City of Irvine
Transportation Design Procedures

February 2007

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INTRODUCTION

The Transportation Design Procedures (TDP's) contained in this document are intended to assist with the design and review of transportation-related features of development projects in the City of Irvine. The TDP's supersede the 1993 Transportation Guidelines (TG's).

The TDP's establish uniform policies and procedures to assist the engineers, architects and designers who prepare plans and those at City Hall who review them. While the TDP's provide a uniform approach to minimize uncertainty, they are intended to be used in conjunction with sound engineering judgment.

This document begins with a full documentation of sources used, followed by a brief description of the City of Irvine's hierarchy of streets, which is the basis for some of the recommended procedures. This is followed by the general procedures for submitting requests for deviations from the Design Procedures.

Subsequent sections describe each Transportation Design Procedure, accompanied, where applicable, by consideration of deviations from the recommendations. Tables, charts, and figures containing parameters, which are required to determine the appropriate Transportation Design Procedures, are also provided.

The Transportation Design Procedures address the following subjects:

TDP-1	Turn lane pocket lengths
TDP-2	Bay taper lengths
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SOURCES

The list of resources used by the original 1993 *Transportation Guidelines* was reviewed where available along with a number of more recent and updated materials. The following sources were reviewed for information:

- ASCE *Residential Streets, 2nd Edition, 1990.*
- AASHTO *Green Book, Geometric Design of Roads and Streets, 2004, 1994 and 1990 (pre-metric).*
- Ackeret, K.W. *"Criteria for the Geometric Design of Triple Left-Turn Lanes."* ITE Journal, December 1994, pp. 27-33.
- Caltrans *Highway Design Manual, 2004.*
- Caltrans *2003 MUTCD California Supplement, 2004.*
- City of Irvine *Standards and Design Manual (including Standard Plans #102, 103, 104, 106, 107, 108, 40).*
- FHA, *Signalized Intersections Informational Guide*, *Chapter 12*, August 2004
- FHA/ITE/AASHTO *Manual for Uniform Traffic Control Devices, (MUTCD) 2003.*
- FHWA, *Roundabouts an Informational Guide, 2000.*
- Florida DOT, *Triple Left-turns at Signalized Intersections*, December 2002
- ITE *A Toolbox for Alleviating Traffic Congestion and Enhancing Mobility, 1997.*
- ITE *Residential Street Design and Traffic Control, 1989.*
- ITE *Traditional Neighborhood Development, Street Design Guidelines, 1997.*
- ITE *Traffic Engineering Handbook, 1999.*
- ITE *Transportation and Land Development, 2002.*
- ITE *Transportation Planning Handbook, 1992.*
- ITE *Guidelines for Driveway Design and Location, 1989.*
- ITE Journal *Queuing Areas for drive through Facilities, pages 38-42, May 1995*.
- J.E. Leish, *"At Grade Intersections a Design Reference Book"*
- Orange County Standard Plan #1107.
- Oregon State University Transportation Research Institute, *Discussion Paper No.10 Left-turn bays, May 1996*
- Robert Crommelin and Associates, *1972, Entrance-Exit Design and Control for Major Parking Facilities*
- TRB *Highway Capacity Manual*, *2000.*
- TRB *Report 395 Capacity and Operational Effects of Mid-block Left-turn Lanes, 1997.*
- UK Department of Transport *Roads and Traffic in Urban Areas (UK), 1987.*
- UK Department of Transport *Design Manual for Roads and Bridges (UK), 1993.*

Sources rolled forward from the 1993 Transportation Guidelines but not reviewed were:

- ITE *Guidelines for Urban Major Street Design, 1989* - Source not located.

ROADWAY NETWORK CLASSIFICATION

The roadway system in the City of Irvine consists of four highway classifications: Major, Primary, Secondary and Commuter, which accommodate the majority of intercity and intracity trips. Additionally, there are three street classifications, Local Collector, Local Street and Private Way, which provide access to the highways. The following highway definitions are derived from the City's General Plan. It should be noted that the prevailing speeds represent typical travel speeds for a given highway classification, not design speeds. Design speeds by highway classification are specified in the City of Irvine's Standard Plans.

Major: Prevailing Speed 55 mph, Design Speed 60 mph

A high-speed highway with restricted access, carrying intermediate range trips to or between non-residential land uses. A Major is designed to provide six or eight lanes of through traffic, with bicycle lanes and emergency parking only. Pedestrian interference with vehicular traffic is minimized.

Primary: Prevailing Speed 50 mph, Design Speed 55 mph

A moderate-speed highway is one with restricted access, which abuts and distributes trips to a variety of land uses. This facility serves primarily short-range trips. A Primary is designed to provide four lanes of through traffic, bicycle lanes and emergency parking only, and may be required to accommodate moderate parallel and perpendicular pedestrian movements.

Secondary: Prevailing Speed 45 mph, Design Speed 50 mph

A moderate-speed highway is one that abuts and distributes trips to similar land uses. The facility serves short-range trips. A Secondary is designed to provide four lanes of through traffic, bicycle lanes, and emergency parking and will have considerable parallel and perpendicular pedestrian movement.

Commuter: Prevailing Speed 30-35 mph, Design Speed 45 mph

A relatively low-speed street abutting similar land uses. The facility primarily collects and distributes trips to and from highways, and carries trips between adjacent land uses. A Commuter provides two lanes of through traffic, emergency parking, and a significant amount of parallel and perpendicular pedestrian traffic.

Community Collector: Prevailing Speed 30 mph, Design Speed 35 mph

These streets do not generally serve residential driveways, but connect local streets within a development.

Local Street: Prevailing Speed 25 mph, Design Speed 25 mph

These streets provide access to immediately adjacent properties. Abutting properties may have a driveway that connects to the street. A low speed street is primarily for access to residential land uses. A Local Street may have parking and a significant amount of parallel and perpendicular pedestrian traffic. While through traffic is possible, it is discouraged by operational controls or by hardscape such as cul-de-sacs.

Private Way: Prevailing Speed 20 mph, Design Speed 20 mph

This is a low-speed roadway for general circulation in residential neighborhoods to access residential units, garages, and parking areas. The maximum Average Daily Traffic on a Private Way shall not exceed 850 trips. When the Average Daily Traffic on a Private Way exceeds 850 trips, the entire Private Way shall be designated a Private Local Street and shall be designed in accordance with City Plan 104 for Residential Local Streets.

REQUEST FOR DEVIATIONS

The Design Procedures should be followed to the maximum extent possible. However, there will be instances when an applicant may have a sound reason for proposing a design different than those listed in the Design Procedure. In that case, the onus is on the applicant to provide sufficient explanation and justification to the City for the request. Each request should be accompanied by a supporting written statement that shall include the following items:

1. Reasons for requesting the deviation;

2. Specific justification for requesting the proposed design feature (i.e. technical, environmental, economic, right of way constraints, community concerns etc.);

3. Explanation of how the proposal meets the intent of the Design Procedure; or does not create any traffic operations or safety issues and,

4. Supporting documentation, which shall include references justifying the rationale for the requested deviation where applicable.

TDP-1 TURN LANE POCKET LENGTHS

Recommendation

The recommended left turn pocket length for signalized intersections, based on the J.E. Leish nomograph for highways[1,2], is presented in Figure 1.1. The parameters used in the recommended J.E. Leish nomograph include the peak hour turn volume, the signal cycle length, and the estimated truck mix.

During the peak hours signalized intersections in the City of Irvine typically have cycle lengths of 120 seconds. The recommended procedure is to provide for a storage length to accommodate all turn arrivals during the peak hour 95 percent of the time using a 120 second signal cycle. The assumed estimated truck mix should be explicitly stated in the analysis. Engineering judgment should be used for cases that may dictate longer left turn pockets than recommended by the nomograph such as for access to schools, sports venues and major shopping centers.

Storage lengths for unsignalized intersections should be designed based on one foot for each vehicle per hour (vph) turning left during the peak hour.

The recommended minimum acceptable design length for a turn pocket length is 150 feet for Major, Primary, and Secondary highways with speeds greater than 45 miles per hour. For Commuter and Local streets and driveways, pocket lengths down to 90 feet are permitted when volumes warrant.

The recommended maximum single turn pocket length shall be 300 feet. Dual left-turn lanes shall, therefore, be implemented when over 300 feet of storage is required or when necessary to provide acceptable levels of service at the intersection. The length of each lane where double left-turns are provided shall be half that of the single left-turn lane. When dual left-turn lanes are required, they shall be provided at time of construction of the project, in order to minimize the amount of signal time dedicated to the left-turn movement.

Discussion and Consideration for Deviation

The length of left pockets at a signalized intersection is based on a number of parameters, including traffic control, physical constraints, turn volume, cycle length, and percentage of trucks in the traffic flow. The purpose of the minimum turn pocket length is to allow the turning vehicle to exit the through movement and decelerate into the turn pocket minimizing the impact to the through traffic movement.

For new designs where pocket lengths are not a constraint, the recommended 95 percent confidence level turn length should be provided. Where a constraint exists, such as limited distance between an adjacent full access driveway and intersections, the 90 percent confidence level shall apply.

It is possible that the application of TDP-1 could adversely affect existing access locations, such as the preclusion of previously possible left-turn access to and from adjacent properties. Any potential impacts of this nature should be taken into account during the application of this TDP.

Where the peak hour left-turn volume exceeds 300, but dual-lefts would be impractical, such as at T-intersections where the opposing approach has no left-turn lanes, the City Traffic engineer and City Engineer shall be consulted.

Where left turn movements are more that 600 vehicles per hour triple-left turn lanes may improve intersection capacity[3]. However, triple left turn lanes require more specific justification and more attention to detail in the design than for double left turn lanes apply and are not appropriate where:

- A high number of vehicle-pedestrian conflicts occur.
- Left-turning vehicles are not expected to evenly distribute themselves among the lanes.
- Channelization may be obscured.
- Sufficient right-of-way is not available to provide for the design vehicle.

In addition, the interaction between vehicles in adjacent lanes and also width of the receiving lanes should be considered. The following are design considerations for triple left-turn lanes provided by Ackeret.[4]:

- Widths of receiving lanes.
- Width of intersection (to accommodate three vehicles abreast).
- Clearance between opposing left-turn movements during concurrent maneuvers.
- Pavement marking visibility.
- Placement of stop bars for left-turning and through vehicles.
- Weaving movements downstream of turn.
- Potential for pedestrian conflict.

An operational analysis of the intersection should be provided[5] which indicates that the provision of a triple left turn lane would correct a situation in which the overall capacity of the intersection would be seriously deficient, and that no other geometric or signal modifications would correct the deficiency. The operational analysis must take into account the effects of adjacent intersections, including:

- Backup from a downstream signal on the receiving roadway
- Relative turning movement distribution at a downstream intersection that would compromise the ability of the receiving lanes to store the left turning vehicles
- Heavy volumes from other approaches that are also accommodated by the roadway that receives the left turns.
- Upstream effects that could make it difficult to distribute the approaching left turns over the three left turning lanes (e.g. a heavy single lane exit ramp from a freeway).

The Highway Capacity Manual should be used for operational analysis only when there are no complicating factors of the type listed above. If there are any upstream or downstream influences, a microscopic simulation should be performed.

Should a triple left turn be included in a previous City adopted mitigation measure, fee program or City CIP, consultation with the City should be initiated to determine the scope of any needed operational analysis that will be required, if any.

Sources

[1] *J.E. Leish, "At Grade Intersections, A Design Reference Book*
[2] *Oregon State University Transportation Research Institute, Discussion Paper No.10 Left-turn bays, Mat 1996.*
[3] *USDOT FHA, Signalized Intersections Informational Guide, Chapter 12, August 2004*
[4] *Ackeret, K.W. "Criteria for the Geometric Design of Triple Left-Turn Lanes." ITE Journal, December 1994, pp. 27-33.*
[5] *Florida DOT, Triple Left-turns at Signalized Intersections, December 2002*

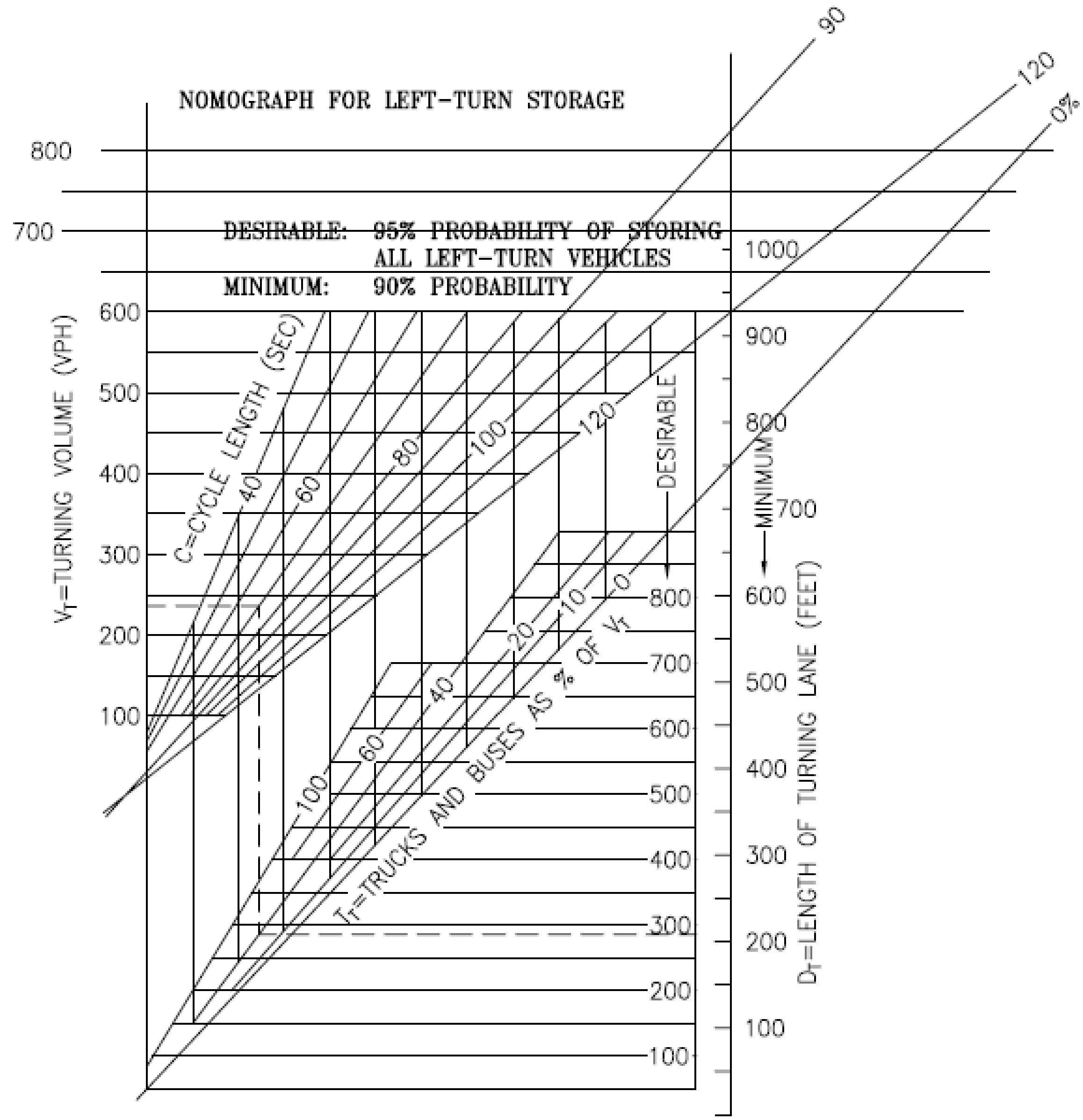


Figure 1.1 – J.E. Leish Nomograph

TDP-2 BAY TAPER LENGTHS

Recommendation

The minimum recommended bay taper lengths for Major, Primary, and Secondary highways within the City of Irvine shall be 150 feet for dual left-turn lanes and 90 feet for single left-turn lanes (City of Irvine Standard Plan No.107).[1] For Commuter and Local streets and driveways, the bay taper may be reduced to 60 feet if the volumes are minimal and the required left-turn pocket length is less than 150 feet. This will require the approval of the City Engineer on a case-by-case basis.

Discussion and Consideration for Deviation

The Caltrans *Highway Design Manual*[2] defines a bay taper as a reversing curve along the left edge of the traveled way, which directs traffic into the left-turn lane. The length of this bay taper should be short to clearly delineate the left-turn move and to discourage through traffic from drifting into the left-turn lane.

For triple left-turn lanes a bay taper length of 180 feet should be provided.

Sources

[1] *City of Irvine Standard Plan No. 107.*
[2] *Caltrans Highway Design Manual, Chapter 4 (p.400-9), 2004.*

TDP-3 LEFT-TURN IN/OUT ACCESS

Recommendations

Figure 3.1 provides the procedures to determine whether left in only or left in/left out access at unsignalized intersection locations will be considered along Major, Primary, Secondary, and Commuter streets. The procedure is based on the volume of vehicles entering and/or exiting a driveway in relationship to the conflicting volumes along the highway.

The recommended procedures are illustrated in Figures 3.1 and 3.2. Figure 3.1 depicts the relationship between the left-turn volume LTV-I for the left-turn volume in and LTV-O for the left-turn volume out and the sum of the conflicting volumes V_C for each scenario. Figure 3.2 presents a graph in which the left-turn in and/or left-turn out is compared to its conflicting volume. The access is considered acceptable when the intersection of the left-turn volume and the sum of the conflicting volumes lie below the line. Access is not recommended when this point is above the line.

This procedure should be used in conjunction with supporting *Highway Capacity Manual*[1] unsignalized intersection levels of service analysis. If left-turn access is proposed for a Major, Primary, or Secondary highway, an accident analysis using the City of Irvine's Traffic Engineering/Circulation Division requirements will be required to determine the projected accident rate. This rate will be used to assist in determining the acceptability of the access and future acceptability of access based on actual operation.

If implementation of restricted left-turn access and/or egress will result in additional u-turns at signalized intersections on either side of the access point, then analysis should be performed (as outlined in TDP 1) to determine whether the left-turn pocket lengths need to be increased.

Discussion and Consideration for Deviation

The ability to provide full or partial access along a highway should commensurate with the ability to safely guide vehicles in and out of the driveways through the highway traffic stream while maintaining acceptable levels of service. One criteria to determine whether acceptable and safe conditions exist for providing full left in and left out access is the *Highway Capacity Manual* unsignalized intersection analysis methodology. The recommended procedure is based on a series of relationships for highways with speeds 45 mph or greater, in which the combination of left-turn volume and conflicting volumes maintains a level of service D or better.

The operation of an unsignalized access location is a function of the distribution of gaps in the traffic stream and the distribution of gaps acceptable to drivers. The length in time for an acceptable gap increases as the speed of the traffic stream being entered or crossed increases, and as the number of lanes that must be crossed increases. Based on this criterion, a full unsignalized driveway access along a Major highway typically cannot be supported because the number of lanes and prevailing speeds along a Major highway conflict with the ability to provide safe access[2]. Because of the significant number of conflicts with left-turn outbound movements, the requirement for permitting outbound lefts is much more stringent than with left-turn in only.

As indicated, the Design Procedures are based on the *Highway Capacity Manual* methodology for determining levels of service. Inherent in that methodology is the assumption that the arriving conflicting vehicles will be random. Because of signalization and the platooning of vehicles, there may be circumstances where it can be demonstrated that the conflicts will not be as great as predicted in the HCM procedures, and a left-turn in/out may be considered. Similarly, however, the platooning could actually result in a greater number of conflicts than a random condition. In summary, the procedure is to provide a determination as to when to consider and when not to consider left-turn access. Accident analysis may need to be provided, particularly if the determination point is near the threshold line.

When a left-turn in only access is allowed along a primary or major arterial with speeds in excess of 45 mph, additional left-turn lane length may be requested to preclude back-up of left-turn vehicles in the through lane.

Other considerations[2, 3] for whether left turn in/outs are appropriate include:

- An existing crash history (where there are crashes that are potentially correctable by adding a left-turn pocket).

- Where the geometric design of the roadway is such that sight distance for left-turn traffic is insufficient for safe completion of a left-turn across opposing traffic.

- High speed of opposing through traffic.

- A high volume of opposing through traffic.

- Where left turning vehicles must cross three or more lanes of opposing through traffic.

Minimum sight distances[4] for a passenger car to complete a left-turn from a major roadway are:

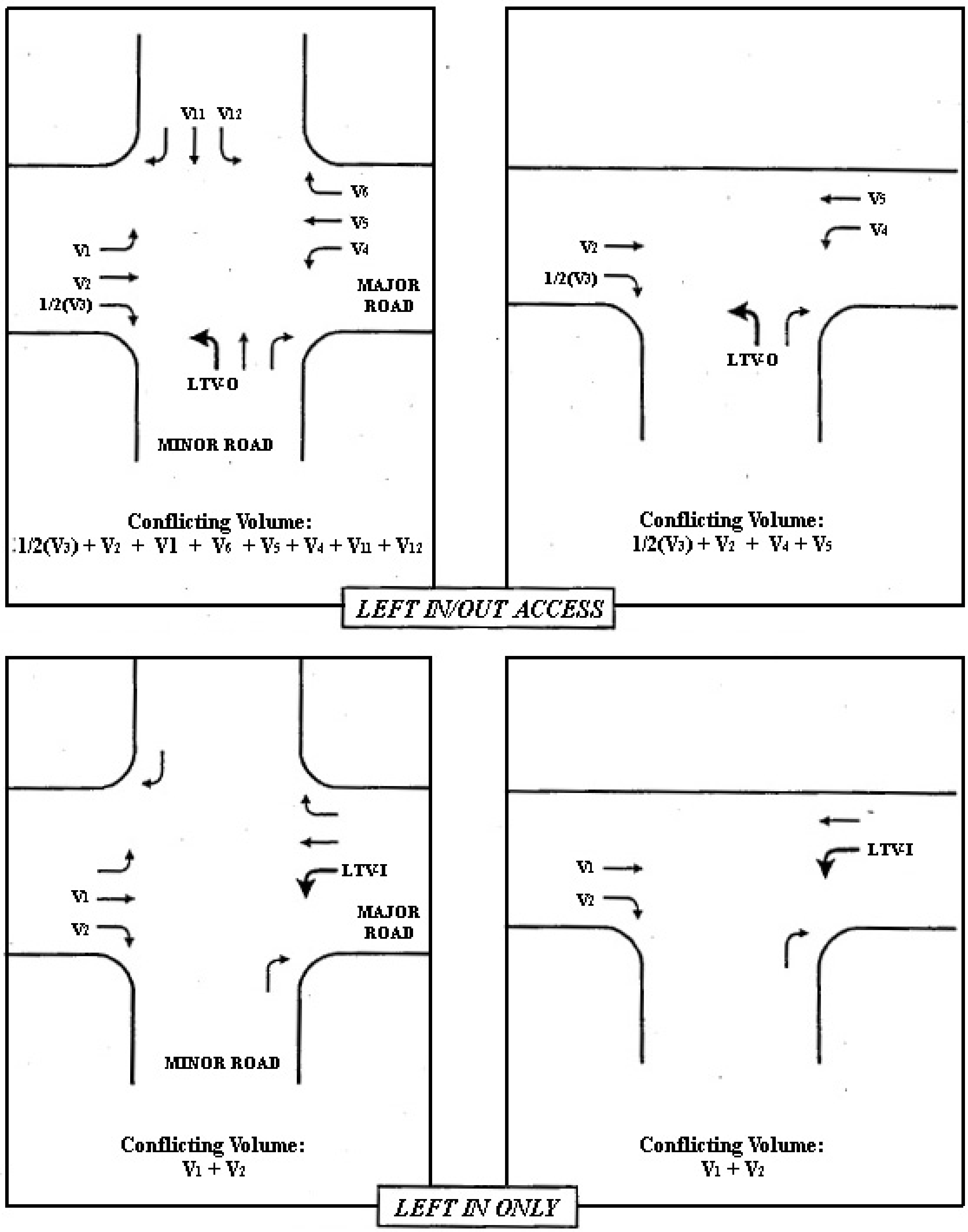
Highway Designation	**Design Speed (V)**	**Min Sight Distance (ft) 2 lane**	**4 lane divided**
Major	60	490	530
Primary	55	445	485
Secondary	50	405	445
Commuter	45	365	400
Local Collector	35	285	315
Local Street	25	205	225

Sight distances should be increased for multilane highways and for other vehicle types in accordance with AASHTO “Intersections Case F – Left turns from the major road”

Sources

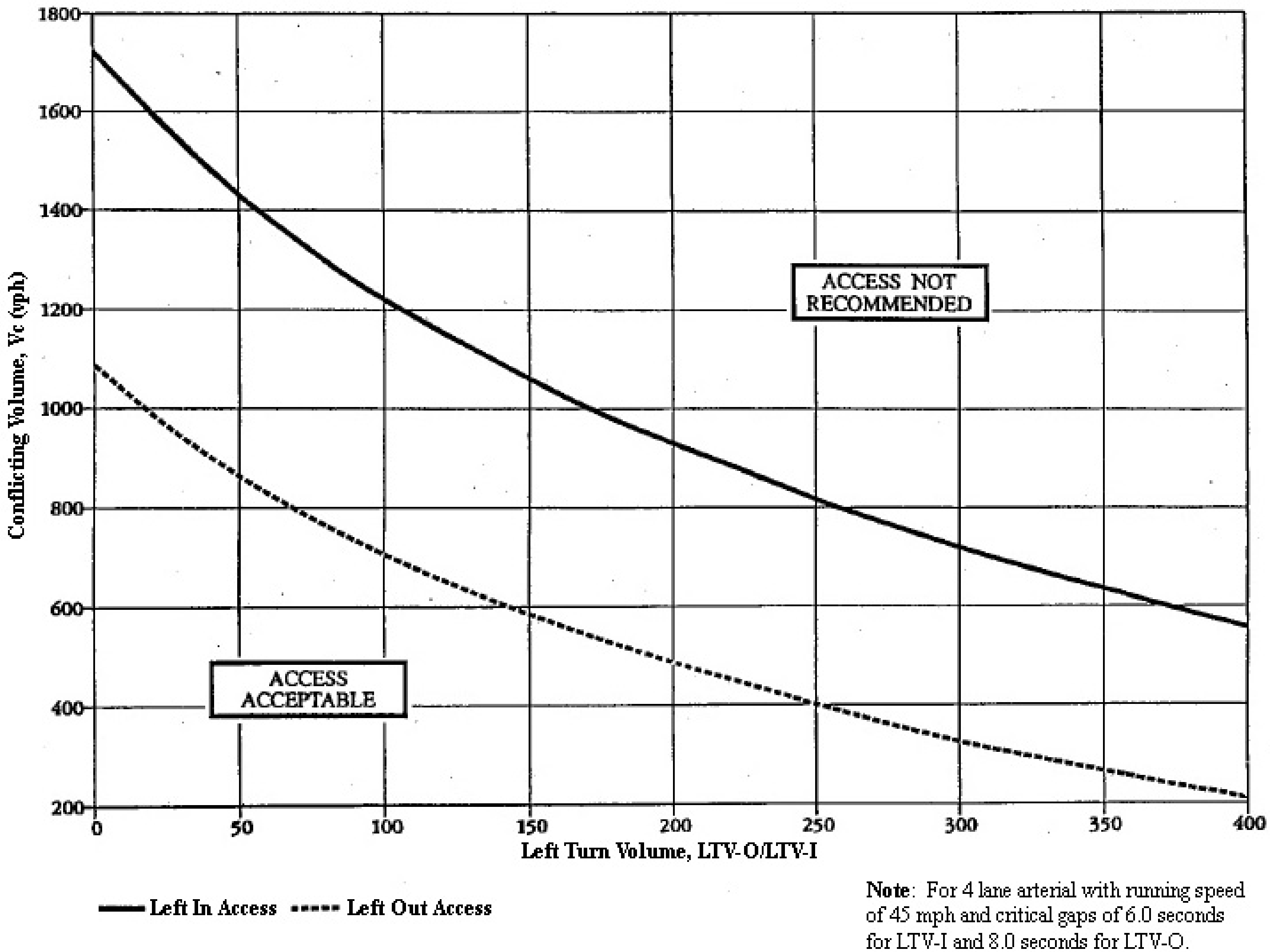
[1] *TRB Highway Capacity Manual. Special Report 209, Chapter 10.*
[2] *ITE Transportation and Land Development. Chapter 14.*
[3] *TRB ,Report 395, Capacity and Operational Effects of Mid-Block Left-turn lanes, 2003.*
[4] *AASHTO Green Book, 2004, Exhibit 9-67 Page 675.*

Left Turn Access Design Procedures



Source: *1985 Highway Capacity Manual, Figure 10-2.* **Figure 3.1**

Left Turn Access Criteria



Note: For 4 lane arterial with running speed of 45 mph and critical gaps of 6.0 seconds for LTV-I and 8.0 seconds for LTV-O.

Source: *Highway Capacity Manual, Special Report 209, TRB, Chapter 10.*

Figure 3.2

TDP-4 RIGHT-TURN LANES AT UNCONTROLLED DRIVEWAYS

Recommendations

Right-turn lanes at driveways are required when the turn volumes and through volumes could conflict and increase the potential for accidents. The following Design Procedures define when right-turn lanes should be provided by highway type.

Recommendations for when right-turn lanes should be provided:

Highway	Right-turn Lane
Major	All
Primary	Peak hourly volume > 100 vehicles
Secondary	Peak hourly volume > 200 vehicles
Commuter	Not required

The length of the right-turn lane should be sufficient to allow a vehicle traveling at the prevailing speed to decelerate before entering the driveway (and not based on storage criteria). The length should also be sufficient to avoid through lane queuing potentially blocking the right-turn lane and an analysis of this should be provided.

The recommended distance to allow the vehicles to decelerate is as follows (the need for a standard 90-foot bay taper is also included) [1]:

Classification	Bay Taper (feet)	Right-turn Lane (feet)	Total (feet)
Major	90	250	340
Primary	90	200	290
Secondary	90	150	240
Commuter	90	150	240
Local Collector	90	150	240
Local Street	N/A	N/A	N/A

The City of Irvine currently permits a defacto right-turn lane provided the curb lane is a minimum of 19’ or where there are existing on-street bike lanes provided, the curb lane width is at least 12’ wide plus a 7’ bike lane.

Discussion and Consideration for Deviation

Two factors affect the need for and the length of right-turn lanes at an unsignalized driveway:

- the need for a right-turn lane is based on the right-turn volume from the highway into the access and
- the length of the right-turn lane is based on the typical operating speeds of the highway from which the right-turn is to be made.

Right-turn lanes are required when the turn volumes and through volumes conflict and create the potential to cause accidents. However, no quantitative measures have been identified to define when right-turn lanes should be provided based on such conflicts. The recommended procedures are provided for consideration and should not replace engineering judgment.

Where a proposed driveway creates a new leg of a signalized intersection, the future intersection level of service may be used to determine whether a right-turn lane is required.

Consistent with their function, access along Major highways should be minimized. However, in the event that access is provided, it is recommended that right-turn lanes be provided for all access locations. On the other hand, Commuter streets which have relatively lower volumes and are designed to interface with access and driveway locations are not required to have right-turn lanes. The procedures for Primary and Secondary highways are recommendations that attempt to balance the speed and volume of the through movement with the right-turn demand.

Source

[1] *Caltrans Highway Design Manual – Table 405.2, 2004.*

TDP-5 FREE RIGHT-TURN LANES AT SIGNALIZED INTERSECTIONS

Recommendations

Free right-turn lanes at highway intersections, properly designed, can accommodate very high vehicle volume concentrations. Their use should be considered when right-turn volumes (existing or projected) exceed 325 vehicles per hour, and when the intersection level of service analysis will result in unacceptable levels of service if the right turn is treated as a conventional signal controlled intersection.

In order to provide for a dedicated free right-turn lane, a traffic island must be installed to separate the free right-turn movement and the through movement. The island also serves as a refuge for pedestrians crossing the highway.[1] Bicycle lanes must also be considered and design guidelines incorporated from the Caltrans Highway Design Manual.[2]

Island sizes and shapes vary from one intersection to another. They should be large enough to command attention. Figure 5.1 presents a design that meets the basic criteria and design elements in a free right-turn design. However, this design is only a guideline to consider. Engineering judgment should prevail as each new design is developed, sensitive to vehicular, bicycle, and pedestrian activity at the location in question.

The bay taper entering the right-turn lane should be 90 feet, consistent with the single left-turn bay taper. The free right-turn lane should begin at a sufficient distance from the intersection to allow the right-turning vehicle not to be queued behind the through movement. The review procedure indicates that this distance should be approximately equal to one foot per each peak hour through vehicle per lane (i.e. a minimum of 300 feet for 300 vehicles/hour).

Exiting the right-turn lane into an existing through lane requires a transition per TDP-6, and a tangent, per TDP-7.

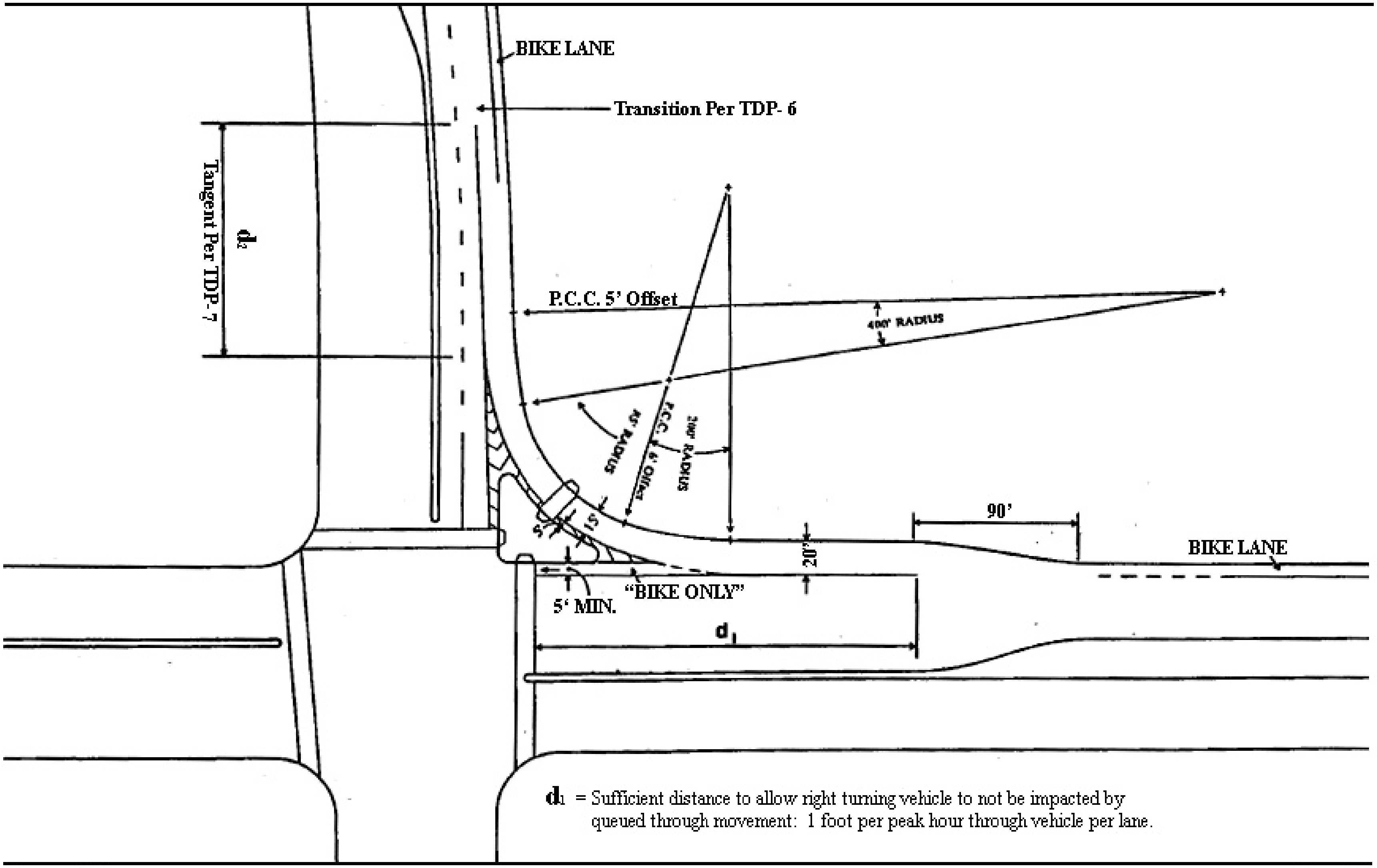
Discussion and Consideration for Deviation

Although free right-turn lanes can be very beneficial in providing increased intersection capacity, they can introduce conflicts, as compared to the conventional signal-controlled separate right-turn lane, when the vehicle exits the right-turn lane and transitions into the through lane. Therefore, each application should carefully consider all the movements that would affect the design, and adjustments made accordingly. The ultimate design must be reviewed and approved by the City Engineer.

Sources

[1] *Caltrans Traffic Manual, Section 6.*
[2] *Caltrans Highway Design Manual, Section 405.4, Chapter 1000, 2004.*

Free Right Turn Lane



d1 = Sufficient distance to allow right turning vehicle to not be impacted by queued through movement: 1 foot per peak hour through vehicle per lane.

Source: *Caltrans*

Figure 5.1

TDP-6 TRANSITIONS

Recommendation

Adequate tapers are needed for lane drops or at locations where traffic must be shifted laterally. The transition should be made on a tangent section whenever possible, and should avoid sight distance restrictions. Whenever feasible, the entire transition should be visible to the driver of a vehicle approaching the narrower section. The design should be such that intersections at-grade within the transition area are avoided. All lane drops should occur in the right lane, instead of the number one or median lane so that traffic merges to the left.

According to the Caltrans Highway Design Manual[1] transitions for lane additions, either for left or right turns or to add a lane to a ramp, should typically occur over a length of 120 feet. Lengths shorter than 120 feet are acceptable where design speeds are below 45 mph.

The minimum transition or lateral shift should be based on the following formulas from ITE[2]:

Highway Designation	**Transition Length**
Major (min 120 ft)	Design Speed x W*
Primary (min 120 ft)	Design Speed x W
Secondary (min 120 ft)	Design Speed x W
Commuter (requires City Engineer approval)	(Design Speed^2 x W)/60
Local Collector (requires City Engineer approval)	(Design Speed^2 x W)/60
Local (requires City Engineer approval)	(Design Speed^2 x W)/60

* W = Width of lateral shift on lane to be dropped

Sources

[1] *Caltrans Highway Design Manual, Section 206, 2004.*

[2] *ITE Guidelines for Urban Major Street Design, page 15, 1989.*

TDP-7 HORIZONTAL ALIGNMENT

Recommendation

Horizontal alignments should provide for safe and continuous operation at uniform design speeds. The minimum radius curves are based on the AASHTO[1] minimum radius formula as follows:

$$R_{min} = \frac{V^2}{15\,(e+f)}$$

Where:

e = rate of roadway super-elevation (in feet per foot)
f = side friction factor
V = vehicle speed (in mph)
R = radius curve (in feet)

The City cross slope (e) on all highways is –0.022. The design speed by highway classification, maximum f, resulting minimum radius of curve and minimum tangent between reverse curves are presented as follows:

Highway Designation	**Design Speed (V)**	**f**	**Minimum Radius of Curve (R_{min})**	**Minimum Tangent Between Reverse Curves**	**Minimum* Stopping Sight Distance (ft)**
Major	60	0.12	2,470	160	570
Primary	55	0.13	1,880	150	495
Secondary	50	0.14	1,420	125	425
Commuter	45	0.15	1,150	100	360
Local Collector	35	0.155	605	100	250
Local	25	0.165	300	50	155
Private Way	20		175	50	115

* Distances in this table should be increased by 20% for sustained downgrades steeper than 3% and longer than 1.25 miles.

If the minimum radius indicated above does not provide the desired lateral clearance to an obstruction, curvature shall be increased in accordance with the minimum stopping-sight distance tables in Chapter 2 of the Caltrans Highway Design manual.[2]

All street centerlines should intersect at a 90-degree angle. Angles deviating more than 15 degrees from a 90-degree angle will not be allowed. Sight distances at intersections shall be in accordance with City of Irvine Standard Plan No. 403[3].

Traveled way widening on some horizontal curves may be required especially if high truck volumes are forecast. AASHTO Guidelines[1] (Chapter 3, page 212) should be used to determine whether widening is necessary (see Table 7.1). A widening of less than two feet is not required.

The use of super-elevation on conventional City streets should be discouraged except where right-of-way constraints make it necessary.

Discussion and Consideration for Deviation

In hillside communities, a deviation may be requested due to physical terrain. A minimum radius of 150 feet may be allowed for short sections on cul-de-sac streets to minimize grading, when permitted by the City Engineer on a case-by-case basis.

In some cases a deviation may be requested due to environmental or right of way constraints, however, the ultimate design must be reviewed and approved by the City Engineer.

There are no minimum street radii along private ways and private courts. However, when a centerline radius less than 300 feet is desired along private ways and courts, the request is subject to further review. This review will require fire truck turning templates and depiction of vehicles parked in all marked curbside parking spaces.

Sources

[1] *AASHTO, Green Book, Geometric Design of Highways and Streets, p. 146, 208-214,445, 2004.*
[2] *Caltrans Highway Design Manual, Chapter 2, Table 201.1, 2004.*
[3] *City of Irvine Standard Plan No. 403.*

US Customary																					
Radius of curve (ft)	Roadway width = 24 ft							Roadway width = 22 ft							Roadway width = 20 ft						
	Design Speed (mph)							Design Speed (mph)							Design Speed (mph)						
	30	35	40	45	50	55	60	30	35	40	45	50	55	60	30	35	40	45	50	55	60
7000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.6	1.6	1.7	1.7	1.8	1.9	1.9
6500	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.7	0.7	0.8	0.8	0.9	1.0	1.6	1.7	1.7	1.8	1.8	1.9	2.0
6000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.6	1.7	1.8	1.8	1.9	2.0	2.0
5500	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.7	0.7	0.8	0.9	0.9	1.0	1.1	1.7	1.7	1.8	1.9	1.9	2.0	2.1
5000	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.7	1.8	1.9	1.9	2.0	2.1	2.1
4500	0.0	0.0	0.0	0.0	0.1	0.1	0.2	0.8	0.8	0.9	1.0	1.1	1.1	1.2	1.8	1.8	1.9	2.0	2.1	2.1	2.2
4000	0.0	0.0	0.0	0.1	0.2	0.2	0.3	0.8	0.9	1.0	1.1	1.2	1.2	1.3	1.8	1.9	2.0	2.1	2.2	2.2	2.3
3500	0.0	0.0	0.1	0.2	0.3	0.3	0.4	0.9	1.0	1.1	1.2	1.3	1.3	1.4	1.9	2.0	2.1	2.2	2.3	2.3	2.4
3000	0.0	0.1	0.2	0.3	0.4	0.5	0.6	1.0	1.1	1.2	1.3	1.4	1.5	1.6	2.0	2.1	2.2	2.3	2.4	2.5	2.6
2500	0.2	0.3	0.4	0.5	0.6	0.7	0.8	1.2	1.3	1.4	1.5	1.6	1.7	1.8	2.2	2.3	2.4	2.5	2.6	2.7	2.8
2000	0.4	0.5	0.6	0.7	0.8	1.0	1.1	1.4	1.5	1.6	1.7	1.8	2.0	2.1	2.4	2.5	2.6	2.7	2.8	3.0	3.1
1800	0.5	0.6	0.8	0.9	1.0	1.1	1.2	1.5	1.6	1.8	1.9	2.0	2.1	2.2	2.5	2.6	2.8	2.9	3.0	3.1	3.2
1600	0.7	0.8	0.9	1.0	1.2	1.3	1.4	1.7	1.8	1.9	2.0	2.2	2.3	2.4	2.7	2.8	2.9	3.0	3.2	3.3	3.4
1400	0.8	1.0	1.1	1.2	1.4	1.5	1.6	1.8	2.0	2.1	2.2	2.4	2.5	2.6	2.8	3.0	3.1	3.2	3.4	3.5	3.6
1200	1.1	1.2	1.4	1.5	1.7	1.8	1.9	2.1	2.2	2.4	2.5	2.7	2.8	2.9	3.1	3.2	3.4	3.5	3.7	3.8	3.9
1000	1.4	1.6	1.7	1.9	2.0	2.2	2.4	2.4	2.6	2.7	2.9	3.0	3.2	3.4	3.4	3.6	3.7	3.9	4.0	4.2	4.4
900	1.6	1.8	2.0	2.1	2.3	2.5		2.6	2.8	3.0	3.1	3.3	3.5		3.6	3.8	4.0	4.1	4.3	4.5	
800	1.9	2.1	2.2	2.4	2.6	2.8		2.9	3.1	3.2	3.4	3.6	3.8		3.9	4.1	4.2	4.4	4.6	4.8	
700	2.2	2.4	2.6	2.8	3.0			3.2	3.4	3.6	3.8	4.0			4.2	4.4	4.6	4.8	5.0		
600	2.7	2.9	3.1	3.3	3.5			3.7	3.9	4.1	4.3	4.5			4.7	4.9	5.1	5.3	5.5		
500	3.3	3.5	3.7	3.9				4.3	4.5	4.7	4.9				5.3	5.5	5.7	5.9			
450	3.7	3.9	4.1					4.7	4.9	5.1					5.7	5.9	6.1				
400	4.2	4.4	4.7					5.2	5.4	5.7					6.2	6.4	6.7				
350	4.8	5.1	5.3					5.8	6.1	6.3					6.8	7.1	7.3				
300	5.6	5.9						6.6	6.9						7.6	7.9					
250	6.8							7.8							8.8						
200	8.5							9.5							10.5						

Notes: Values shown are for WB-50 design vehicle and represent widening in feet. For other design vehicles, use adjustments in Exhibit 3-48.
Values less than 2.0 ft may be disregarded.
For 3-lane roadways, multiply above values by 1.5.
For 4-lane roadways, multiply above values by 2.

Calculated and Design Values for Traveled Way Widening on Open Highway Curves (Two-Lane Highways, One-Way or Two-Way) (Continued)

Source: *AASHTO--Geometric Design of Highways and Streets, p. 212.* **Figure 7.1**

TDP-8 LANE WIDTHS

Recommendations

The City of Irvine Standard Plans[1] No. 101, 102 and 103 provide for the right-of-way and curb to curb design standards for constructing Major, Primary, Secondary highways and Commuter Streets in the City of Irvine. Plan No.104 and Plan No.111 provide design standards that include lane widths for Local Streets and Private Ways. The recommended procedures present the ideal lane widths for typical street sections, both at mid-block and at the intersections.

Lane widths less than 12 feet reduce travel speeds[2]. The following are the recommended lane widths to be used for the design and installation of pavement striping.

Lane	**Recommended**
Through lane (adjacent to raised median)	13 feet
Through lane (adjacent to painted median)	12 feet
Through lane	12 feet[3,4]
Left turn (including double and triple left turn)	10 feet
Curb lane (with a bicycle lane)	20 feet (12 feet vehicle, 8 feet bike lane)
Curb lane (without a bicycle lane)	14 feet
On-street Bicycle lane	8 feet

The recommended lane widths shall apply to all new highway designs.

Discussion and Consideration for Deviation

All lane width requirements and intersection design controls should be evaluated when making the lane width selection.

Minimum lane widths for retrofitting and where restricted right-of-way is a constraint are:

Lane	**Minimum**
Through lane adjacent to raised or striped median	12 feet
Through lane	11 feet
Left turn	10 feet
Curb lane (with a bicycle lane)	17 feet (12 feet vehicle, 5 feet bike lane)
Curb lane (without a bicycle lane)	12 feet
Bicycle lane	5 feet

Sources
[1] *City of Irvine Standard Plans No. 101, 102, 103, 104 and 111.*
[2] *TRB, Highway Capacity Manual, Chapter 12, 2000.*
[3] *Caltrans Highway Design Manual, Section 301.1, 2004.*
[4] *ITE Transportation and Land Development, 4-12, Table 4-2, 2002.*

TDP-9 DISTANCES BETWEEN SIGNALIZED INTERSECTIONS

Recommendations

Consistent separation between signalized intersections along a given highway is one of the key parameters for maintaining signal progression. The recommended distance between intersections is based primarily on cycle length and prevailing speeds.[1] The recommended ideal distance between one signalized intersection and another is as follows:

Classification	Recommended Spacing
Major	1 mile
Primary	1/2 mile
Secondary	1/4 mile
Commuter	1/8 mile

Discussion and Consideration for Deviation

As indicated, the ideal distance between one signalized intersection and another is based primarily on the relationships between two issues: cycle length and speed. In general, the longer the cycle length and the higher the speed, the greater the distance is between the signalized intersections.

Although, the recommended distance between signalized intersections is desirable, it is not practical in all locations.

The following table has been prepared to provide guidance for the minimum spacing for signalized intersections within the City. It should be noted that while these are recommendations, there may be various other factors or extenuating circumstances, where distances less than the minimums presented in the table could be supported. The City recognizes that there are a number of General Plan/Zoning land use objectives, right of way restrictions, physical constraints and/or environmental constraints, which may dictate that the recommended minimum spacing identified in this table can not be met. Any proposal for an access spacing less than from this recommended minimum spacing should clearly document what factors or extenuating circumstances have resulted in the inability to provide the recommended minimum spacing and document what alternatives have been considered. In such deviation cases, the City Traffic Engineers should be consulted and a signal progression analysis provided that demonstrates that the proposed new signalized intersection can be reasonably accommodated without detrimentally impacted traffic flow.

Classification	Recommended Minimum Spacing
Major	1/2 mile
Primary	1/4 mile
Secondary	400 feet*
Commuter	300 feet*

*This distance may need to be increased depending on left-turn storage requirements in accordance with TDP-11 on Corner Clearance.

It should be noted, however, that the distance between intersections should be used in conjunction with the procedures for distances between driveways and intersections, and corner clearance.

Source
[1] *ITE Transportation and Land Development. Chapter 4, pages 4.23 to 4.33, 2002.*

TDP-10 DISTANCE BETWEEN DRIVEWAYS AND INTERSECTIONS

Recommendations

The following table presents the recommended minimum spacing between a driveway and an intersection or two driveways[1, 2]. In addition to the recommended minimum spacing, additional recommendations on Corner Clearance as detailed in TDP-11 will need to be addressed.

Classification	Minimum Separation (feet)
Major	335 feet
Primary	230 feet
Secondary	185 feet
Commuter	150 feet
Local*	105 feet
Private Way*	90 feet

* Minimum separation lengths are not required within single-family detached residential tracts.

The distances between driveways should be measured from the centerline of the driveways, whereas the distance between the intersection and the driveways should be from the curb face of the street to the near side curb face of the driveway.

The minimum distance between unsignalized local street intersections, whether public or private, shall be 105 feet measured from the curb face of the street to the nearside adjacent street, near side curb.

To minimize the impacts of proposed accesses on highways and to ensure the efficient operation of the street system, the recommendations have been developed to identify desirable access spacing. Where necessary, consolidation of drives should be considered to achieve the recommended minimum spacing. These recommendations are not a substitute for engineering judgment. However, in the absence of unusual conditions, these recommendations should apply to all new or reconstructed public or private accesses to the street system.

Discussion and Consideration for Deviation

The circulation system in the City of Irvine is based upon a hierarchy of streets designed to optimize through traffic movement while providing access to abutting properties. The location, frequency and number of accesses taken from a highway or street have a direct effect on the capacity, speed, and overall operating characteristics of the highway.

Figure 10.1 taken from the ITE Transportation Planning Handbook[3] provides an illustration of the relationship between movement and access on the highway system. In general, the functions of movement and access tend to oppose each other, with the higher classification highways used primarily to maintain movement through the street system, while lower classification highways provide access to abutting land uses. The classification of streets is essentially a determination of the degree to which access functions are to be emphasized at the cost of traffic movement or discouraged to improve the movement function.

From the standpoint of delay to vehicles entering the traffic stream and the ability of the traffic stream to absorb vehicles exiting driveways, numerous driveways at close spacing result in delay and cause conflicts that increase the traffic hazard. On the other hand, a scarce number of access points to and from a site generate heavy congestion at the few access locations provided and depending on the location and type of access, may result in U-turns at adjacent signalized intersections, as vehicles maneuver from the highway into the site.

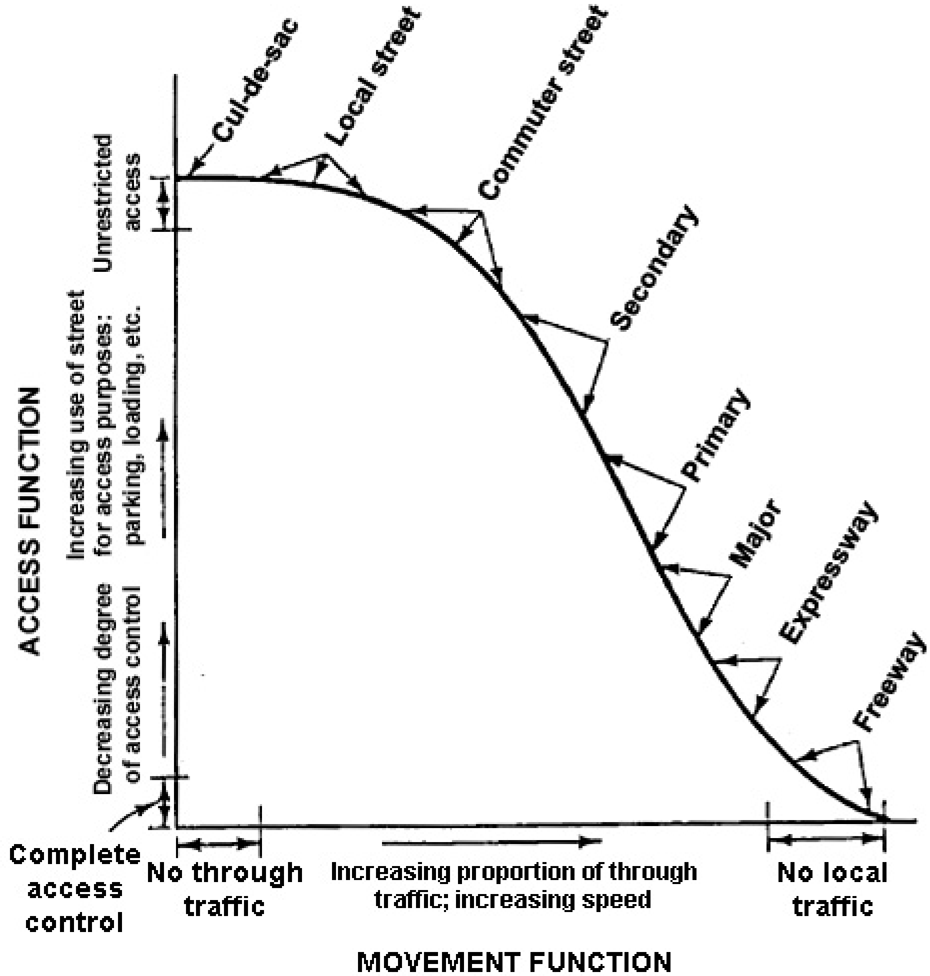
The following recommended procedures are based on research conducted by the Midwest Research Institute[4]. The procedures are designed to maintain control over driveway access to properties abutting highways with the objective of preserving highway capacity and safety.

- If a property width is less than the minimum recommended distance from a street corner or adjacent driveway and access to a side street is not available, the maximum distance between the street corner or adjacent drive should be provided. At a minimum, however, the driveway should be of sufficient distance from the property edge so as to maintain any required setbacks and/or to align with the center of any internal drive aisles that may be provided.
- Where lots are of insufficient size to allow accesses on opposite sides of the street to be aligned, the centerline of the driveways should be offset at the same recommended distances as recommended for adjacent driveways. Furthermore, greater distances may be required if there is a need for left-turn storage lanes (see Corner Clearance section).
- In certain situations, circulation impacts may be greater on the Local highway with the recommended minimum distances than with less than minimum distances, due to local conditions. In these cases, a site specific traffic analysis shall be performed to demonstrate the benefits and impacts of the proposed reduction. This analysis should include but not be limited to: 1) an analysis of the local highway intersection's level of service, and 2) detailed access analysis addressing the character of the highway, including existing and future traffic volumes, speeds, and striping.

Sources

[1] *ITE Guidelines for Urban Major Street Design, Table 12.2, 1989.*
[2] *ITE Guidelines for Driveway Design and Location, Table 2, 1989.*
[3] *ITE Transportation Planning Handbook, Chapter 11, 1992.*
[4] *Technical Guidelines for the Control of Direct Access to Arterial Highway, Volume 2, Midwest Research Institute, page 158 August 1975.*

Schematic Relationship Between Access and Movement Functions of Streets



Source: *Transportation Planning Handbook, ITE Chapter 11, pg. 391.* **Figure 10.1**

TDP-11 CORNER CLEARANCE

Recommendations

Corner clearance is the distance from a signalized intersection to the nearest access (public or private) upstream or downstream of the signalized intersection. Corner clearance requirements are measured as the distance from the curb face of the cross street to the near curb face of the private access drive.

Figure 11.1[1] illustrates the approaching and departing conditions for locating a private or public access. The minimum corner clearance on the approach of a signalized intersection should allow for the following:

1. Right-turn ingress and egress movements (Figure 11.1 – note 1) must not interfere with the right turns at the downstream signalized intersection, i.e. they should be a sufficient distance back from the intersection so as not to interfere with the maximum right-turn queue.
2. Corner clearance (Figure 11.1 -note 2) must be a sufficient distance from the approaching intersection so as to allow right-turn egress to enter the left-turn pocket lane at the approaching intersection.
3. If left-turn ingress and/or egress is to be provided, the minimum corner clearance shall be the minimum back-to-back left-turn pockets plus the bay taper. As Figure 11.1 - note 3 indicates, the minimum distance is the minimum length required to provide 150-foot back-to-back left-turn pockets and the 90-foot transition in between for a total of 390 feet measured from the back of the stop bar or crosswalk.

Corner clearance from a departing intersection should be sufficient to allow the driver to enter the street without interfering with a vehicle approaching from a departing intersection[2]. The minimum corner clearance at the departing intersection should allow for the following:

1. The corner clearance (Figure 11.1 – note 1) must be adequate to allow a vehicle approaching from a departing intersection to stop in the event a vehicle at the access pulls out into the street. (The distance by the highway type under “Distances Between Driveways and Intersections” is recommended.)
2. The corner clearance (Figure 11.1 – note 2) should be of sufficient length to allow vehicles to make a right turn into the access and not significantly impact the vehicles desiring to continue traveling through the intersection. (The distance by the highway type under “Distances between Driveways and Intersections” is recommended.)
3. If left-turn ingress and/or egress is to be provided, the minimum corner clearance shall be the sum of the departing intersection left-turn pocket length and the bay taper (Figure 11.1 – note 3).

Discussion and Consideration for Deviation

Developments that occur on the corner of a signalized intersection may be requested to provide an evaluation of the queuing at the signal in order for staff to approve the driveway location.

See “Discussion and Consideration for Deviation” under TDP-10 Distances between Driveways and Intersections for more information.

Sources

[1] *ITE Guidelines for Driveway Design and Location, 1989.*
[2] *ITE Guidelines for Urban Major Street Design, 1989.*

Schematic Diagram Showing Corner Clearance for Unsignalized Driveways

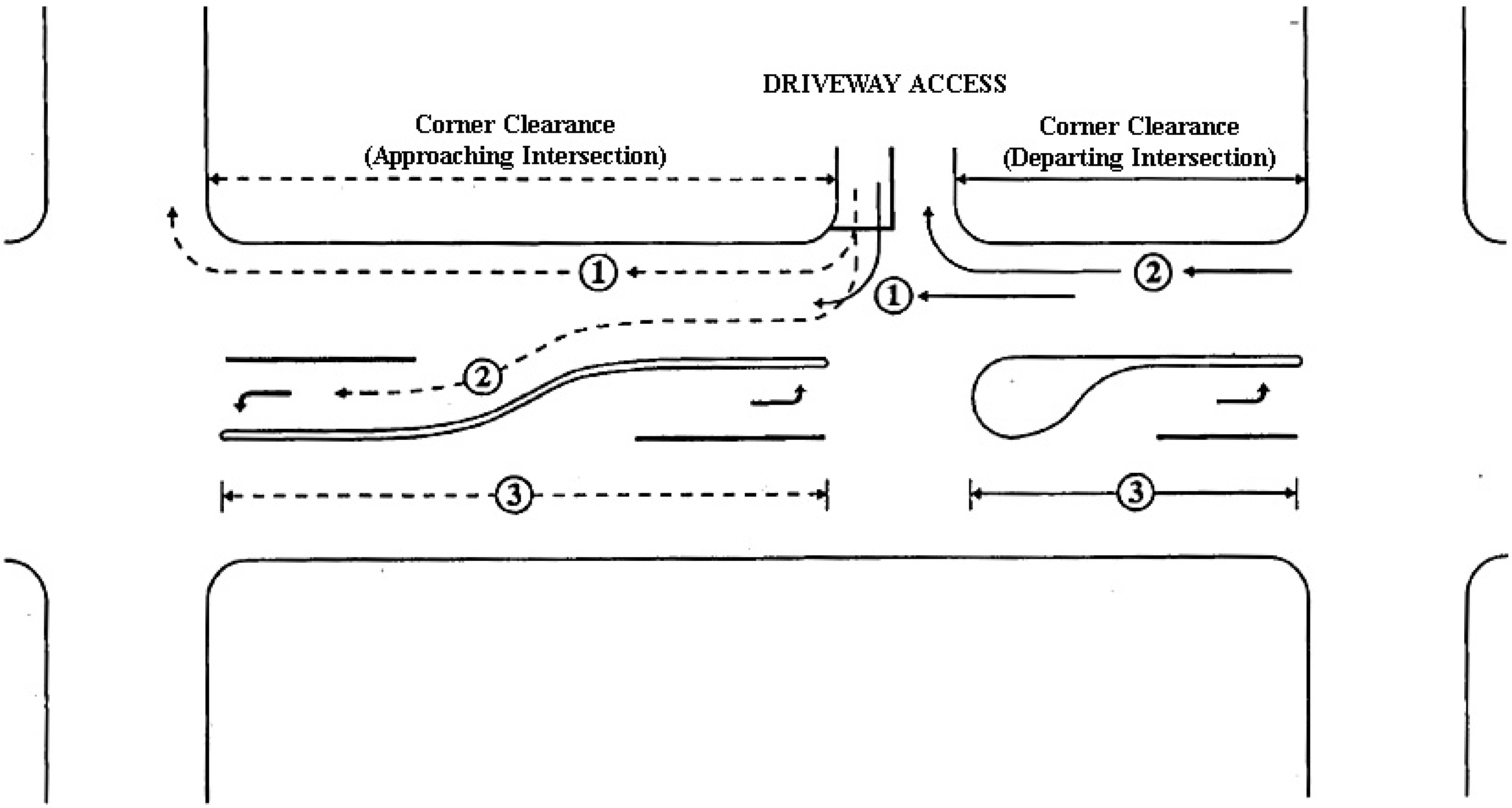


Figure 11.1

TDP-12 SIGNAL WARRANTS

Recommendations

To determine whether installation of a traffic signal is justified at a particular location, an engineering study of roadway traffic and other conditions including pedestrian volumes and intersection location should be performed as outlined in Chapter 4 of the Manual for Uniform Traffic Control Devices (MUTCD). A traffic signal should not be installed unless the engineering study indicates that overall safety and or operation of the intersection would be improved.

All new or modified traffic signals should be installed with the capacity to accommodate eight-phase operations. The City standard traffic signal controller shall be as required in the latest approved "Section 104 Traffic Signals" in the City's Standards and Design Manual. Signal poles capable of accommodating left-turn phasing may be required depending on actual or projected volumes. Traffic signal, controller and equipment design and construction shall be performed in accordance with Section 104 – Traffic Signals in the City of Irvine Standards and Design Manual1.

Both the MUTCD2 and the MUTCD California Supplement3 provide eight different signal warrants that define minimum conditions under which installing traffic control signals might be justified.

Warrant 1 – Eight Hour Vehicular Volume
Warrant 2 – Four Hour Vehicular Volume
Warrant 3 – Peak Hour
Warrant 4 – Pedestrian Volume
Warrant 5 – School Crossing
Warrant 6 – Coordinated Signal System
Warrant 7 – Crash Experience
Warrant 8 – Roadway Network

The Peak Hour signal warrant is generally intended for use at locations where traffic conditions are such that for a minimum of 1 hour of an average day the minor street suffers undue delay when entering of crossing the major street. Examples of such locations include office complexes, industrial complexes and commercial facilities. In the City of Irvine Warrant 3 should be the first Warrant to consider. Figure 12.1 shows graphs of the Warrant 3 major and minor street volume requirements for streets with speeds below 40 mph and above 40 mph.

Engineering judgment and rationale should be applied to a street approach with one lane plus a right-turn lane. In this case, the degree of conflict of minor-street right turn traffic on the major street should be considered. If the right-turn movement enters the major street with minimal conflict then the right turn traffic should not be included in the minor street volume. However, if there is conflict, engineering judgment should be used to determine how much of the right-turn traffic should be included in the minor street volume.

Discussion and Consideration for Deviation

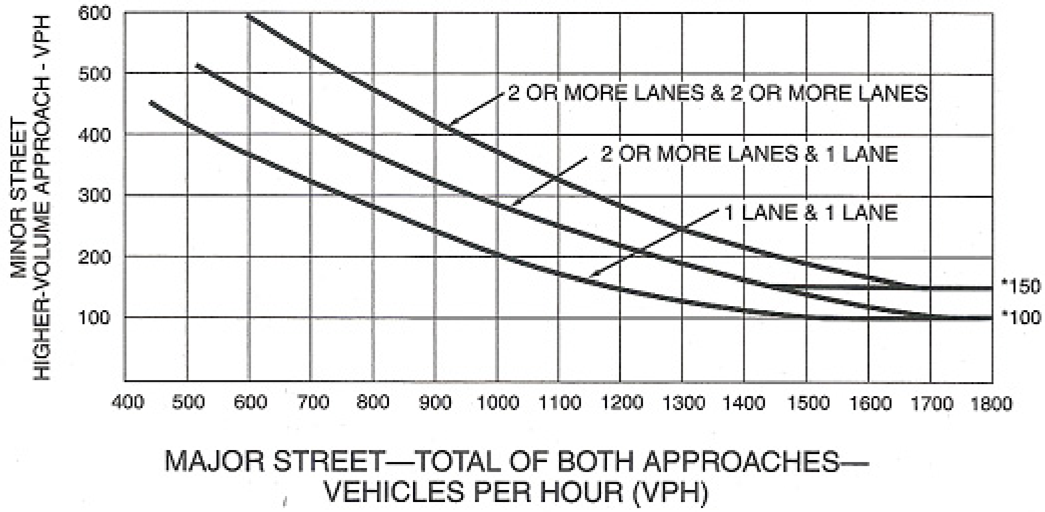
In the event that the relationship between the major and the minor street volumes do not justify the Peak Hour warrant, additional analysis or warrants should be considered.

At a future intersection location where only forecast volumes are available, Average Daily Traffic (ADT) volumes should be estimated at part of a traffic or access study. In these cases, it is advisable to use Table 4C-101 from the MUTCD California Supplement, May 2004 (see Figure 12.2) to determine whether a traffic signal is warranted.

Sources

[1] *City of Irvine Design Manual and Standard Plans – Section 104 Traffic Signals.*
[2] *FHA/ITE/AASHTO - Manual of Uniform Traffic Control Devices (MUTCD) Part 4C, 2003.*
[3] *Caltrans – MUTCD 2003, California Supplement , Part 4C, 2004.*

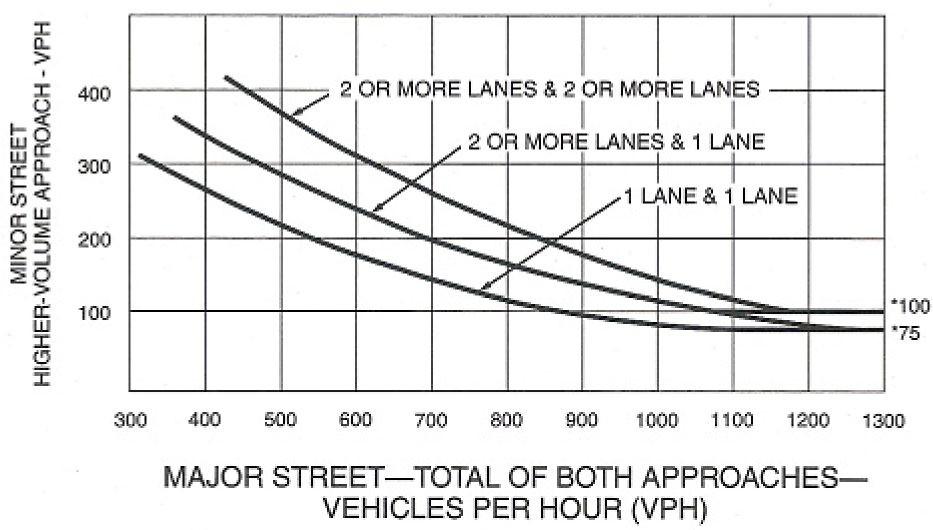
Figure 4C-3. Warrant 3, Peak Hour



*Note: 150 vph applies as the lower threshold volume for a minor-street approach with two or more lanes and 100 vph applies as the lower threshold volume for a minor-street approach with one lane.

Figure 4C-4. Warrant 3, Peak Hour (70% Factor)

(COMMUNITY LESS THAN 10,000 POPULATION OR ABOVE 70 km/h OR ABOVE 40 mph ON MAJOR STREET)



*Note: 100 vph applies as the lower threshold volume for a minor-street approach with two or more lanes and 75 vph applies as the lower threshold volume for a minor-street approach with one lane.

Source: *MUTCD* **Figure 12.1**

Signal Warrants

URBAN.................... RURAL......................

1A - Minimum Vehicular Traffic

Satisfied ________ Not Satisfied _______

Minimum Requirements
EADT

Number of lanes for moving traffic on each approach

Major Sreeet	Minor Street	Vehicles Per Day on Major Street (Total of Both Approaches) URBAN	Vehicles Per Day on Major Street (Total of Both Approaches) RURAL	Vehicles Per Day on Higher-Volume Minor Street Approach (One Direction Only) URBAN	Vehicles Per Day on Higher-Volume Minor Street Approach (One Direction Only) RURAL
1..............................	1..................................	8,000	5,600	2,400	1,680
2 or More..............	1..................................	9,600	6,720	2,400	1,680
2 or More..............	2 or More.................	9,600	6,720	3,200	2,240
1..............................	2 or More.................	8,000	5,600	3,200	2,240

1B - Interruption of Continuous Traffic

Satisfied ________ Not Satisfied _______

Number of lanes for moving traffic on each approach

Major Sreeet	Minor Street	Vehicles Per Day on Major Street (Total of Both Approaches) URBAN	Vehicles Per Day on Major Street (Total of Both Approaches) RURAL	Vehicles Per Day on Higher-Volume Minor Street Approach (One Direction Only) URBAN	Vehicles Per Day on Higher-Volume Minor Street Approach (One Direction Only) RURAL
1..............................	1..................................	12,000	8,400	1,200	850
2 or More..............	1..................................	14,400	10,080	1,200	850
2 or More..............	2 or More.................	14,400	10,080	1,600	1,120
1..............................	2 or More.................	12,000	9,400	1,600	1,120

1A&B - Combinations

Satisfied ________ Not Satisfied _______

No one warrant satisfied, but following warrants fulfilled 60% or more... ____ ____
1 2

	Major Street	Minor Street Approach
1A&B - Combinations	2 Warrants	2 Warrants

Source: *MUTCD 2003 California Supplement* **Figure 12.2**

TDP-13 LEFT-TURN SIGNAL PHASING

Recommendation

The Transportation Design Procedure that determines when left-turn phasing is required is based on Figure 13.1. When the intersection of the left-turn volume and the opposing volume is above the line, left-turn phasing is required. Generally, no left-turn signal phasing is recommended when the point is below the line.

Regardless of whether left-turn phasing is required, all new or modified traffic signals should be installed with the capacity to accommodate eight phase operations. The City standard traffic signal controller shall be as required in the latest approved "Section 104 Traffic Signals" in the City's Standards and Design Manual. Signal poles capable of accommodating left-turn phasing may be required depending on actual/projected volumes. Traffic signal, controller and equipment design and construction shall be performed in accordance with Section 104 – Traffic Signals in the City of Irvine Standards and Design Manual.[1]

Discussion and Consideration for Deviation

The provision of left-turn signal phasing at an intersection should be based on a balance between the need to provide safe, protected left-turns along a high volume highway and the desire to limit needless delays along the highway due to unnecessary left-turn phasing.

Protected left-turn signal phasing exists when a separate interval in the signal cycle is provided to accommodate left-turn movements without conflicting with the through traffic movement and left-turns are prohibited during the rest of the cycle. Although, the *Manual on Uniform Traffic Control Devices* (MUTCD)[2] provides no left-turn phasing warrants, the *Traffic Engineering Handbook*[3] offers suggested a methodology for separate left-turn phasing. The methodology recommends that when the product of the left-turning vehicles and opposing volumes during peak hours exceeds 100,000 on a four-lane street or 50,000 on a two lane street, and the left-turn volume is greater than two vehicles per cycle, separate left-turn phasing should be examined. Figure 13.1 provides a graphic representation of this procedure. Other considerations for installation of left-turn signal phasing include:

- An existing crash history(specifically crashes which are potentially correctable by adding left-turn phasing)
- Where the geometric design of the roadway is such that sight distance for left-turn traffic is insufficient for safe completion of a left turn across opposing traffic
- High speed of opposing through traffic
- A high volume of opposing through traffic
- Where left-turning vehicles must cross three or more lanes of opposing through traffic
- Where there are multiple left-turn lanes

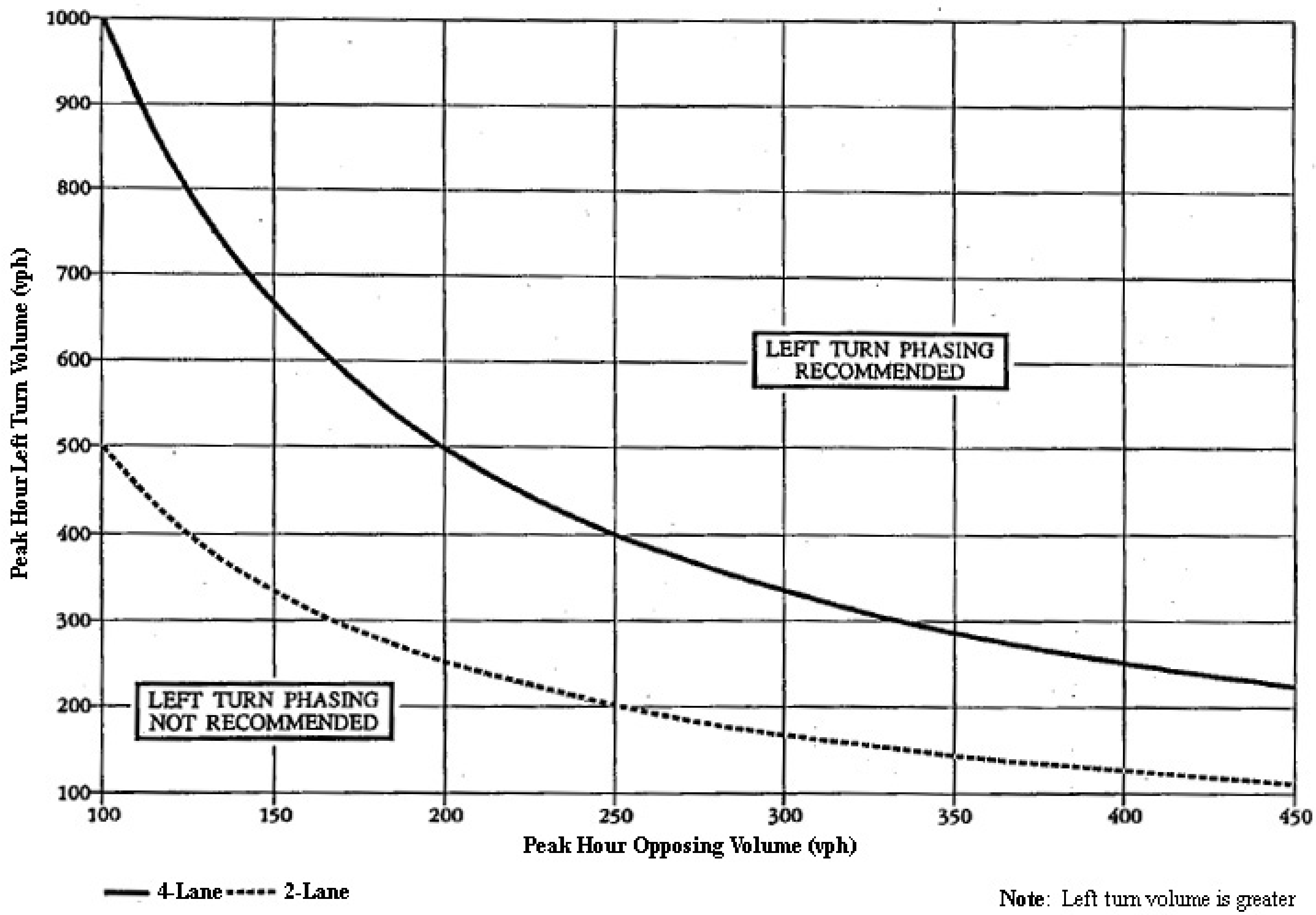
Sources

[1] *City of Irvine Design Manual and Standard Plans – Section 104 Traffic Signals.*

[2] *FHA/ITE/AASHTO – Manual of Uniform Traffic Control Devices, Chapter 4C, 2003.*

[3] *ITE Traffic Engineering Handbook, 4th Edition, p295.*

Left Turn Phasing Guidelines



Note: Left turn volume is greater than 2 vehicles per cycle.

Source: *Traffic Engineering Handbook, 4th Edition, ITE, Chapter 9, pg. 295.*

Figure 13.1

TDP-14 DRIVEWAY LENGTHS

Recommendation

Primary driveways should be of sufficient length to allow vehicles to enter the parking area without causing subsequent vehicles to back out onto City streets. The greater the peak hour traffic demand, the longer the driveway must be. The driveway length should be measured from the back of the sidewalk or the stop bar exiting the site to the near curb line of the first intersecting parking stall or traffic control measure (internal drive aisle or pedestrian crosswalk) located on site. The minimum driveway length for unsignalized intersections should be 25 feet, and should increase at a rate of one foot of storage per peak hour vehicle.[1] Driveway lengths for signalized intersections are based on the minimum right and left-turn pocket lengths presented under TDP-1. The driveway lengths are as follows:

Peak Hour Entering Directional Volumes Per Lane	**Unsignalized Minimum Driveway Length**	**Signalized Minimum Driveway Length**
0-25	25	75
26-50	50	100
51-75	75	125
76-100	100	150
101-125	125	175
126-150	150	200
151-175	175	225
176-200	200	250
> 200	25 additional feet of storage for each additional 25 peak hour vehicles	

Discussion and Consideration for Deviation

The length of the driveway is based on a number of parameters, including traffic control, volumes, and cycle length if signalized. The purpose of the minimum driveway length is to provide entering vehicles sufficient distance before entering a parking aisle to prevent traffic backing onto the highway and to provide exiting vehicles sufficient on-site queuing distance.

It should be noted that there are numerous other traffic and circulation issues that must be addressed when laying out a site plan. These issues would need to be addressed on a case-by-case basis.

For gated entries, a separate gate stacking analysis shall be prepared. Where gates are situated well within the site and guest or visitor parking is outside of the gates, the access shall be evaluated based on TDP-14.

Where parallel parking is desired along an entry driveway, the City Engineer may allow it if an additional width of 10-12 feet is provided for inbound vehicles to bypass any vehicles that are parallel parking and that the parking lane is an additional 8 feet wide. This, however, shall be evaluated on a case-by-case basis.

Source
[1] *ITE Guidelines for Urban Major Street Design, Figure 9.1-9.2, 1989.*

TDP-15 VEHICLE STACKING AND GATE STACKING ANALYSIS

Recommendations

To the extent possible, vehicle stacking and gate-stacking analysis should be based on a study of similar sites. Similar sites should be approved by the City as being “similar” prior to any analyis being performed. Should similar site studies not be available, the following recommendations are provided:

A - Fast Food Queuing Analysis

Sufficient reservoir space should be provided to eliminate conflicts with parking vehicles and to avoid interference with the flow of traffic on the adjacent roadway.

It is recommended that fast food restaurants that have service rates between 3 and 4 minutes per vehicle, as rated by Quick Service Restaurant (QSR) magazine or other service rate study, provide a drive through length to accommodate six vehicles in front of the order menu and three vehicles between the order menu and the pick up window.

For drive through restaurants of a type not previously evaluated, the methodology outlined in ITE Journal May 1995, Queuing Areas for Drive-thru Facilities, by the ITE Technical Council Committee 5D-10[1,] is recommended. The number of vehicles in the queue at both the order menu and at the pick-up window shall be counted during the peak period—generally the lunch hour or PM peak hour. Counts shall be recorded at one-minute intervals or whenever the queue changes, such that there are 60-100 data points.

A sample table is provided below. It includes columns for the frequency of occurrence of each number of vehicles in the queue; the cumulative frequency of occurrence; the probability of the queue length not exceeding that number of vehicles.

Data to be Tabulated for the Fast Food Queuing Analysis

	Queue #1 at Order Menu/Speaker Box			**Queue #2 at Pick-up Window**		
# of veh. in queue (N)	**# of occurrences**	**Cumulative Frequency**	**Probability of queue length not exceeding N**	**# of occurrences**	**Cumulative Frequency**	**Probability of queue length not exceeding N**
0						
1						
2						
3						
4						
Up to highest queue		Total	100%			
	Total			Total		

When all of the data is tabulated for both the queue #1 at the order menu and at the pick-up window, queue #2, the queues equivalent to approximately the 85th percentile should be chosen. The drive through lanes should be designed to accommodate these queues, considering approximately 20 feet per vehicle.

B - Car Wash Queuing Analysis

The analysis for a car wash is similar to the Drive-through restaurant analysis. The hours and days that the counts shall be taken at a car wash should include one weekday (preferably 4-6 PM) and one Saturday (11:00 AM-2:00 PM).

C - Drive-Through Bank Stacking Analysis

A drive through bank with tellers and/or ATM machines may need a stacking analysis if it is located on a pad that was approved for a fast food use or other retail use. A study of two similar sites shall be conducted during the peak hours of the bank or of the roadway, generally the noon or PM peak hour.

The study should contain the following information: fifteen minute traffic volumes, hours of the bank; hours of the drive through teller; hours of the ATM; discussion whether the lanes are for ATM or teller or both; length in feet of the individual lanes; length of the common lane prior to splitting into two lanes.

The length of the drive through lanes for banks with tellers or ATM's should accommodate the maximum number of vehicles observed in the queue at the similar sites. If the queue blocks parking stalls, but is contained within the parking lot, it is recommended that those stalls that would be blocked by the queue, be assigned to employees, so that they are not high-turnover rate stalls. Whenever possible, the design of the drive through should be done such that it is possible for customers to view the number of cars ahead of them in line, particularly when there are bank tellers on duty. In addition, if there are bank tellers, there should be a sign near the entrance stating the hours of the bank teller at the window.

D - Pharmacy Drive-Through Stacking Analysis

For pharmacies with one or two drive through lanes, a stacking analysis shall be performed at a minimum of two similar sites during the peak hours of the pharmacy (if known) or during the peak hours of the roadway (generally the PM peak hour). If customers are required to drop off prescriptions and drive away and return after a specified time, the study will be conducted differently than at a pharmacy where the customers are allowed to wait in the drive through lane after dropping off prescriptions, because the service times will be markedly different.

The study should contain the following information: hours of operation of the drive through lanes; hours of operation of the pharmacy inside the store; hours of operation of the store (if different hours for the pharmacy portion); description of the items permitted to be sold at the drive through window.

The length of the drive through lanes for pharmacies should accommodate the maximum number of vehicles observed in the queue at the similar sites. If the queue blocks parking stalls, but is contained within the parking lot, it is recommended that those stalls that would be blocked by the queue, be assigned to employees, so that they are not high-turnover rate stalls.

E - Gate Stacking at Residential Developments

The reference for this Transportation Review Procedure is County of Orange Standard Plan 1107[2], which states that there should be one foot of stacking for each dwelling unit, in front of the gates. This standard was originally applied for gated entries that were staffed by a guard. With the technology commonly provided, residents are equipped with remote gate operating devices, such that they do not have to stop to speak to a guard, swipe a card, or punch in a code. Only guests would have to stop at the call box or guard shack and phone the resident or have the guard phone the resident to obtain permission to enter.

The distance between the back of the crosswalk and the call box or gate shall be equal to, or greater than, the number of dwelling units served. When two or more gated accesses are provided, the number of residential dwelling units served by each access shall be estimated. The length of the visitor lane should provide approximately between 10 and 20% of the stacking required, but in no cases shall the visitor lane be less than two-car lengths long.

A turn around shall be provided for vehicles that are turned away at the gate. The turn around should have a minimum radius of 38 feet to accommodate trucks and passenger vehicles. Where it is not possible, a minimum radius of 30 feet may be allowed, on a case-by-case basis, by the City Engineer. Exceptions to this rule of providing a turn-around are as follows:

- When all visitor parking is provided outside of the gates and vacant striped-out stalls are provided for turning around at the dead end.
- When all visitor parking is provided at a completely separate location.
- When the parking structure is for residents only, and the gate is situated very close to the street with signage “Residents Only” and the signage depicts where visitors should enter and if a call box is available for a visitor to use to contact the manager and the manager could open the gate to allow the visitor inside the site to turn around.

F - Gate Stacking at Office and Retail Developments

The City currently uses the Crommelin methodology for assessing gate stacking at office developments. This methodology dates from the 1972 Paper, Entrance-Exit Design and

Control for Major Parking Facilities. The City is not aware of a more recent approach. When a more updated method becomes available the City will re-consider the use of the Crommelin methodology

Sources

[1] *ITE Journal, May 1995, pages 38-42.*
[2] *Orange County Standard Plan No. 1107.*
[3] *Robert Crommelin and Associates, 1972, Entrance-Exit Design and Control for Major Parking Facilities*

TDP-16 VERTICAL ALIGNMENT

Steep grades affect truck speeds, overall capacity, and cause operational problems at intersections. For these reasons it is desirable to provide the flattest grades practicable. However, minimum grades on public and private streets shall be 1.0% at centerline unless otherwise approved by the City Engineer.[1]

The maximum grade break shall be 0.5% and in no case shall a grade break exceed 0.5% in a 25-foot length.

Grades exceeding 7% are not desirable. Recommended maximum grades by terrain and road type are[2]:

Type of Terrain	Local	Commuter	Secondary	Primary	Major
Level	8%	7%	6%	5%	5%
Rolling	9%	8%	7%	6%	6%
Mountain	11%	10%	9%	9%	8%

Ramp grades should not exceed 8% however on descending on-ramps and ascending off-ramps 1% steeper is allowed.

Properly designed vertical curves should provide adequate sight distance, safety, comfortable driving, good drainage and pleasant appearance. A proper balance between curvature and grades should be sought. Where possible, vertical curves should be superimposed on horizontal curves. This reduces the number of sight restrictions on the project, makes changes in profile less apparent. At intersections, horizontal and vertical curvature should be as flat as physical conditions permit. Vertical curves shall have the following minimum lengths

Major/Primary/Secondary Highways	200 feet
Commuter Streets	100 feet
Local Collectors	75 feet
Local Streets	50 feet

Stopping and passing sight distances shall be in conformance with the Caltrans Highway design manual[3]

Sight distance on the highway through an under crossing should be at least as long as the minimum stopping sight distance. Lighted under crossings can use Comfort Design Criteria for Sag Vertical Curves with written approval from the City Engineer.

The maximum ramp grades within a parking structure shall be 6% where 90 degree parking is provided and 12% where no parking is provided.

Sources

[1] *City of Irvine Streets/Landscape/ROW Standards & Design Manual.*
[2] *AASHTO, Green Book, Chapter 7, 2004.*
[3] *Caltrans Highway Design Manual, 2004.*

TDP-17 ROUNDABOUTS

The term “modern” roundabout is used in the United States to differentiate modern roundabouts from nonconforming traffic circles or rotaries that have been in use for many years.[1] A modern roundabout operates as a one-way circulatory system around a central island where entry is controlled by pavement markings and priority must be given to traffic approaching from the left inside the roundabout. Entrance roadways that intersect the roundabout along a tangent to the circulatory roadway are not permitted and no traffic is permitted to follow a straight path through the roundabout. Figure 17.1 shows the typical geometric elements of a single-lane modern roundabout

The operating efficiency of a roundabout depends on the ability of drivers to respond to safe opportunities to join the stream of circulating vehicles already in the roundabout. Roundabouts are especially successful when there is a dominant direction of travel but the entering minor street traffic does not require the major street traffic to stop as at a signal.[2]

While roundabouts can perform satisfactorily at intersections with many approach arms if they are well-designed, four-arm roundabouts should generally not be exceeded and the use of roundabouts that exceed four legs should require special study. Although initial construction costs may be greater than for other types of intersection because of the larger land area required, maintenance and vehicle operating costs are likely to be less, as roundabouts permit the free flow of traffic when demand is light and are self regulating. The ability of roundabouts to cope with U-turns can be particularly useful where one or more of the approaches is divided or where U-turns would otherwise be awkward or disruptive.

During uncongested off-peak periods, roundabouts will generally result in less delay than with signal control. However, they are not generally compatible with Urban Traffic Control (UTC) systems, as they cannot respond to UTC center commands (unlike signalized intersections where the phasing can be centrally controlled). They may also be unsatisfactory where there are significant numbers of cyclists or pedestrians, and special provisions may be required (for example grade or mode separation) which can be expensive.

To achieve the desired levels of safety and capacity, vehicle approach speeds should be regulated to appropriate levels. This can be achieved by deflecting the vehicle entry path at the junction approach by using suitably positioned traffic islands on the approaches. Entry-path-curvature radius should not exceed 330 feet to reduce entry speed into the roundabout.

The relationship between the central island diameter and the inscribed circle diameter (ICD) is the most important consideration for the passage of large vehicles at small roundabouts. For example a 60-foot central island diameter requires a minimum ICD of 120 feet to prevent overrunning. For ‘mini’ type roundabouts ICD should not be greater than 100 feet.

Central Island Diameter (ft)[3]	Minimum ICD (ft)
13	92
20	94
26	98
33	101
39	105
46	109
52	113
59	118

Site Category (ft)[4]	Typical Design Vehicle	ICD Range (ft)*
Mini-Roundabout	Single-Unit Truck	45-80 ft
Urban Compact	Single-Unit Truck/Bus	80-100 ft
Urban Single Lane	WB-50	100-130 ft
Urban Double Lane	WB-50	150-180 ft
Rural Single Lane	WB-67	115-130 ft
Rural Double Lane	WB-67	180-200 ft

* Assumes 90-degree angles between entries and no more than four legs.

It is essential that drivers have sufficient visibility of the roundabout they are approaching to be able to stop safely if it is necessary to do so. Drivers should have adequate forward and circulatory visibility and visibility to the left. Roundabouts should preferably be situated on level ground or in sags rather than at or near the crests of hills because it is difficult for drivers to appreciate the layout when approaching on an up gradient.[3] Advance signage should be provided so that motorists understand how to maneuver through the roundabout.

Segregated lanes for right-turning traffic may be adopted where around 50% or more vehicles entering the roundabout leave at the first exit and where sufficient space is available.

Roundabouts should not be used in areas with high pedestrian flows and as a general rule pedestrians should be discouraged from crossing the highway on roundabouts.[3] Guardrails or featured landscaping[4] may be required to channel pedestrian movements away from the roundabout to safer crossing points, but these should not be so far away that the detour discourages their use. Provision should be made for pedestrians to cross the entry and exit arms of a roundabout through the gap in each deflection island. Lowered curbs should be provided and the crossing point should be at least 40 feet from the yield line to minimize the conflict between pedestrians. On wide multilane highways, it may be necessary to provide supplementary refuges between traffic lanes on the flared approaches. At sites with high vehicle and pedestrian flows especially on major streets with high speeds at off-peak times, subways or footbridges may be preferred.

Cyclists are particularly vulnerable at larger roundabouts. Consideration should be given to providing either a separate distinct cycle route away from other traffic or a sign posted alternative route away from the roundabout.

Analysis of roundabouts should be performed using RODEL (or similar) software. RODEL provides proper geometry based on traffic and site conditions. It is based on extensive empirical data collected in Great Britain on hundreds of actual roundabouts and may require additional calibration to adjust for drivers in the USA.

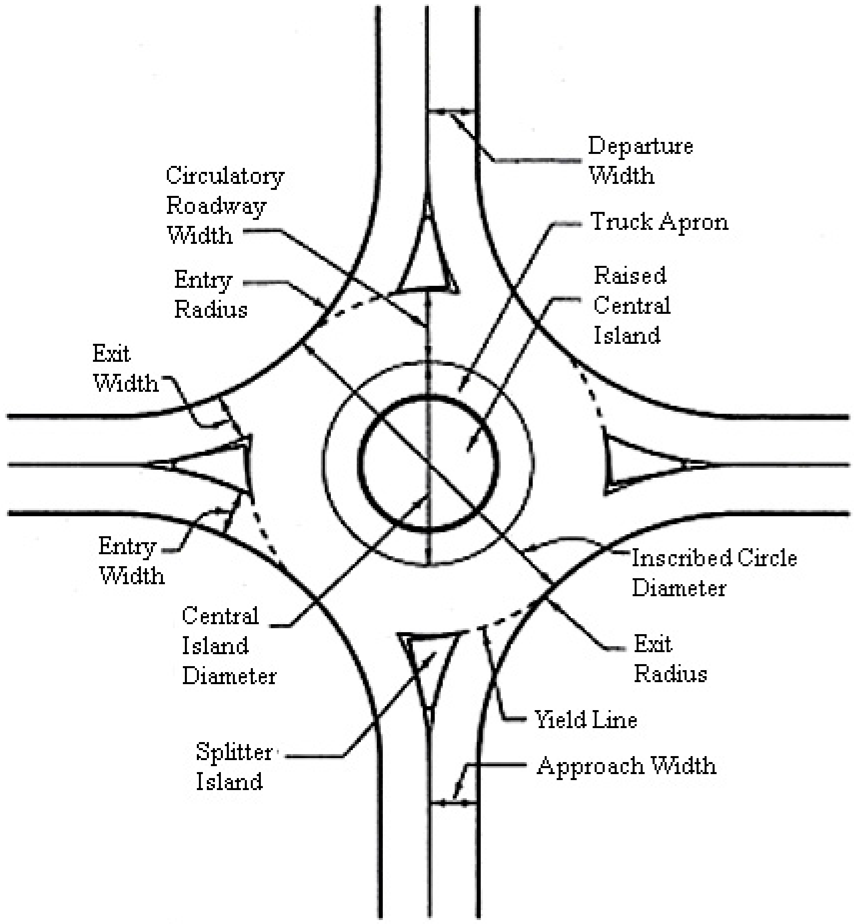
Sources

[1] *AASHTO Green Book, Geometric Design of Highways and Streets, Chapter 9, 2004.*

[2] *UK Department of Transport, Roads and Traffic in Urban Areas, Chapter 40 , 1987.*

[3] *UK Department of Transport, Design Manual for Roads and Bridges, Volume 6, Section 2, Part 3, 1993).*

[4] *FHWA, Roundabouts an Informational Guide, 2000.*



Geometric Elements of a Single-Lane Modern Roundable

Source: *AASHTO* **Figure 17.1**