turn movements impede arterial traffic. This condition can be alleviated in part by signalizing the ramp terminal (subject to progression considerations) and/or increasing the separation distances; this was done at several interchanges in Florida.
3. Often, the arterial roadway functions as a distributor for freeway-to-activity center traffic. This "double loads" the arterial by superimposing short trips and turning movements onto the normal arterial traffic. Alleviating this condition calls for restructuring both street and interchange patterns.

STUDIES AND ANALYSIS

The spacing between ramp terminals and cross-route access points must allow for proper merging, weaving, and diverging of ramp and arterial traffic. The ability to change

TABLE 82 Access separation distances at interchanges

State	Rural	Urban
1. Alabama	300 feet to access	100 feet to access
2. Alberta	425m from signal to access 150m from ramp to access	Same
3. California	125m minimum distance from ramp to nearest intersection	Same
4. Illinois	500 to 700 feet	Same
5. Iowa	200 m rural primary highway 100m other road or street	50m urban
6. Kentucky	300 feet to access	100 feet to access
7. Maryland	Based on geometrics, speeds, volumes, presence of signals and queuing	Same
8. N. Dakota	AASHTO guidelines (300 feet)	AASHTO guidelines (100 feet)
9. Ohio	600 feet for diamond interchange, 1,000 feet for cloverleaf.	
10. Oregon	300 feet from frontage road 500 feet from ramp (suggested)	Same
11. Pennsylvania	AASHTO guidelines (300 feet)	AASHTO guidelines (100 feet)
12. South Carolina	500 feet desirable, 300 feet minimum	300 feet desirable, 150 feet minimum
13. Texas	AASHTO guidelines (300 feet)	AASHTO guidelines (100 feet)
14. Utah	300 feet to access	150 feet to access
15. Virginia	200 feet from entrance ramp	Same
16. West Virginia	300 feet to access	100 feet to access
17. Washington	300 feet to access	300 feet to access
18. Wisconsin	1,000 feet to access (500 feet - minor roads)	500 feet to access
19. Wyoming	300 feet to access	150 feet to access

lanes and the deceleration and storage requirements for left turns also influence access spacing.

A 1968 study identified general principles that apply to most types of interchange development (*143*):

- The most appropriate use of interchange area land (in terms of the regional economy) should be encouraged, consistent with maintaining an efficient and safe traffic facility.
- Land near interchanges should have sufficient depth to provide access to interior tracts, and developments with shallow frontages should be discouraged.
- Land use should be of a type that requires only a minimum number of access points and intersections along the arterial, particularly in the vicinity of ramp entrances and terminals.
- Development with frontage facing away from the arterial and onto service drives and local streets should be encouraged.

The study recommended that "the design of interchange traffic facilities should be coordinated with the simultaneous development of a comprehensive plan for the interchange area," and that "the practice of acquiring property access rights be expanded in critical cross-route problem areas." These findings and recommendations provided the basis for Illinois' access control policies at interchanges.

A background paper on interchange access management prepared for Oregon DOT sets forth suggested guidelines and standards for access spacing at interchanges (*144*). The guidelines resulted from detailed analysis of the various merging, weaving, sight distance, and left-turn storage requirements at various design volumes and speeds. The resulting guidelines for freeway interchanges are summarized in Table 84 for 2-lane and multi-lane roads.

- The nearest major cross route (arterial) intersection with a street on both sides of the interchange should not be less than 1,320 ft.
- The distance to the first access on a 2-lane road ranges from 750 to 1,320 ft. On a 4-lane road, this distance ranges from 750 to 1,320 ft for right turns downstream from off-ramps and 990 to 1,320 ft for median openings and for right-turn entry upstream from on-ramps.

APPLICATION GUIDELINES

Several application guidelines were developed on the basis of the review of previous studies and current experiences found in the case studies and elsewhere. These guide-

TABLE 83 Case studies of access separtation distances at interchanges

Case Study	Separation Distance	Comments
1. I-75 / Fruitville Rd. Sarasota Co., Florida	550 feet before 1,050 feet after	Ramp relocated and signalized to eliminate weave
2. I-75 / Jacaranda Blvd. Sarasota Co., Florida	480 feet before 1,000 feet after	Ramp relocated and signalized to eliminate weave
3. I-275 / 6-Mile Rd. Wayne Co., Michigan	800 feet	Directional crossover allows left-turn entrance only
4. Route 46 / Union Blvd. Passaic, New Jersey	750 feet	Frequent offset driveways, short weaves for left-turns
5. I-495 / Route 454 Suffolk Co., NY	Not applicable	Heavy overlapping left-turns at diamond interchange creates extensive backups
6. I-77 / SR 18 Summit County, Ohio	Varies; 120 to 300 feet	Heavy left-turns require short weaves beyond cloverleaf interchange - with frequent congestion
7. US Route 1 / Route 213 Bucks County, PA	120 feet	Inadequate corner clearance and extensive curb cuts on far side of nearby signalized intersection
8. I-95 / Temple Avenue Chesterfield Co., VA	350 feet	Commercial development within Interchange
9. I-295 / US 360 Richmond, VA	Varies; 600 to 750 feet	Weaving conflicts and backup onto ramp from signalized intersection
10. I-5 & Harrison Avenue Centralia, Washington	300 feet	Frequent curb cuts and short separation distances; congestion at signalized intersections

Source: Urbitran Interviews and Analysis

lines provide a framework and suggest representative values; however, they should be adjusted to reflect local conditions.

There are many different types of interchanges along freeways, expressways, and strategic arterials. They range from diamonds to full cloverleafs and may include direct connections; however, from an access spacing standpoint, they can be categorized as those with free-flowing entrances and exit ramps and those where ramp entrances and terminals are controlled by traffic signals or stop signs. These types and their access management and spacing implications are shown in Figure 49 and described as follows:

- **Ramp Intersections Controlled by Traffic Signals.** Signalized and unsignalized access spacing should reflect established guidelines for the types and operating environments under consideration. The signalized ramp intersection is treated similarly to other signalized intersections; however, queuing from the ramp onto the freeway mainline must be avoided.
- **Ramps with Free-Flow Entry or Exit.** Access separation distances to the first downstream median opening or signalized intersection should consider the various movements and operations involved. These include the merge where the ramp traffic enters the arterial, the weaving movements to enter the median lanes, the transition into left-turn lanes, and the required storage length.

Estimating Separation Distances

Figure 50 illustrates the elements to be considered in computing access separation distances. These include (1) the dis-

TABLE 84 Suggested minimum access spacing standards for 2- and 4-lane cross routes at freeway interchanges, Oregon

2-Lane Cross Routes

Access Type	Area Type		
	Fully Developed Urban (45 mph)	Suburban (45 mph)	Rural (55 mph)
First Access	750	990	1,320
First Major Signalized Intersection	1,320	1,320	1,320

4-Lane Cross Routes

Access Type	Area Type		
	Fully Developed Urban (35 mph)	Suburban (45 mph)	Rural (55 mph)
First Access from Off-Ramp	750	990	1,320
First Median Opening	990	1,320	1,320
First Access Before On-Ramp	990	1,320	1,320
First Major Signalized Intersection	2,640	2,640	2,640

Source: *(144)*

tance required to weave across the through travel lanes, (2) the distance required for transition (i.e., to move) into the left-turn lane or lanes, (3) the distance needed to store left turns with a low likelihood of failure, and (4) the distance from the stop line to the centerline of the intersecting road or drive. In addition, driver perception-reaction distance could be added.

Where only right-turn access is involved (there would be no left turns or median breaks) the relevant distances include weaving and the distance to the centerline.

Weaving Distances

Most weaving analysis has centered on freeway operations. Oregon and Florida studies used a series of curves developed by Jack Leisch (*144, 145, 146*). The values from these curves are listed in Table 85. Weaving distances are given for five speed ranges for weaving volumes from 200 to 2,600 vehicles per hour. Weaving distances of less than 400 ft should generally not be used.

Table 85 may be used to estimate the required weaving distance for a given weaving volume and speed. Alternatively, it may be used to estimate the likely speeds for a given volume and weaving distance. At speeds and volumes normally encountered in urban and suburban areas, weaving distances of 700 to 800 ft will be adequate for most conditions along 2-lane roads. Along multilane roads, weaving distances of 1,200 to 1,600 ft will usually be adequate.

Transition Distance

The transition or "lane change" distance to enter the storage lane depends on the approach speed and the number of lanes to be crossed. A 150- to 250-ft distance appears reasonable.

Left-Turn Storage

Left-turn storage lanes should be adequate to handle the anticipated turning volumes with a low likelihood of over-

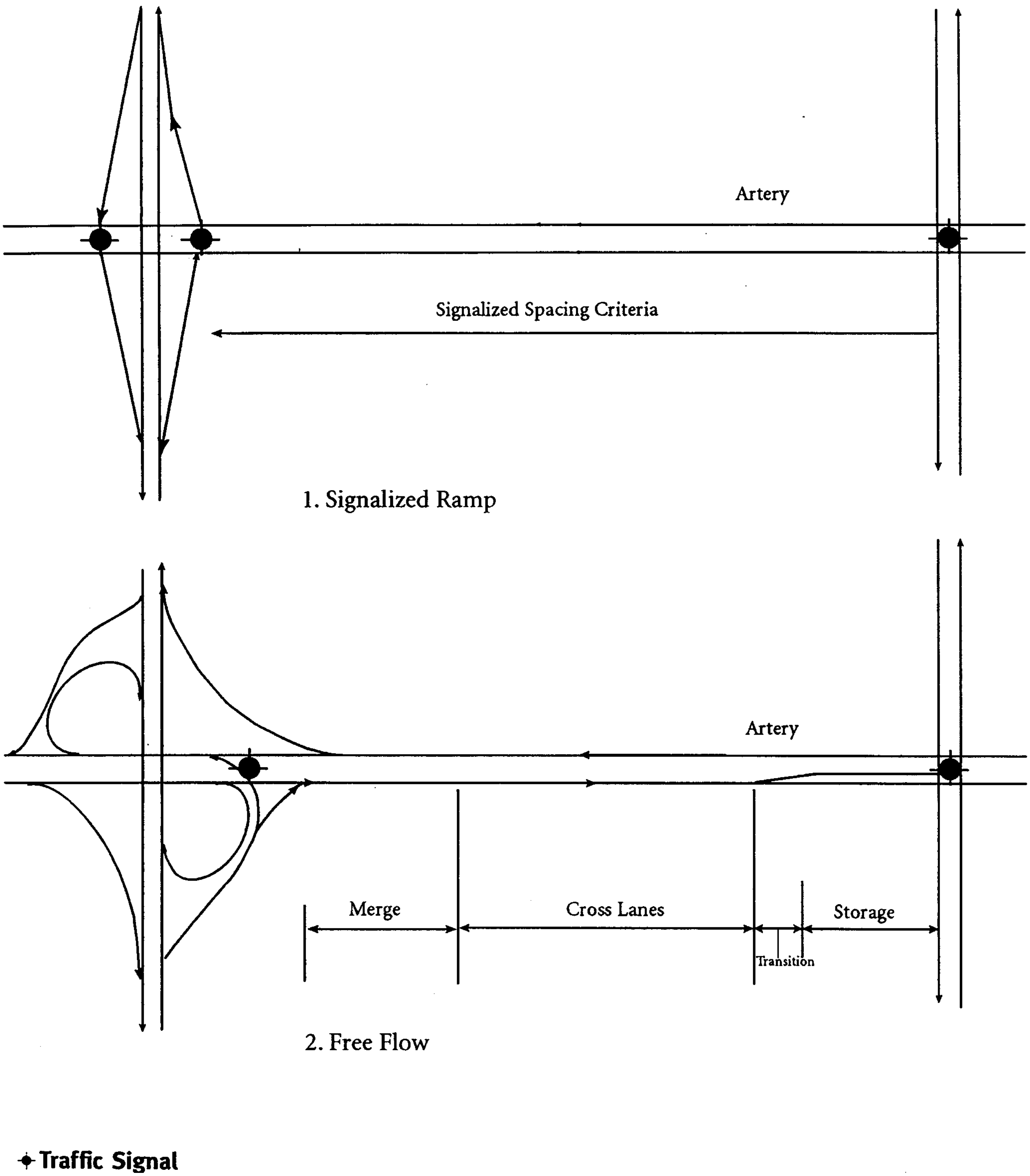
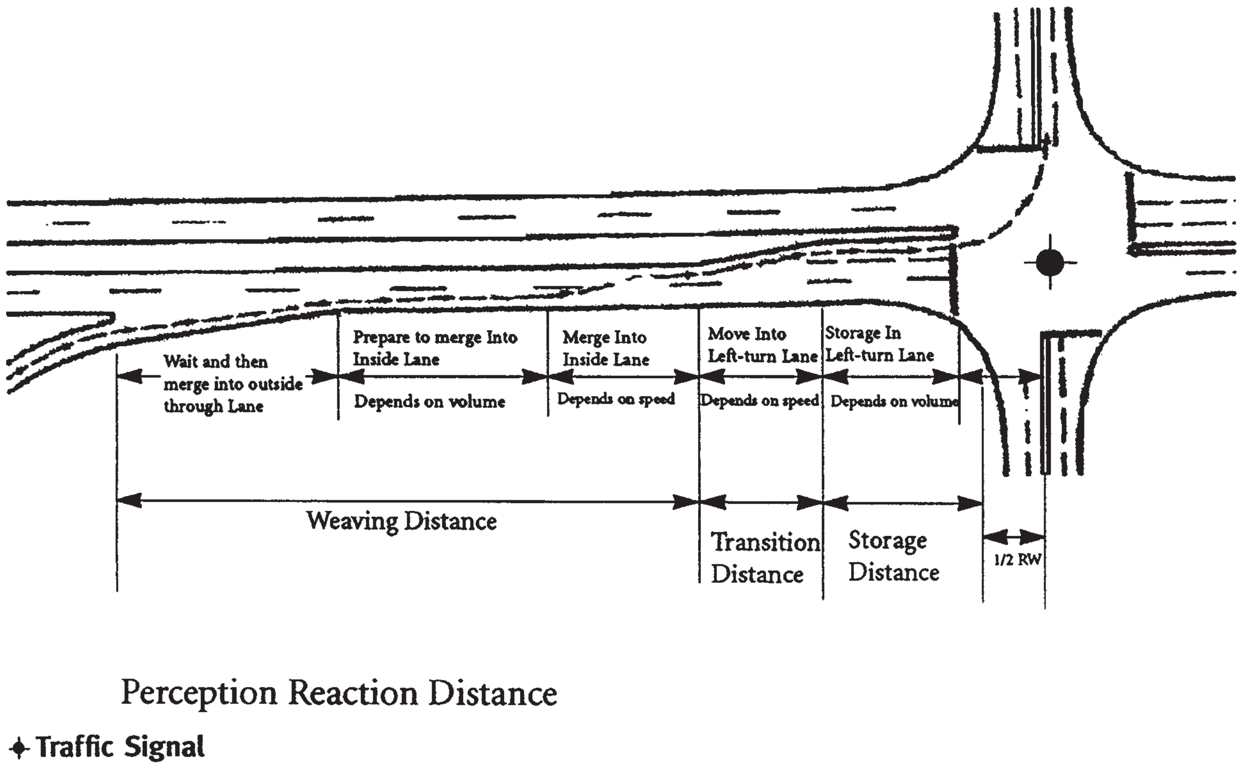


Figure 49. Types of ramp access to/from arterial roads.



Source: *(145)*

Figure 50. Factors influencing access separation distance.

flow or failure. Storage length can be estimated from the following equation:

$$L = \frac{RV\,(25)}{N_c} = R125 \qquad (12)$$

Where V = Left turns per hour (flow rate)
N_c = Cycles per hour
l = Left turns per cycle
R = Randomness factor for less than 5 percent failure
R = 2.0 for random operations (i.e., rural)
R = 1.5 for operations where traffic tends to platoon
L = Length of left-turn storage in feet

Where there are dual left-turn lanes, the resulting value can be reduced by roughly 45 percent. Thus, the length of a single left-turn lane in feet may be estimated to be as much as 50 times the number of left turns per cycle. For dual left-turn lanes, the length of each lane in feet may be estimated to be as much as 28 times the number of left turns per cycle. The actual storage also will depend on the degree of randomness of arriving left-turning vehicles.

Street Width Distance

Where separation distances are measured from the centerline of the road crossing the arterial, an additional distance of half the right of way should be added. This distance will normally approximate 50 ft.

Perception-Reaction Distance

It may be desirable to add a perception-reaction distance. Calculated at 2.5 ft per second, it will probably add roughly 125 ft. These components should be added to obtain the required access separation distance. Table 86 gives suggested access separation distances for various left-turn volumes and weaving distances. The 800-ft weaving distance will generally apply to a single lane of travel in each direction while the 1,200-ft and 1,600-ft weaving distances will generally apply to 2 and 3 lanes of travel in each direction. These “default” values can be used as alternatives to the values cited in Table 85.

Planning Implications

Providing adequate separation distances will reduce related congestion and safety problems; however, turning movements—left turns in particular—and their effects on arterial traffic flow will not disappear. Two approaches are available for alleviating this problem:

- Frontage roads along freeways can be better integrated with ramps at interchanges so that one road rather than two roads intersect the arterial in each direction of

TABLE 85 Estimated weaving distances

Weaving Volume (vph)	Speed (mph) 25-30	35	40	45	50
200	50	100	150	200	400
400	100	200	300	450	800
600	150	300	450	700	1,200
800	200	400	600	950	1,800
1,000	300	500	750	1,200	2,400
1,200	350	600	900	1,450	*
1,400	400	710	1,050	1,700	*
1,600	450	820	1,200	2,050	*
1,800	500	930	1,400	2,400	*
2,000	600	1,040	1,600	*	*
2,200	700	1,150	1,800	*	*
2,400	800	1,270	2,050	*	*
2,600	900	1,400	2,300	*	*

Notes: Use 400 feet for values above solid line.
* Speeds are not attainable.

Source: Adapted from *(146)*.

TABLE 86 Estimated access separation distances (feet)

[3]Left-Turn Storage		[1] Weaving Distance 800 feet [a]	1,200 feet [b]	1,600 feet [c]
Lefts/Lane/ Cycle	Distance (Feet)	Weaving plus Left-Turn Storage Distance		
2	100	900	1,300	1,700
4	200	1,000	1,400	1,800
6	300	1,100	1,500	1,900
8	400	1,200	1,600	2,000
10	500	1,300	1,700	2,100

Additional Distances

(1)	Weaving Distance	See (1) above
(2)	Transition Distance	150-250 ft.
(3)	Left-turn Storage	See (3) above
(4)	Cross Street Distance (1/2 Right-of-Way)	40-50 ft.
(5)	PIEV Distance (Optional)	100-150 ft.

Example: 6 left-turns/cycle. 1,200-foot weaving distance.
Separation distance equals:
1,500 + 250 + 50 + 150 = 1,950 ft.

Notes:
(a) Typically applies to two-lane highways.
(b) Typically applies to four-lane highways.
(c) Typically applies to six-lane highways.

travel. In addition, a continuous system of frontage roads can provide additional property access and reduce reliance on arterial road access.

- Interchange configurations can be developed and modified to provide direct access to major streets or developments, thereby avoiding “double loading” arterials and reducing weaving and turning volumes.

These actions can best be taken in the initial interchange planning and location process as part of a joint land use and transportation planning effort. The product of such an “interchange access management plan” would be more rational arrangements of streets and development, better access separation distances and preservation of mobility and safety over the long term.

CHAPTER **10**

FRONTAGE ROADS (TECHNIQUES 6A AND 6B)

INTRODUCTION

The frontage road, as an access control technique, reduces the frequency and severity of conflicts along the main travel lanes of a highway. Direct property access is provided from the frontage roads and prohibited from the main travel lanes. The resulting spacing between the intersections along the main roadway facilitates the design of auxiliary lanes for deceleration and acceleration. Thus, frontage roads segregate through and local land-service traffic, thereby protecting the through travel lanes from encroachment, conflicts, and delays.

Frontage roads, however, may require more circuitous access to adjacent land developments. They may also complicate the operations at signalized intersections thereby reducing some of the overall benefits achieved. How well they function depends on how well these considerations are reflected in the design and operations. Unless they are carefully designed and selectively applied, both in new and retrofit situations, frontage roads may not achieve the desired results.

Frontage roads generally are, but need not be, parallel to the roadway for through traffic. They may be provided on one or both sides of the main highway. They may be continuous, or they may extend for short sections only. They may operate one-way. Figure 51 illustrates the different types of frontage roads normally found.

The design of a frontage road is affected by the type of service it is intended to provide. Where a frontage road is continuous and passes through highly developed areas, its primary function is that of general service, and it assumes the character of an important street. At the other extreme, where a frontage road is only a few blocks long, follows an irregular pattern, borders the rear and side of buildings, or serves only scattered development, traffic will be light and operation will be local in character.

From an operational and safety standpoint, one-way frontage roads are preferable to two-way roads. The safety advantage in reducing vehicular and pedestrian conflicts on intersecting streets often compensates for any inconvenience to local traffic. Where frontage roads parallel a freeway and accommodate traffic from slip ramps, the efficiency and safety associated with one-way frontage roads greatly surpasses those of two-way frontage roads.

Two-way frontage roads may be appropriate in partially developed areas where the adjoining street system is so irregular or so disconnected that one-way operation would introduce considerable added travel distance and cause undue inconvenience. Two-way frontage roads also may be necessary for suburban or rural areas where points of access to the through facility are infrequent, where only one frontage road is provided, where roads or streets connecting with the frontage roads are widely spaced, or where there is no parallel street within reasonable distance of the frontage roads.

FREEWAY FRONTAGE ROADS

Frontage roads along freeways and expressways are used in many urban, suburban, and even rural settings to maintain the integrity of the local street system and provide access to adjacent development. The frontage roads can be integrated with the interchange and ramping system to alleviate congestion on interchanging arterials near major streets and activity centers. If desired, frontage roads can increase the connectivity and access opportunities for developments that front along freeways. Figures 52-A, B, and C illustrate freeway frontage road/interchange concepts.

Freeway frontage roads generally operate one-way in developed areas and are integrated with ramping patterns; diamond interchanges are common—sometimes with U-turn loops provided just short of the interchanges to permit reversal of direction before the traffic signals.

ARTERIAL FRONTAGE ROADS

Fully developed frontage roads effectively control access to the through lanes on an arterial street, provide access to adjoining property, separate local from through traffic, and permit circulation of traffic on each side of the arterial. They may be used in conjunction with grade separation structures at major cross streets, in which case the arterial takes on many of the operating characteristics of a freeway. Frontage roads and grade separations afford the ultimate in access control in densely developed areas. Figure 53 (*147*) shows how a frontage road becomes an integral part of upgrading an arterial roadway while simultaneously serving adjacent development.

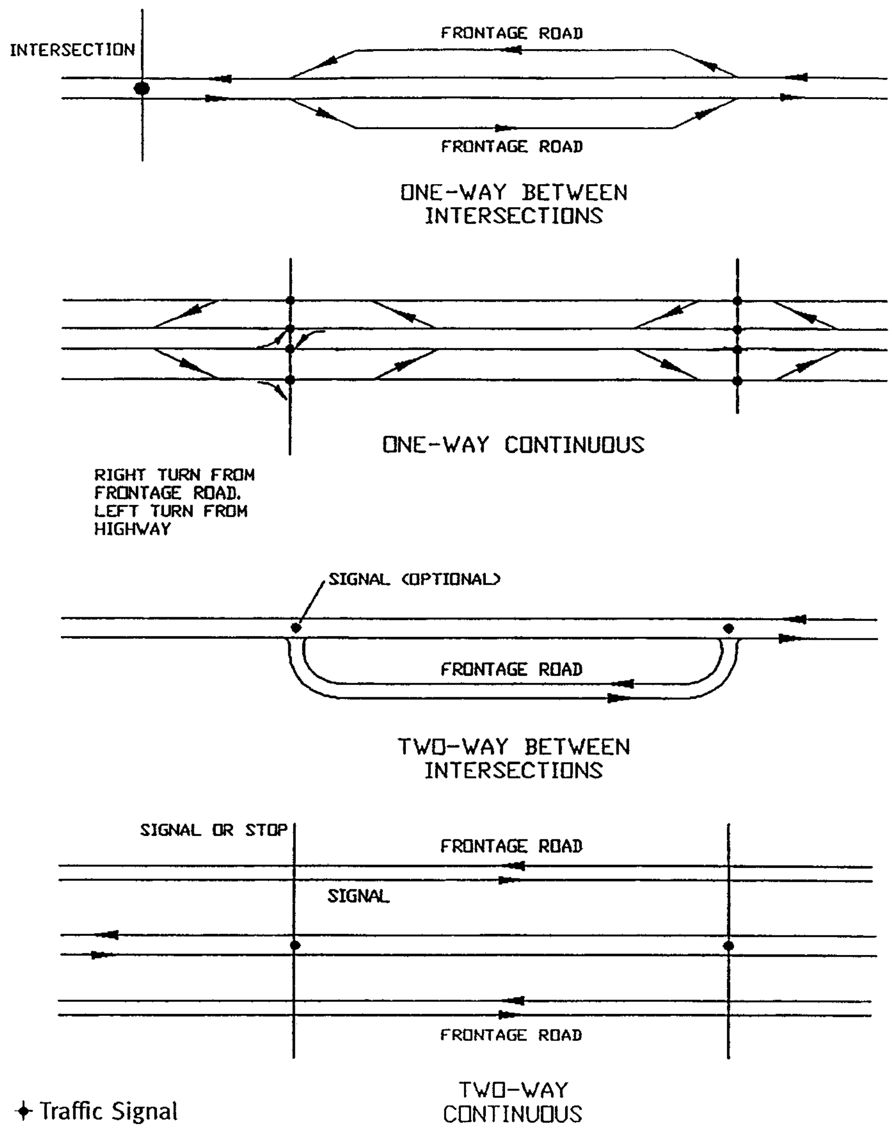


Figure 51. Types of frontage roads.

Frontage roads along arterials must be carefully designed to avoid increasing conflicts at junctions and delays on intersecting roads. The following planning and design guidelines should be considered in installing arterial frontage roads in both new developments and retrofit situations (*148*):

1. Frontage roads, especially for "retrofit" situations, should operate one-way and should enter or leave the mainline lanes as merging or diverging movements. There should be no signalized junctions along the arterial or the frontage road in this area (Figure 54).
2. The separation of frontage roads at cross streets should be maximized to ensure sufficient storage for crossroad traffic between the frontage roads and the arterial. The absolute minimum separation should be 150 ft, where two-way frontage roads are provided. This dimension is about the shortest acceptable length for placing signs and other traffic control devices. Greater distances are needed to provide adequate left-turn storage and to separate operation of the two intersections. Spacing of at least 300 ft (preferably more) enables turning movements to be made from the main lanes onto the frontage roads without seriously disrupting arterial traffic and thereby minimizes the potential of wrong-way entry onto the through lanes of the predominant highway.
3. "Reverse" frontage roads, with developments along each side, are desirable in developing urban areas. A desirable separation distance is 600 ft with a minimum distance of 300 ft. The frontage road may operate either one-way or two-way (Figure 55).
4. Frontage roads that can be terminated at each block operate well with respect to the arterial roadway and the cross street. This type of design should be considered where continuity of the frontage road is not needed.

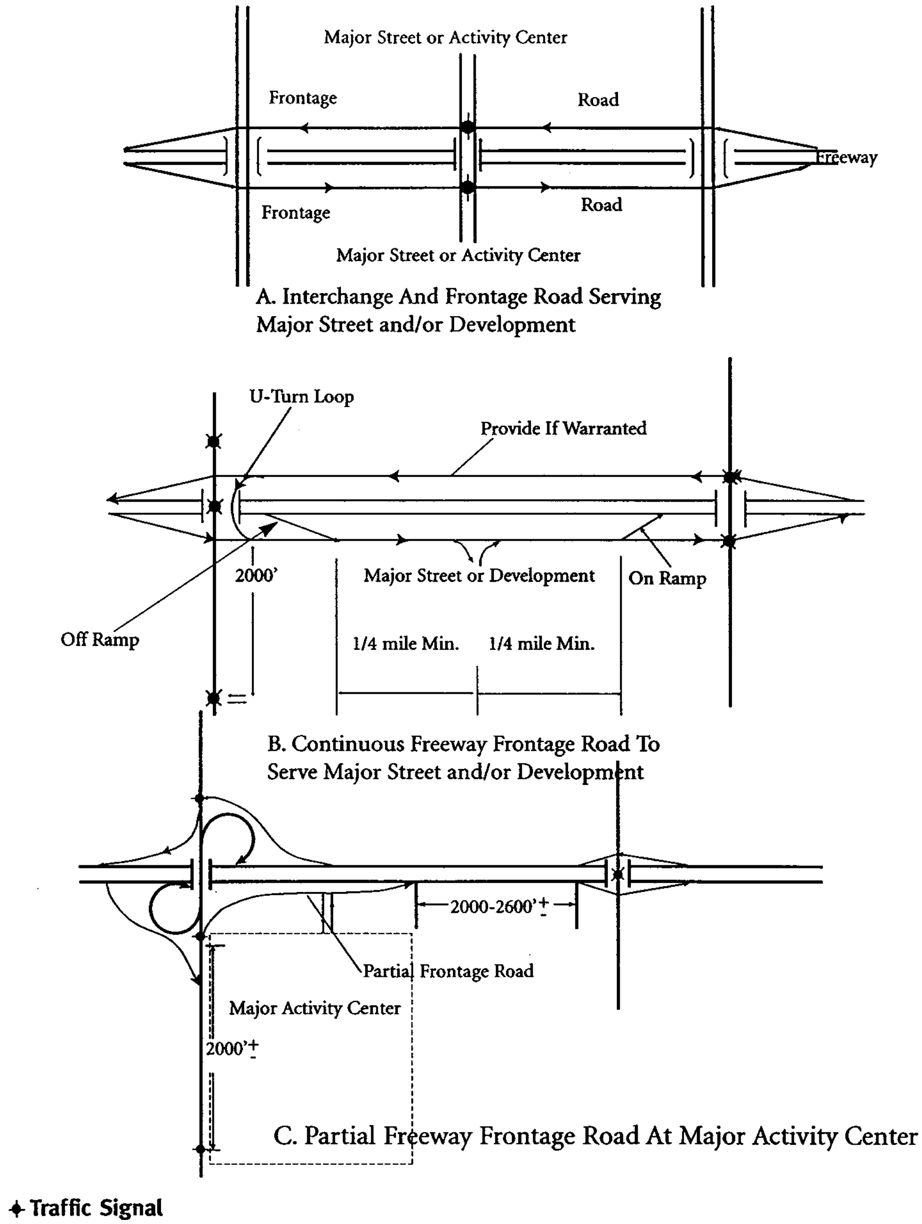
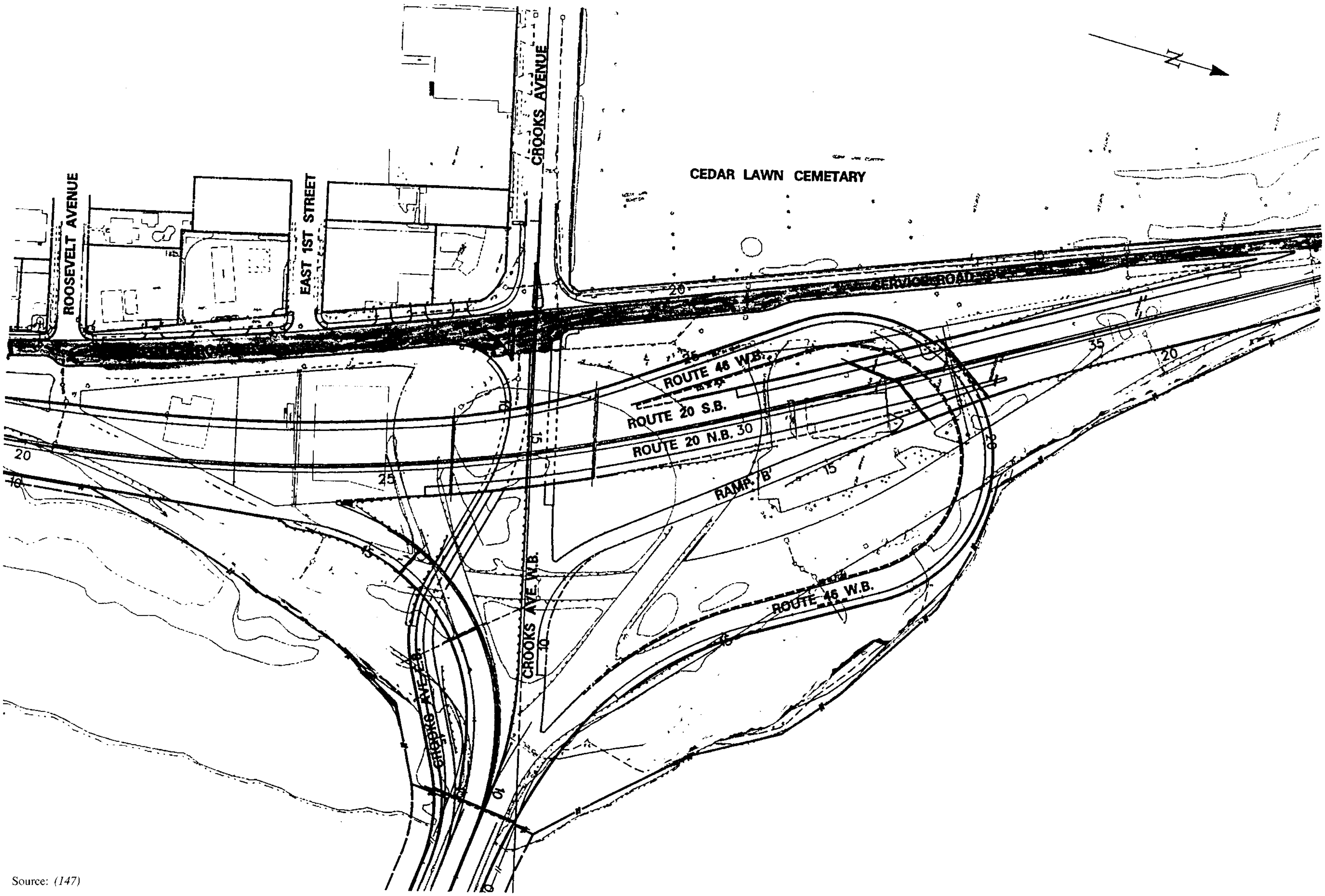


Figure 52. Freeway frontage road/interchange concept.

5. Where major activity centers front along an arterial roadway, frontage roads should be incorporated into the ring road or otherwise eliminated.
6. A minimum outer separation of 20 ft should be used to provide space for pedestrian refuge and safe placement of traffic control devices and landscaping.
7. Pedestrian and bicycle movements should use the frontage roads. Parking may be permitted where the frontage roads traverse residential areas.

The reverse frontage road concepts can be adapted to and incorporated into community and subdivision designs for newly developing areas. Access along arterial streets would be limited to specifically designated locations that fit the signal progression pattern. A series of collector roads would intersect the arterials at selected locations and link the arterial roads with the surrounding residential and commercial areas. A series of "loop" access roads would link each community circulation system with the collector street. These "reverse frontage roads" would serve developments on each side.

This concept has several desirable attributes from both a land development and access management perspective: (1) it reduces "strip" developments along arterials and the attendant marginal interference; (2) it allows traffic signals only at locations that permit optimum progression because the need for other signals is eliminated; (3) it provides a logical graduation of traffic movements from arterials or collectors



Source: *(147)*

Figure 53. Crooks Avenue interchange.

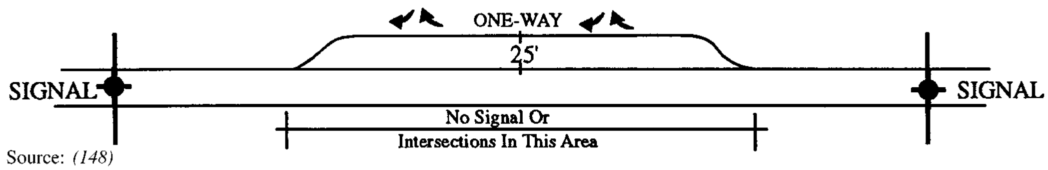


Figure 54. Arterial frontage road concept for retrofit conditions.

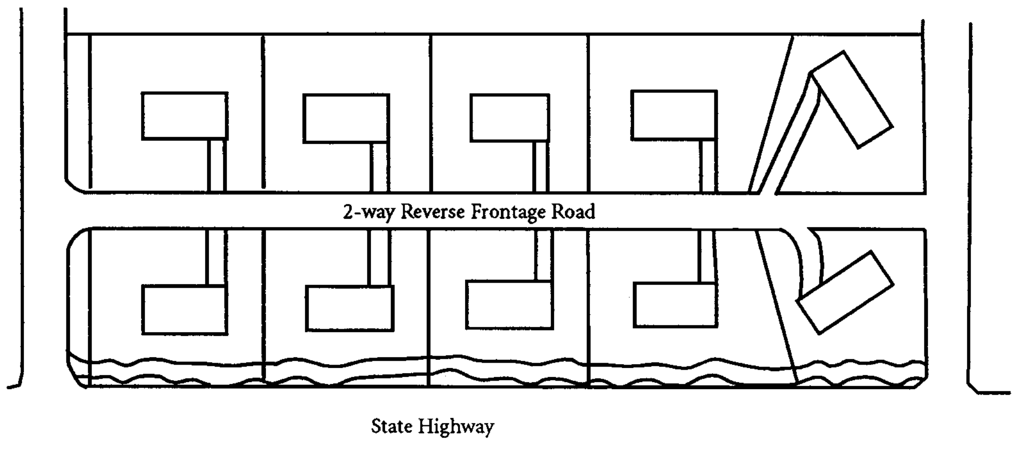


Figure 55. Illustrative reverse frontage road concept.

to local streets; (4) it permits a cohesive internal design of residential and commercial areas; and (5) it permits future upgrading of arterials to expressway standards (*149*).

One-Way Frontage Roads Operations

One-way arterial frontage roads with narrow separation distances from the main travel lanes are found in several urban areas. Frontage roads such as along the Grand Concourse and Queens Boulevard in New York City have mid-block slip ramps to and from the main travel lanes. Left turns from the outer roads and right turns from the main roadways are restricted.

An illustrative service road concept for high-volume suburban roads that draws on these experiences is shown in Figure 56. The key features include

- Minimum ½-mi spacings between crossroads,
- Approximately 1,400 ft of relatively unrestricted property access,
- Basic 200-ft arterial road right of way,
- Acceleration and deceleration lanes for slip ramps,
- Left-turn bays from service roads and right-turn bays from the arterial at intersections,
- Dual left-turn lanes (as needed) from arterial, and
- Protected signal phases for left turns.

When two major arterials with frontage roads intersect, a grade separation may be necessary. Alternatively, it may be necessary to break the continuity of one or both sets of frontage roads to simplify traffic signal operations.

Two-Way Frontage Roads

Two-way frontage roads with wide setback distances provide opportunities for property access from both sides of the frontage roads and enable left turns to be redirected and removed from the main intersection to allow two-phase signal operations. As shown in Figure 57, this is accomplished by prohibiting movements on the frontage road across the intersecting street. The same principle can be applied where grades are separated between the two roadways.

APPLICATION GUIDELINES

Frontage roads along arterials that interchange with freeways may be desirable to reduce left turns and weaving, avoid double loading of arterial roads, and improve property access. Frontage roads should be closely coordinated with the supporting street system, especially in areas where development is allowed or planned.

The applications of frontage roads will depend on traffic and land-use needs as well as property availability. Frontage

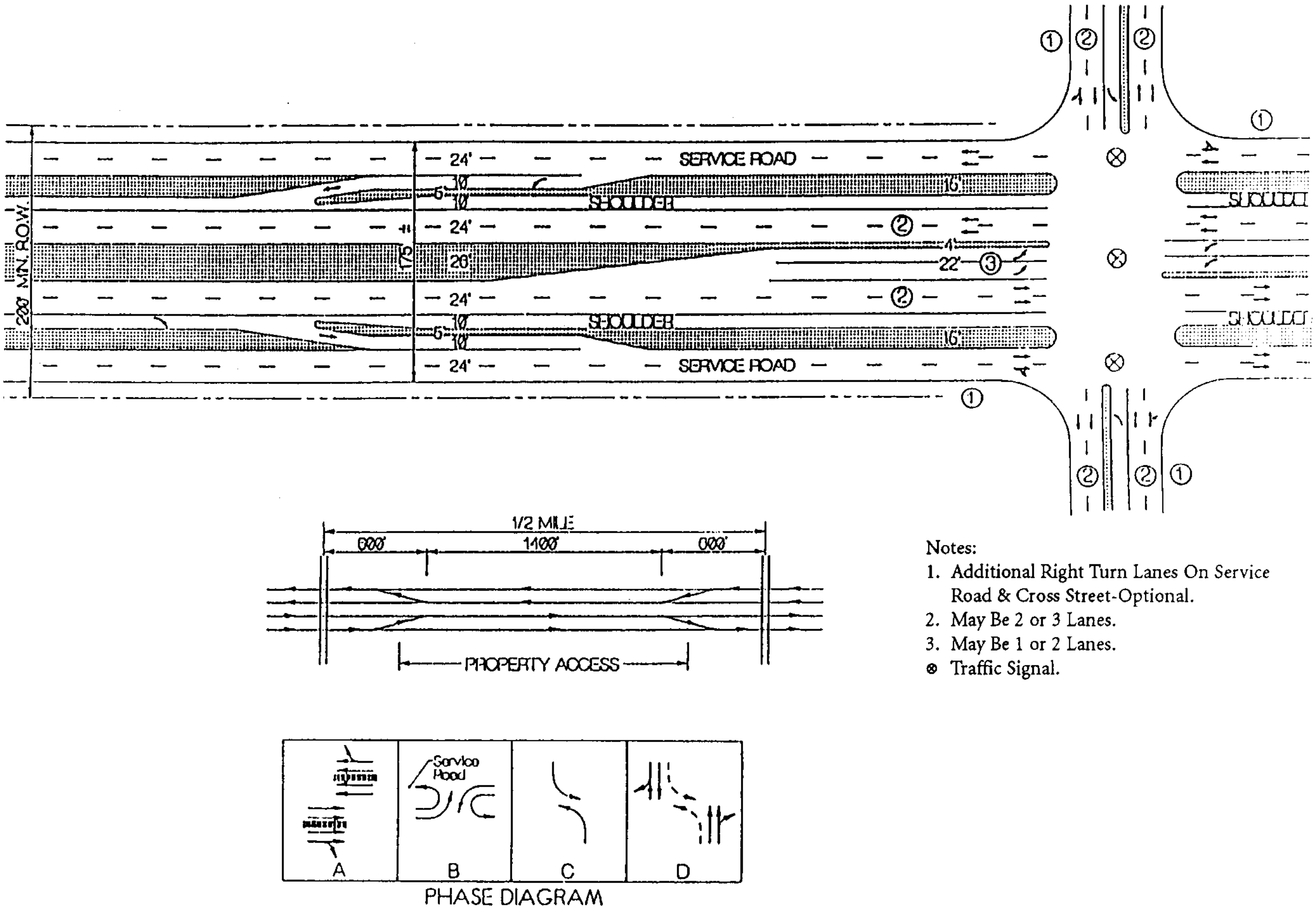


Figure 56. Service road concept for suburban strategic arterial.

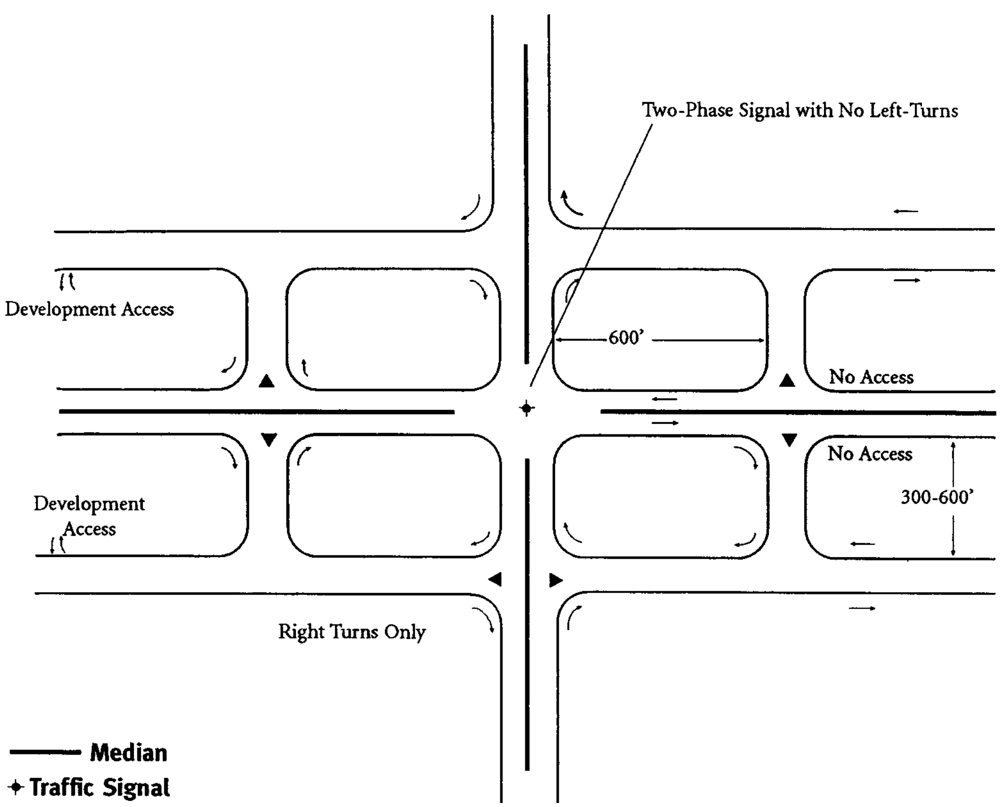
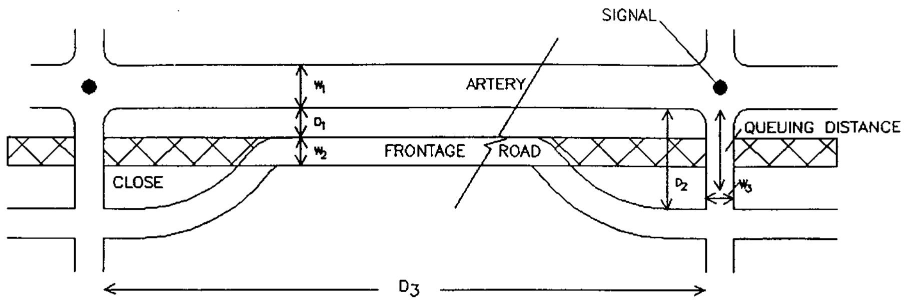


Figure 57. Intersection concept for two-way reverse frontage roads (backage).



D1 - Minimum midblock separation
D2 - Minimum separation at intersection
W1 - Arterial road width
D3 - Signal progression artery
W2- Service road width
W3- Cross road width

Figure 58. Key variables in relocating frontage roads at intersections.

roads may allow closer access spacings than would be practical from main travel lanes to enhance local access and to better integrate with overall community designs.

A minimum midblock width (D-1 in Figure 58) of 30 ft is necessary to enable shoulders to be provided on both the main highway and the frontage areas to allow for drainage between the roadway. This minimum distance can be maintained at intersections of one-way frontage roads where right turns from the main road and left turns from the frontage road are prohibited.

The preferred alternative to restricting turns is to locate the frontage roads a considerable distance from intersecting crossroads to lengthen the spacing between successive intersections along the crossroads. This permits the intersections between the cross street and the frontage road to be well removed from the cross street intersection with the main lanes. Separate signal phases can be used to relieve some of the conflicts between the various movements; however, this can be done only at the expense of increased delay to most of the traffic.

Accordingly, outer separations at intersections of 150 ft or more between the arterial and frontage roads is desirable in urban areas wherever practical and feasible. In rural areas, a minimum separation of 300 ft (D-2 in Figure 57) is desirable.

The 150-ft dimension reflects the following considerations:

1. It is about the minimum acceptable length needed for placing signs and other traffic control devices to give proper direction to traffic on the cross street.
2. It usually affords acceptable storage space on the cross street in advance of the main intersection to avoid blocking the frontage road. Under high traffic volume conditions, a queuing analysis should be made to ensure that the frontage road intersection is located beyond peak-hour traffic queues on the crossroad.
3. It enables turning movements to be made from the main lanes onto the frontage roads without seriously disrupting the orderly movement of traffic.
4. It facilitates U-turns between the main lanes and the two-way frontage road. (Such a maneuver is geometrically possible with a somewhat narrower separation, but it is extremely difficult with commercial vehicles.)
5. It alleviates the problem of wrong-way entry onto the through lanes or the arterial.
6. It separates points of conflict between the frontage traffic and the main highway traffic.

Narrower separations may be acceptable where frontage road traffic is very light, where frontage roads operate one-way only, or where some movements can be prohibited.

Frontage roads reduce marginal frictions, allow public agencies complete control of access to the arterial, and accommodate parking and loading. Frontage road design must address potential effects at major crossroad intersections, especially when the distances between the frontage road and arterial are short, the intersections are signalized, and the storage distances on the crossroad are inadequate. When commercial development occurs along frontage roads, the resulting traffic volumes may create congestion and increase accidents as a result of low-capacity overlapping maneuver areas, close conflict points, and complex movements needed to enter and leave the main travel lanes. For these reasons, arterial frontage roads must be very carefully designed to protect both arterial and crossroad operations.

CHAPTER **11**

CONCLUSIONS AND RECOMMENDATIONS

This research was performed to develop methods to predict and analyze the safety and traffic operations effects of selected access management techniques for different roadway variables and traffic volumes. Accordingly, more than 100 individual techniques were identified and grouped according to policy and roadway design features. The more significant techniques—generally those that relate to access spacing or median/left-turn treatments were analyzed in terms of their safety, travel time/delay, and capacity. In some cases, economic effects were also quantified. Where effects could not be quantified, case studies of current practices were assembled and analyzed.

ACCESS EFFECTS

The key findings and conclusions relating to effects were as follows:

1. The spacing of traffic signals affects both safety and operations. Data on the safety effects of traffic signals suggest that accident rates rise as signal density increases, but the information is limited, and the effects of intersecting volumes are not clearly identified. Long and uniform signal spacing permits effective progression at desired travel speeds. As signal frequency increases, progression efficiency is reduced, with a corresponding increase in delays. In general, there is a 2- to 3-mph drop in speeds for each traffic signal per mile added.
2. Accident rates rise as the density of unsignalized access connections per mile increases. The patterns are generally consistent among states.
3. The number of impacted through vehicles traveling in the curb lane increases as high-volume driveways are spaced closer together. The likelihood of spillbacks across a driveway rises with either an increase in the traffic volumes entering driveways and/or the driveway density.
4. Corner clearances (i.e., driveway setback distances on the near and far side of intersections) vary widely among states and communities. In most cases, access drives are located within the functional areas of intersections and/or within the normal queuing distances. Safety and operations improve when corner clearance is increased.
5. Several decades of research have documented the safety and operational benefits associated with installing TWLTLs or nontraversable medians on undivided highways. Raised medians result in lower accident rates than TWLTLs. They make it possible to reduce the frequency of conflicting movements which, in turn, also improves safety.
6. The provision of left-turn storage lanes improves both safety and capacity by removing turning traffic from the through lanes. The safety benefits have been well documented, and several studies have clearly quantified the gains in capacity.
7. Indirect left turns or U-turns are increasingly used as an alternative to direct left turns. They make it possible to prohibit left turns from driveway connections onto multi-lane highways and to eliminate traffic signals that do not fit into time-space patterns. When incorporated into intersection designs, they allow left turns to be rerouted and signal phasing to be simplified. Safety, capacity, and travel time benefits have been reported.
8. Access spacing or setback distances on arterial roadways near freeway interchanges are generally inadequate for the weaving and left-turn storage movements that must be accommodated. Often, problems are compounded by locating frontage roads too close to ramp terminals.
9. Frontage roads along freeways—when properly integrated with interchanges—can reduce arterial left turns and weaving movements as well as improve access to development. Frontage roads along arterials reduce marginal frictions, but they can increase conflicts at junctions and delays on intersecting roads unless carefully designed. Therefore, arterial frontage roads should be used selectively.

POLICY IMPLICATIONS

Access management techniques—access spacing in particular—can be addressed through both retrofit (corrective) and policy actions. Access separation distances should be established as part of access management programs, site retrofit actions, and community zoning ordinances. Advance purchase of right of way and/or access rights is also desirable.

The basic policy issues are to (1) classify roads as determined by the transportation plan, (2) establish access and geometric standards for each class of roadway, (3) limit access along major arterials, and (4) consider restricting left turns where access is provided along arterials.

Comprehensive access management codes should indicate where access is allowed or denied for various classes of roads, specify allowable spacings for signalized and unsignalized connections, and set forth permit procedures and requirements. Codes may define or limit the application of specific techniques.

There should be a sufficient network of supporting local and collector streets that provide direct access to adjacent developments. These secondary streets should connect to arterials at appropriate and well-spaced locations. They make it possible to minimize direct property access on major arterials.

Access should be provided from strategic and primary arterials only when reasonable access cannot be provided from other roadways. In such cases, access should be limited to right turns wherever possible.

Left-turn and cross egress should be separated and placed at locations that fit into overall signal coordination patterns with high efficiency.

Sound land use and development planning is essential to permit effective arterial traffic flow while allowing attractive property access. Access spacing standards (including corner clearance requirements) should be established in advance of actual development. Zoning, subdivision, and access spacing requirements should be consistent.

Where land remains to be subdivided or platted, larger frontages should be encouraged. In New Jersey for example, the amended Municipal Land Use Law restricts municipalities from approving the subdivision of lots along state highways where the proposed lots would not conform with State Access Code spacing requirements. Colorado law prohibits the approval of subdivision requests that require access that is inconsistent with the state's access regulations.

The size of corner lots in developing areas should be adequate to meet safety and queuing requirements. Adequate corner clearances require lot frontage of at least 150 to 250 ft—these distances are also consistent with desired minimum unsignalized access spacings.

Better coordination of land use, interchange geometry, and arterial streets is essential to avoid "double loading arterials" with left turns, weaving movements, and traffic congestion. Strategically placed frontage roads can play an integral role in this effort. It is equally important to anticipate future developments in the quadrants of an interchange and to formulate appropriate access concepts that preserve the arterial street system while serving these developments. A suitable supporting street system is essential.

Median width and opening policies are essential design elements. Raised medians are more effective than painted channelization from an access management perspective. Wide medians that allow indirect U-turns in lieu of direct left turns should be considered for new arterials where space permits because they improve safety and simplify intersection operations and signal timing/coordination.

Any access control or management plan must be done systemwide to avoid shifting problems. Many access management techniques deal with a single location (e.g., closing a median at a driveway). Some techniques (e.g., a continuous median) may transfer problems to other locations upstream or downstream from the location under consideration. In such cases, broader analyses of benefits and effects are essential.

RESEARCH DIRECTIONS

Several needs emerged from the research effort: (1) expanding and refining the safety database relating to access density; (2) quantifying the effects of median closures—both signalized and unsignalized—and their upstream and downstream effects; and (3) assembling more information on driver selection of roadside businesses based on accessibility considerations, such as the proportion of pass-by traffic, repeat traffic, and destination trips by direction of approach.

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APPENDIX A

RECOMMENDED CLASSIFICATION OF ACCESS MANAGEMENT TECHNIQUES BY POLICY AND ROADWAY FEATURE

A. POLICY (MANAGEMENT ELEMENTS)

A-1 Administrative and Regulating Procedures (Access Code) Spacing Requirements

A1-1 State policies/plans/programs for access.
A1-2 Local policies/plans/programs for access.
A1-3 Regulate minimum spacing of traffic signals (1a).
A1-4 Regulate minimum spacing of driveways (1b).
A1-5 Regulate minimum corner clearance (1c).
A1-6 Regulate minimum property setback from roads.
A1-7 Optimize driveway spacing in the permit authorization stage.
A1-8 Regulate maximum number of driveways per property frontage.
A1-9 Consolidate access for adjacent properties.
A1-10 Require highway damages for extra driveways.
A1-11 Deny access to small frontage.
A1-12 Require access on collector street (when available) in lieu of additional driveway on highway.
A1-13 Establish access separation distances at freeway interchanges (1d).
A1-14 Regulate driveway construction and maintenance.

A-2 Zoning and Subdivision Regulations

A2-1 Land use/zoning regulation.
A2-2 Designate the number of driveways regardless of future subdivision of a property.
A2-3 Require adequate internal design and circulation plan.
A2-4 Redesign internal road system.
A2-5 Key allowable trip generation to access spacing.

A-3 Purchase of Access Rights

A3-1 Buy abutting properties.
A3-2 Acquire land.
A3-3 Acquire easements to provide alternate access routes.

B. DESIGN

B-1 Interchanges

B1-1 Build interchange (at major intersection or activity center).
B1-2 Modify freeway ramps to improve access.
B1-3 Build freeway frontage road.

( ) indicates priority techniques.

B-2 Frontage Roads

B2-1 Construct a local service or frontage road to provide access to individual parcels.
B2-2 Construct a bypass road.
B2-3 Build a reverse frontage road.
B2-4 Locate or relocate an intersection of a parallel frontage road and crossroad further from arterial/crossroad intersection (6b).

B-3 Medians - Left Turns

B3-1 Install median barrier with no direct left-turn ingress or egress.
B3-2 Install restrictive median with left-turn deceleration lanes (2a).
B3-3 Install restrictive median on undivided highway (2a).
B3-4 Replace continuous TWLTL with restricted median (2b).
B3-5 Close existing median openings (2c).
B3-6 Replace full median opening with median designed for left turns from the major roadway (2d).
B3-7 Install channelizing islands to prevent left-turn deceleration lane vehicles from returning to the through lanes.
B3-8 Install median channelization to control the merge of left-turn egress vehicles.
B3-9 Provide left-turn deceleration/storage lane where none exists (3a).
B3-10 Install left-turn acceleration lane (3b).
B3-11 Install continuous TWLTLs (3c).
B3-12 Install alternating left-turn lane.
B3-13 Install isolated median and deceleration lane to shadow and store left-turning vehicles.
B3-14 Install left-turn deceleration lane in lieu of right-angle crossover.
B3-15 Install median storage for left-turn egress vehicles.
B3-16 Increase storage capacity of existing left-turn deceleration lane.
B3-17 Channelize left-turn lanes across wide medians.
B3-18 Provide U-turns as alternative to direct left turns (3d).
B3-19 Provide jughandle and eliminate left turns along a highway (3e).
B3-20 Construct flyover to accommodate left-turn egress/and ingress movements.
B3-21 Prohibit left turns.
B3-22 Build left-turn connecting roads.

B-4 Right Turns

B4-1 Install right-turn acceleration lane (4a).
B4-2 Install continuous right-turn lane (4b).
B4-3 Install right-turn deceleration lane (4c).
B4-4 Install channelizing islands to prevent driveway vehicles from backing onto the highway (5c).
B4-5 Install channelizing islands to move ingress merge point laterally away from the highway.
B4-6 Move sidewalk-driveway crossing laterally away from highway.

B-5 Access/Driveway Location - Retrofit

1. Consolidation

B5-1-1 Consolidate driveway access for adjacent properties (5a).

( ) indicates priority techniques.

B5-1-2 Consolidate existing access whenever separate parcels are assembled under one purpose, plan, entity, or usage.
B5-1-3 Encourage connections between adjacent properties (even when each has highway access).

2. *Reorientation of Access*

B5-2-1 Encourage connections between adjacent properties (even when each has highway access).
B5-2-2 Require access on collector street (when available) in lieu of additional driveway on highway.
B5-2-3 Relocate or reorient access.

3. *Relocation*

B5-3-1 Coordinate driveways on both sides of street (align opposing driveways or establish minimum offset) (5d).
B5-3-2 Locate a new driveway opposite an intersection or driveway and install a traffic signal where warranted and properly spaced.
B5-3-3 Install two one-way driveways in lieu of one two-way driveway.
B5-3-4 Install two two-way driveways with limited turns in lieu of one standard two-way driveway.
B5-3-5 Install two one-way driveways in lieu of two two-way driveways.
B5-3-6 Install two two-way driveways with limited turns in lieu of two standard two-way driveways.

B-6 Traffic Controls

B-6-1 Install traffic signal at high-volume driveways.
B-6-2 Install traffic signals to slow highway speeds and meter traffic for larger gaps.
B-6-3 Restrict parking on the roadway next to driveways to increase driveway turning speeds.
B-6-4 Provide reversible operation of access drive.
B-6-5 Implement curbside loading controls.
B-6-6 Prohibit left-turn driveway maneuvers on an undivided highway.
B-6-7 Install one-way operations on the highway.
B-6-8 Replace curb parking with off-street parking.

B-7 Access/Driveway Design

B-7-1 Widen right through lane to limit right-turn encroachment onto the adjacent lane to the left.
B-7-2 Install channelizing island to prevent left-turn deceleration lane vehicles from returning to the through lane.
B-7-3 Channelize driveways to discourage or prevent left-turn maneuvers (5b).
B-7-4 Install barrier to prevent uncontrolled access along property frontages (5c).
B-7-5 Install median channelization to control the merge of left-turn egress vehicles.
B-7-6 Install driveway channelizing island to prevent left-turn driveway encroachment conflicts.
B-7-7 Install driveway channelizing island to prevent right-turn deceleration lane vehicles from returning to the through lanes.

( ) indicates priority techniques.

B-7-8 Install driveway channelizing island to control the merge area of right-turn egress vehicles.
B-7-9 Regulate the maximum width of driveways.
B-7-10 Install visual cues of the driveway.
B-7-11 Improve driveway sight distance.
B-7-12 Regulate minimum sight distance.
B-7-13 Optimize sight distance in the permit authorization stage.
B-7-14 Increase the effective approach width of the driveway (horizontal geometrics).
B-7-15 Improve the vertical geometrics of the driveway.
B-7-16 Increase the turning speed of right-angle median crossovers by increasing the effective approach width.
B-7-17 Install additional exit lane on driveway.
B-7-18 Require two-way driveway operation where internal circulation is not available.
B-7-19 Control driveway design elements.
B-7-20 Install barrier to prevent uncontrolled access along property frontage (5c).
B-7-21 Provide full driveway access with steady flow in one direction of travel on arterial road.
B-7-22 Design driveways so signals impact only one side of artery at any one location.
B-7-23 Widen driveways to improve storage.

( ) indicates priority techniques.

APPENDIX B

EFFECTS OF MULTIPLE DRIVEWAYS ON FACILITIES WITH POSTED SPEEDS OF 35 TO 55 MPH

Table B-1

PERCENTAGE OF RIGHT LANE THROUGH VEHICLES INFLUENCED AT OR BEYOND ANOTHER DRIVEWAY
POSTED SPEED = 35 MPH

Driveway Spacing (ft)	No. of Driveways per 1/4 Mi., n	Right-Turn-In Volume per Driveway, R (vph)							
		R < 30		30 < R < 60		60 < R < 90		R > 90	
		Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$
100	13.2	2.4%	27.3%	7.5%	64.2%	12.2%	82.1%	21.8%	96.1%
125	10.6	2.4%	22.5%	7.5%	56.0%	12.2%	74.7%	21.8%	92.5%
150	8.8	2.4%	19.1%	7.5%	49.5%	12.2%	68.2%	21.8%	88.5%
175	7.5	2.4%	16.4%	7.4%	44.0%	12.1%	62.1%	21.6%	84.0%
200	6.6	2.2%	13.9%	7.1%	38.3%	11.5%	55.4%	20.6%	78.1%
225	5.9	2.0%	11.2%	6.3%	31.8%	10.3%	47.2%	18.4%	69.7%
250	5.3	1.5%	7.7%	4.8%	22.7%	7.8%	34.7%	13.8%	54.5%
275	4.8	1.1%	5.3%	3.5%	15.9%	5.8%	24.8%	10.3%	40.7%
300	4.4	0.8%	3.6%	2.6%	11.1%	4.3%	17.6%	7.7%	29.6%
325	4.1	0.6%	2.6%	2.0%	8.0%	3.3%	12.8%	5.9%	22.0%
350	3.8	0.5%	1.8%	1.5%	5.6%	2.5%	9.0%	4.4%	15.6%
375	3.5	0.3%	1.2%	1.1%	3.7%	1.7%	6.0%	3.1%	10.5%
400	3.3	0.2%	0.7%	0.7%	2.3%	1.1%	3.7%	2.0%	6.6%
425	3.1	0.1%	0.5%	0.5%	1.4%	0.8%	2.4%	1.4%	4.2%
450	2.9	0.1%	0.3%	0.3%	0.9%	0.5%	1.5%	0.9%	2.6%
475	2.8	0.1%	0.2%	0.2%	0.5%	0.3%	0.8%	0.5%	1.4%
500	2.6	0.0%	0.1%	0.1%	0.2%	0.1%	0.4%	0.3%	0.7%
525	2.5	0.0%	0.0%	0.0%	0.1%	0.1%	0.2%	0.1%	0.3%

Table B-2

PERCENTAGE OF RIGHT LANE THROUGH VEHICLES INFLUENCED AT OR BEYOND ANOTHER DRIVEWAY
POSTED SPEED = 40 MPH

Driveway Spacing (ft)	No. of Driveways per 1/4 Mi., n	Right-Turn-In Volume per Driveway, R (vph)							
		R < 30		30 < R < 60		60 < R < 90		R > 90	
		Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$
100	13.2	2.4%	27.3%	7.5%	64.2%	12.2%	82.1%	21.8%	96.1%
125	10.6	2.4%	22.5%	7.5%	56.0%	12.2%	74.7%	21.8%	92.5%
150	8.8	2.4%	19.1%	7.5%	49.5%	12.2%	68.2%	21.8%	88.5%
175	7.5	2.4%	16.6%	7.5%	44.3%	12.2%	62.5%	21.8%	84.3%
200	6.6	2.3%	14.4%	7.3%	39.5%	12.0%	56.9%	21.4%	79.5%
225	5.9	2.2%	12.4%	7.0%	34.7%	11.4%	51.0%	20.4%	73.8%
250	5.3	2.0%	10.2%	6.4%	29.4%	10.4%	44.0%	18.6%	66.2%
275	4.8	1.6%	7.5%	5.1%	22.1%	8.3%	33.9%	14.8%	53.5%
300	4.4	1.2%	5.3%	3.9%	16.0%	6.4%	25.1%	11.4%	41.1%
325	4.1	1.0%	3.8%	3.0%	11.7%	4.9%	18.6%	8.8%	31.2%
350	3.8	0.8%	2.9%	2.4%	8.8%	3.9%	14.0%	7.0%	24.0%
375	3.5	0.6%	2.1%	1.9%	6.6%	3.2%	10.7%	5.6%	18.4%
400	3.3	0.5%	1.6%	1.5%	4.8%	2.4%	7.8%	4.4%	13.7%
425	3.1	0.4%	1.1%	1.1%	3.4%	1.8%	5.5%	3.2%	9.6%
450	2.9	0.2%	0.7%	0.8%	2.3%	1.3%	3.7%	2.3%	6.5%
475	2.8	0.2%	0.5%	0.5%	1.5%	0.9%	2.4%	1.6%	4.3%
500	2.6	0.1%	0.3%	0.4%	1.0%	0.6%	1.7%	1.1%	3.0%
525	2.5	0.1%	0.2%	0.3%	0.7%	0.4%	1.1%	0.8%	1.9%
550	2.4	0.1%	0.1%	0.2%	0.4%	0.3%	0.6%	0.5%	1.1%
575	2.3	0.0%	0.1%	0.1%	0.2%	0.1%	0.3%	0.3%	0.6%

Table B-3

PERCENTAGE OF RIGHT LANE THROUGH VEHICLES INFLUENCED AT OR BEYOND ANOTHER DRIVEWAY
POSTED SPEED = 45 MPH

Driveway Spacing (ft)	No. of Driveways per 1/4 Mi., n	Right-Turn-In Volume per Driveway, R (vph)							
		R < 30		30 < R < 60		60 < R < 90		R > 90	
		Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$
100	13.2	2.4%	27.3%	7.5%	64.2%	12.2%	82.1%	21.8%	96.1%
125	10.6	2.4%	22.5%	7.5%	56.0%	12.2%	74.7%	21.8%	92.5%
150	8.8	2.4%	19.1%	7.5%	49.6%	12.2%	68.2%	21.8%	88.5%
175	7.5	2.4%	16.6%	7.5%	44.4%	12.2%	62.6%	21.8%	84.4%
200	6.6	2.4%	14.6%	7.5%	40.0%	12.2%	57.5%	21.7%	80.1%
225	5.9	2.3%	12.9%	7.3%	36.0%	11.9%	52.6%	21.3%	75.5%
250	5.3	2.2%	11.3%	7.0%	32.0%	11.5%	47.5%	20.5%	70.2%
275	4.8	2.1%	9.7%	6.6%	27.9%	10.8%	42.1%	19.2%	64.1%
300	4.4	1.8%	7.8%	5.8%	23.0%	9.4%	35.3%	16.8%	55.5%
325	4.1	1.5%	5.8%	4.6%	17.5%	7.5%	27.3%	13.5%	44.4%
350	3.8	1.2%	4.4%	3.8%	13.5%	6.1%	21.2%	11.0%	35.4%
375	3.5	1.0%	3.4%	3.1%	10.3%	5.0%	16.5%	8.9%	28.0%
400	3.3	0.8%	2.6%	2.5%	8.0%	4.1%	12.9%	7.3%	22.1%
425	3.1	0.7%	2.1%	2.1%	6.5%	3.5%	10.4%	6.2%	18.0%
450	2.9	0.6%	1.6%	1.8%	5.1%	2.9%	8.2%	5.2%	14.4%
475	2.8	0.5%	1.3%	1.4%	3.9%	2.3%	6.3%	4.2%	11.1%
500	2.6	0.4%	0.9%	1.1%	2.9%	1.8%	4.7%	3.2%	8.3%
525	2.5	0.3%	0.7%	0.8%	2.1%	1.4%	3.4%	2.5%	6.1%
550	2.4	0.2%	0.5%	0.6%	1.5%	1.0%	2.5%	1.8%	4.4%
575	2.3	0.2%	0.4%	0.5%	1.1%	0.8%	1.8%	1.4%	3.2%
600	2.2	0.1%	0.3%	0.4%	0.8%	0.6%	1.3%	1.1%	2.3%
625	2.1	0.1%	0.2%	0.3%	0.6%	0.4%	0.9%	0.8%	1.6%
650	2.0	0.1%	0.1%	0.2%	0.4%	0.3%	0.6%	0.5%	1.1%
675	2.0	0.0%	0.1%	0.1%	0.2%	0.2%	0.4%	0.3%	0.7%
700	1.9	0.0%	0.0%	0.1%	0.1%	0.1%	0.2%	0.2%	0.4%
725	1.8	0.0%	0.0%	0.0%	0.1%	0.1%	0.1%	0.1%	0.2%
750	1.8	0.0%	0.0%	0.0%	0.0%	0.0%	0.1%	0.1%	0.1%

Table B-4

PERCENTAGE OF RIGHT LANE THROUGH VEHICLES INFLUENCED AT OR BEYOND ANOTHER DRIVEWAY
POSTED SPEED = 50 MPH

		Right-Turn-In Volume per Driveway, R (vph)							
		R < 30		30 < R < 60		60 < R < 90		R > 90	
Driveway Spacing (ft)	No. of Driveways per 1/4 Mi., n	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$
100	13.2	2.4%	27.3%	7.5%	64.2%	12.2%	82.1%	21.8%	96.1%
125	10.6	2.4%	22.5%	7.5%	56.0%	12.2%	74.7%	21.8%	92.5%
150	8.8	2.4%	19.1%	7.5%	49.6%	12.2%	68.2%	21.8%	88.5%
175	7.5	2.4%	16.6%	7.5%	44.4%	12.2%	62.6%	21.8%	84.4%
200	6.6	2.4%	14.7%	7.5%	40.1%	12.2%	57.7%	21.8%	80.3%
225	5.9	2.4%	13.1%	7.4%	36.5%	12.1%	53.2%	21.7%	76.1%
250	5.3	2.3%	11.7%	7.3%	33.0%	11.9%	48.9%	21.3%	71.8%
275	4.8	2.3%	10.4%	7.1%	29.8%	11.6%	44.7%	20.7%	67.2%
300	4.4	2.2%	9.2%	6.8%	26.6%	11.1%	40.4%	19.8%	62.1%
325	4.1	2.0%	8.0%	6.4%	23.4%	10.4%	35.9%	18.5%	56.4%
350	3.8	1.8%	6.5%	5.6%	19.5%	9.1%	30.2%	16.3%	48.8%
375	3.5	1.5%	5.2%	4.7%	15.5%	7.7%	24.4%	13.7%	40.3%
400	3.3	1.3%	4.1%	4.0%	12.6%	6.5%	19.9%	11.6%	33.4%
425	3.1	1.1%	3.3%	3.4%	10.1%	5.5%	16.2%	9.9%	27.5%
450	2.9	0.9%	2.7%	2.9%	8.2%	4.7%	13.2%	8.4%	22.7%
475	2.8	0.8%	2.2%	2.5%	6.8%	4.1%	11.0%	7.3%	19.0%
500	2.6	0.7%	1.8%	2.2%	5.7%	3.6%	9.1%	6.4%	15.9%
525	2.5	0.6%	1.5%	1.9%	4.7%	3.1%	7.6%	5.5%	13.3%
550	2.4	0.5%	1.2%	1.6%	3.8%	2.6%	6.2%	4.7%	10.9%
575	2.3	0.4%	1.0%	1.3%	3.1%	2.2%	5.0%	3.9%	8.8%
600	2.2	0.4%	0.8%	1.1%	2.4%	1.8%	3.9%	3.2%	6.9%
625	2.1	0.3%	0.6%	0.9%	1.9%	1.5%	3.0%	2.6%	5.4%
650	2.0	0.2%	0.5%	0.7%	1.4%	1.2%	2.3%	2.1%	4.1%
675	2.0	0.2%	0.4%	0.6%	1.1%	0.9%	1.8%	1.7%	3.2%
700	1.9	0.1%	0.3%	0.5%	0.9%	0.7%	1.4%	1.3%	2.5%
725	1.8	0.1%	0.2%	0.4%	0.7%	0.6%	1.1%	1.1%	2.0%
750	1.8	0.1%	0.2%	0.3%	0.5%	0.5%	0.8%	0.8%	1.5%
775	1.7	0.1%	0.1%	0.2%	0.4%	0.4%	0.6%	0.6%	1.1%
800	1.7	0.1%	0.1%	0.2%	0.3%	0.3%	0.4%	0.5%	0.8%
825	1.6	0.0%	0.1%	0.1%	0.2%	0.2%	0.3%	0.3%	0.5%
850	1.6	0.0%	0.0%	0.1%	0.1%	0.1%	0.2%	0.2%	0.3%
875	1.5	0.0%	0.0%	0.0%	0.1%	0.1%	0.1%	0.1%	0.2%
900	1.5	0.0%	0.0%	0.0%	0.0%	0.0%	0.1%	0.1%	0.1%
925	1.4	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.1%	0.1%
950	1.4	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

Table B-5

PERCENTAGE OF RIGHT LANE THROUGH VEHICLES INFLUENCED AT OR BEYOND ANOTHER DRIVEWAY
POSTED SPEED = 55 MPH

Driveway Spacing (ft)	No. of Driveways per 1/4 Mi., n	Right-Turn-In Volume per Driveway, R (vph)							
		R < 30		30 < R < 60		60 < R < 90		R > 90	
		Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$	Single Driveway, P_2	Multiple Driveways, At Least Once per 1/4 Mi., $1 - (1 - P_2)^n$
100	13.2	2.4%	27.3%	7.5%	64.2%	12.2%	82.1%	21.8%	96.1%
125	10.6	2.4%	22.5%	7.5%	56.0%	12.2%	74.7%	21.8%	92.5%
150	8.8	2.4%	19.1%	7.5%	49.6%	12.2%	68.2%	21.8%	88.5%
175	7.5	2.4%	16.6%	7.5%	44.4%	12.2%	62.6%	21.8%	84.4%
200	6.6	2.4%	14.7%	7.5%	40.1%	12.2%	57.7%	21.8%	80.2%
225	5.9	2.4%	13.2%	7.5%	36.6%	12.2%	53.4%	21.8%	76.3%
250	5.3	2.4%	11.9%	7.4%	33.5%	12.1%	49.5%	21.6%	72.4%
275	4.8	2.3%	10.7%	7.3%	30.6%	12.0%	45.7%	21.3%	68.4%
300	4.4	2.3%	9.7%	7.2%	28.0%	11.7%	42.3%	21.0%	64.5%
325	4.1	2.2%	8.7%	7.0%	25.5%	11.4%	38.8%	20.3%	60.3%
350	3.8	2.1%	7.8%	6.7%	23.0%	10.9%	35.4%	19.5%	55.9%
375	3.5	2.0%	6.9%	6.3%	20.5%	10.3%	31.8%	18.4%	51.1%
400	3.3	1.8%	5.9%	5.7%	17.7%	9.4%	27.7%	16.7%	45.3%
425	3.1	1.6%	4.8%	5.0%	14.7%	8.1%	23.1%	14.5%	38.5%
450	2.9	1.4%	4.0%	4.4%	12.3%	7.1%	19.5%	12.8%	33.0%
475	2.8	1.2%	3.4%	3.8%	10.3%	6.3%	16.5%	11.2%	28.1%
500	2.6	1.1%	2.8%	3.4%	8.7%	5.5%	13.9%	9.9%	23.9%
525	2.5	1.0%	2.4%	3.0%	7.3%	4.9%	11.8%	8.7%	20.5%
550	2.4	0.8%	2.0%	2.7%	6.3%	4.3%	10.1%	7.8%	17.6%
575	2.3	0.8%	1.7%	2.4%	5.4%	3.9%	8.7%	7.0%	15.2%
600	2.2	0.7%	1.5%	2.1%	4.6%	3.5%	7.5%	6.2%	13.2%
625	2.1	0.6%	1.3%	1.9%	4.0%	3.1%	6.5%	5.6%	11.4%
650	2.0	0.5%	1.1%	1.7%	3.4%	2.8%	5.5%	4.9%	9.8%
675	2.0	0.5%	0.9%	1.5%	2.9%	2.4%	4.7%	4.3%	8.2%
700	1.9	0.4%	0.8%	1.3%	2.4%	2.1%	3.9%	3.7%	6.9%
725	1.8	0.3%	0.6%	1.1%	2.0%	1.8%	3.2%	3.2%	5.7%
750	1.8	0.3%	0.5%	0.9%	1.6%	1.5%	2.6%	2.7%	4.7%
775	1.7	0.2%	0.4%	0.8%	1.3%	1.3%	2.1%	2.2%	3.8%
800	1.7	0.2%	0.3%	0.6%	1.1%	1.0%	1.7%	1.9%	3.1%
825	1.6	0.2%	0.3%	0.5%	0.9%	0.9%	1.4%	1.6%	2.5%
850	1.6	0.1%	0.2%	0.5%	0.7%	0.8%	1.2%	1.3%	2.1%
875	1.5	0.1%	0.2%	0.4%	0.6%	0.6%	1.0%	1.1%	1.7%
900	1.5	0.1%	0.2%	0.3%	0.5%	0.5%	0.8%	1.0%	1.4%
925	1.4	0.1%	0.1%	0.3%	0.4%	0.4%	0.6%	0.8%	1.1%
950	1.4	0.1%	0.1%	0.2%	0.3%	0.3%	0.5%	0.6%	0.8%

APPENDIX C

EXCERPT TABLES FROM *NCHRP REPORT 395*

Table 20. Conversion from an undivided cross section to a raised-curb median (business & 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.

Table 21. Conversion from an undivided cross section to a raised-curb median (residential & 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.

Table 22. Conversion from an undivided cross section to a TWLTL (business & 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.

Table 23. Conversion from an undivided cross section to a TWLTL (residential & 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.

Table 24. Conversion from a TWLTL to a raised-curb median (business & 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.

Table 25. Conversion from a TWLTL to a raised-curb median (residential & 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
		90						TWLTL
	42,500	30						
		60					Stay with existing	
		90					TWLTL	
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		
		90				existing TWLTL		

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

Table E-13. Annual delay to major-street left-turn and through vehicles for the raised-curb median treatment (hours/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.

Table E-14. Annual delay to major-street left-turn and through vehicles for the TWLTL treatment (hours/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.

Table E-15. Annual delay to major-street left-turn and through vehicles for the undivided cross section (hours/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.

Table E-17. Annual accident frequency for the raised-curb median treatment (acc/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 that subject street segment is located (expressed as a percentage).

3 - Shaded areas denote traffic volume levels that exceed the range of the database used to calibrate the safety model.

Table E-18. Annual accident frequency for the TWLTL treatment (acc/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 that subject street segment is located (expressed as a percentage).

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

Table E-19. Annual accident frequency for the undivided cross section (acc/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 that subject street segment is located (expressed as a percentage).

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