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16. Abstract

Intersections are an important part of a highway facility because the efficiency, safety, speed, cost of operation, and capacity of the facility depend on their design to a great extent. Each intersection involves through- or cross-traffic movements on one or more of the highways and may involve turning movements between these highways. Such movements may be facilitated by various geometric designs and traffic controls, depending on the type of intersection. The main objective of intersection design is to facilitate the convenience, comfort, and safety of people traversing the intersection while enhancing the efficient movement of motor vehicles, buses, trucks, bicycles, and pedestrians. In order to design intersections that are both functional and effective, designers need current information regarding intersection design that is easily accessible and in a user-friendly format. The prime objective of the Texas Department of Transportation Project 0-4365 is to produce this reference document, the *Urban Intersection Design Guide*, to provide this information.

This document is presented in two volumes:

- Volume 1 – Guidelines and
- Volume 2 – Applications.

This project is designed to provide TxDOT and other interested parties with useful and practical information on operations and design for intersections.

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URBAN INTERSECTION DESIGN GUIDE: VOLUME 2 – APPLICATIONS

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DISCLAIMER

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Overview

The following application discusses ultimate design considerations. The *Urban Intersection Design Guide*, Chapter 1, Section 1 <link> presents additional discussion on intersection planning and development.

Background

A city of approximately 100,000 is experiencing rapid growth to the south of the city. A two-lane state highway (with shoulders) (referred to as McCullum Road) currently exists that extends from the city's central business district (CBD) (where it is wider) southward for many miles beyond the city's extraterritorial jurisdiction. The state has planned to widen this arterial south of the city in the future and expects to let the widening project in about 10 years. The new cross section of the arterial will be consistent with the city's typical cross section for its major arterial streets, which consists of two 28-ft-wide [8.5 m] roadway sections (measured face-to-face) with an 18-ft-wide [5.5 m] median.

A major developer has decided to plan and construct a large residential/golf course development, called Twin Oaks Estates, south of town and adjacent to the east side of McCullum Road where the arterial currently has a two-lane cross section. Figure 1-1 illustrates the current cross section of McCullum Road, which consists of two 12-ft-wide [3.7 m] travel lanes and two 10-ft-wide [3.0 m] shoulders. The developer desires to work with the city and the state to plan the entrance to the proposed subdivision so that the ultimate McCullum Road cross section can be constructed without affecting the subdivision's entrance. In addition, the developer plans to open the subdivision to development within 2 years, long before the McCullum Road widening project will be completed.

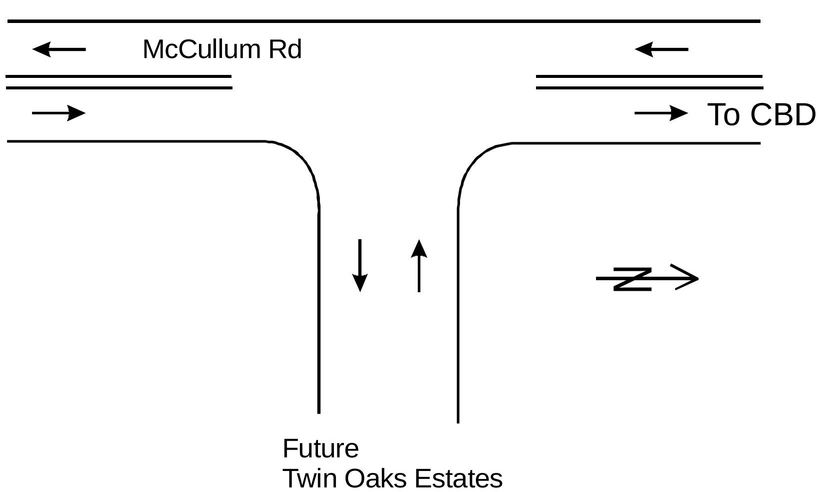

Figure 1-1. Location of McCullum Road and Twin Oaks Intersection.

Issues Considered

Although the final construction plans for the McCullum Road widening project have not been completed, sufficient planning had been done to provide a reasonable estimate of the elevation of McCullum Road where Twin Oaks Boulevard will intersect the arterial. The intersection would be located where sight distance will not be an issue so the location of the intersection would be approved by the state. Also, a preliminary drainage design for McCullum Road had been conducted so the developer's engineer would be able to plan the entrance road in a manner consistent with the proposed drainage plan and optimum placement of curb ramps. The developer also requested the state to provide a median opening on McCullum Road to access Twin Oaks Boulevard, and a separated left-turn lane on southbound McCullum Road to serve the entrance. The state was able to grant the request because the location of the median opening would be relatively consistent with the state's planned spacings of median openings.

The subdivision entrance road was to be designed with the assumption that the planned ultimate cross section of McCullum Road would be constructed, so the intent was to place the subdivision entrance signs, landscaping, and lighting at a location where they would remain permanent. Hence, the subdivision entrance would be placed some distance east of the existing location of McCullum Road, and a temporary extension of Twin Oaks Boulevard from the subdivision entrance to McCullum Road had to be designed and constructed.

Because of the isolated location of the intersection, the numbers of pedestrians and bicyclists expected in the interim were almost non-existent. Hence, the designers did not consider temporary bicycle facilities necessary or cost-effective. Temporary pedestrian facilities to connect the subdivision entrance to McCullum Road were considered appropriate. However, permanent bicycle and pedestrian facilities were considered and planned for the ultimate design of both Twin Oaks Boulevard and McCullum Road.

Lighting of the intersection was considered important because of the intersection's isolated location along a high-speed, rural highway. The city anticipated that the intersection would be signalized at some point in time because of the number of vehicles that would be expected to be generated by the large development and the expectations of high volumes on McCullum Road that would exist in the future. Signals may be warranted at the interim intersection (and considered necessary for safety and operational considerations) before McCullum Road is widened. Utilities need to be located away from pedestrian routes, curb ramps, and landings.

Design Selected

The design selected for the intersection of McCullum Road and Twin Oaks Boulevard is shown in Figure 1-2. The state's plan to widen McCullum Road included using the existing section as the location of the southbound travel lanes and widening the highway on the east side. Hence, the ultimate entrance to Twin Oaks Estates would be positioned about 50 ft [15 m] east of McCullum Road. Twin Oaks Boulevard was constructed to its ultimate cross section (with portland cement concrete) to the edge of the entrance where the curb return of the future intersection would begin. Bicycle lanes on Twin Oaks Boulevard ended at the same location. Temporary sidewalks were extended from the permanent sidewalks at the subdivision entrance to McCullum Road.

Twin Oaks Boulevard was extended westward from the end of the concrete surface to McCullum Road with a compacted base and asphaltic concrete surface consistent with city specifications. Intersection returns were constructed with 25-ft [7.6 m] radii, because the existing shoulders and a relatively flat area adjacent to the shoulders provided additional space for turning movements. A culvert had to be constructed beneath Twin Oaks Boulevard to accommodate roadside drainage.

In anticipation of a future traffic signal installation, a 4-inch [10.2 cm] conduit was placed below Twin Oaks Boulevard extending from the north to the south sides of the roadway. Ground boxes also were installed on both sides of Twin Oaks Boulevard at the terminals of the conduit. Anticipating that temporary traffic signals would be installed at the intersection before McCullum Road is widened, utility poles were installed on both the northeast and southeast corners of the intersection outside of the clear zone for McCullum Road. Luminaire arms and luminaires were installed on both utility poles to provide nighttime illumination.

A wide pavement area existed between the subdivision entrance and the existing McCullum Road. In order to provide delineation of this area and help to keep motorists from traveling on the wrong side of Twin Oaks Boulevard, large buttons and reflectorized pavement markers were installed west of the subdivision entrance essentially to extend the median to the intersection.

Because traffic on McCullum Road currently travels at high speed, the state decided to restripe the McCullum Road approaches to Twin Oaks Boulevard so that a separated left-turn lane could be provided. The restriping required narrowing the shoulders from 10 to 4 ft [3.0 to 1.2 m] in width; however, experience with similar rural intersections in this section of the state revealed that rear-end accidents at intersections on high-speed rural highways where left turns were frequent were reduced when separated left-turn lanes were provided.

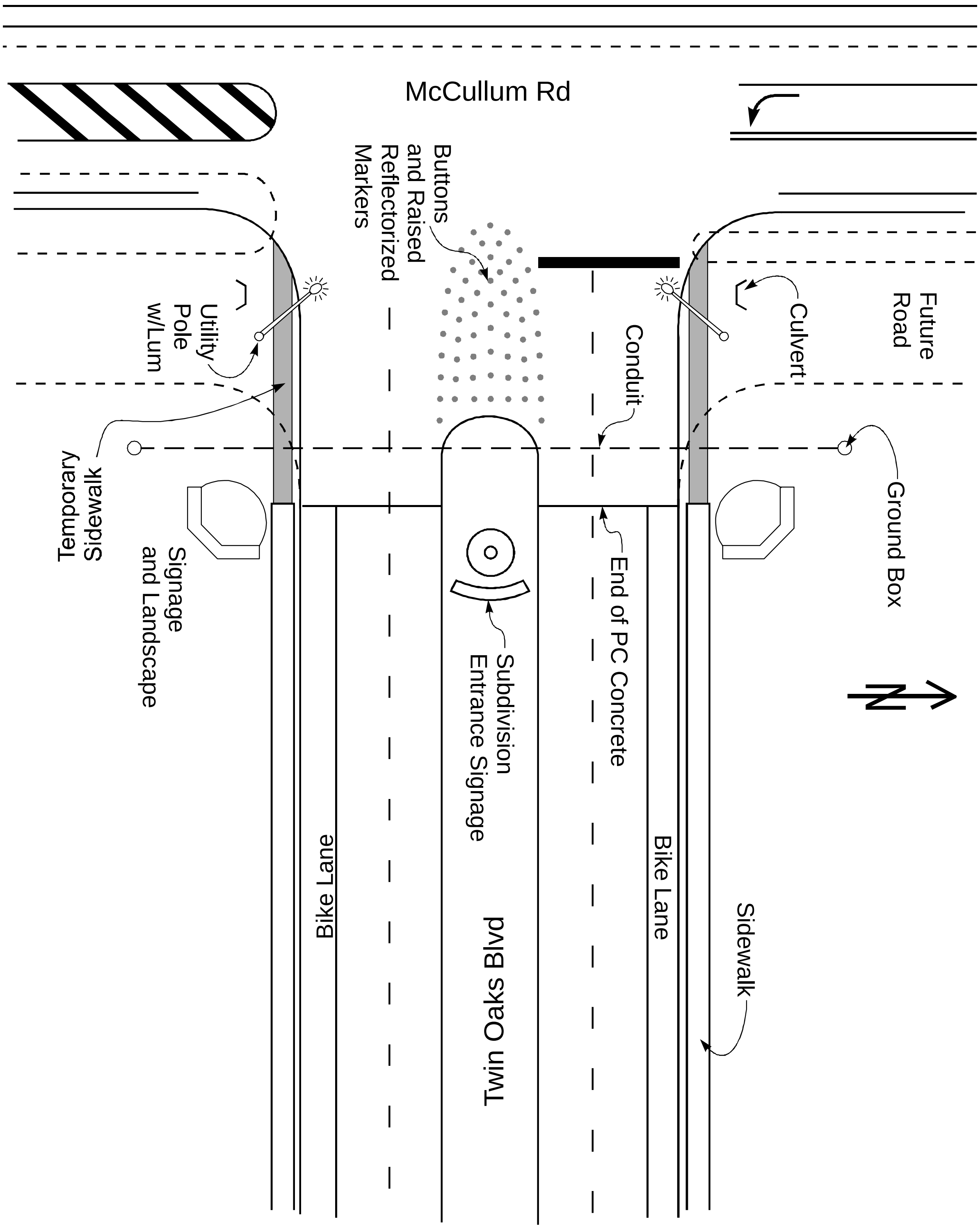

Figure 1-2. McCullum Road and Twin Oaks Intersection.

Application 2
Roundabouts

Overview

Interest in roundabouts as a form of traffic control in North America has been growing. (Discussion on other intersection types are included in the *Urban Intersection Design Guide,* Chapter 1, Section 2 <link>.) Roundabouts guide traffic flow with a raised island constructed in the center of an intersection to create a one-way circular flow of traffic. Figure 1-3 shows the basic geometric elements of a roundabout. They have been used to lower travel speeds, to reduce crash frequency by reducing the number of conflict points, and to provide an alternative to traffic signal installation. Current research is investigating how best to accommodate pedestrians at this type of intersection. The decision to use a roundabout should be based on an engineering analysis that considers the needs of all modes of travel. Discussions with the TxDOT Design Division should occur early in the decision process when considering a roundabout in a project.

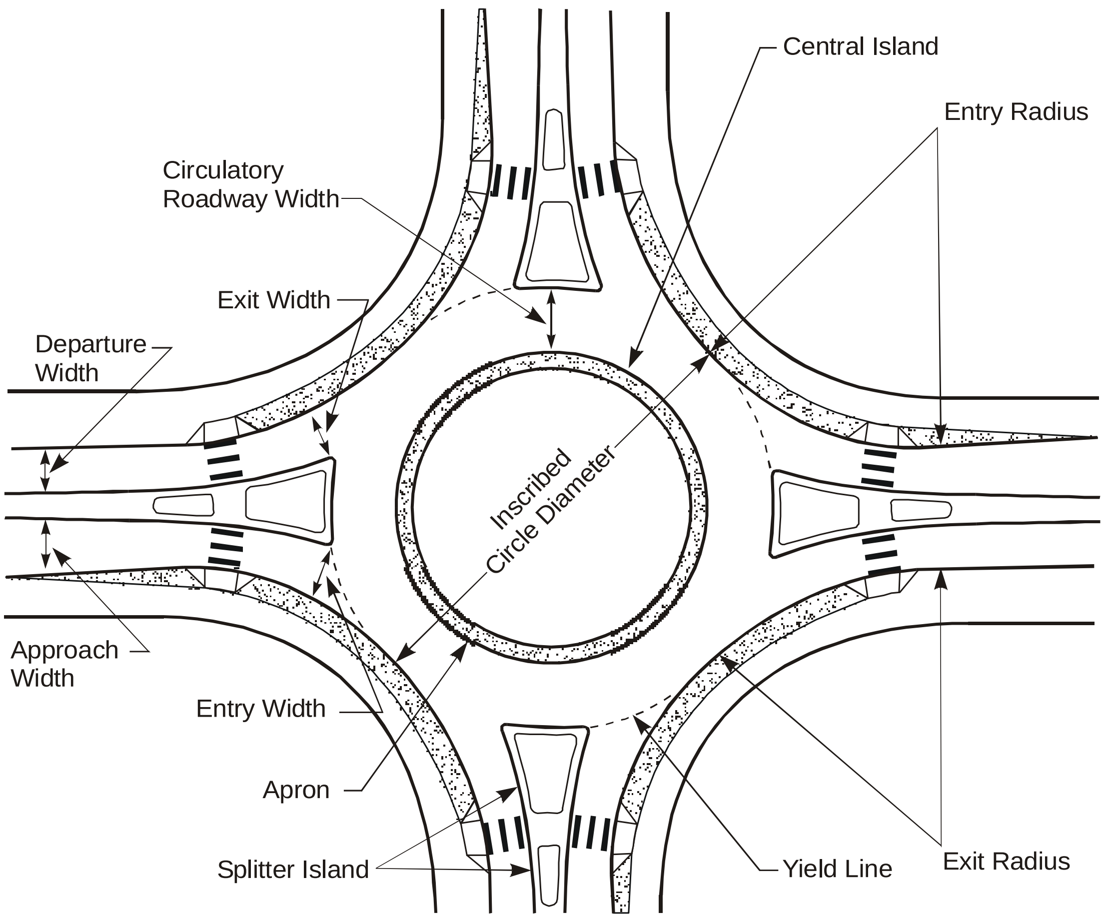

Figure 1-3. Example of Roundabout.[1]

[1] Roundabouts: An Informational Guide. Federal Highway Administration, McLean, Virg., January 1998.

Characteristics

Roundabouts perform best at intersections with similar traffic volumes on each approach leg and at intersections with heavy left-turning volumes. Roundabouts reduce the severity and frequency of intersection crashes by the nature of their design. Roundabouts resolve vehicle conflicts by means of priority control; the key operational feature of roundabouts is that entering vehicles yield to the circulating traffic. Traffic interactions are based on gap acceptance; entering traffic must wait for a gap in the traffic stream to enter.

Modern roundabouts range in size from mini-roundabouts with inscribed circle diameters as small as 50 ft [15.2 m], to compact roundabouts with inscribed circle diameters between 100 and 115 ft [30.5 and 35.1 m], to large roundabouts, often with multilane circulating roadways and more than four entries up to 500 ft [152.4 m] in diameter. The greater speeds permitted by larger roundabouts, with inscribed circle diameters greater than 250 ft [76.2 m], may reduce their benefits to some degree.[2]

Roundabouts eliminate left turns at intersections, which reduces the opportunity for crashes. Roundabouts contain only four merging conflict points, compared with 24 merging/crossing conflict points at intersections controlled by STOP signs or traffic signals (see Figure 1-4). The driver needs to decide when to enter the circulating stream, when to leave the circulating stream, and how fast to travel while circulating so that other drivers may enter the circulating stream without causing a conflict or crash.

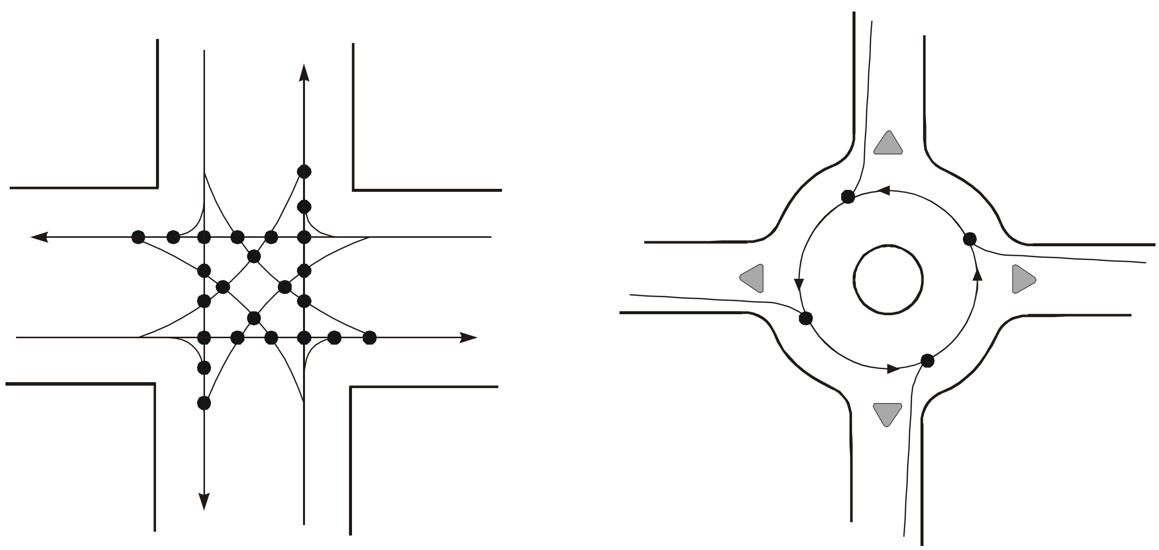

Figure 1-4. Comparison of Conflict Points at a Traditional Four-Leg Intersection and a Roundabout.[3]

[2] American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets.* AASHTO, Washington, D.C., 2001.

[3] Parham, A., and K. Fitzpatrick. "Handbook of Speed Management Techniques." Report FHWA/TX-99/1770-2. Texas Department of Transportation, Austin, Tex., September 1998.

Figure 1-5 shows examples of roundabouts. Additional photographs are available at the Center for Transportation Research and Training Web site.[4] Figure 1-6 illustrates a roundabout warning sign approaching a roundabout. The *TMUTCD*[5] states that the Circular Intersection (W2-6) sign accompanied by an educational word message plaque may be installed in advance of a circular intersection.

Roundabouts at interchange ramp termini may result in fewer delays and crashes and may be less costly when compared to other conventional interchange designs. A modern roundabout interchange is a freeway-to-street interchange or a street-to-street interchange that contains at least one roundabout. Unlike interchanges regulated by traffic signals, modern roundabout interchanges do not require long storage and turning lanes over or under a bridge, which is an expensive element of the interchange.

[4] Center for Transportation Research and Training. Kansas State University. February 6, 2003. http://www.ksu.edu/roundabouts/home.htm. Accessed February 2004.

[5] *Texas Manual on Uniform Traffic Control Devices for Streets and Highways.* Texas Department of Transportation. 2003. http://www.dot.state.tx.us/TRF/mutcd.htm. Accessed January 2004.

(A)

(B)

Figure 1-5. Examples of Roundabouts.[4]

Figure 1-6. Roundabout Advance Warning Sign Example.

Comparison with Traffic Circles

Roundabouts are similar to traffic circles, but they have design and operational characteristics that result in better performance. In general, traffic circles have smaller diameters than roundabouts. The typical residential traffic circle is approximately 20 ft [6.1 m] in diameter, with roadway approach widths of 30 ft [9.1 m] or more.[6] The key operational feature of roundabouts is that traffic must yield at entry to the traffic that is already within roundabouts. Roundabouts and traffic circles can be compared as follows:[7]

- Vehicles entering a roundabout on all approaches are required to yield to vehicles within the circulating roadway. Traffic circles sometimes employ stop or signal control to give priority to entering vehicles.
- The circulating vehicles are not subjected to any other right-of-way conflicts, and weaving is kept to a minimum. This provides the means by which the priority is distributed and alternated among vehicles. A vehicle entering as a subordinate vehicle immediately becomes a priority vehicle until it exits the roundabout. Some traffic circles impose control measures within the circulating roadway or are designed with weaving areas to resolve conflicts between movements.
- The speed at which a vehicle is able to negotiate the circulating roadway is controlled by the location of the central island with respect to the alignment of the right entry curb. This feature is responsible for the improved safety record of roundabouts. Some large

[6] *Neighborhood Traffic Control*. North Central Section, Institute of Transportation Engineers. December 1994.

[7] Courage, K., and L.T. Hoang. "Design and Evaluation of Modern Roundabouts in Florida." Resource Papers for the 1997 ITE International Conference, March 23-26, 1997.

traffic circles provide straight paths for major movements or are designed for higher speeds within the circulating roadway. Some small traffic circles do not achieve adequate deflection for speed control because of the small central island diameter.

- No parking is allowed on the circulating roadway of a roundabout.
- No pedestrian activities take place on the central island. Pedestrians are not expected to cross the circulating roadway. Some larger traffic circles provide for pedestrian crossing to, and activities on, the central island.
- Roundabouts are designed to properly accommodate specified design vehicles. Some smaller traffic circles are unable to accommodate large vehicles, usually because of right-of-way restraints.
- Roundabouts have raised splitter islands on all approaches. Splitter islands are an essential safety feature, required to separate traffic moving in opposite directions and to provide refuge for pedestrians. They are also an integral part of the deflection scheme.
- When pedestrian crossings are provided across the approach roads, they are placed approximately one car length back of the entry point. Some traffic circles accommodate pedestrians in other places, such as the yield point.
- The entry deflection is the result of physical features of a roundabout. Some traffic circles rely on pavement markings to promote deflection.

Roundabouts and Pedestrians

The *Green Book*[2] states that pedestrian crossing locations at roundabouts should achieve a balance among pedestrian convenience, pedestrian safety, and roundabout operations. The further a pedestrian crossing is from the roundabout, the more likely it is that pedestrians will choose a shorter route that may present unintended conflicts. Both crossing location and crossing distance are important considerations. Crossing distance should be minimized to reduce exposure to pedestrian-vehicle conflicts. Location of pedestrian crosswalks at the yield line is discouraged, as drivers may be distracted from pedestrian movements by watching for appropriate gaps in the traffic stream to merge into the circulating roadway. Crosswalks should be located to take advantage of the splitter island. The pedestrian refuge in the island should be level with the street grade to avoid the use of ramps at the refuge. Crossings should also be located at a distance from the yield line that is approximately an even increment of a vehicle length to reduce the likelihood that vehicles will be queued across the crosswalk.

Roundabouts are difficult for persons with visual disabilities to cross as a pedestrian. There are two main problems — determining where to cross and determining when it is safe to cross. Determining where to cross is made difficult by the very nature of a roundabout its circular geometry. Determining when to cross near a roundabout is made difficult because the information available at traditional intersections is not available. Specifically, there is no surge of parallel traffic movement to communicate to the visually impaired pedestrian that a gap in cross traffic is available. Therefore, to comply with the Americans With Disabilities Act (ADA), additional information needs to be provided to communicate to the pedestrian with a visual disability where and when the crossing should be made. At this time, specific requirements for how to provide this communication have not been developed, but this does not alleviate the designer's responsibility to do so.

The U.S. Access Board[8] has developed recommendations for accessible design of roundabout crossings. Some of the more general items are similar to those noted by the *Green Book,*[2] such as a street-grade pedestrian refuge. Other items of interest include aligning the crosswalk with the ramp from the sidewalk, including landscaping or small barriers to prevent pedestrians from crossing a roundabout at a non-crosswalk location, and including pedestrian signals at the crossings. Another idea is to use raised crossings at roundabouts to help ensure that traffic slows, particularly on the roundabout exit. NCHRP Project 03-78 entitled *Crossing Treatments at Roundabouts and Channelized Turn Lanes for Pedestrians with Vision Disabilities* was funded to develop some alternative treatments that would make these crossings accessible without hindering the operation of all roundabouts with a signal requirement.

Summary

Advantages of roundabouts include that they:

- can noticeably reduce vehicle speeds,
- reduce potential for vehicular crashes,
- can increase capacity,
- reduce the number of conflict points at an intersection,
- provide an orderly and continuous flow of traffic,
- provide landscaping opportunities, and
- are effective at multileg intersections.

Disadvantages of roundabouts include that they:

- may be restrictive for some larger service and emergency vehicles unless central island is mountable,
- require pedestrians and bicyclists to adjust to less traditional crossing patterns,
- may require some parking removal on approaches to accommodate vehicles' deflected paths,
- may result in drivers being unfamiliar with operation initially, and
- require additional maintenance if landscaped.

[8] United States Access Board. Draft Guidelines for Accessible Public Rights-of-Way. June 17, 2002. http://www.access-board.gov/rowdraft.htm. Accessed September 2003.

Application 3
Alternative Intersection Designs

Overview

At each particular location, selecting an intersection type is influenced by:

- functional class of intersecting streets;
- design level of traffic;
- number of intersecting legs;
- topography;
- access requirements;
- traffic volumes, patterns, and speeds;
- all modes to be accommodated;
- availability of right of way; and
- desired type of operation.

Any of the basic intersection types can vary greatly in scope, shape, and degree of channelization.[2] Basic intersection types are discussed in the *Urban Intersection Design Guide*, Chapter 1, Section 2 <link>; however, there are also a number of alternatives for intersection design. Following is overview information on a number of innovative intersection designs. Before an innovative design is pursued, the designer should coordinate with the Design and Traffic Operations Division for additional guidance.

Unconventional Left-Turn Alternative Designs

Hummer[9,10] provided information on unconventional left-turn alternatives for urban and suburban arterials. The alternatives are focused on treating left turns to and from arterials, reducing delay to through vehicles, and reducing or separating the number of conflict points. Hummer notes that by their nature as unconventional solutions and rerouting certain movements, the alternatives all have the potential to cause more driver confusion than a conventional arterial. However, this can be offset by using the alternatives on a section of the arterial and developing appropriate legible and understandable traffic control devices.

Detailed studies on the operation and safety benefits of the alternatives are not available; however, Hummer noted that the unconventional alternatives, where the number of unprotected conflicting movements has been reduced, are theoretically safer than conventional arterials. Simulation tools can be used to determine the benefits of different

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[9] Hummer, J.E. “Unconventional Left-Turn Alternatives for Urban and Suburban Arterials – Part One.” *ITE Journal*, September 1998.
[10] Hummer, J.E. “Unconventional Left-Turn Alternatives for Urban and Suburban Arterials – Part Two.” *ITE Journal*, November 1998.

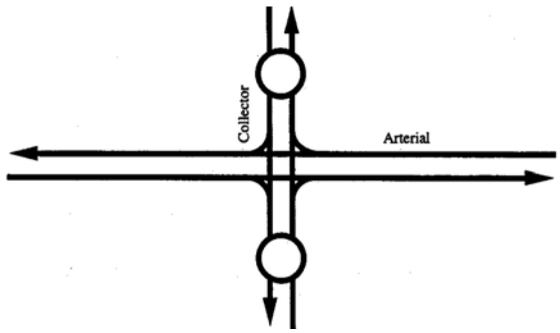
design alternatives, including unconventional alternatives, for a selected arterial. Table 1-1 summarizes characteristics of locations that may be suited for an unconventional intersection. The following figures summarize the information Hummer provided on the seven alternatives:

- Figure 1-7 Bowtie
- Figure 1-8 Superstreet
- Figure 1-9 Paired Intersection
- Figure 1-10 Jughandle
- Figure 1-11 Continuous Flow Intersection
- Figure 1-12 Continuous Green T

Table 1-1. Summary of Alternative Designs.[9,10]

Alternative	Applicable Traffic Volume			Extra right of way needed
	Left turns from arterial	Left turns from minor street	Minor Street through	
Median U-Turn	low-medium	low-medium	any	30-ft-wide [9.1 m] along arterial
Bowtie	low-medium	low-medium	low-medium	two circles up to 300 ft [91.4 m] in diameter on minor street
Superstreet	any	low-medium	low-medium	30-ft-wide [9.1 m] along arterial
Paired Intersection	any	any	low	two 80-ft-wide [24.4 m] parallel collectors
Jughandle	low-medium	low-medium	any	two 400-ft [122.0 m] by 300-ft [91.4 m] triangles at intersection
Continuous Flow	any	any	any	two 400-ft [122.0 m] by 300-ft [91.4 m] rectangles at intersection
Continuous Green T	any	low-medium	none	no extra

BOWTIE



Description. The bowtie alternative, inspired by "raindrop" interchange designs common in Great Britain, is a variation of the median U-turn alternative with the median and the directional crossovers on the cross street. To overcome the disadvantage of requiring a wide right of way on the cross street, the bowtie uses roundabouts on the cross street to accommodate left turns instead of directional crossovers across a wide median, as shown in the figure above. Left turns are prohibited at the main intersection, which therefore requires only a two-phase signal. Vehicles yield upon entry to the roundabout, but if the roundabout has only two entrances as shown above, the entry from the main intersection does not have to yield. The roundabout diameter, including the center island and circulating roadway, varies from 90 to 300 ft [27 to 91 m] depending on the speed of traffic on the approaches, the volume of traffic served, the number of approaches, and the design vehicle. The distance from the roundabout to the main intersection could vary from 200 to 600 ft [61 to 183 m], trading off spillback against extra travel distance for left-turning vehicles. The arterial may have a narrow median. Arterial U-turns are difficult, having to travel through both roundabouts and through the main intersection three times, so midblock left turns should be accommodated directly along the arterial.

Variations. A three-legged version of the bowtie is possible but would require much extra right of way. It would likely be inferior to a three-legged median U-turn or jughandle except in cases where an agency was later phasing in a fourth leg of the intersection.

History. A few agencies have installed roundabouts on cross streets in an evolutionary manner, but no agency to the author's knowledge has consciously designed a complete bowtie alternative. Raindrop interchanges, similar to diamond interchanges but with roundabouts instead of signalized or stop-controlled ramp terminals, have been in use successfully in Great Britain for years. A few raindrop interchanges have been designed and built in the United States recently, most notably in Vail, Colorado.

Advantages. The advantages of the bowtie over conventional multiphase signalized intersections include:

- reduced delay for through arterial traffic,
- reduced stops for through arterial traffic,
- easier progression for through arterial traffic,
- fewer threats to crossing pedestrians, and
- reduced and separated conflict points.

Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

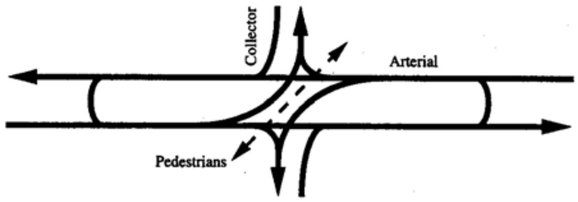
- driver confusion,
- driver disregard for the left-turn prohibition at the main intersection,
- increased delay for left-turning traffic and possibly cross-street through traffic,
- increased travel distances for left-turning traffic,
- additional right of way for the roundabouts, and
- difficult arterial U-turns.

When to Consider. Agencies should consider the bowtie alternative where there are generally high arterial through volumes and moderate to low cross-street through volumes and moderate to low left-turn volumes. If the left-turn volume is too high, the extra delay and travel distance for those drivers, and the spillback potential, will outweigh the savings for arterial through traffic. Likewise, if the cross-street through volume is too high, delays caused by the roundabout will outweigh the savings for the arterial through traffic. Arterials with narrow or nonexistent medians and no prospects of obtaining extra right of way for widening are good candidates for the bowtie. Developers may be convinced with certain incentives to build roundabouts into site plans. The distances between signals should be long so that the extra right-of-way costs for the roundabouts do not overwhelm the savings elsewhere.

Incidentally, roundabouts rarely make sense directly on multilane arterials. Roundabout capacity cannot easily be expanded by widening beyond two lanes, so roundabouts rarely work at intersections between multilane arterials. However, roundabouts are generally inappropriate for intersections between arterials and collectors or local streets because of the extra delay to larger numbers of arterial vehicles.

Figure 1-7. Alternative Designs – Bowtie.[9]

SUPERSTREET



Description. A superstreet is another extension of the median U-turn concept that provides the best conditions for through arterial movements short of interchanges. The superstreet alternative, shown above, requires cross-street through movements and left turns to and from the arterial to use the directional crossovers. Four-approach intersections become two independent three-approach intersections. This independence allows each direction of the arterial to have its own signal timing pattern, including different cycle lengths if desired, so that engineers can achieve "perfect" progression in both directions at any time with any intersection spacing. Pedestrians can make a relatively safe but slow two-stage crossing of the arterial as shown above. Other design details of the superstreet are identical to median U-turns.

Variations. One variation, at an intersection with a low-volume cross street, is to dispense with the directional crossovers for left turns from the arterial at the intersection. Another variation is to reverse the direction of the crossovers at the intersection to allow left turns to the arterial in cases where those are the heavier volume movements. However, these crossovers create difficult merges for the left on the arterial.

History. Richard Kramer, the long-time traffic engineer in Huntsville, Alaska, USA, conceived of the superstreet alternative and published a paper on it in 1987. To the author's knowledge, nobody has implemented the full superstreet, but some agencies have severed cross-street through movements and built directional crossovers on arterials in a piecemeal fashion.

Advantages. The advantages of the superstreet over a conventional multiphase signalized intersection include:

- reduced delay for through arterial traffic and for one pair of left turns (usually left turns from the arterial),
- reduced stops for through arterial traffic,
- "perfect" two-way progression at all times with any signal spacing for through arterial traffic,
- fewer threats to crossing pedestrians, and
- reduced and separated conflict points.

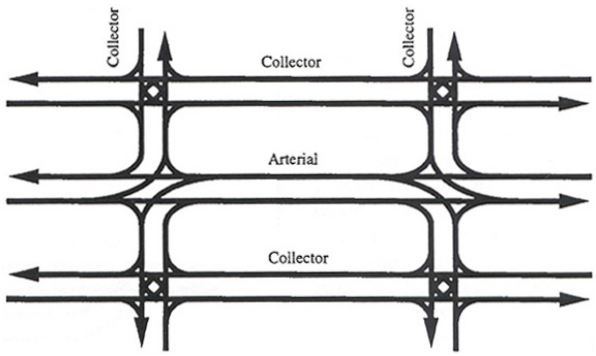
Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

- driver and pedestrian confusion,
- increased delay for cross-street through traffic and for one pair of left turns (usually left turns to the arterial),
- increased travel distances for cross-street through traffic and for one pair of left turns,
- increased stops for cross-street through traffic and for one pair of left turns,
- a slow two-stage crossing of the arterial for pedestrians, and
- additional right of way along the arterial.

When to Consider. Consider a superstreet where high-arterial through volumes conflict with moderate to low cross-street through volumes. This will be the case for many suburban arterials where roadside development generates most of the conflicting traffic. One should also consider a superstreet where close to 50/50 arterial through-traffic splits exists for most of the day, but uneven street spacings remove any chance of establishing two-way progression. As for median U-turns, arterials with narrow medians and no prospects for obtaining extra rights of way for widening are poor candidates for the superstreet.

Figure 1-8. Alternative Designs – Superstreet.[9]

PAIRED INTERSECTION



Descriptions. The paired intersection alternative uses directional crossovers (see above). The alternative employs directional crossovers for left turns from the arterial at one intersection of the pair and directional crossovers for left turns to the arterial at the second member of the pair. Complete circulation throughout the corridor requires that continuous two-way collector roads are parallel to the arterial, are set back at least several hundred feet from the arterial to avoid spillback, and provide developable parcels fronting the arterial. The intersections between the cross streets and the parallel collector roads may be stop-controlled or signal-controlled depending on the traffic volumes and other usual factors. If developments along the arterial have access from the parallel collector roads, then the arterial median does not have to be wide enough to accommodate U-turns by all vehicles. Like in a superstreet, pedestrians in the paired intersection alternative can make a relatively safe but slow two-stage crossing of the arterial.

Variations. Directional crossovers accommodating left turns to the arterial can operate with a signal controlling both directions of the arterial, but this could make two-way progression suboptimal with poor signal spacing. A variation that preserves perfect two-way progression as in the superstreet is to have the crossover end in a merge onto the arterial, which requires several hundred feet for an acceleration lane and a median that is at least 30 ft or so wide.

History. Agencies have been prohibiting turns from or onto arterials while relying on parallel streets for circulation for years, especially in downtown areas. Designers also have been channeling left turns into a development through one driveway and left turns out of the development through another driveway for years. However, Edison Johnson, a traffic engineer with the City of Raleigh, North Carolina, USA, was the first to conceive of the complete paired intersection alternative (with directional crossovers and parallel collector streets) in the late 1980s when asked to work on developing an arterial where complete conversion to a freeway was not politically acceptable. The design, which appeared in a consultant's report in 1992, is slowly being phased in by the city as the area develops.

Advantages. The advantages of the paired intersection alternative over an arterial with conventional multiphase signalized intersections include:

- reduced delay for through arterial traffic and for some left turns;
- reduced stops for through arterial traffic;
- easier progression for through arterial traffic, and with the left merge variation "perfect" two-way progression at all times with any signal spacing;
- fewer threats to crossing pedestrians; and
- reduced and separate conflict points on the arterial.

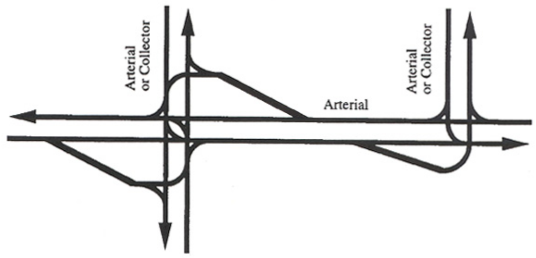
Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

- driver and pedestrian confusion,
- increased delay for cross-street through traffic and for some left-turning traffic,
- increased travel distances for cross-street through traffic and for some left-turning traffic,
- a slow two-stage crossing of the arterial for pedestrians,
- additional right of way for the parallel collector roads, and
- additional construction, maintenance, and operation costs for the parallel collector roads.

When to Consider. The paired intersection alternative is worth considering for arterials with high through-traffic volumes and low cross-street through volumes. In addition, the means to build and operate the parallel collector roads must be available. In developed corridors, good parallel streets must exist and the environment on them must allow increased traffic. In such circumstances, a one-way pair may be a superior alternative anyway. In developing corridors, agencies may be able to convince developers to pay for a portion of the cost of the collectors, and the agencies should ensure that parcels access the collectors.

Figure 1-9. Alternative Designs – Paired Intersection.[9]

JUGHANDLE



Description. The jughandle alternative uses ramps diverging from the right side of the arterial to accommodate all turns from the arterial. In the four-approach jughandle intersection shown above, the ramps are prior to the intersection. Left turns from the arterial use the ramp, then turn left on the cross street at the ramp terminal. Ramp terminals are typically stop-controlled for left turns and yield-controlled for channelized right turns. In modern jughandles ramp terminals are several hundred feet from the main intersection to ensure that queues from the signal on the cross street do not block the terminal. Since no U-turns or left turns are allowed directly from the arterial, the median may be narrow. The signal at the main intersection may need a third phase, for left turns from the cross street, if the volume is heavy.

If agencies use jughandles as the only way drivers can make left turns and U-turns along a section of arterial, all turns will be made from the right lane. This could decrease driver confusion, decrease lane changes, and increase travel speeds in the left lane.

Variations. If left turns from the ramp terminal are difficult, agencies can use loop ramps beyond the main intersection to accommodate left turns from the arterial. The travel distances for the left-turning vehicles are longer with a loop ramp, but loop ramps allow an easier right turn onto the cross street at the ramp terminal. Agencies also can employ loop ramps beyond the intersection for left turns from the cross street to avoid the third-signal phase. Jughandles for three-approach intersections and jughandles exclusively for U-turning traffic use ramps which curve back to meet the arterial as shown above.

History. The New Jersey Department of Transportation has used jughandles for years on hundreds of miles of heavy-volume arterials and continues to build new jughandle intersections.

Advantages. The advantages of the jughandle alternative over conventional multiphase signalized intersections include:

- ♦ reduced delay for through arterial traffic,
- ♦ reduced stops for through arterial traffic,
- ♦ easier progression for through arterial traffic,
- ♦ narrower right of way needed along the arterial, and
- ♦ reduced and separated conflict points.

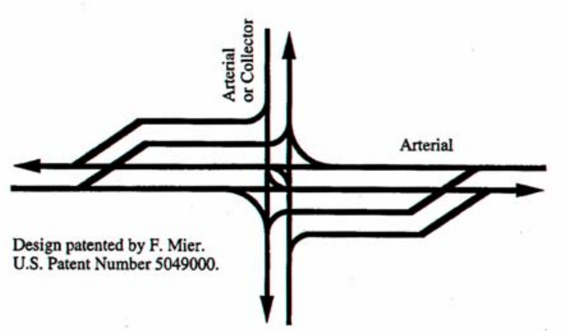
Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

- ♦ driver confusion;
- ♦ driver disregard for left-turn prohibitions at the main intersection;
- ♦ increased delay for left turns from the arterial, especially if queues of cross-street vehicles block the ramp terminal;
- ♦ increased travel distances for left turns from the arterial;
- ♦ increased stops for left turns from the arterial;
- ♦ pedestrians must cross ramps and the main intersection;
- ♦ additional right of way for ramps;
- ♦ additional construction and maintenance costs for ramps; and
- ♦ lack of access to arterial for parcels next to ramps.

When to Consider. Designers should consider jughandles on arterials with high through volumes, moderate to low left-turn volumes, and narrow rights of way. The distances between signals should be long so that the extra right of way and other costs for the ramps do not overwhelm the savings elsewhere.

Figure 1-10. Alternative Designs – Jughandle.[9]

CONTINUOUS FLOW INTERSECTION



Description. The continuous flow intersection features a ramp to the left of the arterial upstream of the main intersection to handle traffic turning left from the arterial, as shown above. Usually, high volumes will justify a signal at the crossover where the ramp begins. Engineers can easily coordinate this two-phase signal with the signal at the main intersection. A single signal controls the main intersection and the left-turn ramp/minor-street intersection. The major breakthrough with this design is that arterial through traffic and traffic from this left-turn ramp can move during the same signal phase without conflicting. This allows, in effect, protected left turns with a two-phase signal. The cross-street stop bar must be set back beyond the left-turn ramp, which probably means more lost time and longer clearance intervals for the cross-street signal phase(s). Right turns are removed from conflicts near the intersection with ramps. U-turns on the arterial are possible at the left-turn crossover if the median is wide enough. Without provisions for U-turns the arterial median may be narrow. The left-turn ramp usually crosses the opposing traffic 300 ft [92 m] or so from the cross street to balance the various higher costs of a longer ramp against the chance of spillback from the main intersection blocking the signal at the crossover.

Franciso Mier of El Cajon, Calif., USA, holds the U.S. patent, #5049000, for the continuous flow intersection. Agencies wishing to implement the design must contact Mier to obtain the rights.

Variations. If left turns to the arterial are heavy at the continuous flow intersection as shown above, a third signal phase may be needed at the main intersection. To avoid the third phase, designers can use left-turn ramps in three or all four quadrants of the intersection.

History. Mier obtained his patent in 1987. With co-authors, he has published articles evaluating the concept in general and has written several reports evaluating the concept in particular locations. The first continuous flow intersection in the United States, with ramps in a single quadrant at a T-intersection, was opened in 1994 in Long Island, N.Y., USA, at an entrance to Dowling College. Several others have opened recently in Mexico. Early reports on the operation of these intersections are favorable.

Advantages. The advantages of the continuous flow intersection over a conventional multiphase signalized intersection include:

- reduced delay for through arterial traffic,
- reduced stops for through arterial traffic,
- easier progression for through arterial traffic,
- narrower right of way needed along the arterial, and
- reduced and separated conflict points.

With ramps in three or four quadrants these advantages may extend to the cross street as well.

Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

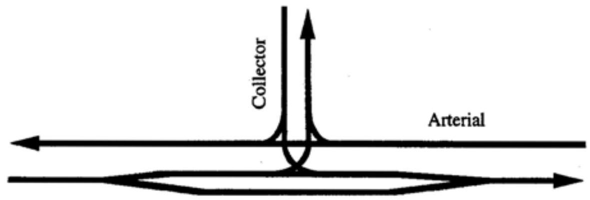
- driver and pedestrian confusion;
- increased stops for left turns from the arterial;
- restricted U-turn possibilities;
- pedestrians must cross ramps and the main intersection (and pedestrians must cross the four-quadrant design in a slow two-stage maneuver);
- additional right of way for ramps;
- additional construction, maintenance, and operation costs for ramps and extra signals;
- lack of access to the arterial for parcels next to ramps; and
- the costs of obtaining the rights to use the design.

If left turns from the arterial experience more delay than at comparable conventional intersections, the extra delay is likely to be small in magnitude.

When to Consider. Agencies should consider the continuous flow intersection on arterials with high through volumes and little demand for U-turns. The designer must have some right of way available along the arterial near the intersection and must be able to restrict access to the arterial for parcels near the intersection. Like the bowtie and jughandle alternatives, the extra right of way and other costs will be hard to justify if installations are too close together.

Figure 1-11. Alternative Designs – Continuous Flow Intersection.[10]

CONTINUOUS GREEN T



Description. While the other unconventional alternatives discussed worked for both three-approach and four-approach intersections, the continuous green T alternative only works for three-approach intersections (see above). The two through lanes on the top of the T are controlled differently. The median lane is subject to the standard two-phase or (more likely) three-phase signal, which also controls opposing through traffic; left turns from the arterial; and turns from the cross street. However, the shoulder lane receives a steady green signal. Pavement marking directs traffic turning left from the cross street into the median lane. Pedestrians must seek signal protection between the two through lanes. This area can be narrow and should not present a hazardous fixed object. Agencies should make the separation visible and tactile with raised reflectors or rumble strips. The separation should extend several hundred feet upstream and downstream from the intersection to minimize last-minute weaves. Agencies can use more than one continuous through lane, but dual left-turn lanes from the cross street would mean dual signal-controlled through lanes on the top of the T and would put great pressure on the remaining continuous through lane(s). The arterial should have a raised median of some type, at least for the length of the through-lane separation, to stop vehicles from turning left or from driveways and thereby crossing the through-lane separation.

Variations. The main variation to the continuous green T as shown above is to have all through lanes on the top of the T get a steady green signal while left turns from the cross street are channelized into a merging lane in the median. The merging lane must be lengthy to minimize conflicts. This variation requires a slightly wider median than the minimal 16-ft [4.9 m] median (i.e., one exclusive turn-lane wide) required for the continuous green T-intersection as shown above. The island channelizing the left turns should be very positive; some agencies use curbs with pavement markings and reflectors. A merge from the left is a difficult driving maneuver, so while this variation rewards higher volumes of arterial through traffic, it will break down with higher left-turn volumes.

History. Several districts of the Florida Department of Transportation have used the continuous green T alternative shown with no major apparent problems. A large number of agencies use the variation described above.

Advantages. The advantages of the continuous green T-intersection over a conventional multiphase signalized T-intersection include:

- reduced delay for through arterial traffic in one direction, and
- reduced stops for through arterial traffic in one direction.

It is very unlikely that the through movement at the top of the T is a critical movement that controls signal timing. If that movement should happen to be critical, however, removing it from the domain of the signal would lead to reduced delay for all other movements at the intersection.

Disadvantages. The disadvantages of the alternative relative to conventional intersections include:

- driver and pedestrian confusion,
- driver disregard of the separation between the through lanes,
- no signal protection for pedestrians to cross the arterial,
- increased lane changing conflicts before and after the separation of the through lanes, and
- restricted access to parcels adjacent to the continuous green through lane(s).

Driveways along the continuous green through lanes(s) pose two potential problems. First, through drivers in the continuous green lanes may not expect to slow for anything in those lanes, even a right-turning vehicle. Second, drivers turning left onto the arterial from the minor street may try to merge into the continuous green through lane or pass through the lane separation to get to a driveway.

When to Consider. Of the unconventional alternatives discussed in these features, the continuous green T has the most restrictive niche. Engineers should consider it at signalized three-approach intersections with moderate to low left-turn volumes from the minor-street and high arterial through volumes, where there are no crossing pedestrians and few drivers choose one of the two continuous green lanes.

Figure 1-12. Alternative Designs – Continuous Green T.[10]

Quadrant Roadway Intersection

Reid[11] also proposed another unconventional intersection design alternative B the “quadrant roadway intersection” (QRI) design. The QRI design removes left-turn movements from main arterial/cross-street intersections through use of an additional roadway in one intersection quadrant. Figure 1-13 shows a typical QRI design and Figure 1-14 shows the left-turn patterns. By routing all left-turn movements from the arterial and cross street to the quadrant roadway, the main arterial and cross-street intersection can operate with a simple two-phase signal. The spacing of the QRIs from the main intersection is a trade-off between left-turn travel distance and time versus available storage for the westbound left-turn movement. In the analysis of the QRI, a 91.8-ft [28 m] spacing was selected for both QRIs from the main intersection. Other considerations for QRI include:

- potential uses of the land within the quadrant roadway such as service station or convenience store served by right-in/right-out driveways,
- additional advance signing needs,
- design modifications for missed left-turn opportunities (consider additional median U-turns beyond the main intersection),
- preservation of signal operation at each intersection (a fourth intersection leg cannot be developed at either end of the quadrant roadway because these signals must function as T-intersections), and
- restriction of driveways between intersections to preserve left-turn storage for the main intersection approaches.

[11] Reid, J.D. “Using Quadrant Roadways to Improve Arterial Intersection Operations,” *ITE Journal*, June 2000.

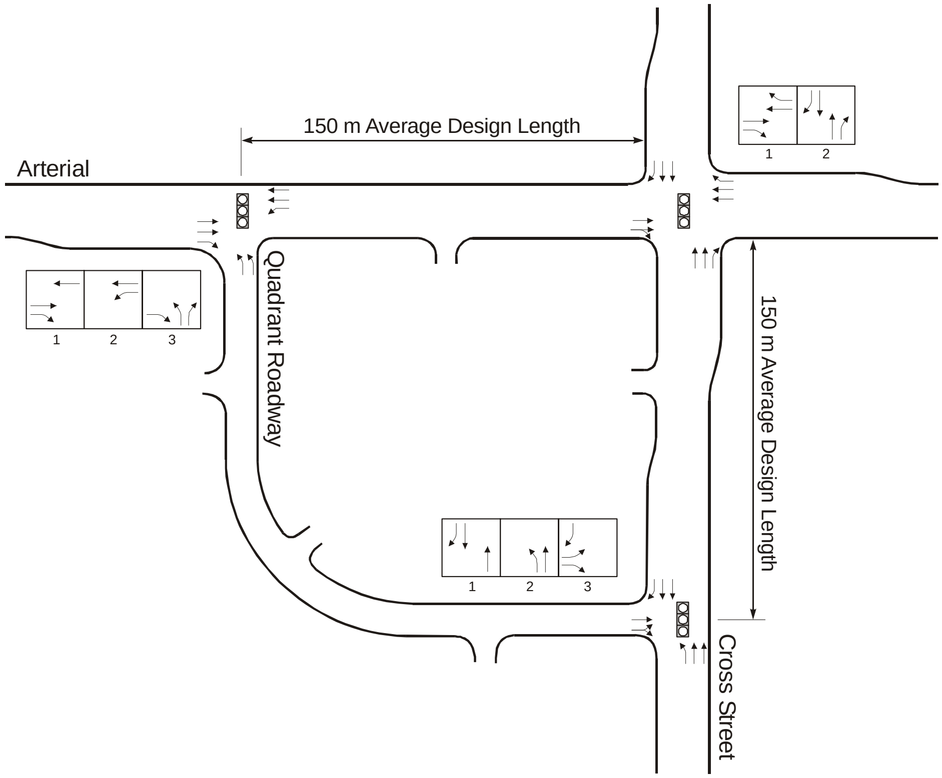
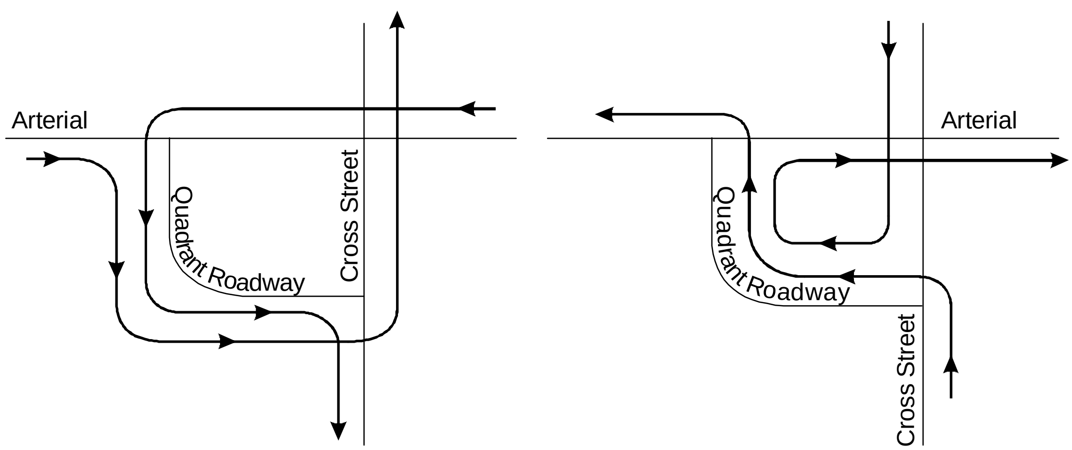


Figure 1-13. QRI Design.[11]



(A) Left-Turn Pattern from the Arterial (B) Left-Turn Pattern from the Cross Street

Figure 1-14. QRI Left-Turn Pattern.[11]

A CORSIM experiment was conducted that showed improved stopped-delay and system travel time for QRIs as compared to typical arterial intersections. The author noted that while driver expectations may be violated at QRIs, designs similar to QRI have been successfully implemented in the field. Based on his analysis, the advantages and disadvantages listed below were identified.

Advantages were:

- creates more progression opportunities by allowing a larger progression bandwidth due to two-phase signal operation at the main intersection;

- reduces total intersection system delay;
- reduces queuing, especially for the worst approach movements, by greater than 120 percent in level of service conditions;
- fewer vehicle conflict points at the main intersection and a probable reduction in left-turn or head-on collisions; and
- narrower intersection widths (by eliminating dual turn lanes) reduce vehicle clearance and pedestrian crossing times.

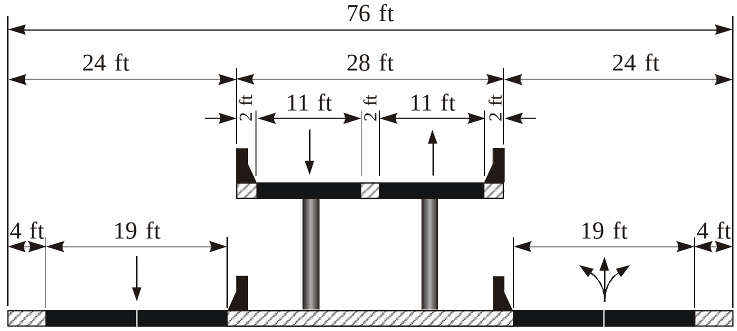
Disadvantages were:

- increased left-turn travel distance and the potential for increased left-turn travel times and stops;
- greater possibility of driver confusion, error at critical (intersection) locations, and missed left-turn opportunities;
- nonconformity of left-turn patterns for each approach of the same intersection;
- additional advance signing requirements; and
- additional right of way required for the quadrant roadway.

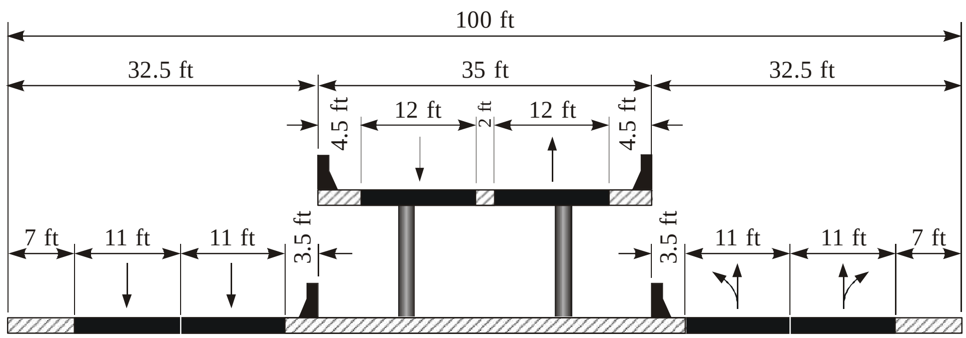
Flyovers

Bonilla[12] examined the benefits of the flyover, which is defined as a grade-separated structure that allows arterial through traffic to go over a crossing arterial or collector without slowing down or stopping for an at-grade signal. He states that capacity per lane is generally that of arterial through lanes, about 1750 vph. His economic evaluation showed that congested intersections with an approach volume averaged over 20 years of 50,000 vehicles per day or more would justify a simple arterial flyover. The minimum right of way for urban arterial flyovers is listed in Table 1-2, and Figure 1-15 shows minimum cross sections. Safety considerations for flyovers require a smooth transition from at-grade arterial lanes to the flyover. The physical split between exiting intersection-bound traffic and the through traffic must be logical, simple, and anticipated.

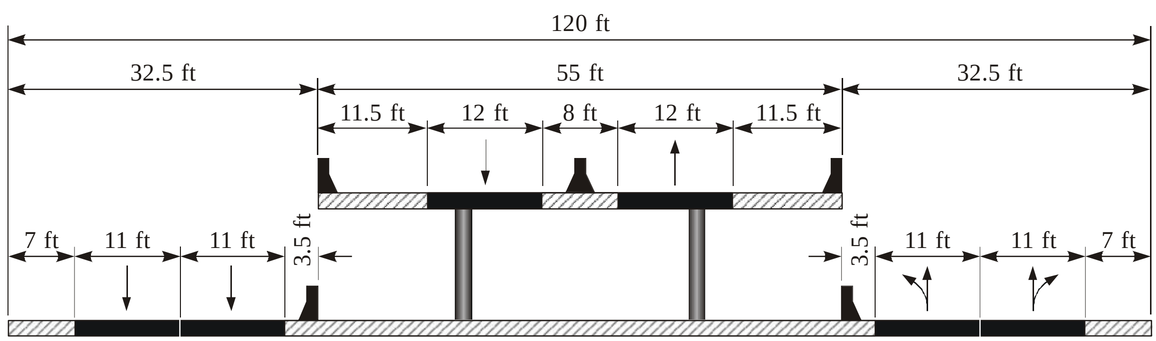
[12] Bonilla, C.R. Physical Characteristics and Cost-Effectiveness of Arterial Flyovers, *Transportation Research Record* 1122, Transportation Research Board, Washington, D.C., 1987.



(A) Marginal



(B) Low Type



(C) High Type

Figure 1-15. Minimum Cross Section and Right of Way for a Two-Lane Flyover.[11]

Table 1-2. Minimum Right of Way for Urban Arterial Flyovers.[12]

	Right of Way by Number of Lanes, ft [m]		
	Two Lanes	Four Lanes	Six Lanes
Marginal	76 [23.2]	98 [29.9]	
Low Type	100 [30.5]	120 [36.6]	140 [42.7]
High Type	120 [36.6]	144 [43.9]	168 [51.2]

Merging traffic from the at-grade intersection with traffic from the flyover may require somewhat longer tapers, similar to those used for arterial lane drops. Another challenge with the design is tying the vertical alignment with existing grade before reaching the next cross street. Bonilla proposed the following warrants for flyovers:

- The intersection is a bottleneck and conventional traffic engineering measures cannot resolve the capacity problem.
- A minimum of four through lanes already exists and maximum use of the intersection right of way has been made. The sum of critical lane volumes approaches or exceeds 1200 vph.
- It is time-consuming, expensive, or contrary to public objectives to obtain additional right of way. A minimum right of way of 100 ft [30 m] is available.
- Impact to adjacent properties and minor streets limited to right turn only is not severe.
- The accident rate is significantly larger than for nearby intersections on the same arterial.

Echelon

Another concept for an uncontrolled access urban arterial interchange is the echelon interchange.[13] The echelon interchange elevates one-half of a divided highway as it approaches the point of intersection, resulting in two grade-separated intersections. The two grade-separated intersections operate in the same manner as two one-way pair intersections (see Figure 1-16). Miller and Vargas[13] concluded that the echelon interchange will sometimes, but not always, out-perform traditional grade separation designs in signalized networks. It offers two important possible advantages: (a) it will not overpower the adjacent signalized intersection to the extent free-flow movements might; and (b) it offers the planner/designer significant flexibility and more discretionary options relative to its layout and its attendant land use.

[13] Miller, N.C., and J.E. Vargas. "The Echelon Interchange," 1997 *ITE Compendium*. (1997).

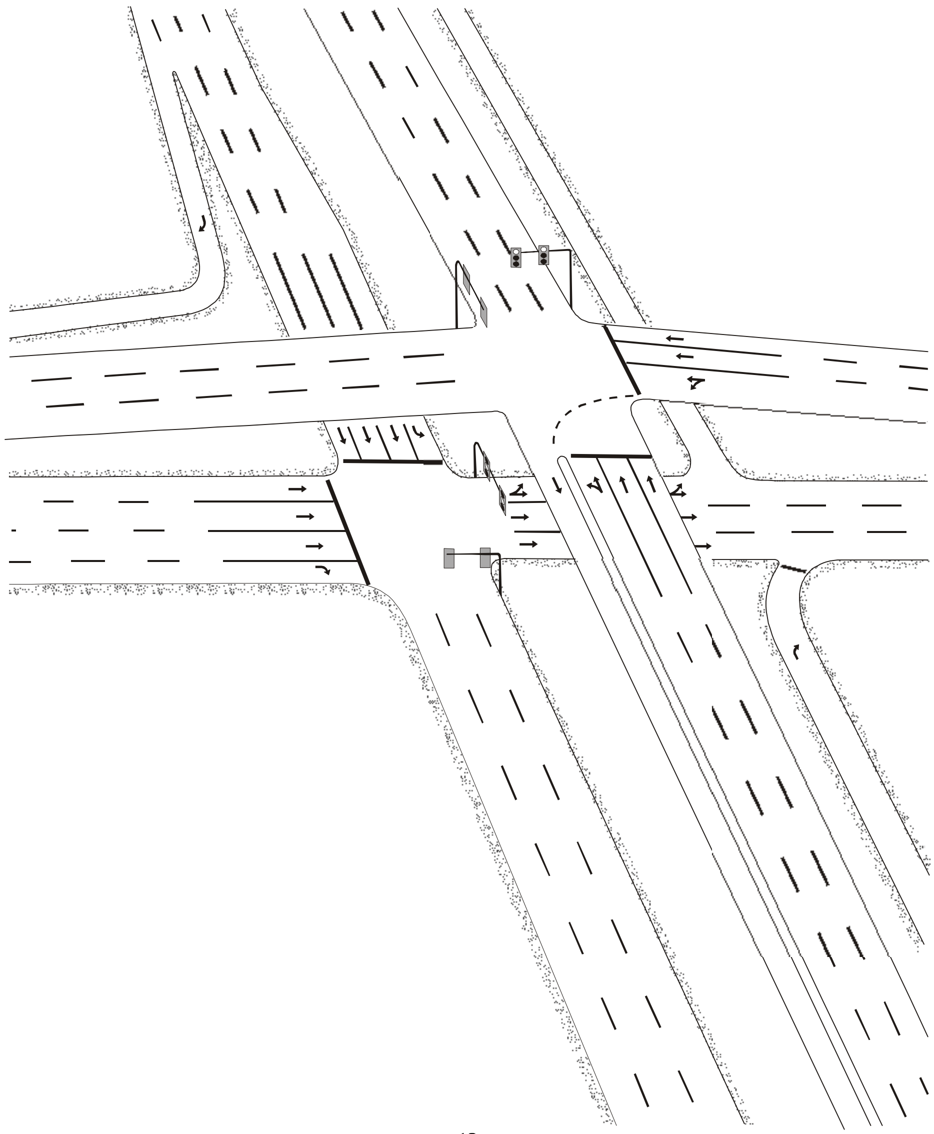


Figure 1-16. Echelon Interchange.[13]

Chapter 2
Design Control and Criteria

Contents:

Application 1

Pedestrian Features Checklist

Overview

The following presents a checklist on pedestrian features. The *Urban Intersection Design Guide*, Chapter 2, Section 2 presents additional information on pedestrians. Information on sidewalks and pedestrian treatments is in the *Urban Intersection Design Guide*, Chapter 5, Section 1 and Chapter 7, respectively.

Background

During an intersection design, several elements are competing for attention. Following is a checklist that can be used to review a design to assist in identifying whether pedestrians are adequately considered within the design. The objective of the checklist is to encourage consideration of pedestrians throughout the design. It is easier to incorporate pedestrian-friendly features early in the design rather than trying to retrofit after the design is nearly complete or after the intersection is constructed.

Checklist

Checklist For Pedestrian Features At An Urban Intersection

***Checklist Key**

Y = Yes or Acceptable N = No or Needs Improvement I = Irrelevant to Site

*Y	N	I	**SIDEWALKS**
			Is a sidewalk present on at least one side of road?
			Are sidewalks on both sides of road?
			Is sidewalk continuous (e.g., no gaps)?
			Is sidewalk wide enough to meet current accessibility requirements?
			Is sidewalk wide enough to meet current demand?
			Is sidewalk wide enough to meet future demand?
			Is cross slope on sidewalk < 2 percent (required by *Americans with Disabilities Act Accessibility Guidelines* (ADAAG))?
			Is the pedestrian path clear of obstructions, such as street lights, utility poles, newspaper stands, trash receptacles, etc.?

Checklist For Pedestrian Features At An Urban Intersection

***Checklist Key**

Y = Yes or Acceptable N = No or Needs Improvement I = Irrelevant to Site

*Y	N	I	SIDEWALKS (Continued)
			Is there street furniture in the pedestrian path that could affect movement (e.g., benches, etc.)?
			Is condition of sidewalks well maintained (e.g., are tree roots causing upheave)?
			Is there adequate separation between the pedestrian path and traffic (greater separation desired on higher functional class roads)?
			Is adequate information provided for the pedestrian (e.g., signs, clearly defined pedestrian route, etc.)?
			Are signs of adequate size or lighted as needed for the pedestrian population expected for the area?
			Is adequate street lighting of the sidewalk present?
			Is desirable street furniture present (e.g., benches, water fountain, etc.)?
			Are accessible driveway crossings present?
			Are pedestrians visible to drivers (and vehicles visible to pedestrians) exiting driveway?

*Y	N	I	CROSSING
			Can pedestrians (including children and wheelchair users) see approaching vehicles?
			Can vehicles in each lane clearly see pedestrians?
			Is waiting area paved?
			Is waiting area large enough to accommodate anticipated demand?
			Does waiting area meet accessibility requirements?
			Is waiting area clear of street furniture, fixtures, or other obstacles?

Checklist For Pedestrian Features At An Urban Intersection

***Checklist Key**

Y = Yes or Acceptable N = No or Needs Improvement I = Irrelevant to Site

*Y	N	I	**CROSSING (Continued)**
			Are relevant traffic control devices (signs, markings, signals) present? If so, are they clearly visible (or audible) to the pedestrian (e.g., pedestrian head located in-line with crosswalk, etc.)?
			Is the length of the crossing (i.e., width of cross street) appropriate for the ability or comfort of the pedestrian?
			Is the pedestrian push button in the desired location?
			Does the crossing reflect pedestrian desired lines?
			Are traffic control devices at the crossing appropriate or is a more restrictive control needed?
			Is the amount of time a pedestrian waits at an unsignalized intersection to cross the street reasonable?
			Is the pedestrian crossing interval at a signalized intersection adequate?
			Are pedestrian-oriented signal treatments (e.g., scramble phases, no right turn on red, etc.) needed?
			Are additional features (e.g., longer crossing time) that would be activated by an extended button press at a signalized intersection needed?
			Are detectable warnings present where needed (e.g., on curb ramps, at flush transitions)?
			Is curb ramp and landing present?
			Does curb ramp meet accessibility requirements?
			Is the foot of the curb ramp contained within the crosswalk markings?
			Is median refuge island large enough to accommodate anticipated demand?
			Is the median refuge island accessible?
			Does the median refuge island limit visibility for the pedestrian?

Checklist For Pedestrian Features At An Urban Intersection

***Checklist Key**

Y = Yes or Acceptable N = No or Needs Improvement I = Irrelevant to Site

*Y	N	I	**CROSSING (Continued)**
			Does the median refuge island include signal activation (e.g., pedestrian push button) if crossing is signalized?
			Do pedestrians need to be physically directed to cross in a preferred location?
			Would stormwater drainage (e.g., flow rate, ponding, snow, ice accumulating, etc.) near the crossing affect pedestrians?

*Y	N	I	**INTERACTING**
			Can drivers see waiting pedestrians?
			Can drivers see crossing pedestrians?
			Do through drivers appropriately yield the right of way to pedestrians?
			Do left-turning drivers appropriately yield the right of way to pedestrians?
			Do right-turning drivers appropriately yield the right of way to pedestrians?
			Is right turn on red an issue?
			Is the posted or operating speed on the roadway an issue?
			Is red-light running a frequent problem?
			Do drivers realize that a pedestrian may be crossing (e.g., is crosswalk clearly marked and clearly visible, are advance signs of crossing present)?
			Is transit stop located where pedestrians are likely to be walking, waiting, or crossing?
			Is the speed of the turning vehicle (which is influenced by the corner radius) incompatible with pedestrian usage in the area?
			Do features of the median refuge island limit visibility to the pedestrian?
			Does on-street parking affect pedestrian movement?

***Checklist For Pedestrian Features At An Urban Intersection**

***Checklist Key**

Y = Yes or Acceptable N = No or Needs Improvement I = Irrelevant to Site

*Y	N	I	**EXPERIENCE**
			Do landscaping, art, and/or vista enhance experience?
			Does landscaping interfere with walking/crossing (e.g., planters consume too much of available walking area, trees/bushes limit view to or from pedestrians, etc.)?
			Is the interaction between development and walking/waiting areas positive?
			Is the area maintained?
			Are amenities appropriate for types of pedestrians using the facility (e.g., school children, disabled, older, etc.)?
			Is walking route comfortable and convenient (sidewalk direct without unnecessary horizontal or vertical changes)?
			Is waiting area comfortable?
			Is alternative route available if sidewalk is blocked?
			Does the pedestrian route have good drainage (e.g., ponds are not forming near the route)?
			Would pedestrians feel secure?
			Would pedestrians feel safe?

Checklist For Pedestrian Features At An Urban Intersection

***Checklist Key**

Y = Yes or Acceptable N = No or Needs Improvement I = Irrelevant to Site

*Y	N	I	**POLICY**
			Is location part of a sidewalk plan?
			Is there support by municipality?
			Is there support by local residents or businesses?
			Is there support by community groups?
			Is there evidence of pedestrian travel (e.g., beaten down path, etc.)?

Application 2
Safety Study Example

Overview

The following application presents a safety study. The *Urban Intersection Design Guide*, Chapter 2, Section 4 presents additional information on urban intersection safety.

Background

A young engineer was assigned the task of developing suggestions for improvements to an intersection reported as having a crash problem. The intersection of interest is in a rapidly developing area of the district. Two two-lane state highways intersect at a 90-deg angle with one highway having an Average Daily Traffic (ADT) of 11,000 and the other having an ADT of 6000. The intersection has a signal and no turn lanes on any of the approaches. A sketch of the intersection is shown in Figure 2-1.

The engineer identified two resources that could assist in the evaluation:

- ITE's *Manual of Transportation Engineering Studies*,[1] and
- TxDOT's *Treatments for Crashes on Rural Two-Lane Highways in Texas* (TxDOT Report 4048-2).[2]

The engineer used the procedure presented in these documents as a guide for the assignment along with several other references. While the TxDOT report had a rural focus, the steps presented were applicable to safety studies on all facility types. The following is the safety study format that was used:

- Identify Crash Characteristics,
- Gather Existing Conditions,
- Collect Additional Field Data,
- Assess Situation and Select Treatments, and
- Implement and Evaluate.

Steps undertaken within each of the above elements are discussed in the following sections.

[1] Hummer, J.E., "Traffic Accident Studies" chapter, *Manual of Transportation Engineering Studies*. Institute of Transportation Engineers. Prentice Hall, Englewood Cliffs, NJ, 2000.

[2] Fitzpatrick, K., A.H. Parham, and M.A. Brewer. *Treatments for Crashes on Rural Two-Lane Highways in Texas*. Report FHWA/TX-02/4048-2. Texas Transportation Institute, College Station, TX, April 2002.

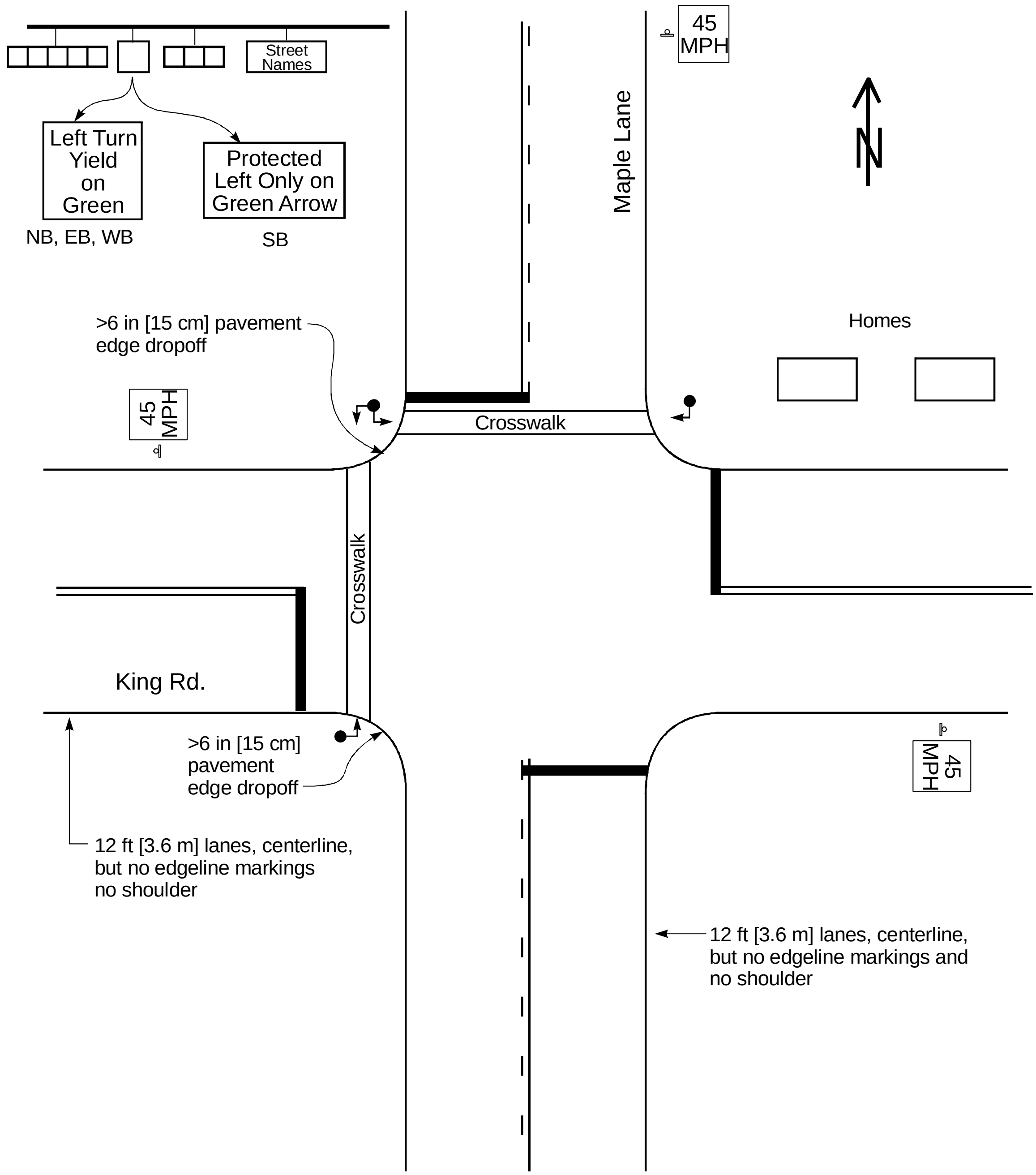


Figure 2-1. Condition Diagram.

Identify Crash Characteristics

For this location, a citizen reported that the intersection needed improvements due to the number of crashes occurring. In other situations sites may be identified by other agencies (e.g., police) or through a comprehensive review of a region's crash data. The engineer identified the following resources as being needed for the evaluation:

- ♦ Texas crash data (available from the District Traffic Section) and
- ♦ crash narratives (ordered from Department of Public Safety [DPS] after crashes are identified or may be available from local law enforcement).

Documents that could assist with the safety study include the following:

- ♦ ITE "Traffic Accident Studies" chapter of the *Manual of Transportation Engineering Studies*,[1]
- ♦ *Treatments for Crashes on Rural Two-Lane Highways in Texas*[2] (TxDOT Report 4048-2),
- ♦ *Texas Manual on Uniform Traffic Control Devices*[3] (*TMUTCD*),
- ♦ TxDOT *Roadway Design Manual*[4] (available on the web),
- ♦ AASHTO *Roadside Design Guide*,[5]
- ♦ NCHRP Synthesis 295: Statistical Methods in Highway Safety Analysis,[6] and
- ♦ NCHRP Report 440: *Accident Mitigation Guide for Congested Rural Two-Lane Highways*.[7]

The following steps were used to identify the crash characteristics at the site:

- ♦ ***Crash Data.*** An initial step in a safety study is to request the crash data for the site. The junior engineer was told to obtain 3 years of data since this is the most common time frame used in these types of studies. Time frames of less than 2 years may be necessary, but the smaller sample size may not be representative of conditions at the location and the user may need to adjust for the regression-to-the-mean condition (see NCHRP Synthesis 295[6] for additional information on regression-to-the-mean). Several

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[3] *Texas Manual on Uniform Traffic Control Devices for Streets and Highways*. Texas Department of Transportation. 2003. http://www.dot.state.tx.us/TRF/mutcd.htm. Accessed February 17, 2003.
[4] *Roadway Design Manual*. Texas Department of Transportation. 2002. http://manuals.dot.state.tx.us/dynaweb/coldesig/rdw/@Generic__BookView
[5] AASHTO. *Roadside Design Guide*. American Association of State Highway and Transportation Officials, Washington, D.C., 2002.
[6] Persaud, B.N. *Statistical Methods in Highway Safety Analysis.* NCHRP Synthesis 295. Transportation Research Board, Washington, D.C., 2001.
[7] Fitzpatrick, K., D.W. Harwood, I.B. Anderson, and K. Balke. *Accident Mitigation Guide for Congested Rural Two-Lane Highways*. National Cooperative Highway Research Program Project 440, November 1999.

factors are associated with each crash in the Texas crash database. The analyst may not be interested in all the factors and may want to limit which data fields are pulled. For example, the database includes ADT for several years, and the analyst may only be interested in the ADTs for the years under study. Examples of factors that may be of interest include: collision type, severity of injury, road surface conditions, weather, object struck, traffic control, month, day of week, time of day, light conditions, first harmful event, roadway condition, alignment, curve, number of vehicles involved, other factors, and direction of travel. The crash data for this study were obtained from the District Traffic Section. Starting in 2005, the department's new Crash Record Information System (CRIS) will replace the existing Department of Public Safety database.

- ***Summary Report.*** Once the crash data were available, a summary report of the crashes was prepared. This report, shown in Table 2-1, will assist in the evaluation.
- ***Collision Diagram.*** A collision diagram was developed to identify the patterns of crashes and is shown in Figure 2-2. Examples of collision diagrams are contained in several documents including NCHRP Report 440: *Accident Mitigation Guide for Congested Rural Two-Lane Highways*[7] and ITE "Traffic Accident Studies" chapter of the *Manual of Transportation Engineering Studies*.[1]
- ***ADT.*** The approach volumes for each roadway were identified for the intersection. The calculated crash rates for the intersection could be compared to district averages to assist in the determination of whether improvements should be programmed.
- ***Crash Narratives.*** The crash narratives from the Department of Public Safety were requested. The information from the crash database used to produce the summary and collision diagram was compared with the information contained in the narratives. The narratives provided additional information, such as conditions during the crash involving a driving while intoxicated (DWI) charge.

Table 2-1. Crashes at Maple and King.

Acc Num	Collision Type	Severity	Surface	Weather	Time of Day	Month	Day	Light Condition	No. of Veh
6019	same 1 str 2 stop	non-injury	wet	clear	6:00 AM	Jan	Mon	dawn	2
6217	opp 1 str 2 LT	possible	dry	clear	3:00 PM	Sept	Mon	daylight	2
6295	opp 1 str 2 LT	nonincap	wet	fog	8:00 PM	Dec	Mon	daylight	2
6146	opp 1 str 2 LT	possible	wet	raining	6:00 AM	July	Fri	daylight	2
6162	same 1 str 2 stop	possible	wet	clear	9:00 AM	July	Wed	daylight	2
6163	opp 1 bck 2 stop	possible	dry	clear	10:00 AM	July	Tues	daylight	2
7195	same 1 str 2 stop	non-injury	dry	clear	11:00 AM	Aug	Sun	daylight	2
7165	opp 1 str 2 LT	possible	dry	clear	7:00 PM	July	Fri	daylight	2
7262	ang both str	possible	wet	raining	11:00 AM	Nov	Wed	daylight	3
7263	same 1 str 2 stop	possible	wet	raining	11:00 AM	Nov	Wed	daylight	2
8024	opp 1 str 2 LT	possible	dry	clear	8:00 AM	Feb	Tues	daylight	2
8088	same 1 str 2 stop	nonincap	wet	raining	6:00 PM	April	Mon	daylight	3
8124	opp 1 str 2 LT	possible	dry	clear	3:00 PM	June	Wed	daylight	2
8257	same 1 str 2 stop	nonincap	wet	raining	2:00 PM	Nov	Sat	daylight	3
6245	opp both str	incap	wet	raining	1:00 PM	Oct	Tues	daylight	2
8005	opp 1 str 2 LT	non-injury	dry	clear	8:00 PM	Jan	Thurs	dark not lighted	2
8057	same 1 str 2 stop	possible	wet	raining	12:00 PM	March	Sat	daylight	2
8106	same 1 str 2 stop	nonincap	dry	clear	6:00 PM	May	Mon	daylight	3
8175	opp 1 str 2 LT	possible	dry	clear	6:00 AM	Aug	Tues	daylight	3
9277	opp both str	nonincap	dry	clear	10:00 PM	Nov	Tues	dark not lighted	2
9085	rear end	non-injury	dry	clear	5:00 PM	April	Fri	daylight	4
9200	rear end	non-injury	wet	raining	3:00 PM	Aug	Fri	daylight	2
9256	same 1 str 2 stop	non-injury	wet	raining	10:00 AM	Oct	Sat	daylight	2
9257	same 1 str 2 stop	non-injury	wet	raining	4:00 PM	Oct	Sat	daylight	2
9293	same 1 str 2 stop	possible	wet	raining	4:00 PM	Dec	Sat	daylight	4
9157	ang both sir	nonincap	wet	raining	5:00 PM	July	Wed	daylight	2
9001	opp 1 str 2 LT	possible	dry	clear	6:00 PM	Jan	Sat	dark not lighted	2
9094	opp 1 str 2 LT	nonincap	wet	clear	11:00 AM	April	Mon	daylight	2
9286	same 1 str 2 stop	possible	wet	clear	9:00 AM	Dec	Sat	daylight	4
9055	same 1 str 2 stop	possible	wet	clear	4:00 PM	March	Fri	daylight	3
9068	SIN str	non-injury	wet	raining	5:00 PM	March	Sat	daylight	1

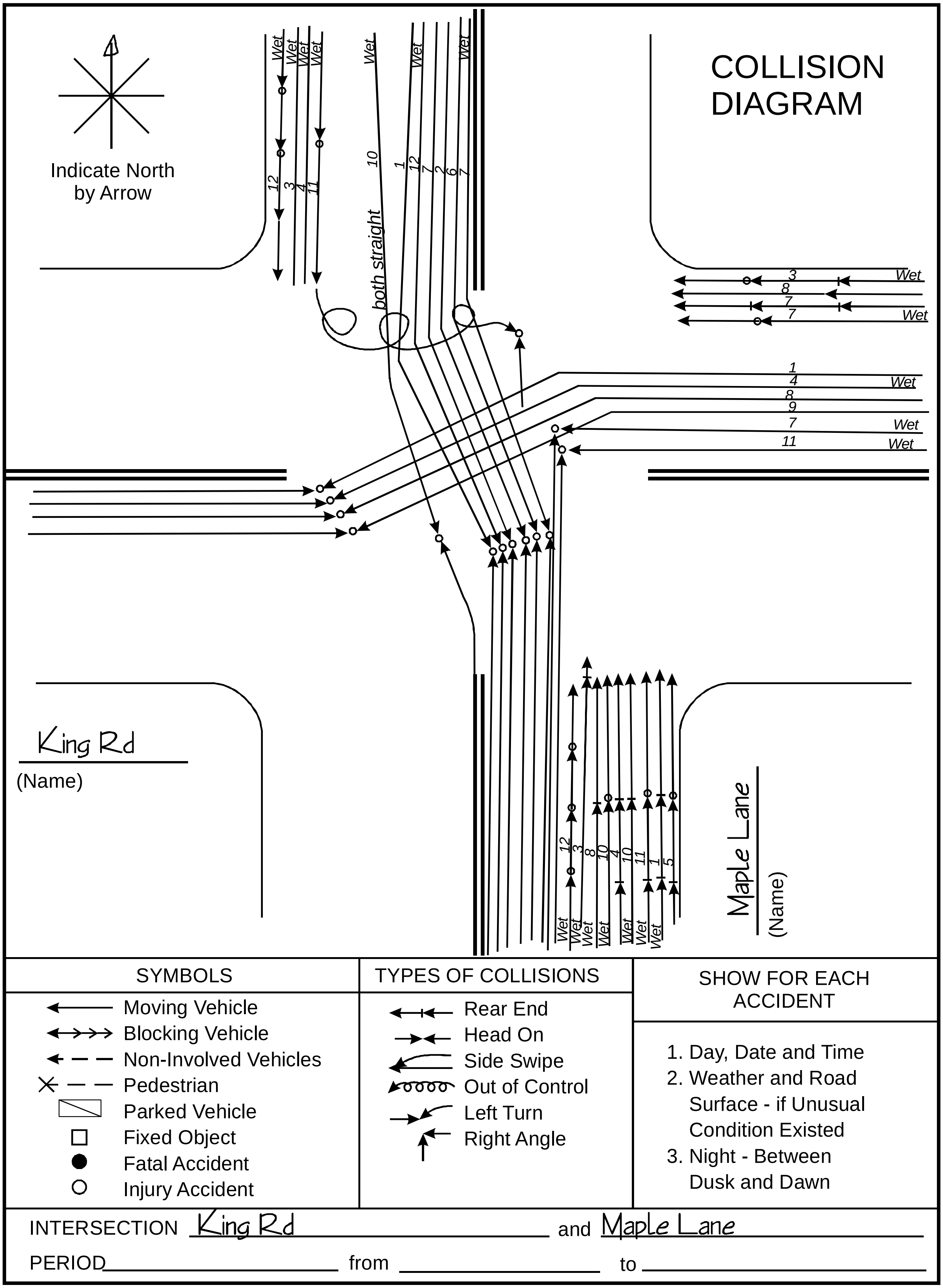


Figure 2-2. Collision Diagram.

Gather Existing Conditions

The next effort in the process was to gather information on the in-field condition of the site. Following are the steps that were followed:

- ***Field Methodology.*** At each location, the review team performed the following:
  - filmed a drive-through video of all approaches to record existing conditions from a driver's perspective,
  - drew a condition diagram (see Figure 2-1),
  - took pictures,
  - observed traffic, and
  - noted driver behavior.
- ***Checklists.*** To assist with field operations, three groups of questions or checklists were used at each site (these checklists are shown in Table 2-2, Table 2-3, and Table 2-4). Observations recorded on the checklists included the following:
  - High volumes produced queues (5 to 10 vehicles long) during red indications.
  - Queues that formed cleared during the next green indication.
  - Pavement markings were worn and needed replacing.
  - No left-turn bays were present; however a left-turn indication would appear during the cycle on the southbound approach. Is that left-turn signal pretimed or activated due to standing queue? (Need to request signal timing plan.)
  - Edge drop-offs are present, especially on the westbound approach.
  - Several large trucks were observed on the westbound approach.
  - Subdivisions are being constructed on several approaches.
- ***Findings.*** The findings from the field, information in the crash narratives obtained from the Department of Public Safety, and information from the crash database used to produce the summary and collision diagrams were compared.

Table 2-2. Basic Field Observations.[7]

Operational Problem Symptoms	Physical Inventory Parameters (Supplement Construction Plans)
♦ Length of vehicle queues ♦ Erratic vehicle maneuvers ♦ Vehicles experiencing difficulty in making turning movements ♦ Vehicles experiencing difficulty in making merging or weaving movements ♦ Evidence of unreported crashes such as damaged guardrail, skid marks, or tire tracks off of the pavement ♦ Pedestrians on roadway ♦ Pedestrian-vehicle conflicts	♦ Sight distance restrictions ♦ Pavement and shoulder conditions ♦ Signal visibility ♦ Signs, including speed limits ♦ Curb radii ♦ Pavement markings ♦ Lighting ♦ Driveway locations ♦ Fixed objects and roadside design

Table 2-3. Questions to Consider During the Field Observation.[7]

a. Are the crashes caused by physical conditions of the road or adjacent property, and can the condition be eliminated or corrected?
b. Is a blind corner responsible? Can it be eliminated? If not, can adequate measures be taken to warn the motorists?
c. Are the existing signs and pavement markings doing the job for which they were intended? Is it possible they are, in any way, contributing to causes of crashes, rather than contributing to crash prevention?
d. Is traffic properly channelized to minimize the occurrence of crashes?
e. Would crashes be prevented by the prohibition of any single traffic movement, such as a minor left-turn movement?
f. Can part of the traffic be diverted to other thoroughfares where the crash potentialities are not as great?
g. Are night crashes out of proportion to daytime crashes, based on traffic volume, indicating need for special nighttime protection, such as street lighting, signal control, or reflectorized signs or marking?
h. Do conditions show that additional traffic laws or selective enforcement are required?
i. Is there a need for supplemental studies of traffic movement, such as driver observance of existing control devices, speed studies of vehicles approaching the crash location, and others?
j. Is parking in the area contributing to crashes? If so, perhaps reduction of the width of approach lanes or sight obstructions in advance of the intersection resulting from the parking are causing the crashes.
k. Are there adequate advance warning signs of route changes so that the proper lanes may be chosen by approaching motorists well in advance of the area, thus minimizing the need for lane changing near the crash location?

Table 2-4. On-Site Observation Report. [7]

Location________________________________ Control______________________

Date__________________________________ Time________________________

Operational Checklist:	**No**	**Yes**
a. Do obstructions block the drivers' view of opposing vehicles?	______	______
b. Do drivers respond incorrectly to signals, signs, or other traffic control devices?	______	______
c. Do drivers have trouble finding the correct path through the locations?	______	______
d. Are vehicle speeds too high? Too low?	______	______
e. Are there violations of parking or other traffic regulations?	______	______
f. Are drivers confused about routes, street names, or other guidance information?	______	______
g. Can vehicle delay be reduced?	______	______
h. Are there traffic flow deficiencies or traffic conflict patterns associated with turning movements?	______	______
i. Would one-way operation make the location safer?	______	______
j. Is this volume of traffic causing problems?	______	______
k. Do pedestrian movements through the location cause conflicts?	______	______
l. Are there other traffic flow deficiencies or traffic conflict patterns?	______	______
Physical Checklist:		
a. Can sight obstructions be removed or lessened?	______	______
b. Are the street alignment or widths inadequate?	______	______
c. Are curb radii too small?	______	______
d. Should pedestrian crosswalks be relocated? Repainted?	______	______
e. Are signs inadequate as to usefulness, message, size, conformity, and placement? (See *MUTCD*)	______	______
f. Are signals inadequate as to placement, conformity, number of signal heads, or timing? (See *MUTCD*)	______	______
g. Are pavement markings inadequate as to their clearness or location?	______	______
h. Is channelization (island or paint markings) inadequate for reducing conflict areas, separating traffic flows, and defining movements?	______	______
i. Does the legal parking layout affect sight distance, through or turning vehicle paths, or traffic flow?	______	______
j. Do speed limits appear to be unsafe or unreasonable?	______	______
k. Is the number of lanes insufficient?	______	______
l. Is street lighting inadequate?	______	______
m. Are driveways inadequately designed or located?	______	______
n. Does the pavement condition (potholes, washboard, or slick surface) contribute to crashes?	______	______

Comments: ____

____

____

Collect Additional Field Data

The previous efforts identified potential trends; however, additional information was needed to better define the condition at the site. Table 2-5 lists supplemental traffic studies that can be considered to further define the nature of operational or safety problems, isolate the cause of the problem, and help identify appropriate solutions. The junior engineer decided to perform the following additional studies:

- ***Intersection Sight Distance.*** The available sight distance for each approach was measured. Because the junior engineer observed several large trucks on the east/west approaches, a combination truck was used in the analysis. Using the procedure for Intersection Sight Distance, Case D, the engineer used an 11.5-sec time gap (combination truck) with no adjustments (the intersection is level and does not have multiple lanes). The design speed was assumed to be 50 mph [80 km/h]. Therefore, the intersection sight distance for combination trucks is 845 ft [258 m]. For each corner, sight distances much greater than 845 ft [258 m] are present. The approaches are straight, on level grades, and with minimum vegetation growth or development near the intersection.
- ***Review of Signal Timing Plan.*** The review of the signal timing plan revealed that the signal was on a fixed-time cycle.

Table 2-5. Supplementary Engineering Studies.[7]

Supplementary Study	Purpose of Study	Symptom of Operational Study Problem that Indicates Study Needed
Capacity Studies	To determine operating condition and pinpoint bottlenecks.	♦ Congestion and delays
Travel Time and Delay Studies	To determine location and extent of delay and average travel speeds.	♦ Intersection congestion ♦ Other congestion along roadway ♦ Rear-end crashes during peak periods
Speed Studies	To determine actual vehicle speeds, actual speed profiles, and adequacy of legal and advisory speed limits.	♦ Extremely high or low speeds observed during on-site visits ♦ Run-off-road crashes ♦ Rear-end crashes near intersection
Traffic Conflict and Erratic Maneuver Studies	To supplement traffic crash data and identify potential crash problems.	♦ Hazardous driver actions observed during on-site visits ♦ Public complaints of safety problems not evident in crash data
Traffic Signal Studies	To determine need for and design of traffic signals; to identify improper phasing, timing, or interconnect strategy; and to identify unwarranted signals.	♦ Right-angle crashes at unsignalized intersections ♦ Excessive delay at Stop sign controlled intersections ♦ Excessive delay at existing signalized intersections
Sight Distance Studies	To determine adequacy of the length of highway visible to the driver.	♦ Rear-end crashes at horizontal curves, crest vertical curves, or decision points ♦ Right-angle crashes at uncontrolled intersections ♦ Turning crashes at intersection
Turning Radius Studies	To determine adequacy of existing curb radii.	♦ Sideswipe crashes involving vehicles traveling in opposite directions ♦ Rear-end crashes in right-turn lanes ♦ Evidence of large vehicles' encroachment on curb or shoulder
Skid Resistance Studies	To determine the coefficient of tire-pavement friction.	♦ Run-off-road or skidding crashes under wet-pavement conditions

Assess Situation and Select Treatments

The next series of steps assessed the condition at the site and selected appropriate safety treatment(s). The following steps were completed.

- ***Identify Crash Patterns and Conditions Present at the Site.*** The following crash patterns were identified:
  - Of the crashes, 61 percent occurred on wet pavement, and several narratives included comments regarding wet pavement.
  - Approximately one-third of the crashes involved left-turning vehicles and about one-half involved rear-end crashes.
  - Most crashes occurred during the day (i.e., nighttime crashes not an issue).
  - Less than 10 percent of the crashes involved DWI.
- ***Identify Potential Mitigation Measures.*** Suggested measures are contained in *Treatments for Crashes on Rural Two-Lane Highways in Texas*[2] along with other documents. The junior engineer identified potential treatments that would later be reviewed by a safety review team. The safety review team for the district has individuals with many different backgrounds, including both engineering and enforcement.
- ***Select Safety Treatment(s) for Site.*** The safety treatments selected for this site included the following:
  - Add left-turn bays on all approaches.
  - Retime signals to consider left-turn bays.
  - Resurface pavement (to improve skid resistance).
  - Repair pavement edge drop-offs.

Implement and Evaluate

The final elements in a safety study are to implement the selected improvements and, subsequently, to evaluate their effectiveness. The objective of an effectiveness evaluation is to compare the actual effects of the project with its predicted effects. Feedback from the evaluation of completed projects will enable the anticipated effects of planned projects to be more accurately quantified in the future. The junior engineer plans to compare the crash behavior after the treatments are installed with the data currently available.

Chapter 3
Design Elements

Contents:

Application 1
ISD, Case A

Overview

The sight triangles for an intersection with no control should allow the driver of a vehicle to see an approaching vehicle and have enough time to stop before reaching the intersection. Discussion on intersection sight distance is included in the *Urban Intersection Design Guide*, Chapter 3, Section 1 <link>.

Example at an Uncontrolled Location

Problem. Determine the sight triangles for all four legs of the intersection of Blythe and Franklin illustrated in Figure 3-1. There is no control at the intersection, therefore ISD, Case A is appropriate. Approach sight triangles are determined, but departure sight triangles are not determined for intersections with no control.

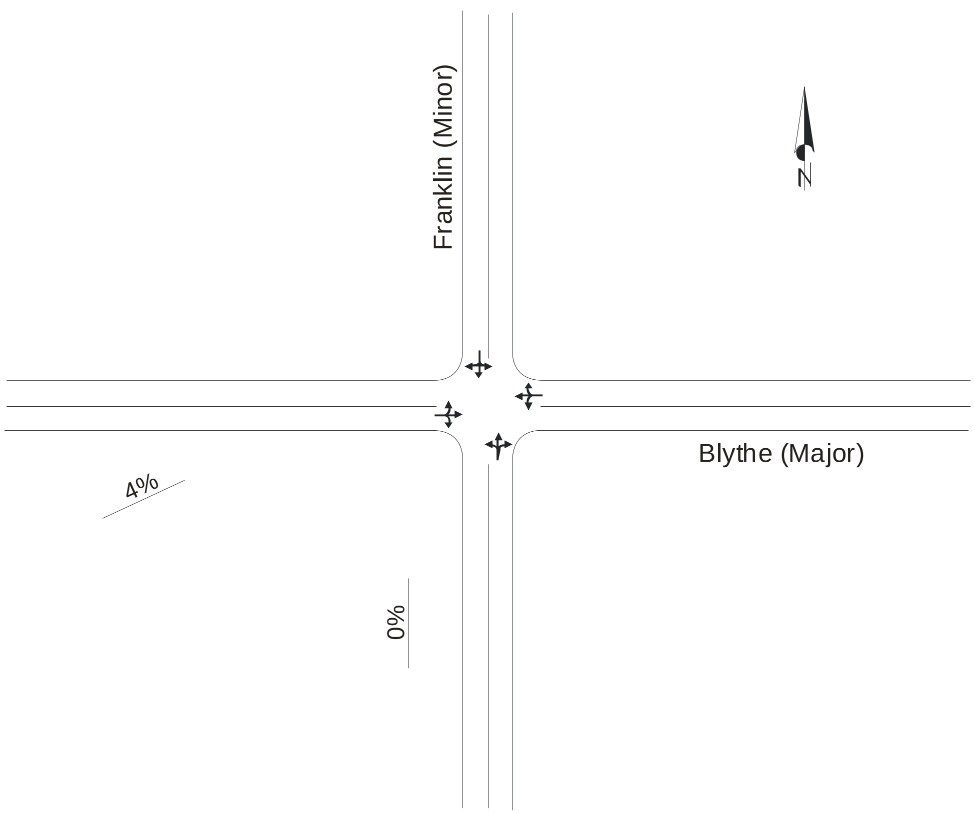


Figure 3-1. Blythe and Franklin Intersection.

Known Information. The information known for this site includes:

- Design speed on Franklin is 35 mph [56 km/h].
- Design speed on Blythe is 40 mph [64 km/h].

- Grade for Franklin is 0 percent.
- Grade for Blythe is 4 percent.

Solution. Following is the solution for this example:

- *Step 1: Identify needed adjustments.*

The grade for both northbound and southbound approaches on Franklin is 0 percent, which allows the use of *Green Book* Exhibit 9-51 (reproduced as Table 3-1). However, for Blythe, the approach is +4 percent, which is greater than the cutoff for using Exhibit 9-51 (3 percent grade). So, Exhibit 9-53 (reproduced as Table 3-2) must be used to determine the adjustment factor for this approach.

- *Step 2: Determine the sight triangle length for both approaches for Franklin.*

Since the design speed on Franklin is 35 mph [56 km/h], the length of the sight triangle leg "a" for both the northbound and southbound approaches is 165 ft [50 m], as shown in Table 3-1.

Table 3-1. Length of Sight Triangle Leg–Case A–No Traffic Control (Reproduction of *Green Book* Exhibit 9-51).

US Customary		Metric	
Design Speed (mph)	**Length of Leg (ft)**	**Design Speed (km/h)**	**Length of Leg (m)**
15	70	20	20
20	90	30	25
25	115	40	35
30	140	50	45
35	***165***	***60***	***55***
40	***195***	***70***	***65***
45	220	80	75
50	245	90	90
55	285	100	105
60	325	110	120
65	365	120	135
70	405	130	150
75	445		
80	485		

- *Step 3: Determine the sight triangle length for the eastbound approach for Blythe.*

For the eastbound approach, you need to find the adjustment factor for the sight distance from Table 3-2 and multiply that by the base sight distance length from Table 3-1. The adjustment factor for a +4 percent grade on a facility with a design speed of 40 mph [64 km/h] is 0.9, as highlighted on Table 3-2. The base length of the sight triangle leg "b" for this approach is 195 ft [59 m] as found in Table 3-1. Thus, the adjusted length of the sight triangle leg for the eastbound approach is as follows:

Adjusted sight triangle leg "b" length = base length × adjustment factor

Adjusted sight triangle leg "b" length = 195 ft [59 m] × 0.9 [0.9]

Adjusted sight triangle leg “b” length = 176 ft [54 m]

Table 3-2. Adjustment Factors for Sight Distance Based on Approach Grade (Reproduction of *Green Book* Exhibit 9-53).

US Customary														
Approach Grade (%)	**Design Speed (mph)**													
	15	20	25	30	35	***40***	45	50	55	60	65	70	75	80
-6	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.2
-5	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2
-4	1.0	1.0	1.0	1.1	1.1	***1.1***	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
-3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	1.0	1.0	***0.9***	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
Metric														
Approach Grade (%)	**Design Speed (km/h)**													
	20	30	40	50	60	***70***	80	90	100	110	120	130		
-6	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2		
-5	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2		
-4	1.0	1.0	1.0	1.1	1.1	***1.1***	1.1	1.1	1.1	1.1	1.1	1.1		
-3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0		
+4	1.0	1.0	1.0	1.0	0.9	***0.9***	0.9	0.9	0.9	0.9	0.9	0.9		
+5	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9		
+6	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9		

- ♦ *Step 4: Determine the sight triangle length for the westbound approach for Blythe.*

For the westbound approach, you also need to find the adjustment factor for the sight distance from Table 3-2 and multiply that by the base sight distance length from Table 3-1. The adjustment factor in this case is for a –4 percent grade. With a design speed of 40 mph [64 km/h] the adjustment factor is 1.1, as indicated by the highlighted text in Table 3-2. The base length of the sight triangle leg “b” for this approach is 195 ft [59 m] as found in Table 3-1. Thus, the adjusted length of the sight triangle leg for the eastbound approach is as follows:

Adjusted sight triangle leg “b” length = base length × adjustment factor

Adjusted sight triangle leg “b” length = 195 ft [59 m] × 1.1 [1.1]

Adjusted sight triangle leg “b” length = 215 ft [66 m]

- ♦ *Step 5: Illustrate findings.*

Figure 3-2 and Figure 3-3 illustrate the approach sight triangle legs as calculated above. Note that each triangle provides clear viewing of traffic approaching for adjoining legs of the intersection. Also note that the triangles are shifted depending on whether the driver is viewing traffic approaching from the left or the right. As noted previously, departure sight triangles are not determined for intersections with no control.

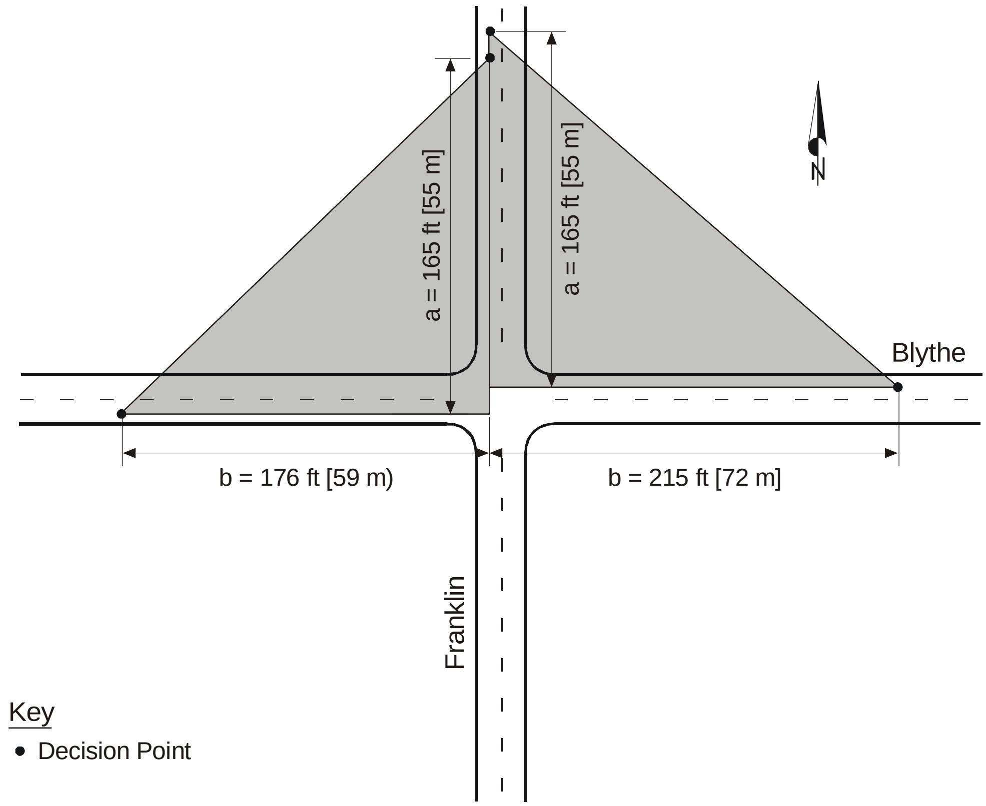


Figure 3-2. Case A – Sight Triangles for Southbound Approach.

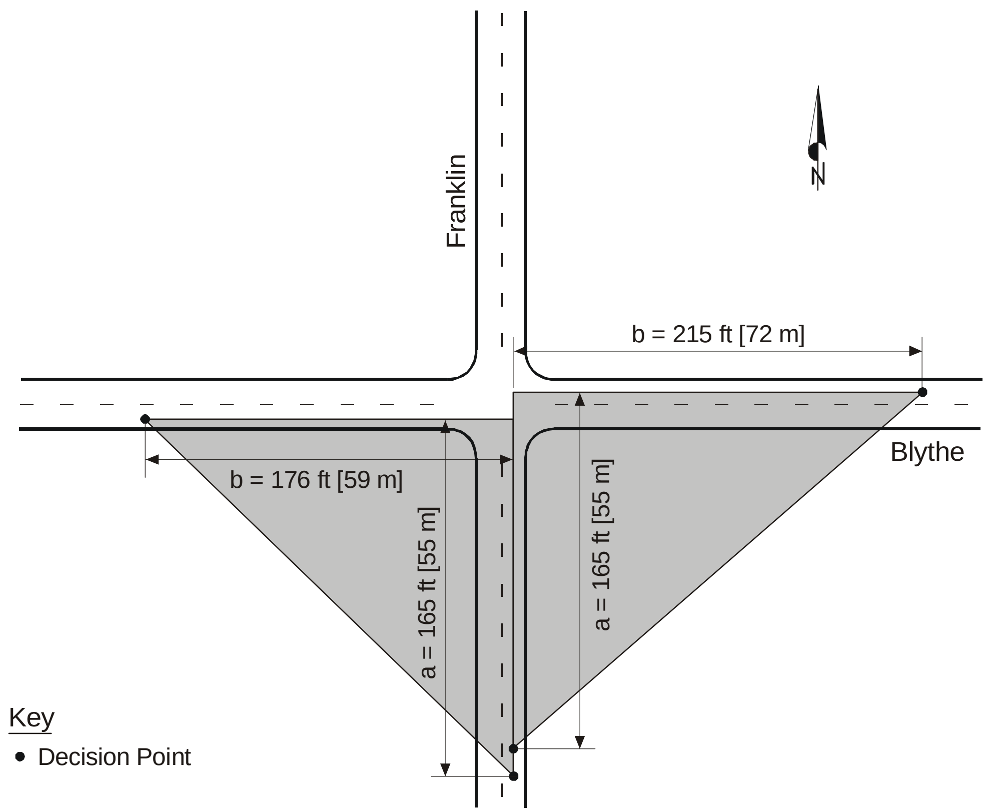


Figure 3-3. Case A – Sight Triangles for Northbound Approach.

Application 2
ISD, Case B1

Overview

The departure sight triangles for intersections with stop control on the minor road and left turns from a minor road to a major road should allow the driver of a vehicle to see approaching vehicles and choose gaps in the traffic that allow them to accelerate and complete a left turn without unduly interfering with major-road traffic operations. Discussion on intersection sight distance is included in the *Urban Intersection Design Guide,* Chapter 3, Section 1 <link>.

Single-Unit Truck Turning Left

Problem. Determine the required sight distance and departure sight triangles for a vehicle turning left from Forbes Boulevard onto Skinner Drive from either the northbound or southbound directions as illustrated in Figure 3-4. Traffic on Forbes is stop-controlled.

Because a large proportion of the vehicles in the light industrial area are single-unit trucks, that vehicle is used as the design vehicle.

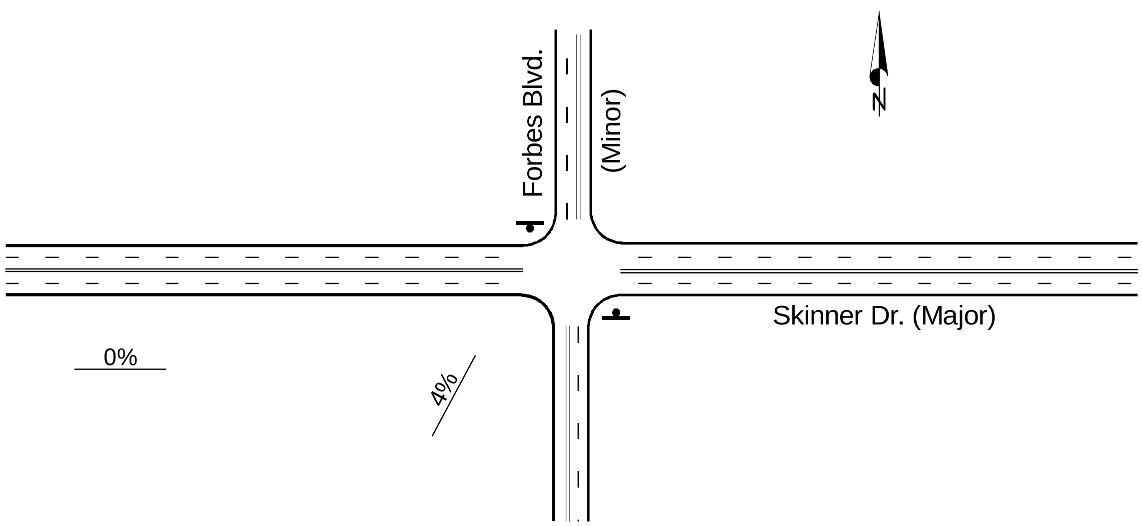


Figure 3-4. Forbes and Skinner Intersection.

Known Information. The information known for this site includes:

- Design speed on Skinner Drive is 40 mph [64 km/h].
- Grade for Forbes Blvd. is 4 percent.
- Grade for Skinner Drive is 0 percent.
- No medians are present on the approaches.
- Lane widths are 12 ft [3.7 m].

Solution. Following is the solution for this example:

- *Step 1: Identify needed adjustments.*

For Forbes, the southbound approach has a grade of +4 percent, which exceeds 3 percent, thereby requiring adjustments in the time gap. The northbound approach of –4 percent does not require adjustments in the time gap.

As shown in Figure 3-4, Skinner has two lanes in each direction, thereby requiring additional adjustments to the time gap listed in Table 3-3.

Table 3-3. Time Gap for Case B1–Left Turn from Stop (Reproduction of *Green Book* Exhibit 9-54).

Design Vehicle	Time gap (sec) at design speed of major road (t_g)
Passenger car	7.5
Single-unit truck	***9.5***
Combination truck	11.5

Note: Time gaps are for a stopped vehicle to turn right or left onto a two-lane highway with no median and grades 3 percent or less. The table values require adjustment as follows:

- For multilane highways: For left turns onto two-way highways with more than two lanes, add 0.5 second for passenger cars or ***0.7 second*** for trucks for each additional lane, from the left, in excess of one, to be crossed by the turning vehicle.
- For minor road approach grades: If the approach grade is an upgrade that exceeds 3 percent, add ***0.2 second*** for each percent grade for left turns.

- *Step 2: Determine the sight triangle length "a" for a left turn from the southbound approach of Forbes Blvd.*

Based on the *Green Book* procedure, the "a" length for the sight triangle for vehicles turning left is the following:

"a" leg length = distance between major-road travel way and front of vehicle
+ distance between front of vehicle and driver's eye
+ distance to middle of lane of interest

a_R = "a" leg length to vehicles approaching from the right

a_R = distance from major-road travel way and front of vehicle
+ distance between front of vehicle to driver's eye
+ 2.5 lane widths (crossing two lanes before merging with major road through traffic)

$a_R = 6.5$ ft [2.0 m] + 8 ft [2.4 m] + 2.5×12 ft [3.7 m]

$a_R = 45$ ft [14 m]

When practical, it is desirable to increase the distance from the edge of the major road to the front of the vehicle. The *Green Book* recommends using a 10-ft [3 m] dimension rather than 6.5 ft [2.0 m] for the distance between major road travelway and front of vehicle when available. In this example, that would bring the a_R dimension to 48 ft [14.6 m].

a_L = "a" leg length to vehicles approaching from the left

a_L = 6.5 ft [2.0 m] + 8 ft [2.4 m] + 0.5 × 12 ft [3.7 m]

a_L = 21 ft [6 m]

- *Step 3: Determine the sight triangle length "b" for a left turn from the southbound approach of Forbes Boulevard.*

The equation for the "b" leg length is as follows:

US Customary	Metric
$ISD = 1.47\ V_{major}t_g$	$ISD = 0.278\ V_{major}t_g$

The initial time gap to be used in this equation is 9.5 sec for single-unit trucks, as highlighted in Table 3-3. However, because the grade on the southbound approach is +4 percent, the t_g should be increased by 0.8 sec since +4 percent is greater than +3 percent (0.2 sec × 4 percent = 0.8 sec). Furthermore, because Skinner has two lanes in each direction, rather than one, the t_g should be increased by 0.7 sec for each additional lane the truck must cross. In this case, the adjustment factor is 0.7 sec since the truck must cross one additional lane.

Thus, the t_g value is as follows:

t_g = base t_g + grade adjustment + major road width adjustment

t_g = 9.5 sec = (4 percent × 0.2 sec) + (1 lane × 0.7 sec)

t_g = 9.5 sec + 0.8 sec + 0.7 sec

t_g = 11.0 sec

Therefore, the "b" leg length is as follows:

US Customary	Metric
$ISD = 1.47\ V_{major}t_g$	$ISD = 0.278\ V_{major}t_g$
ISD = 1.47 (40 mph) (11.0 sec)	ISD = 0.278 (70 km/h) (11.0 sec)
ISD = 647 ft	ISD = 241 m

- *Step 4: Determine the sight triangle length "a" for a left turn from the northbound approach of Forbes Blvd.*

As noted above, the a_R length for the sight triangle is 45 ft [14 m]. The time gap to be used in this equation is 9.5 sec for single-unit trucks (see Table 3-3). No adjustment for grade needs to be made for the northbound approach (it has a downgrade of 4 percent). The adjustment for the number of lanes remains the same, thereby making the adjusted time gap the following:

t_g = base t_g + major road width adjustment

t_g = 9.5 sec + 0.7 sec

$t_g = 10.2$ sec

The "b" leg length is calculated as follows:

US Customary	Metric
$ISD = 1.47\ V_{major}t_g$	$ISD = 0.278\ V_{major}t_g$
ISD = 1.47 (40 mph) (10.2 sec)	ISD = 0.278 (70 km/h) (10.2 sec)
ISD = 600 ft	ISD = 199 m

- *Step 5: Illustrate findings.*

Figure 3-5 and Figure 3-6 illustrate the resulting sight triangles for a left turn from the minor road (Forbes Blvd.) in the southbound and northbound directions, respectively.

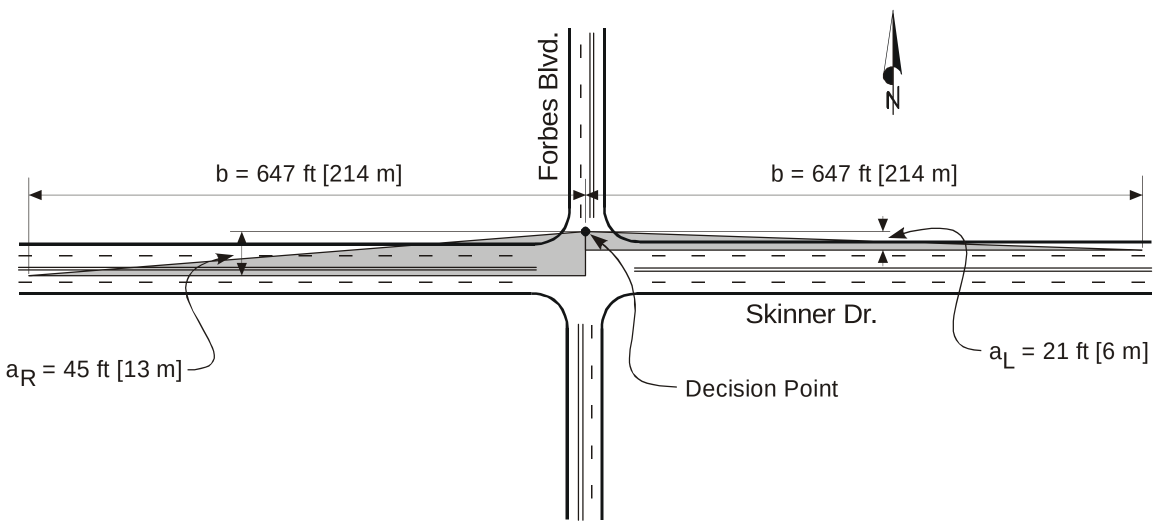


Figure 3-5. Case B1 – Sight Triangles for Left-Turn Movement from Southbound Minor Road for a Single-Unit Truck.

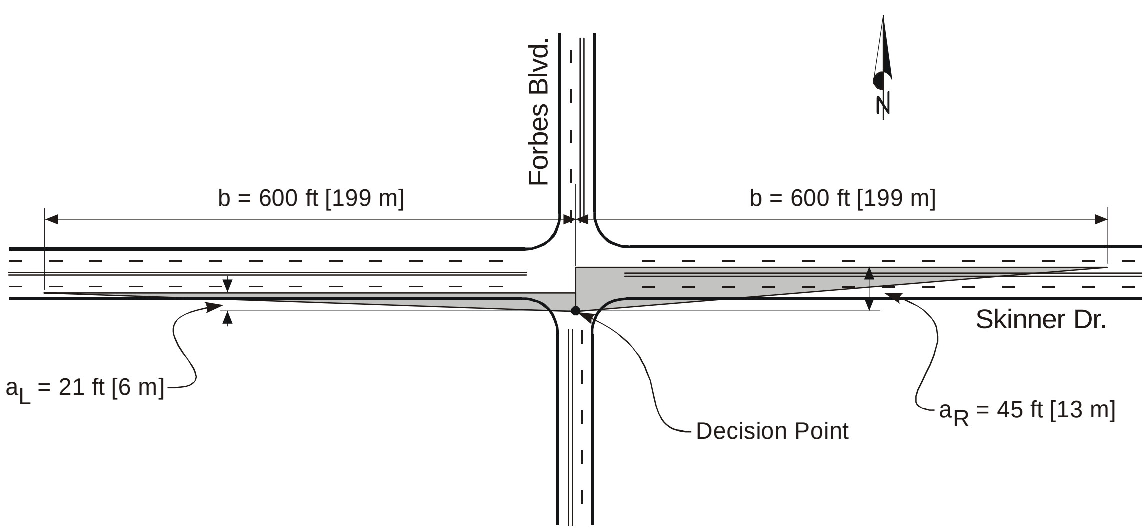


Figure 3-6. Case B1 – Sight Triangles for Left-Turn Movement from Northbound Minor Road for a Single-Unit Truck.

Application 3
ISD, Case B2

Overview

The departure sight triangles for intersections with stop control on the minor road and a right turn from the minor road are similar to the left-turn triangles except that the time gaps required can be reduced by 1 sec. Discussion on intersection sight distance is included in the *Urban Intersection Design Guide,* Chapter 3, Section 1 <link>.

Example 1: Passenger Car Turning Right

Problem. Determine the required sight distance and departure sight triangles for a passenger car turning right from Forbes Blvd. onto Skinner Drive from either the northbound or southbound directions (see Figure 3-7). Traffic on Forbes is stop-controlled.

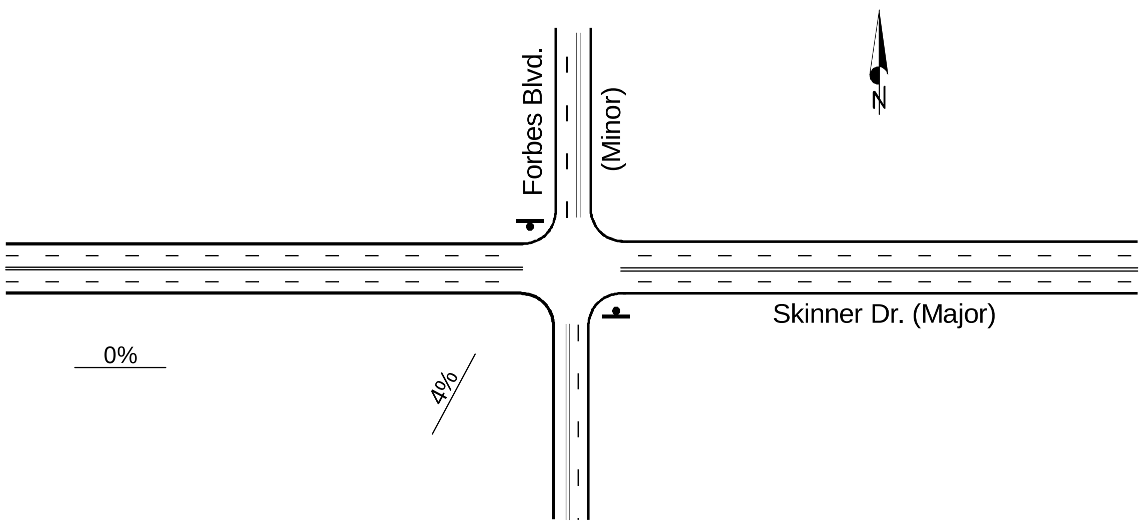


Figure 3-7. Forbes and Skinner Intersection.

Known Information. The information known for this site includes:

- Design speed on Skinner Drive is 40 mph [64 km/h].
- Grade for Forbes Blvd. is 4 percent.
- Grade for Skinner Drive is 0 percent.
- No medians are present on the approaches.
- Lane widths are 12 ft [3.7 m].

Solution. Following is the solution for this example:

- *Step 1: Identify needed adjustments.*

The potential adjustments are listed in Table 3-4. For Forbes, the grade of 4 percent exceeds 3 percent, thereby requiring adjustments in the time gap. The adjustment for multilane highways is only used when crossing the major road, not when turning right onto the major road. Therefore, no multilane adjustment is needed in this example.

Table 3-4. Time Gap for Case B2-Right Turn from Stop (Reproduction of *Green Book* Exhibit 9-57).

Design Vehicle	Time gap (sec) at design speed of major road (t_g)
Passenger car	***6.5***
Single-unit truck	8.5
Combination truck	***10.5***

Note: Time gaps are for a stopped vehicle to turn right onto or cross a two-lane highway with no median and grades 3 percent or less. The table values require adjustment as follows:

- For multilane highways: For crossing a major road with more than two lanes, add 0.5 second for passenger cars or 0.7 second for trucks for each additional lane to be crossed and for narrow medians that cannot store the design vehicle.
- For minor road approach grades: If the approach grade is an upgrade that exceeds 3 percent, add ***0.1 second*** for each percent grade.

- *Step 2: Determine the sight triangle length for a right turn from Forbes Blvd.*

Based on the AASHTO procedure, the “a” length for the sight triangle for vehicles turning right is the following:

“a” leg length = distance from major-road traveled way and front of vehicle
+ distance between front of vehicle and driver’s eye + 0.5 lane width

a = 6.5 ft [2.0 m] + 8 ft [2.4 m] + 0.5 × 12 ft [3.7 m]

a = 21 ft [6 m]

The equation for the “b” leg length is as follows:

US Customary	Metric
$ISD = 1.47\ V_{major} t_g$	$ISD = 0.278\ V_{major} t_g$

Southbound. The initial time gap to be used in this equation is 6.5 for passenger cars, as highlighted in Table 3-4. Because the grade on Forbes is +4 percent, the t_g should be increased by 0.4 sec (0.1 sec × 4 percent = 0.4 sec) since +4 percent is greater than +3 percent.

Thus, the t_g value is as follows:

t_g = base t_g + grade adjustment

t_g = 6.5 sec + 0.4 sec

t_g = 6.9 sec

Therefore, the "b" leg length is as follows:

US Customary	Metric
ISD = 1.47 $V_{major}t_g$	ISD = 0.278 $V_{major}t_g$
ISD = 1.47 (40 mph) (6.9 sec)	ISD = 0.278 (70 km/h) (6.9 sec)
ISD = 406 ft	ISD = 134 m

Northbound. Because northbound vehicles are on a –4 percent grade, no adjustment is required for the approach grade. Thus, the t_g value is 6.5 sec. Therefore, the "b" leg length is:

US Customary	Metric
ISD = 1.47 $V_{major}t_g$	ISD = 0.278 $V_{major}t_g$
ISD = 1.47 (40 mph) (6.5 sec)	ISD = 0.278 (70 km/h) (6.5 sec)
ISD = 382 ft	ISD = 126 m

♦ *Step 3: Illustrate findings.*

Figure 3-8 illustrates the resulting sight triangle for a right turn from the minor road (Forbes Blvd.) in the northbound or southbound directions.

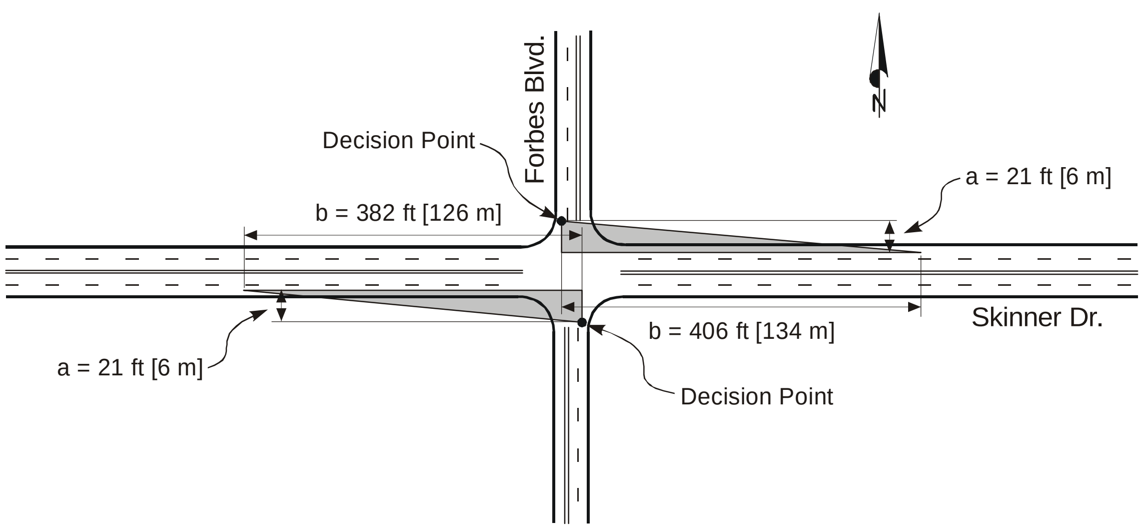


Figure 3-8. Case B2 – Sight Triangles for Right-Turning Passenger Cars.

Example 2: Combination Truck Turning Right on Northbound Approach

Problem. A processing plant is located south of the Forbes and Skinner intersection. The combination trucks that are leaving the intersection turn right toward a nearby interstate. Determine the required sight distance and departure sight triangles for a combination truck turning right from Forbes Blvd. onto Skinner Drive for the northbound direction.

Known Information. The information known for this site is listed above at the beginning of this application.

Solution. Following is the solution for this example:

- *Step 1: Identify needed adjustments.*

For northbound Forbes, the grade of –4 percent does not require an adjustment in the time gap. The adjustment for multilane highways is only used when crossing the major road, not when turning right onto the major road. Therefore, no multilane adjustment is needed in this example.

- *Step 2: Determine the sight triangle length for a right-turning combination truck from the northbound approach of Forbes Blvd.*

As noted above in Example 1, the "a" length for the sight triangle is 21 ft [6 m]. The time gap to be used in this equation is 10.5 sec for combination trucks, as highlighted in Table 3-4.

Because the grade on Forbes is –4 percent, the t_g used is the base t_g, 10.5 sec.

The "b" leg length is as follows:

US Customary	Metric
ISD = 1.47 $V_{major}t_g$	ISD = 0.278 $V_{major}t_g$
ISD = 1.47 (40 mph) (10.5 sec)	ISD = 0.278 (70 km/h) (10.5 sec)
ISD = 617 ft	ISD = 204 m

- *Step 3: Illustrate findings.*

Figure 3-9 illustrates the resulting sight triangles for a right turn for a combination truck from the minor road (Forbes Blvd.) in the northbound direction.

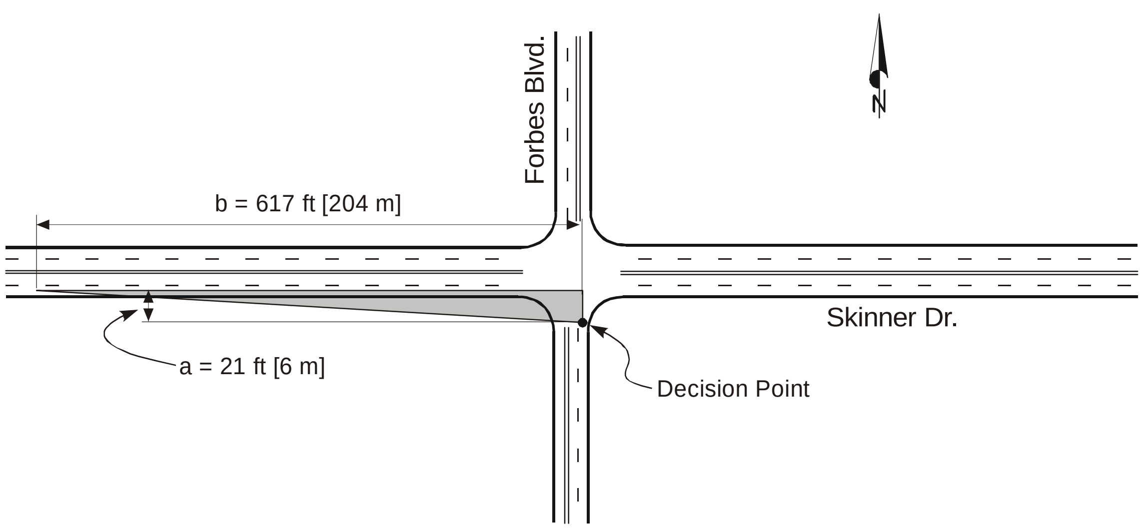


Figure 3-9. Case B2 – Sight Triangle for Right-Turning Combination Truck on Northbound Approach.

Application 4
ISD, Case B3

Overview

When vehicles are crossing the major road from a stop-controlled approach, the sight triangles provided for right and left turns should be sufficient; however, the following situation should be addressed if necessary:

- where left and/or right turns are not permitted from a particular approach and the crossing maneuver is the only legal maneuver;
- where the crossing vehicle would cross the equivalent width of six or more lanes;
- where substantial volumes of heavy vehicles cross the roadway and steep grades that might slow the vehicle while its back portion is still in the intersection are present on the departure roadway on the far side of the intersection.

Discussion on intersection sight distance is included in the *Urban Intersection Design Guide,* Chapter 3, Section 1 <link>.

Crossing a Six-Lane Highway

Problem. Determine the required sight distance and departure sight triangles for a passenger car crossing Cook Avenue from either the northbound or southbound direction of Sender Drive (see Figure 3-10).

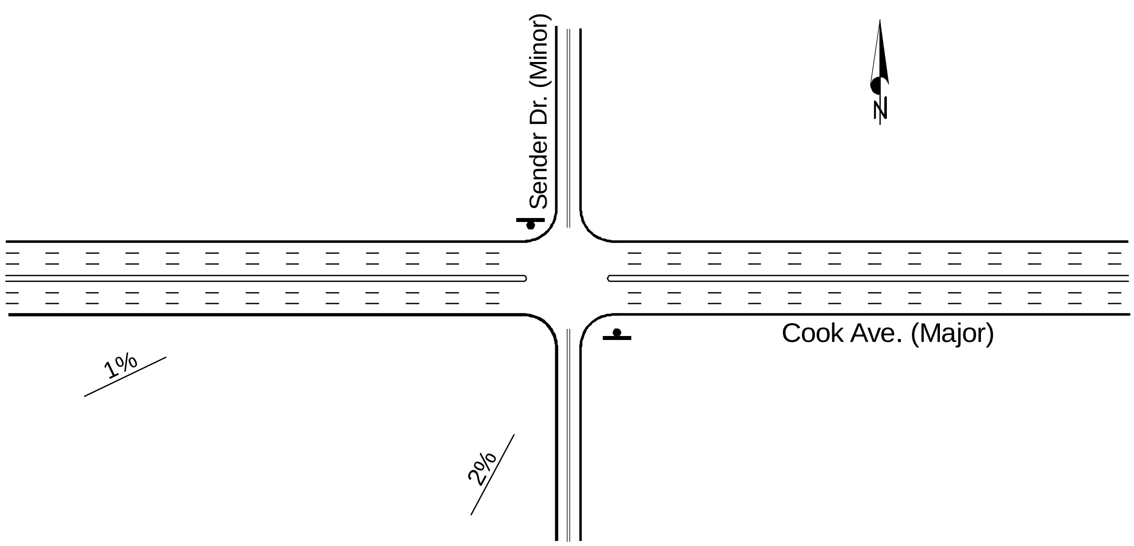


Figure 3-10. Cook Avenue and Sender Drive Intersection.

Known Information. The information known for this site includes:

- Design speed on Cook Avenue is 45 mph [72 km/h].
- Grade on Sender Drive is 2 percent.

- Grade on Cook Avenue is 1 percent.
- Median on Cook Avenue is 5 ft [1.5 m] wide.
- Lane widths are 12 ft [3.7 m].
- Median is not wide enough to store vehicles.

Solution. Following is the solution for this example:

- *Step 1: Identify needed adjustments.*

Both approaches on Sender are less than 3 percent; therefore, there is no need for adjustments in the time gap.

An adjustment will be needed due to the number of lanes to be crossed. The time gap values in Table 3-5 are for a two-lane highway. Because Cook has three lanes in each direction, the width of Cook will require an adjustment in the time gap value.

Table 3-5. Time Gap for Case B3 – Crossing Maneuver (Reproduction of *Green Book* Exhibit 9-57).

Design Vehicle	Time gap (sec) at design speed of major road (t_g)
Passenger car	***6.5***
Single-unit truck	8.5
Combination truck	10.5

Note: Time gaps are for a stopped vehicle to turn right onto or cross a two-lane highway with no median and grades 3 percent or less. The table values require adjustment as follows:

- For multilane highways: For crossing a major road with more than two lanes, add ***0.5 second*** for passenger cars or 0.7 second for trucks for ***each additional lane*** to be crossed and for ***narrow medians*** that cannot store the design vehicle.
- For minor road approach grades: If the approach grade is an upgrade that exceeds 3 percent, add 0.1 second for each percent grade.

- *Step 2: Determine the sight triangle length for a crossing maneuver from the southbound approach of Sender Drive.*

Based on the *Green Book* procedure, the "a" length for the sight triangle for vehicles crossing the road is the following:

"a" leg length = distance between major-road travel way and front of vehicle
+ distance between front of vehicle and driver's eye
+ distance to middle of lane of interest

The lane of interest for traffic approaching from the right would be to the far median lane for that direction and for traffic approaching from the left would be the near curb lane for that direction. Selecting these lanes rather than the lane farthest from the subject approach (i.e., the far curb lane) will result in more of the sight triangle covering the roadside rather than the roadway, which is the more critical concern. For this example, the minor road vehicle needs to cross 3.5 lanes to turn into the outside lane for the westbound approach.

a_R = "a" leg length to vehicles approaching from the right

a_R = distance from major-road travel way and front of vehicle
+ distance between front of vehicle and driver's eye

+ 3.5 (lane width) + median width

a_R = 6.5 ft [2.0 m] + 8 ft [2.4 m] + 3.5 (12 ft [3.7 m]) + 5 ft [1.5 m]

a_R = 62 ft [19 m]

a_L = “a” leg length to vehicles approaching from the left

a_L = distance from major-road travel way and front of vehicle
+ distance between front of vehicle and driver’s eye
+ 0.5 (lane width)

a_L = 6.5 ft [2.0 m] + 8 ft [2.4 m] + 0.5 (12 ft [3.7 m])

a_L = 21 ft [6 m]

The equation for the “b” leg length is as follows:

US Customary	Metric
ISD = 1.47 $V_{major}t_g$	ISD = 0.278 $V_{major}t_g$

The initial time gap to be used in this equation is 6.5 sec for passenger cars, as highlighted in Table 3-5. However, because the vehicle must cross more than two lanes, the multilane highway adjustment is needed. The adjustment adds 0.5 sec for each additional lane to be crossed and for narrow medians that cannot store the design vehicle. In this example, the passenger car must cross four additional lanes and the median. Because the approaches are on a less than 3 percent grade, no grade adjustment is needed.

Thus, the t_g value is as follows:

t_g = base t_g + grade adjustment + multilane highway adjustment (4 additional lanes + median)

t_g = 6.5 sec + 0.0 sec + 0.5 sec (5)

t_g = 9.0 sec

Therefore, the “b” leg length is as follows:

US Customary	Metric
ISD = 1.47 $V_{major}t_g$	ISD = 0.278 $V_{major}t_g$
ISD = 1.47 (45 mph) (9.0 sec)	ISD = 0.278 (80 km/h) (9.0 sec)
ISD = 595 ft	ISD = 200 m

- *Step 3: Determine the sight triangle length for a crossing maneuver from the northbound approach of Sender Drive.*

Sight distances for northbound Sender Drive are similar to southbound Sender Drive.

- *Step 4: Illustrate findings.*

Figure 3-11 illustrates the sight distances for the northbound minor road approach.

- *Step 5: Identify landscaping limits in median.*

Landscaping is being considered for the 5-ft [1.5 m] median, so the distance from the intersection where only low-growing plants should be used was needed. Also debated was whether the sight distance should be to the median lane of the westbound direction or to the curb lane since this will have an impact on the length of low-growing plants. The *Green Book* does not provide guidance on which major road lane to use in the analysis. All of the examples included in the *Green Book* use two-lane highways; therefore, a designer would need to decide which lane would be most appropriate. If we assume that the minor road driver is turning into the median lane, then the sketch shown in Figure 3-11 would reflect the needed sight distance for the intersection. The distance of low-growing plants would be 547 ft [167 m] as illustrated in Figure 3-12A. The landscape limit was also checked for the scenario of assuming the intersection sight distance is to a vehicle in the curb lane of the major road (see Figure 3-12B). In that scenario only 394 ft [120 m] of the median would need to have low-growing plants, and taller growing plants could be planted beyond that point. Assuming that the intersection sight distance is to the curb lane rather than the median lane could result in the minor road driver not seeing a major road vehicle in the median lane at the 595 ft [181 m] distance. Therefore, the engineers designing this intersection selected low-growing plants for the initial 547 ft [167 m] of the median.

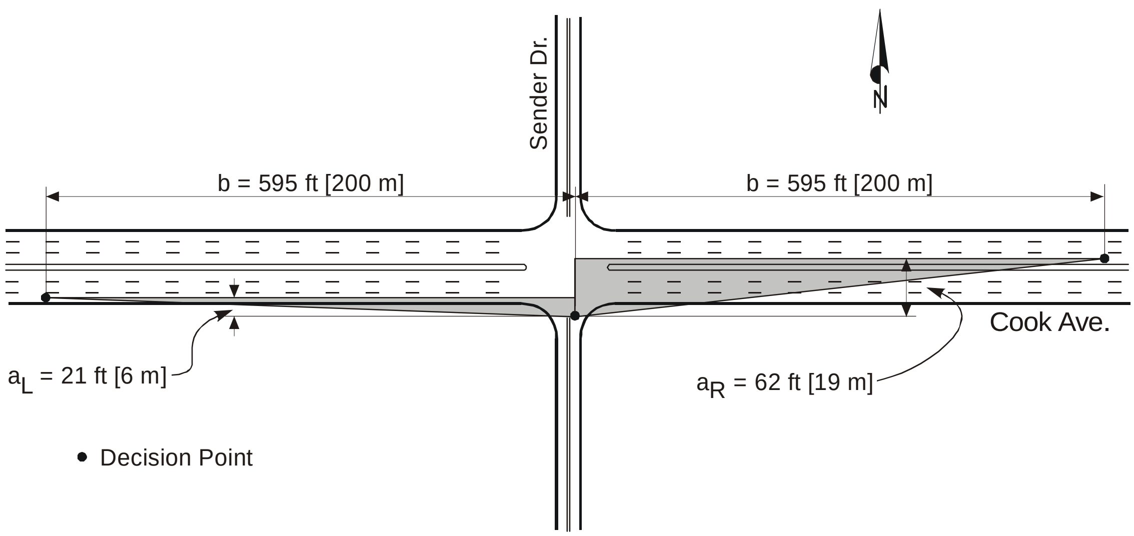
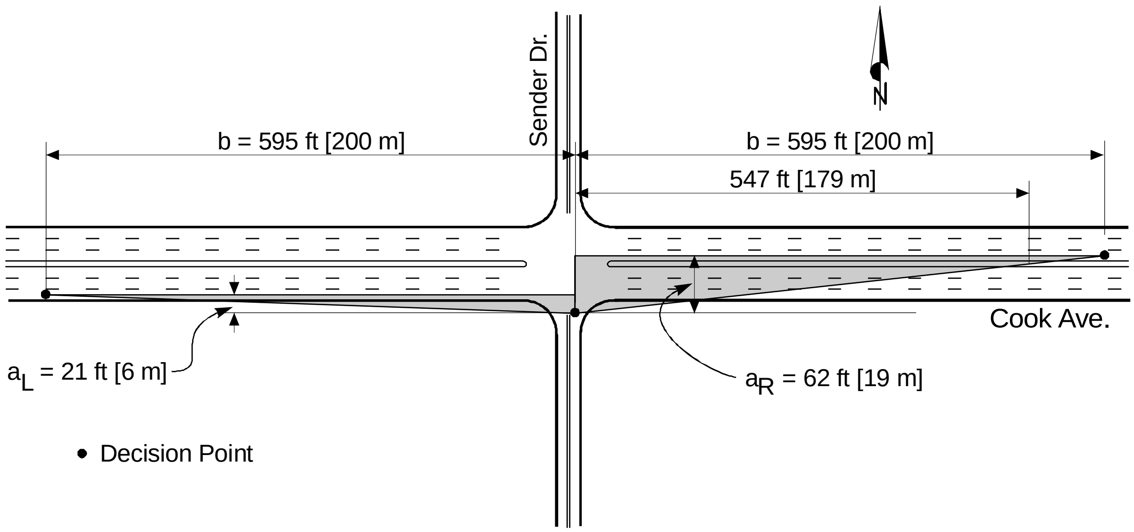
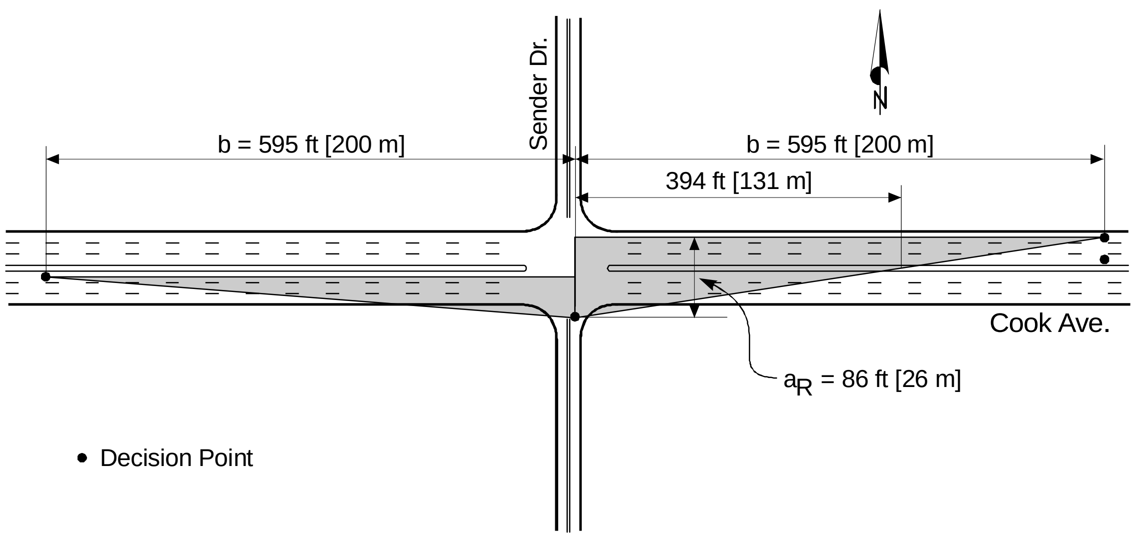


Figure 3-11. Case B3 – Sight Triangles for Crossing Maneuver from One of the Minor Road Approaches.



(A) Major-Road Vehicle in Far Median Lane



(B) Major-Road Vehicle in Far Curb Lane

Figure 3-12. Sight Distance Through Median.

Application 5
ISD, Case C1

Overview

The length of the leg of the sight triangle along the minor roadway for Case C (intersections with yield control on the minor road crossing maneuver from the minor road) is based on the same assumptions as those for Case A (see Chapter 3, Application 1 <link>) except that minor-road vehicles that do not stop are assumed to decelerate to 60 percent of the minor roadway design speed rather than 50 percent. Discussion on intersection sight distance is included in the *Urban Intersection Design Guide*, Chapter 3, Section 1 <link>.

Crossing at a Yield-Controlled Intersection

Problem. Determine the sight triangle for a crossing maneuver for a passenger car from the minor road at the intersection of Bluebonnet Lane and Cherry Grove (see Figure 3-13).

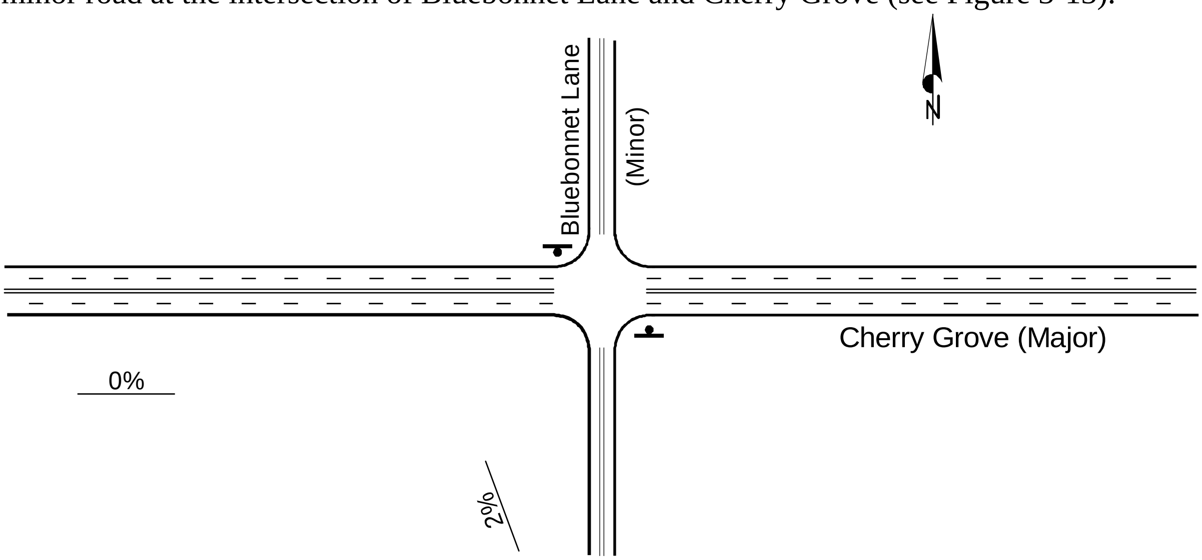


Figure 3-13. Cherry Grove and Bluebonnet Lane Intersection.

Known Information. The information known for this site includes:

- Design speed on Cherry Grove is 40 mph [64 km/h].
- Design speed on Bluebonnet Lane is 40 mph [64 km/h].
- Grade on Cherry Grove is 0 percent.
- Grade for Bluebonnet Lane is 2 percent.
- Cherry Grove is a four-lane undivided highway with a width of 48 ft [14.6 m].
- Bluebonnet Lane is a two-lane highway with no median.

Solution. The solution is provided below:

- *Step 1: Identify needed adjustments.*

The northbound grade for Bluebonnet Lane is 2 percent, which is less than the 3 percent threshold for adjusting the ISD values. Therefore, no adjustment for the approach grade is necessary.

- *Step 2: Determine the minor road leg length for a crossing maneuver for both approaches of Bluebonnet Lane.*

The minor road leg length is provided in Table 3-6 as a function of the design speed of the minor road. For a minor road with a design speed of 40 mph [64 km/h], the "a" length of the leg on the minor road is 235 ft [72 m].

- *Step 3: Determine the major road leg length for a crossing maneuver for both approaches of Bluebonnet Lane.*

The major road length is calculated using the following equation:

US Customary

$b = 1.47_{major}t_g$ Where $t_g = t_a + (w + L_a)/(0.88\ V_{minor})$

Metric

$b = 0.278\ V_{major}t_g$ Where $t_g = t_a + (w + L_a)/(0.167\ V_{minor})$

Where:

b = length of leg of sight triangle along the major road, ft or m

t_g = travel time to reach and clear the major road, sec

t_a = travel time to reach the major road from the decision point for a vehicle that does not stop, sec

w = width of the intersection to be crossed, ft or m

L_a = length of the design vehicle, ft or m

V_{minor} = design speed of minor road, mph or km/h

V_{major} = design speed of major road, mph or km/h

For this example, Cherry Grove is 48 ft [14.6 m] wide and the length of a passenger car is 19 ft [5.8 m]. The travel time is 4.9 sec for a 40 mph [64 km/h] design speed [or 5.1 sec for a 70 km/h design speed] as shown in Table 3-6.

Table 3-6. Case C1 – Crossing Maneuvers from Yield-Controlled Approaches – Length of Minor Road Leg and Travel Times (Reproduction of *Green Book* Exhibit 9-60).

US Customary				
	Minor-road approach		**Travel time (t_g) (seconds)**	
Design Speed (mph)	**Length of Leg[a] (ft)**	**Travel Time t_a[a,b] (seconds)**	**Calculated Value**	**Design Value[c,d]**
15	75	3.4	6.7	6.7
20	100	3.7	6.1	6.5
25	130	4.0	6.0	6.5
30	160	4.3	5.9	6.5
35	195	4.6	6.0	6.5
40	***235***	***4.9***	6.1	6.5
45	275	5.2	6.3	6.5
50	320	5.5	6.5	6.5
55	370	5.8	6.7	6.7
60	420	6.1	6.9	6.9
65	470	6.4	7.2	7.2
70	530	6.7	7.4	7.4
75	590	7.0	7.7	7.7
80	660	4.3	7.9	7.9
Metric				
	Minor-road approach		**Travel time (t_g) (seconds)**	
Design Speed (km/h)	**Length of Leg[a] (m)**	**Travel Time t_a[a,b] (seconds)**	**Calculated Value**	**Design Value[c,d]**
20	20	3.2	7.1	7.1
30	30	3.6	6.2	6.5
40	40	4.0	6.0	6.5
50	55	4.4	6.0	6.5
60	65	4.8	6.1	6.5
70	***80***	***5.1***	6.2	6.5
80	100	5.5	6.5	6.5
90	115	5.9	6.8	6.8
100	135	6.3	7.1	7.1
110	155	6.7	7.4	7.4
120	180	7.0	7.7	7.7
130	205	7.4	8.0	8.0

[a] For minor-road approach grades that exceed 3 percent, multiply the distance or the time in this table by the appropriate adjustment factor.

[b] Travel time applies to a vehicle that slows before crossing the intersection but does not stop.

[c] The value of t_g should equal or exceed the appropriate time gap for crossing the major road from a stop-controlled approach.

[d] Values shown are for a passenger car crossing a two-lane highway with no median and grades 3 percent or less.

Thus, the travel time to cross Cherry Grove is:

US Customary

$t_g = t_a + (w + L_a)/(0.88\ V_{Minor})$

$t_g = 4.9\ sec + (48\ ft + 19\ ft)/(0.88)\ (40\ mph)$

$t_g = 6.80\ sec$

Metric

$t_g = t_a + (w + L_a)/(0.167\ V_{Minor})$

$t_g = 5.1\ sec + (14.4\ m + 5.8\ m)/(0.167)\ (70\ km/h)$

$t_g = 6.83\ sec$

The value for t_g should be checked to ensure that its value meets or exceeds t_g for a stop-controlled approach (note c from Table 3-6). The values for t_g for a stop-controlled approach are in the *Green Book* Exhibit 9-57 and reproduced as Table 3-7. From Table 3-7, t_g for a passenger car is 6.5 sec. The adjustment for crossing a major road with more than two lanes is to add 0.5 seconds for each lane. Therefore:

$t_g = 6.5\ sec + 2 \times 0.5\ sec$

$t_g = 7.5\ sec$

Table 3-7. Time Gap for Case B2 – Right Turn from Stop (Reproduction of *Green Book* Exhibit 9-57).

Design Vehicle	Time gap (sec) at design speed of major road (t_g)
Passenger car	***6.5***
Single-unit truck	8.5
Combination truck	10.5

Note: Time gaps are for a stopped vehicle to turn right onto or cross a two-lane highway with no median and grades 3 percent or less. The table values require adjustment as follows:

- *For multilane highways:* For crossing a major road with more than two lanes, add ***0.5 second*** for passenger cars or 0.7 second for trucks for each additional lane to be crossed and for narrow medians that cannot store the design vehicle
- *For minor road approach grades:* If the approach grade is an upgrade that exceeds 3 percent, add 0.1 seconds for each percent grade.

Because t_g for a stop-controlled approach (7.5 sec) exceeds t_g for yield control (6.8 sec) at the 40 mph [64 km/h] design speed, the 7.5 sec should be used to determine the sight distance along the major road (Cherry Grove). As a result, the major road leg length is as follows:

US Customary	Metric
$b = 1.47\ V_{major} t_g$	$b = 0.278\ V_{major} t_g$
$b = 1.47\ (40\ mph)\ (7.5\ sec)$	$b = 0.278\ (70\ km/h)\ (7.5\ sec)$
$b = 441\ ft$	$b = 146\ m$

- *Step 4: Illustrate findings.*

Figure 3-14 illustrates the resulting sight triangles for a crossing maneuver from the minor road for the northbound direction from a yield control. The southbound direction would have similar sight distances.

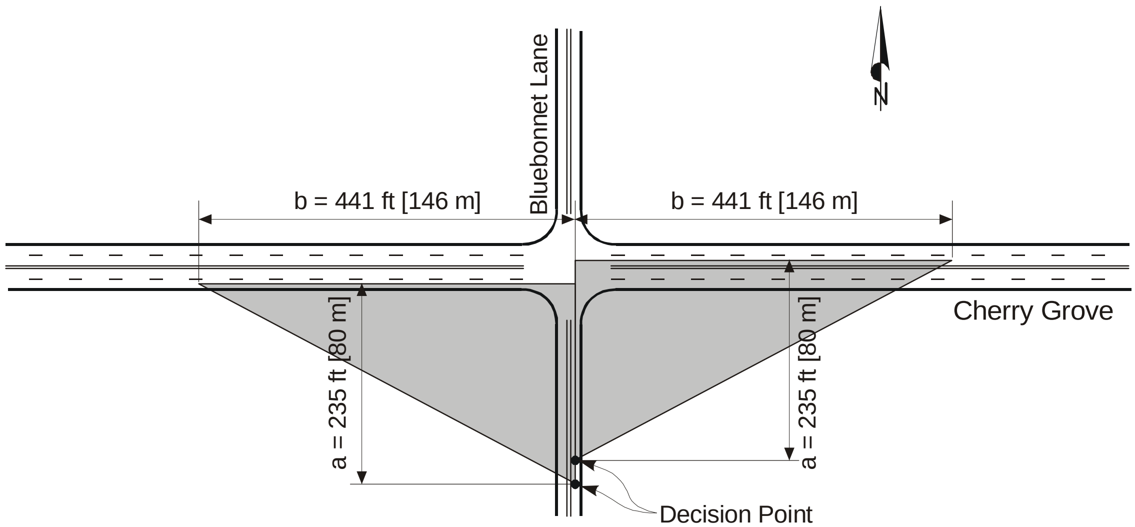


Figure 3-14. Case C1 – Sight Triangles for Crossing Maneuver from Minor Road with Yield Control for Northbound Approach (Southbound Would Be Similar).

Application 6
ISD, Case C2

Overview

Discussion on intersection sight distance is included in the *Urban Intersection Design Guide,* Chapter 3, Section 1 <link>. This application presents an example for Case C2, intersections with yield control on the minor road, left or right turn from the minor road.

Drivers approaching Yield signs are permitted to enter or cross the major road without stopping, if there are no potentially conflicting vehicles on the major road. Figure 3-1 (A) of the *Urban Intersection Design Guide* <link> shows an illustration of the approach sight triangles. Per the *Green Book,* the length of the leg of the approach sight triangle along the minor roadway to accommodate left and right turns without stopping is 82 ft [25 m]. This is based on the assumption that drivers will slow to a turning speed of 10 mph [16 km/h].

The length of the leg of the approach sight triangle along the major roadway is shown in Table 3-8 for passenger cars. The lengths are slightly longer than the values for the stop-controlled case. The longer sight distances represent additional travel time needed at a yield-controlled intersection (3.5 sec) minus the lower acceleration time needed (3.0 sec) since the turning vehicle is accelerating from 10 mph [16 km/h] rather than from a stop condition. The net is a 0.5 sec increase in travel time.

Departure sight triangles like those used in the stop-controlled cases should also be provided to accommodate vehicles that have stopped to traffic. However, since approach sight triangles for turning maneuvers at yield-controlled approaches are larger than the departure sight triangles used at stop-controlled intersections, no specific check of departure sight triangles at yield-controlled intersections should be needed.

Table 3-8. Intersection Sight Distance – Case C2 – Left or Right Turn at Yield-Controlled Intersection (Reproduction of *Green Book* Exhibit 9-64).

US Customary			
		Length of leg, Passenger cars	
Design Speed (mph)	**Stopping Sight Distance (ft)**	**Calculated (ft)**	**Design (ft)**
15	80	176.4	180
20	115	235.2	240
25	155	294.0	295
30	200	352.8	355
35	250	411.6	415
40	305	470.4	***475***
45	360	529.2	530
50	425	588.0	590
55	495	646.8	650
60	570	705.6	710
65	645	764.4	765
70	730	823.2	825
75	820	882.0	885
80	910	940.8	945
Metric			
		Length of leg, Passenger cars	
Design Speed (km/h)	**Stopping Sight Distance (m)**	**Calculated (m)**	**Design (m)**
20	20	44.5	45
30	35	66.7	70
40	50	89.0	90
50	65	111.2	115
60	85	133.4	135
70	105	155.7	***160***
80	130	177.9	180
90	160	200.2	205
100	485	222.4	225
110	220	244.6	245
120	250	266.9	270
130	285	289.1	290
Note: Intersection sight distance shown is for a passenger car making a right or left turn without stopping onto a two-lane road.			

Single-Unit Truck Turning at a Yield-Controlled Intersection

Problem. Determine the sight triangle for a southbound left-turn vehicle for a single-unit truck at the intersection of Bluebonnet Lane and Cherry Grove (see Figure 3-15).

Known Information. The information known for this site includes:

- Design speed on Cherry Grove is 40 mph [64 km/h].
- Design speed on Bluebonnet Lane is 40 mph [64 km/h].
- Grade on Cherry Grove is 0 percent.

- Grade for Bluebonnet Lane is 2 percent.
- Cherry Grove is a four-lane undivided highway with a width of 48 ft [14.6 m].
- Design vehicle for left turns on the southbound approach is a single-unit truck.

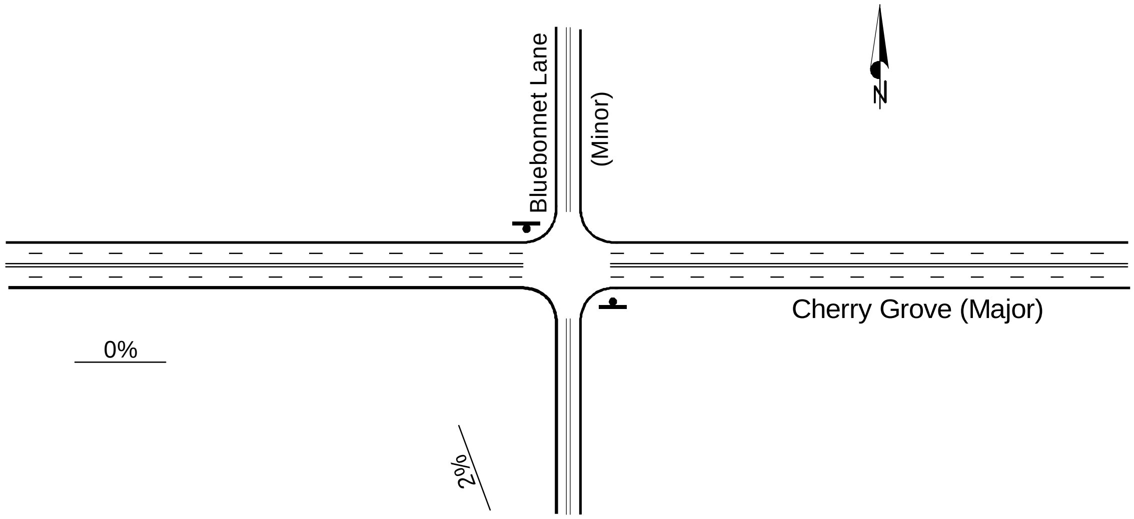


Figure 3-15. Cherry Grove and Bluebonnet Lane Intersection.

Solution. The solution is provided below:

- *Step 1: Identify needed adjustments.*

The approach grades are less than 3 percent so no adjustments are needed for grade. The vehicle is crossing more than one lane during the left turn so an adjustment for number of lanes will be needed.

- *Step 2: Determine the minor road leg length.*

The minor road leg length is 82 ft [25 m] as stated in the *Green Book*.

- *Step 3: Determine the major road leg length for a left-turning truck onto Cherry Grove from Bluebonnet Lane.*

When the major road is a two-lane highway and the vehicle is a passenger car, then the values in Table 3-8 can be used. (If the major road in this example had two lanes, then the sight distance would have been 475 ft [145 m] as shown in Table 3-8.) Because the major road is a multilane highway, an adjustment is needed to the travel time. The major road length is calculated using the same equations as used for determining the distance for a left turn from a minor road at a stop-controlled intersection:

US Customary	Metric
$b = 1.47\ V_{major} t_g$	$b = 0.278_{major} t_g$

The base value for t_g for a single-unit truck is 10.0 sec as shown in Table 3-9.

Table 3-9. Time Gap for Case C2 – Left or Right Turn (Reproduction of *Green Book* Exhibit 9-63).

Design Vehicle	Time gap (sec) at design speed of major road (t_g)
Passenger car	8.0
Single-unit truck	***10.0***
Combination truck	12.0

Note: Time gaps for a vehicle to turn right or left onto a two-lane highway with no median. The table values require adjustments for multilane highways as follows:

- For left turns onto two-way highways with more than two lanes, add 0.5 second for passenger cars and ***0.7 second*** for trucks for each additional lane, from the left, in excess of one, to be crossed by the turning vehicle.
- For right turns, no adjustment is necessary.

Recall that we need to make a multilane roadway adjustment to this travel time. In the case of Cherry Grove, in order to turn left the single-unit truck will have to cross one additional lane when compared to a two-lane highway. Thus, the adjustment for the t_g is as follows:

$t_g = t_g$ (base) + multilane adjustment

$t_g = 10.0$ sec + 0.7 sec

$t_g = 10.7$ sec

As a result, the major road leg length is as follows:

US Customary	Metric
$b = 1.47\ V_{major}\ t_g$	$b = 0.278\ V_{major}\ t_g$
b = 1.47 (40 mph) (10.7 sec)	b= 0.278 (70 km/h) (10.7 sec)
b = 629 ft	b = 208 m

- *Step 4: Illustrate findings.*

Figure 3-16 illustrates the resulting sight triangles for a left-turn, single-unit truck from the minor road for the southbound direction at a yield-controlled intersection.

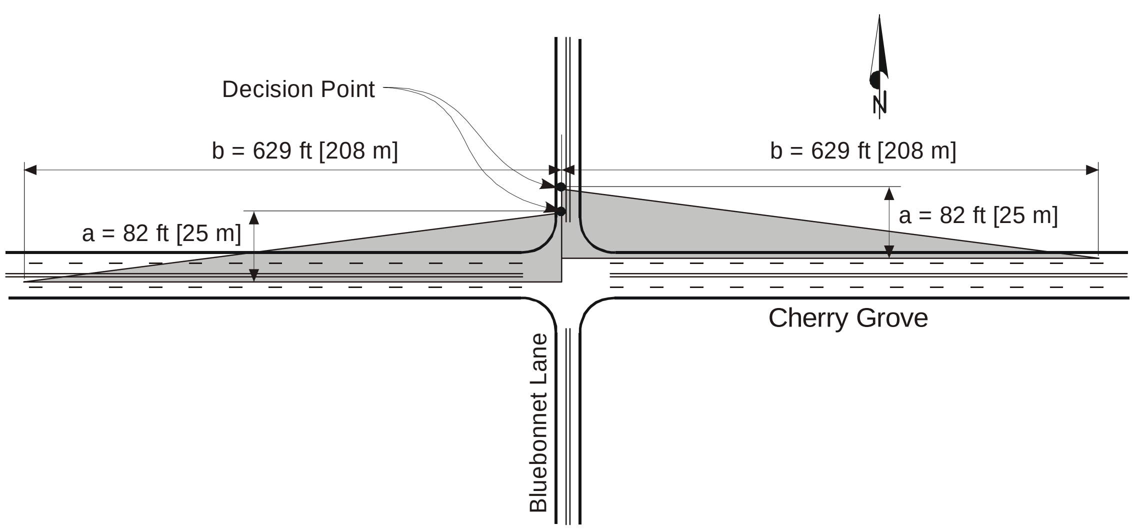


Figure 3-16. Case C2 – Sight Triangles for Left-Turn, Single-Unit Truck from Minor Road with Yield Control.

Application 7

ISD, Case D

Overview

Generally, there are no required sight triangles for signalized intersections, although the first vehicle at one approach should be visible to the first vehicles on all the other approaches, and left-turning vehicles should have sufficient sight distance to select gaps in oncoming traffic and complete their left turns.

If the signal will be placed on flashing mode (yellow for the major roadway and red for the minor roadway) then the appropriate sight triangles for stop control should be provided on the minor road approaches.   If right turns on red are allowed, sight triangles to the left should be provided from each approach.

Discussion on intersection sight distance is included in the *Urban Intersection Design Guide*, Chapter 3, Section 1 <link>.

Example 1: Sight Distance for Flashing Operations

Problem. The intersection of Jersey and Brighton will be signalized and will be placed in flashing mode at night, with Jersey having a flashing yellow and Brighton have a flashing red. Determine the sight triangles for a left turn for a passenger car from Brighton onto Jersey from both the northbound and southbound directions. Figure 3-17 is a schematic of the intersection.

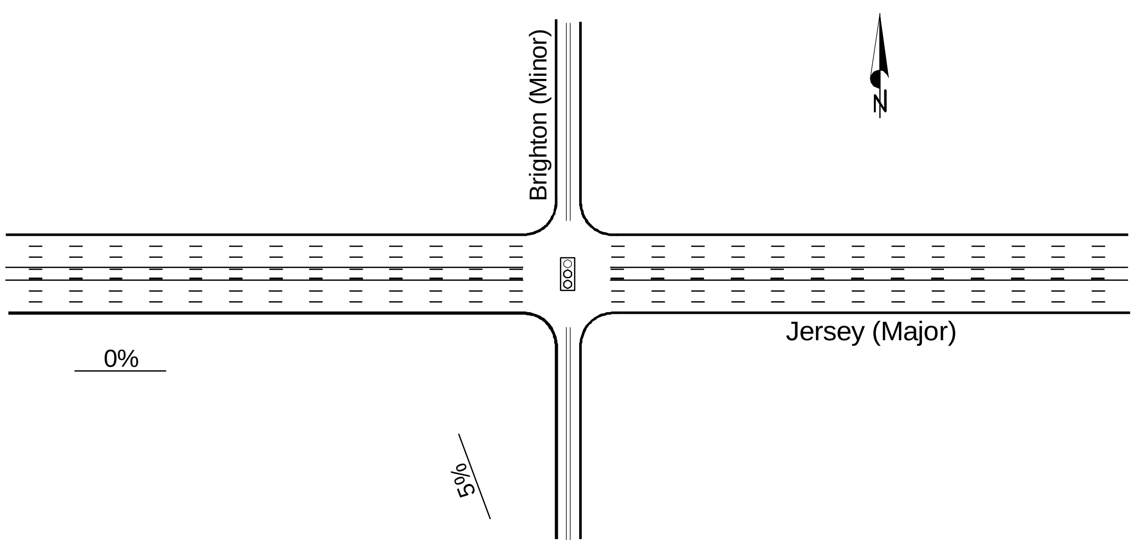


Figure 3-17. Jersey and Brighton Intersection.

Known Information. The following information is known about the site:

- Design speed on Jersey is 45 mph [72 km/h].
- Design speed on Brighton is 30 mph [48 km/h].

- Grade for Jersey is 0 percent.
- Grade for Brighton is 5 percent.
- Jersey has six lanes and a two-way left-turn lane (TWLTL) that is 14 ft [4.3 m] wide.
- Brighton has two lanes and no median.
- Lane widths are 12 ft [3.7 m] on both streets.

Solution. The solution is provided below:

- *Step 1: Identify needed adjustments.*

For northbound Brighton, the grade is +5 percent, which exceeds 3 percent, thereby requiring adjustments in the time gap.

- *Step 2: Determine the minor road sight triangle length for a left turn from northbound Brighton.*

Based on the *Green Book* procedure, the "a" length for the sight triangle is the following:

"a" leg length = distance between major-road travel way and front of vehicle
+ distance between front of vehicle and driver's eye
+ distance to middle of lane of interest

a_L = "a" leg length to vehicles approaching from the left

a_L = distance from major-road traveled way
+ distance from front of the vehicle to the driver's eye
+ 0.5 lane width

a_L = 6.5 ft [2.0 m] + 8 ft [2.4 m] + 0.5 × 12 ft [3.7 m]
a_L = 21 ft [6 m]

a_R = "a" leg length to vehicles approaching from the right

a_R = distance from major-road traveled way
+ distance from front of the vehicle to the driver's eye
+ 3.5 lane width + median

a_R = 6.5 ft [2.0 m] + 8 ft [2.4 m] + 3.5 × 12 ft [3.7 m] + 14 ft [4.3 m]
a_R = 71 ft [22 m]

- *Step 3: Determine the major road sight triangle length for a left turn from northbound Brighton.*

The equation for the "b" leg length is as follows:

US Customary: $ISD = 1.47\ V_{major}t_g$

Metric: $ISD = 0.278\ V_{major}t_g$

The initial time gap to be used in this equation is 7.5 for passenger cars, as highlighted in Table 3-10. The necessary adjustment for the grade is 0.2 sec for each percent grade (5 percent). Thus, the adjustment is 1.0 sec. Because Jersey has three lanes in each direction and a left-turn lane, the t_g should be increased by 0.5 sec for each additional lane the car must cross. In this case, the adjustment factor is 1.5 sec since the vehicle must cross three additional lanes.

Table 3-10. Time Gap for Case B1 – Left Turn from Stop (Reproduction of *Green Book* Exhibit 9-54).

Design Vehicle	Time gap (sec) at design speed of major road (t_g)
Passenger car	***7.5***
Single-unit truck	9.5
Combination truck	11.5

Note: Time gaps are for a stopped vehicle to turn right or left onto a two-lane highway with no median and grades 3 percent or less. The table values require adjustment as follows:

- For multilane highways: For left turns onto two-way highways with more than two lanes, add ***0.5 second*** for passenger cars or 0.7 second for trucks for each additional lane, from the left, in excess of one, to be crossed by the turning vehicle.
- For minor road approach grades: If the approach grade is an upgrade that exceeds 3 percent, add ***0.2 second*** for each percent grade for left turns.

Thus, the t_g value is as follows:

t_g = base t_g + grade adjustment + major road width adjustment

t_g = 7.5 sec + 1.0 sec (grade adjustment) + 1.5 sec (width adjustment)

t_g = 10.0 sec

Therefore, the "b" leg length is as follows:

US Customary	Metric
$ISD = 1.47\ V_{major}t_g$	$ISD = 0.278\ V_{major}t_g$
ISD = 1.47 (45 mph) (10.0 sec)	ISD = 0.278 (70 km/h) (10.0 sec)
ISD = 662 ft	ISD = 195 m

- *Step 4: Illustrate findings.*

Figure 3-18 illustrates the sight triangles for a left turn from the minor road (Brighton) in the northbound direction.

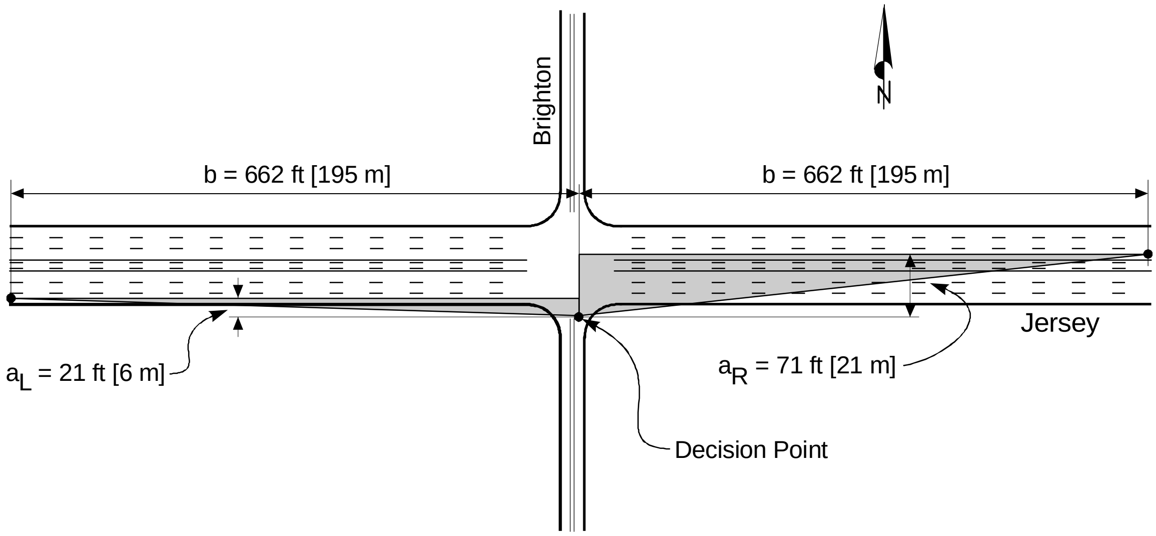


Figure 3-18. Case D – Sight Triangles for Left Turn from Minor Road with Traffic Signal Control in Flashing Mode, Northbound Approach.

- ♦ *Step 5: Determine the major road sight triangle for a left turn from southbound Brighton.*

A similar method to that used for northbound Brighton is used. Because the approach grade is –5 percent, however, the grade adjustment for t_g must be re-examined. No grade adjustment is necessary because it is a downgrade.

Thus, the t_g value is as follows:

t_g = base t_g + major road width adjustment

t_g = 7.5 sec + 1.5 sec (width adjustment)

t_g = 9.0 sec

Therefore, the "b" leg length is as follows:

US Customary	Metric
ISD = 1.47 $V_{major}t_g$	ISD = 0.278 $V_{major}t_g$
ISD = 1.47 (45 mph) (9.0 sec)	ISD = 0.278 (70 km/h) (9.0 sec)
ISD = 595 ft	ISD = 175 m

- ♦ *Step 6: Illustrate findings.*

Figure 3-19 illustrates the sight triangles for a left turn from the minor road (Brighton) in the southbound direction.

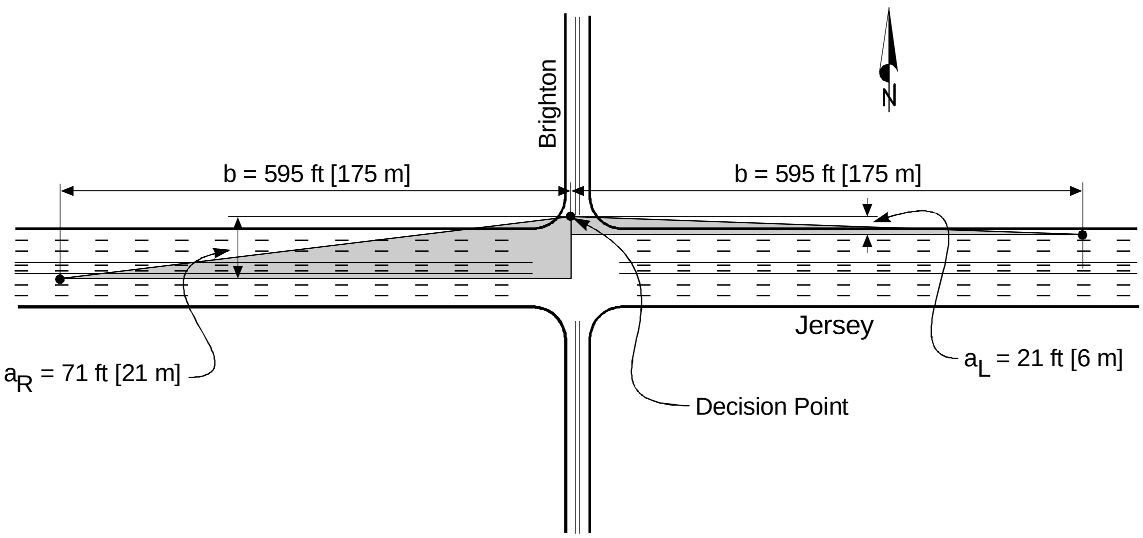


Figure 3-19. Case D – Sight Triangle for Left Turn from Minor Road with Traffic Signal Control in Flashing Mode, Southbound Approach.

Example 2: Sight Distance for Right Turn on Red

Problem. Right turns on red are allowed at the intersection of Vista and Fourth Street. Determine the sight triangles for a right turn for a passenger car from Vista onto Fourth Street for both the northbound and southbound directions. Figure 3-20 shows a schematic of the intersection.

Known Information. The following information is known about the site:

- Design speed on Fourth Street is 50 mph [80 km/h].
- Design speed on Vista is 30 mph [48 km/h].
- Grade for Fourth Street is 1 percent.
- Grade for Vista is 2 percent.
- Fourth Street has four lanes and a two-way left-turn lane.
- Vista has two lanes and no median.
- The width of the two-way left-turn lane is 14 ft [4.3 m].
- Lane widths are 12 ft [3.7 m].

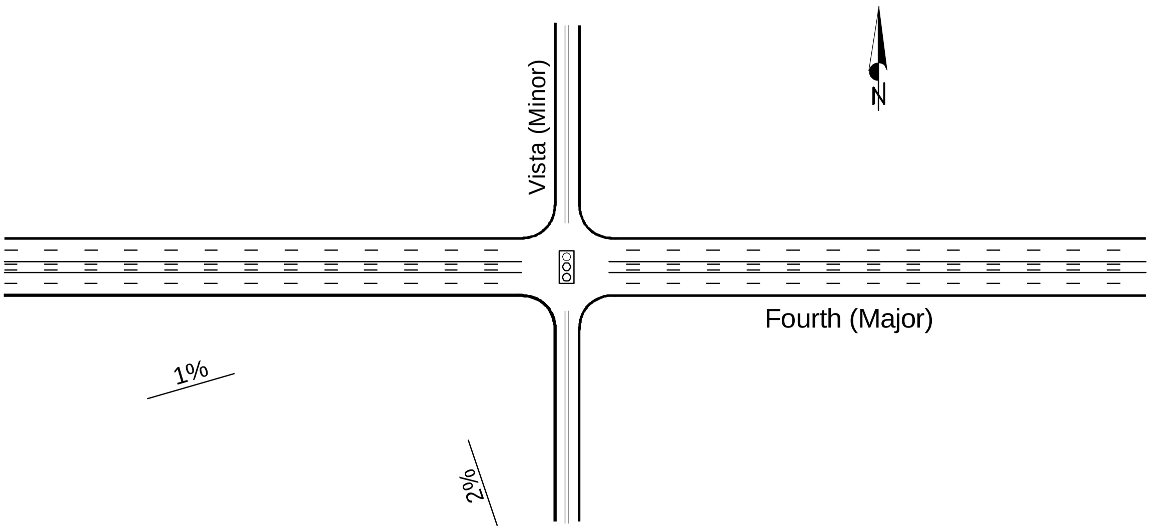


Figure 3-20. Fourth and Vista Intersection.

Solution for Northbound Vista. The solution is provided below:

- *Step 1: Identify needed adjustments.*

For Fourth Street, the approach grades are 1 percent, which is below the 3 percent limit. Therefore, no adjustments in the time gap are needed.

- *Step 2: Determine the minor road sight triangle length for a right turn from Vista.*

Based on the *Green Book* procedure, the "a" length for the sight triangle is the following:

"a" leg length = distance between major-road travel way and front of vehicle
+ distance between front of vehicle and driver's eye
+ distance to middle of lane of interest

a_L = "a" leg length to vehicles approaching from the left

a_L = distance from major-road traveled way
+ distance from front of the vehicle to the driver's eye
+ 0.5 lane width

$a_L = 6.5$ ft [2.0 m] + 8 ft [2.4 m] + 0.5 × 12 ft [3.7 m]

$a_L = 21$ ft [6 m]

- *Step 3: Determine the major road sight triangle length for a right turn from Vista.*

The equation for the "b" leg length is as follows:

US Customary	Metric
$ISD = 1.47\ V_{major} t_g$	$ISD = 0.278\ V_{major} t_g$

The initial time gap to be used in this equation is 6.5 sec for passenger cars, as highlighted in Table 3-11. Because the grade on the southbound approach is within the 3 percent limit, no adjustment for the grade is necessary. There are also no adjustments for number of lanes for a right turn.

Thus, the t_g value is as follows:

t_g = base t_g + adjustments

t_g = 6.5 sec + 0 sec

t_g = 6.5 sec

Therefore, the "b" leg length is as follows:

US Customary	Metric
ISD = 1.47 $V_{major}t_g$	ISD = 0.278 $V_{major}t_g$
ISD = 1.47 (45 mph) (6.5 sec)	ISD = 0.278 (80 km/h) (6.5 sec)
ISD = 478 ft	ISD = 145 m

- *Step 4: Illustrate findings.*

Figure 3-21 illustrates the sight triangle for a right turn from the northbound minor road (Vista); because the conditions are similar for southbound Vista, the ISD sight triangles would have similar dimensions as those for the northbound approach.

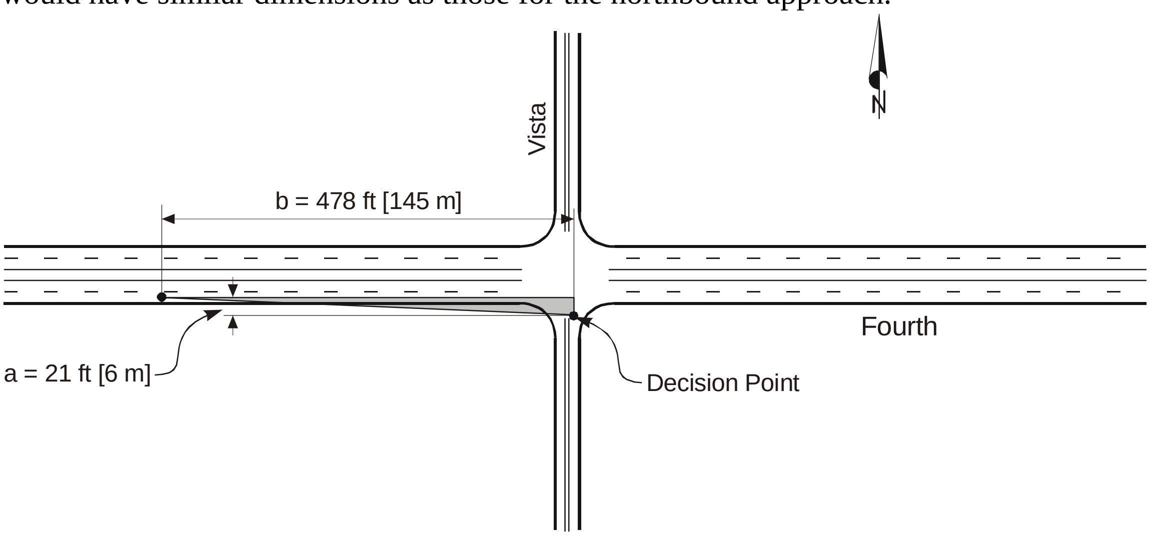


Figure 3-21. Case D – Sight Triangles for a Right Turn from Minor Road with Traffic Signal Control for Right Turn on Red.

Table 3-11. Time Gap for Case B2 – Right Turn from Stop (Reproduction of *Green Book* Exhibit 9-57).

Design Vehicle	Time gap (sec) at design speed of major road (t_g)
Passenger car	***6.5***
Single-unit truck	8.5
Combination truck	10.5

Note: Time gaps are for a stopped vehicle to turn right onto or cross a two-lane highway with no median and grades 3 percent or less. The table values require adjustment as follows:

- For multilane highways: For crossing a major road with more than two lanes, add 0.5 second for passenger cars or 0.7 second for trucks for each additional lane to be crossed and for narrow medians that cannot store the design vehicle.
- For minor road approach grades: If the approach grade is an upgrade that exceeds 3 percent, add 0.1 second for each percent grade.

Application 8
ISD, Case F

Overview

Drivers turning left across oncoming traffic of a major roadway require sufficient sight distance to determine when it is safe to turn left across the lanes used by opposing traffic. Sight distance is based on a left turn by a stopped vehicle, since a vehicle that turns left without stopping would need less sight distance. Discussion on intersection sight distance is included in the *Urban Intersection Design Guide*, Chapter 3, Section 1 <link>.

Example 1: Left Turn from Two-Lane Highway

Problem. Determine the sight distance necessary for a passenger car to turn left from Elmo to Bird. Figure 3-22 is a schematic of the intersection.

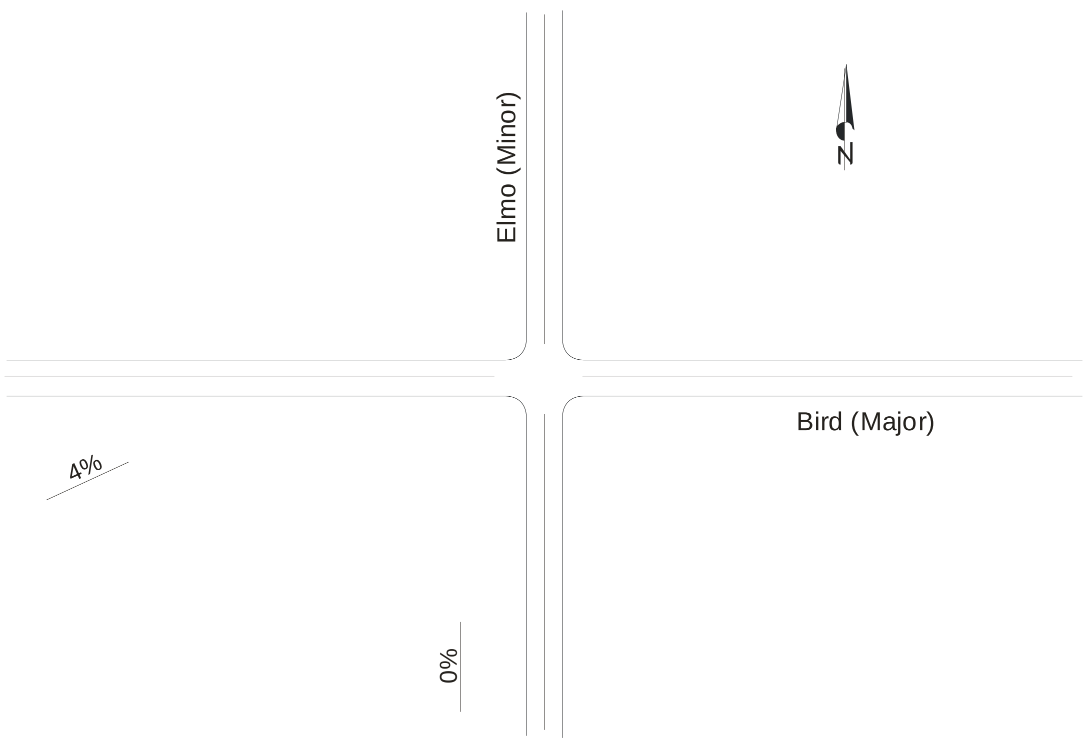


Figure 3-22. Bird and Elmo Intersection.

Known Information. The information known for this intersection includes:

- Bird is the major roadway with no traffic control.
- Elmo has stop control.
- Both Bird and Elmo are two-lane highways.
- Design speed on Bird is 40 mph [64 km/h].

- Design speed on Elmo is 35 mph [56 km/h].
- Grade for Bird is 4 percent.
- Grade for Elmo is 0 percent.
- No medians are present on the approaches.
- Lane width is 12 ft [3.7 m].

Solution. The solution is provided below:

- *Step 1: Identify needed adjustments.*

Potential adjustment includes number of lanes to cross and vehicle type. Since both Bird and Elmo are two-lane highways, there is no adjustment for number of additional lanes to cross. The design vehicle is a passenger car; therefore, no adjustment for vehicle type is needed.

- *Step 2: Determine the sight distance for a left turn from the major highway.*

The values in Table 3-12 can be used to determine the needed sight distance since no adjustments for number of lanes or vehicle type are needed. For Bird, with a design speed of 40 mph [64 km/h], the sight distance is 325 ft [99 m].

- *Step 3: Illustrate findings.*

Figure 3-23 illustrates the resulting sight distance for a left turn from Bird to Elmo.

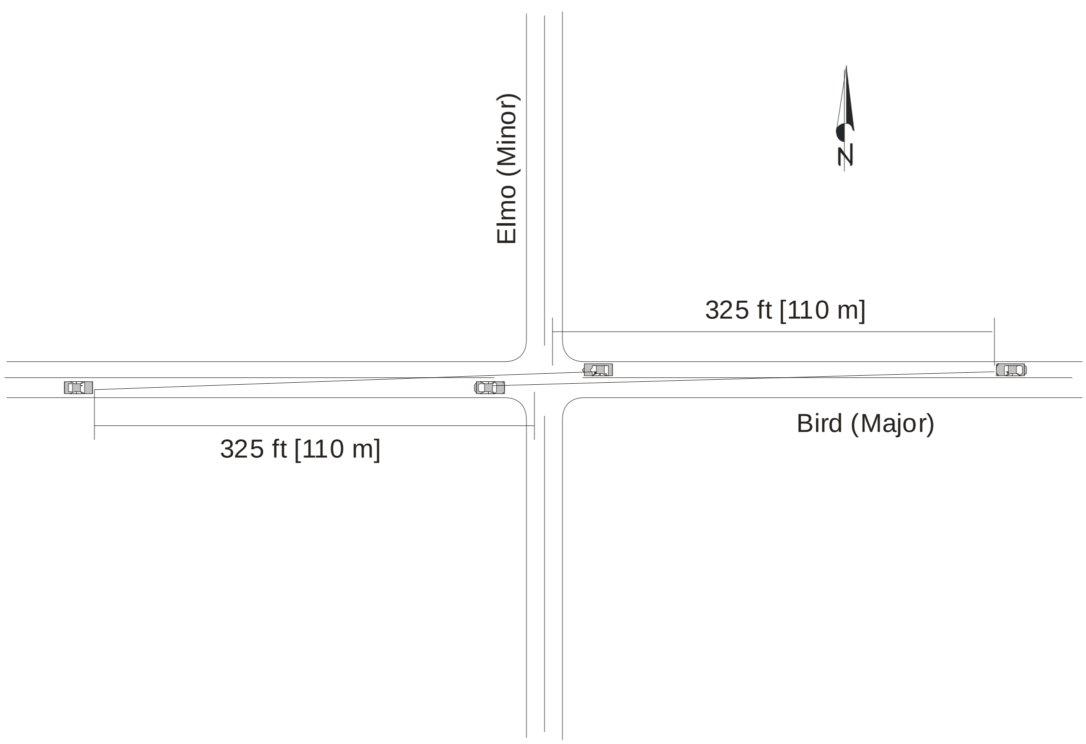


Figure 3-23. Case F – Sight Distance for Left Turns from Two-Lane Major Road.

Table 3-12. Intersection Sight Distance – Case F – Left Turn from Major Road (Reproduction of *Green Book* Exhibit 9-67).

US Customary			
		Intersection Sight Distance	
		Passenger Cars	
Design Speed (mph)	**Stopping Sight Distance (ft)**	**Calculated (ft)**	**Design (ft)**
15	80	121.3	125
20	115	161.7	165
25	155	202.1	205
30	200	242.6	245
35	250	283.4	***285***
40	305	323.4	***325***
45	360	363.8	365
50	425	404.3	405
55	495	444.7	445
60	570	485.1	490
65	645	525.5	530
70	730	566.0	570
75	820	606.4	610
80	910	646.8	650
Metric			
		Intersection Sight Distance	
		Passenger Cars	
Design Speed (km/h)	**Stopping Sight Distance (m)**	**Calculated (m)**	**Design (m)**
20	20	30.6	35
30	35	45.9	50
40	50	61.2	65
50	65	76.5	80
60	85	91.7	***95***
70	105	107.0	***110***
80	130	122.3	125
90	16	137.6	140
100	185	152.9	155
110	220	168.2	170
120	250	183.5	185
130	285	198.8	200
Note: Intersection sight distance shown is for a passenger car making a left turn from an undivided highway. For other conditions and design vehicles, the time gap should be adjusted and the sight distance recalculated.			

Example 2: Left Turn from Six-Lane Highway

Problem. Determine the sight distance necessary for a combination truck to turn left from Elm onto Hazel (see Figure 3-24).

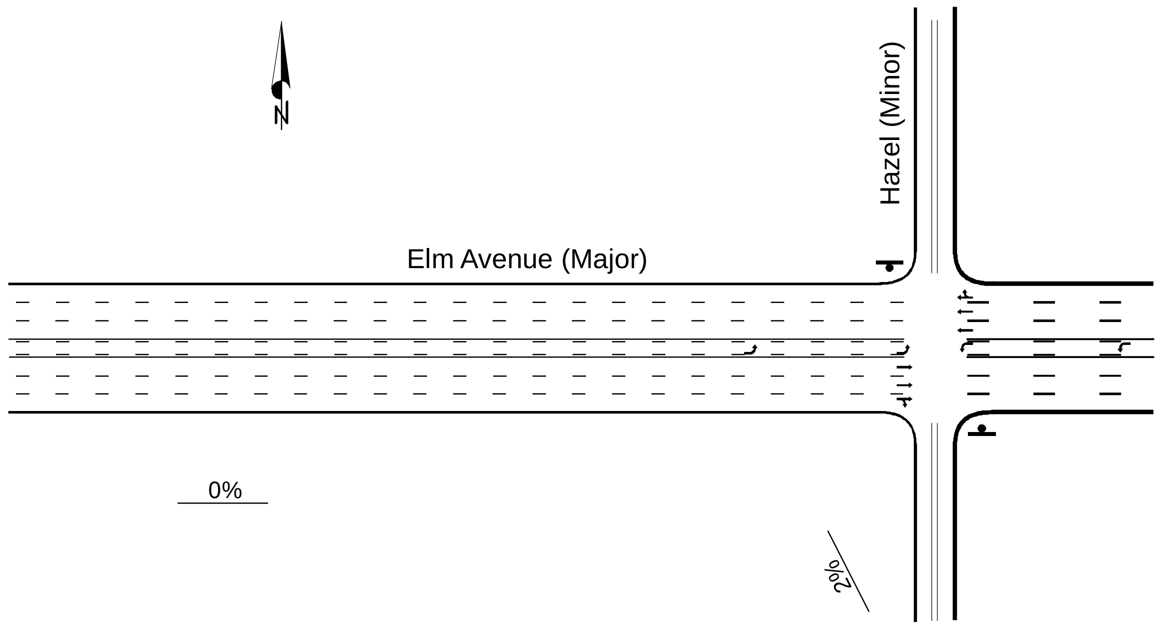


Figure 3-24. Elm and Hazel Intersection.

Known Information. The following information is known about the site:

- Design speed on Elm Avenue is 45 mph [72 km/h].
- Design speed on Hazel is 30 mph [48 km/h].
- Grade for Elm Avenue is 0 percent.
- Grade on Hazel is 2 percent.
- Elm Avenue has six lanes and a two-way left-turn lane.
- Hazel has two lanes and no median.
- The width of the TWLTL on Elm is 14 ft [4.3 m].
- Lane widths are 12 ft [3.7 m].

Solution. The solution is provided below:

- *Step 1: Identify needed adjustments.*

Because Elm Avenue has three lanes in each direction as well as a left-turn lane, adjustments to the time gap are needed.

- *Step 2: Determine the sight distance of a left turn from the eastbound approach of Elm.*

The sight distance required for the turning vehicle is based on the following equation:

US Customary	Metric
$ISD = 1.47_{major}t_g$	$ISD = 0.278\ V_{major}t_g$

In this case, the base t_g is 7.5 sec as highlighted in Table 3-13. Since the vehicle has to cross three opposing lanes, the adjustment factor is 1.4 sec (two additional lanes at 0.7 sec per lane) to account for crossing the additional lanes. The total t_g is as follows:

t_g = 7.5 sec + 1.4 sec (width adjustment)

t_g = 8.9 sec

Table 3-13. Time Gap for Case F – Left Turn from Major Road (Reproduction of *Green Book* Exhibit 9-66).

Design Vehicle	Time gap (sec) at design speed of major road (t_g)
Passenger car	5.5
Single-unit truck	6.5
Combination truck	***7.5***
Adjustment for multilane highways: ♦ For left-turning vehicles that cross more than one opposing lane, add 0.5 second for passenger cars or ***0.7 second*** for trucks for ***each additional lane*** to be crossed.	

Therefore, the sight distance required for the turning vehicle is as follows:

US Customary	Metric
ISD = 1.47 V_{major} t_g	ISD= 0.278 V_{major} t_g
ISD = 1.47 (45 mph) (8.9 sec)	ISD = 0.278 (70 km/h) (8.9 sec)
ISD = 589 ft	ISD = 173 m

♦ *Step 3: Illustrate findings.*

Figure 3-25 illustrates the resulting sight distance for a left turn from the major road for a combination truck.

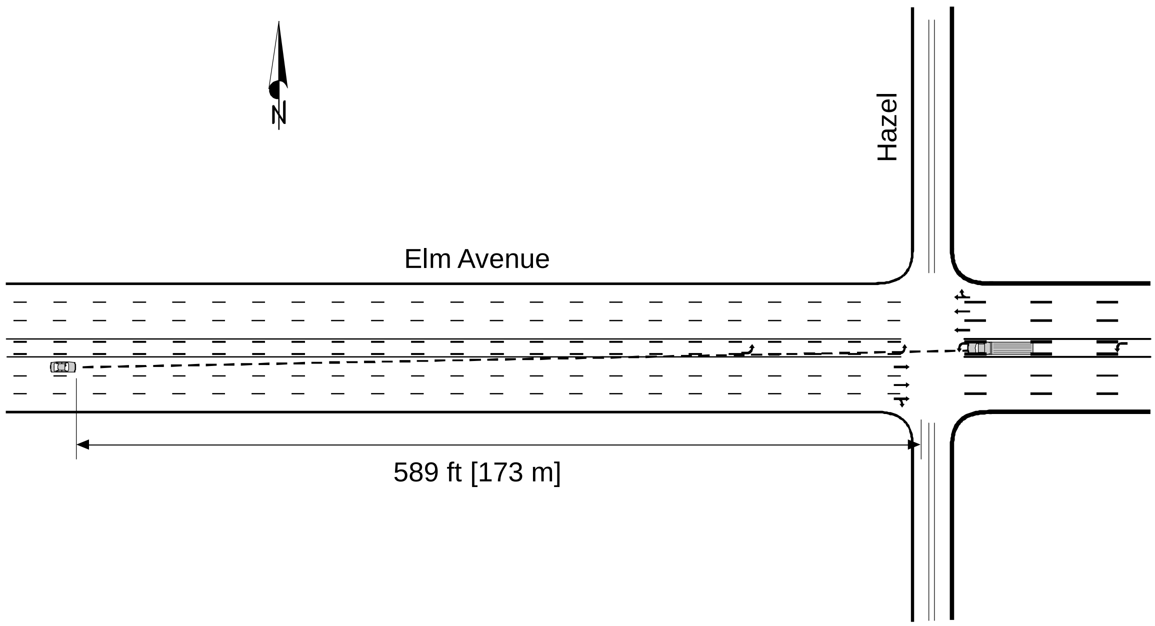


Figure 3-25. Case F – Sight Distance for Combination Truck Turning Left from a Six-Lane Highway.

Application 9

Example of a Superelevation Design at an Intersection

Overview

The use of superelevation on an urban roadway may present a problem with connections to crossing roadways because of the necessity to match grades and provide a smooth ride through the intersection. If the intersection is signalized or is expected to be signalized in the future a smooth ride should be ensured both on the major and minor roadways. The *Urban Intersection Design Guide*, Chapter 3, Section 2 <link> provides information on superelevation.

Background

Problem. The superelevation on the major roadway due to the downstream horizontal curve must be integrated with the vertical profile on the minor roadway at the intersection between the two roadways. Figure 3-26 shows the intersection layout.

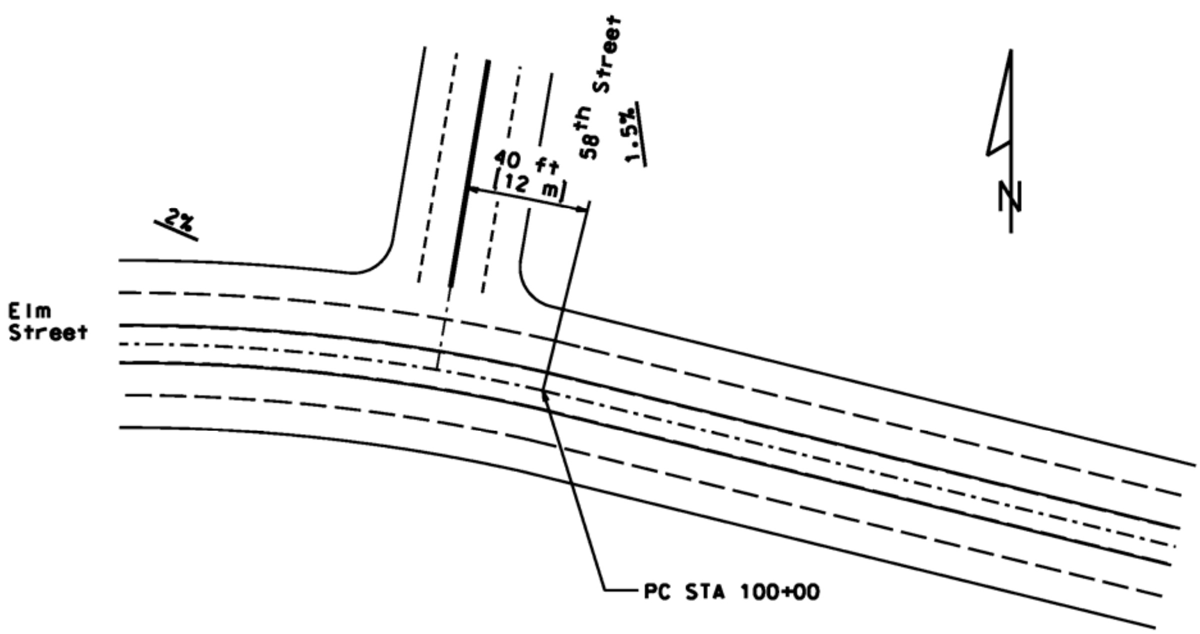


Figure 3-26. Intersection Layout.

Known Information. The information known for this intersection includes:

- Elm is the major roadway with no traffic control.
- 58th Street has stop control.
- Elm Street is a five-lane roadway with two lanes per direction and a TWLTL.
- 58th Street is a four-lane roadway with two lanes per direction.
- Horizontal curve on Elm has a radius of 700 ft [213 m].

- 58th Street has a 1.5 percent downgrade toward Elm Street.
- Elm Street has a 2 percent downgrade toward 58th Street.
- Maximum superelevation to be used is 4 percent (based on urban location).
- Design speeds are 45 mph [72 km/h] for both roadways.
- 58th Street is expected to be extended across Elm Street in the future as the city expands, with a resulting four-legged intersection.
- Normal cross slope of 2 percent exists on both roadways.
- Lane widths are:
  - 58th Street: four lanes at 12 ft [3.7 m] each, and
  - Elm Street: four lanes at 12 ft [3.7 m] each, TWLTL at 14 ft [4.3 m].

Proposed Design

The solution is provided below:

- *Step 1: Identify need for superelevation.*

From the *Roadway Design Manual,*[1] a design speed of 45 mph [72 km/h] is considered to be low speed (see Chapter 2 – Basic Design Criteria Section, Section 2 – Traffic Characteristics, Traffic Speed <link>). Checking Table 2-5 in the *Roadway Design Manual* (and reproduced in this document as Table 3-14), the provision of superelevation is required because the radius of 700 ft [213 m] is less than the value of 940 ft [287 m] shown in bold. The value (in bold) indicates the minimum radius that may be provided with adverse superelevation equal to the 2 percent cross slope on the roadway.

[1] Texas Department of Transportation. *Texas Roadway Design Manual*. February 2004. http://manuals.dot.state.tx.us:80/docs/coldesig/forms/rdw.pdf. Accessed August 30, 2002.

Table 3-14. Minimum Radii and Superelevation Transition Lengths for Limiting Values of e and f for Low-Speed Urban Streets (Reproduction of *Roadway Design Manual* Table 2-5 <link>).

US Customary					
Design Speed (mph)	**Max. e**	**Max. f**	**C**	**Min. R (ft)**	**Superelevation Transition Length,[1] L (ft)**
15	0.04	0.330	4.25	40	55
20	0.04	0.300	4.00	80	75
25	0.04	0.252	3.75	145	80
30	0.04	0.221	3.50	230	90
35	0.04	0.197	3.25	345	100
40	0.04	0.178	3.00	490	115
45	***0.04***	***0.163***	***2.75***	***665***	***125***
15	-0.02[2]	0.350	1.25	10	Not Required
20	-0.02[2]	0.312	1.20	25	Not Required
25	-0.02[2]	0.252	1.15	55	Not Required
30	-0.02[2]	0.214	1.10	105	Not Required
35	-0.02[2]	0.186	1.05	175	Not Required
40	-0.02[2]	0.163	1.00	270	Not Required
45	***-0.02[2]***	***0.163***	***2.75***	***940***	***Not Required***
Metric					
Design Speed (km/h)	**Max. e**	**Max. f**	**C**	**Min. R (m)**	**Superelevation Transition Length,[1] L (m)**
20	0.04	0.350	1.25	10	15
30	0.04	0.312	1.20	20	20
40	0.04	0.252	1.15	45	25
50	0.04	0.214	1.10	80	25
60	0.04	0.186	1.05	125	30
70	***0.04***	***0.163***	***1.00***	***190***	***30***
20	-0.02[2]	0.350	1.25	10	Not Required
30	-0.02[2]	0.312	1.20	25	Not Required
40	-0.02[2]	0.252	1.15	55	Not Required
50	-0.02[2]	0.214	1.10	105	Not Required
60	-0.02[2]	0.186	1.05	175	Not Required
70	***-0.02[2]***	***0.163***	***1.00***	***270***	***Not Required***

[1] L based on two-lane roadway rotated about centerline. For rotation about a pavement edge, or for multilane streets, the design L is determined by multiplying the above tabulated L value times the number of lanes between the rotation axis and edge of pavement. Thus for four-and six-lane streets, with the axis of rotation about the centerline, the design L is double and triple, respectively, the tabulated L.

[2] Normal crown maintained.

- ♦ *Step 2: Determine superelevation rate.*

As shown in Figure 3-27 (Figure 2-2 of the *Roadway Design Manual* <link>), a radius of 700 ft [213 m] and a 45-mph [70 km/h] design speed results in a superelevation rate of 3 percent.

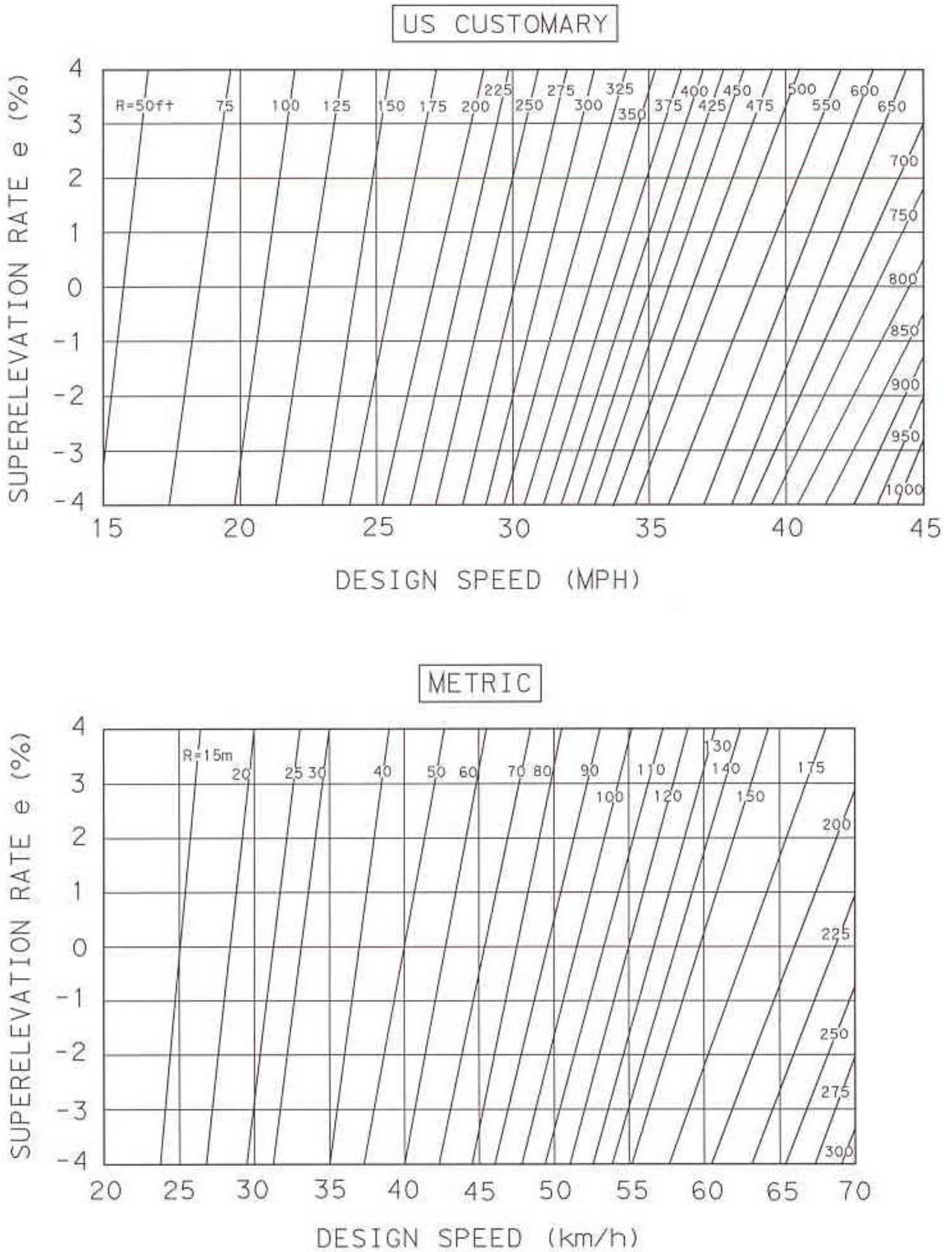


Figure 3-27. Relationship of Radius, Superelevation Rate, and Design Speed for Low-Speed Urban Street Design (Reproduction of Roadway Design Manual Figure 2-2 <link>).

- ♦ *Step 3: Determine superelevation transition lengths.*

From the *Roadway Design Manual*, the length of superelevation transition for these design parameters is 125 ft [38 m] (see Table 3-14), although note 1 of that table states that the length should be increased by a factor based on the number of lanes. The transition length as adjusted for the number of lanes (2.5) is 312 ft [95 m]. The adjustment factor was based on rotating the five-lane section about the centerline. On many urban streets the section is rotated about the gutter elevation on the inside of the curve, which would have resulted in a factor of 5.

In general, two-thirds of the transition takes place on the tangent, with the remaining one-third of the transition occurring within the horizontal curve, as illustrated in Figure 2-1 of the *Roadway Design Manual* <link>. Figure 3-28 shows the placement of the superelevation transition on Elm Street.

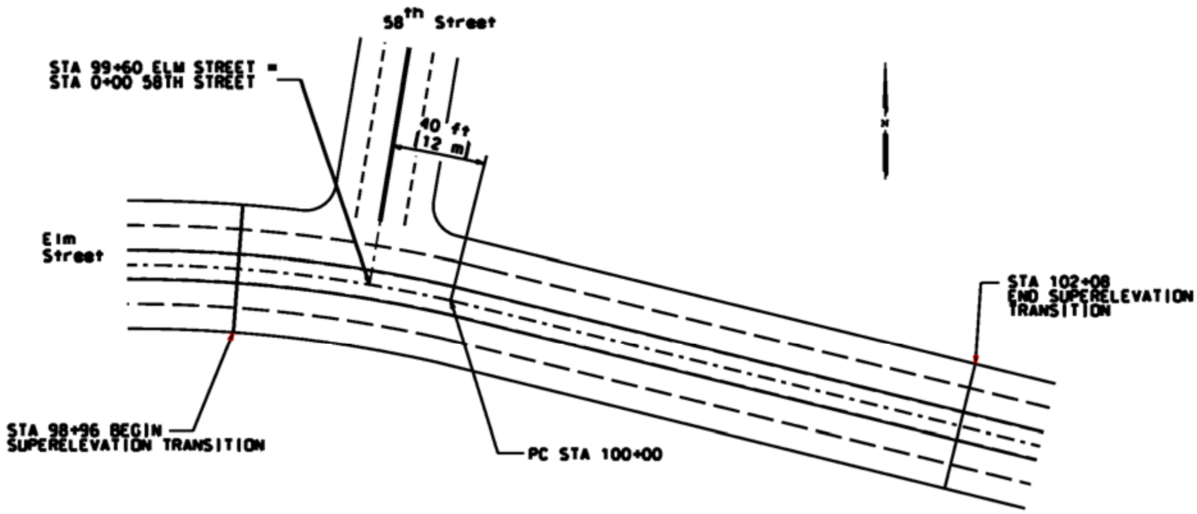


Figure 3-28. Layout Showing Superelevation Transition Placement.

- *Step 4: Tie superelevation on Elm with grade on 58th Street.*

The beginning and end of the superelevation transition should be adjusted so that the intersection falls within an area where the superelevation cross slopes can be made to match the grade of the intersecting roadway. If this is not possible, the *Roadway Design Manual*[1] allows for grade changes without vertical curves as long as the algebraic difference in grade does not exceed 1 percent. With this in mind, the superelevation transition can be placed such that the algebraic difference between the grades of the centerline of the minor road at the intersection with the major road and the cross slope of the major road is less than 1 percent.

As designed, the profile of 58th Street is shown in Figure 3-29. The change in grade of the 58th Street profile is 1 percent as it crosses the centerline of Elm Street. A small change in the beginning point for the superelevation transition could reduce this grade change, resulting in a smoother profile for 58th Street. By beginning the Elm Street superelevation transition earlier, the grade change at the centerline of Elm Street can be reduced. The revised superelevation layout is shown in Figure 3-30. The improved profile of 58th Street is shown in Figure 3-31, with a smaller grade change of 0.7 percent.

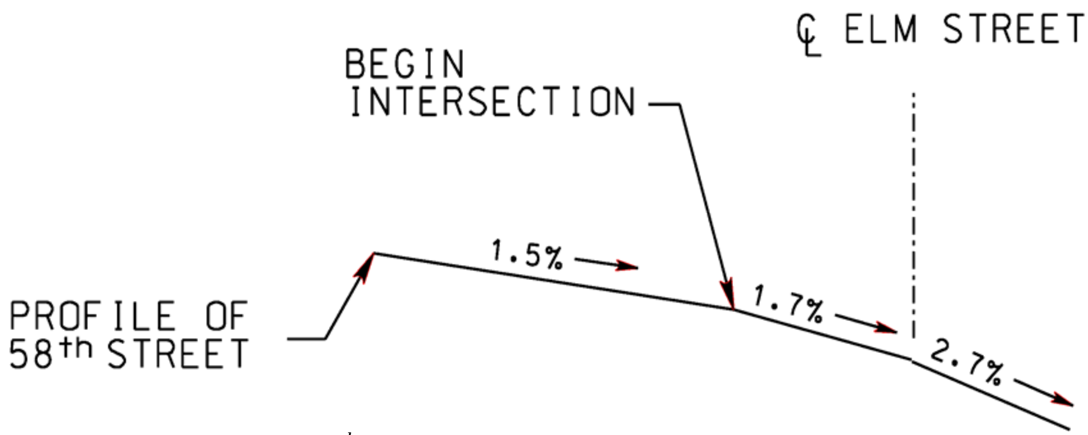


Figure 3-29. Profile of 58th Street as It Meets Elm Street's Cross Slope.

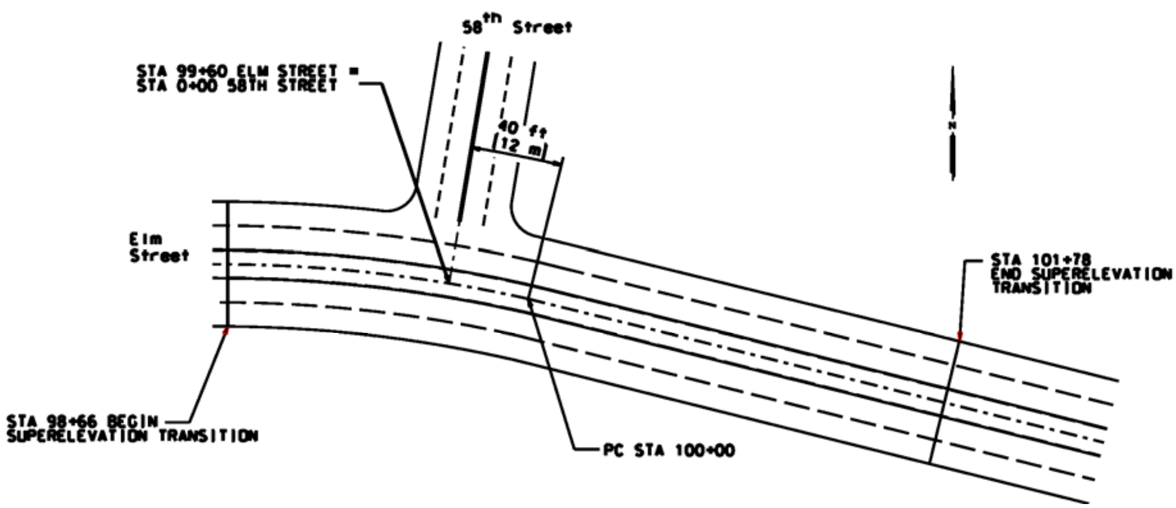


Figure 3-30. Revised Layout of Superelevation Transition.

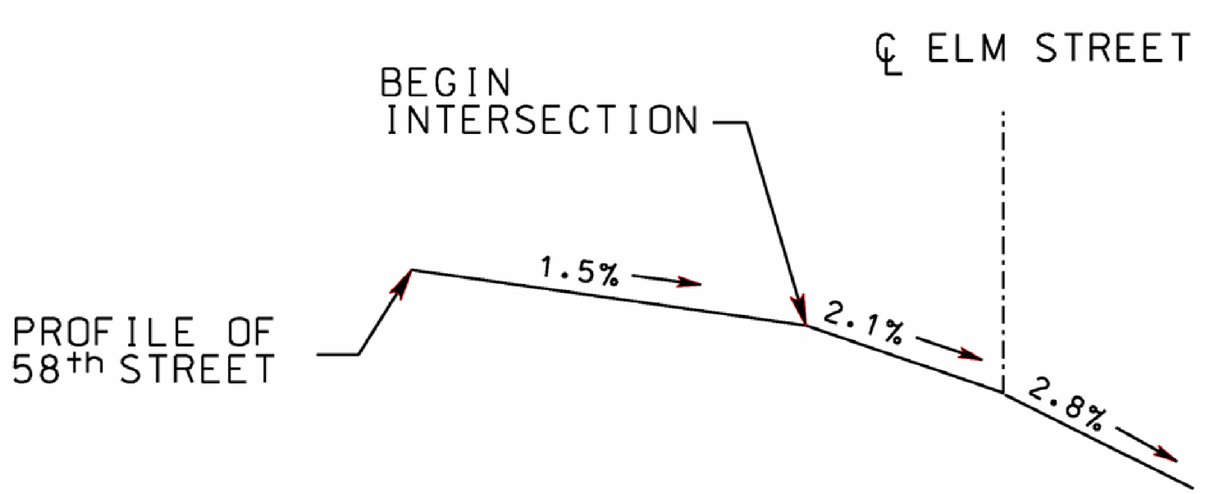


Figure 3-31. Revised Profile of 58th Street as It Meets Elm Street's Cross Slope.

♦ *Step 5: Design intersection cross section of 58th Street.*

Since the drainage design chosen was to warp the 58th Street cross section to meet the grades on Elm Street, the cross section of 58th Street will be controlled by this design. Elevations must be cal

culated at the north edge of pavement of Elm Street to establish the controlling elevations for 58th Street (see Figure 3-32).

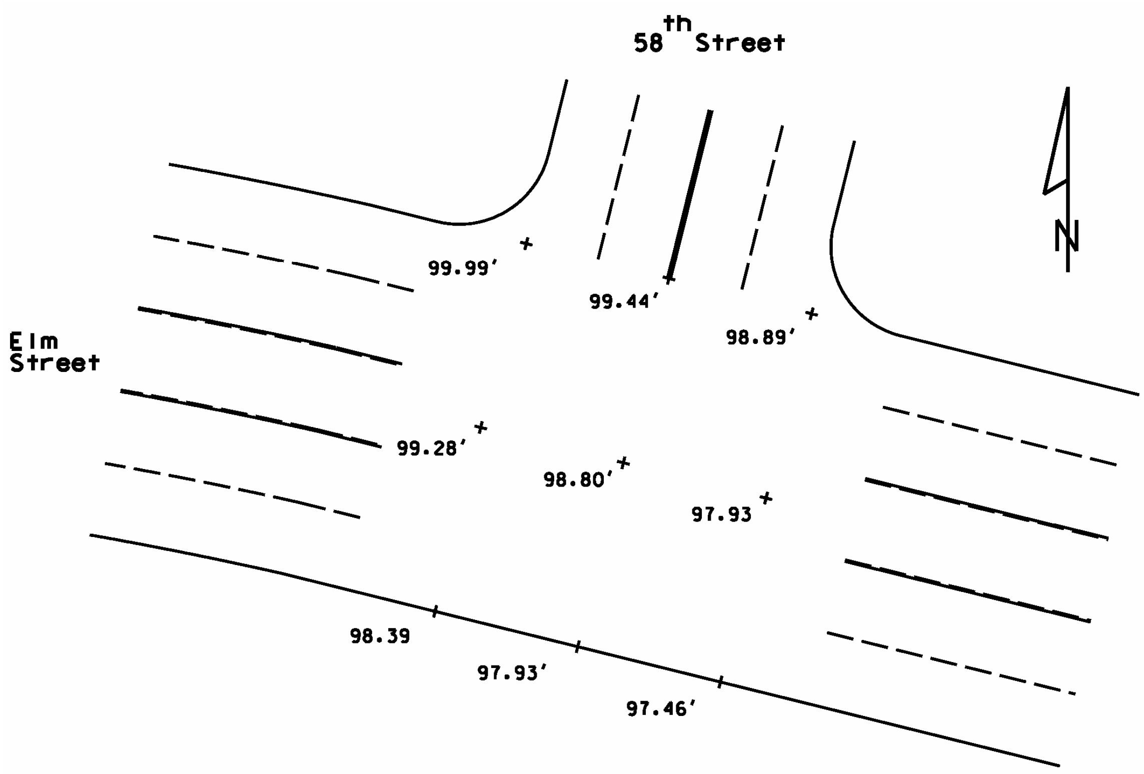


Figure 3-32. Elevations in Intersection.

These elevations can then be used to establish a cross section of 58th Street directly as it intersects with Elm Street (shown in Figure 3-33). Fifty-eighth Street's typical cross section next must be transitioned to meet the proposed cross section at Elm Street.

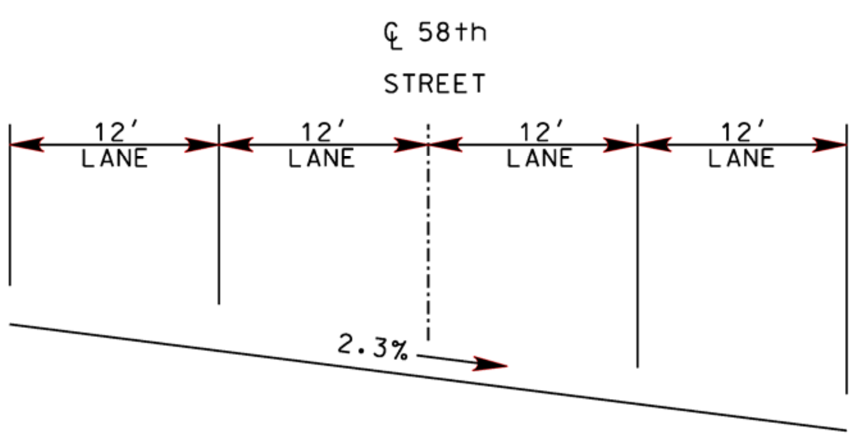


Figure 3-33. Cross Section of 58th Street Entering Intersection (STA 0+31).

Table 3-15 shows how the cross sections on 58th Street were modified to meet the grades on Elm Street. Note how the cross slopes are modified from a typical slope at STA 3+00 to the controlling slopes at STA 0+31.0.

Table 3-15. Cross Sections of 58th Street.

STA	West EOP (ft)	Cross Slope	CL Elevation (ft)	Cross Slope	East EOP (ft)	Remarks
3+00	104.32	-2.0%	104.80	-2.0%	104.32	
2+00	102.66	-0.6%	102.81	-2.1%	102.30	
1+00	101.08	1.1%	100.81	-2.2%	100.28	
0+31	**99.99**	**2.3%**	**99.44**	**-2.3%**	**98.89**	**Gutter**
0+00	*99.28*	*2.0%*	*98.80*	*-2.0%*	*98.32*	Centerline Intersection
0-31	98.39	1.9%	97.93	-2.0%	97.46	Gutter

- *Step 6: Contour area and review for drainage issues.*

Once the profiles, cross sections, and transition areas have been designed, the intersection and curve can be drawn with contour lines to aid in the review process. An illustration of the contoured area (as shown in Figure 3-34) can greatly aid in the identification of errors or problematic drainage areas. It is also generally a good idea to provide this contour layout in the Plans, Specifications, and Estimates (PS&E) to be sure the grading plan is clearly communicated to the contractor.

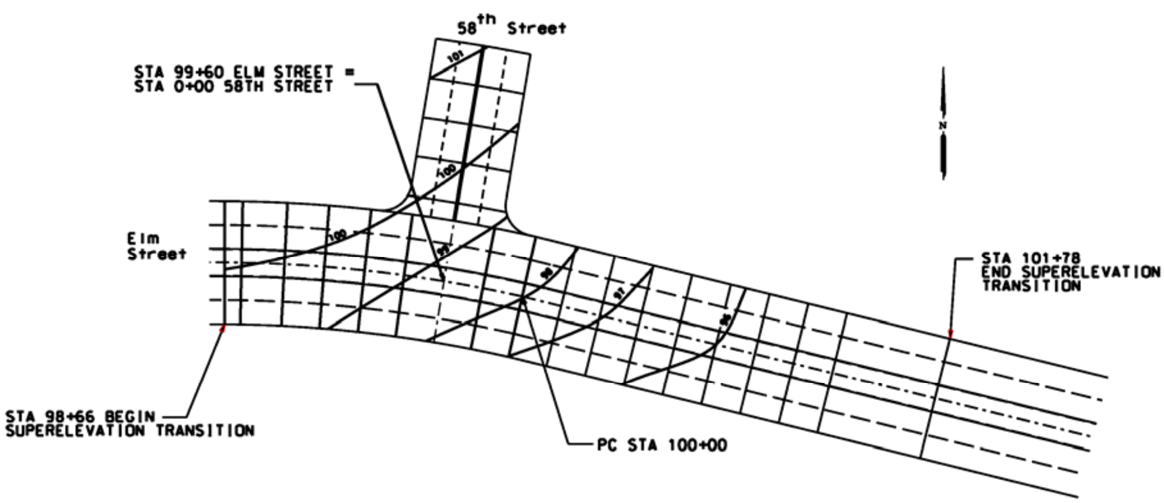


Figure 3-34. Contour of Intersection.

Application 10

Right-Turn Radius Selection Influences

Overview

The design of the right-turn radii is affected by a number of issues, including:

- design vehicle,
- available right of way,
- intersection skew,
- use of free-flow right-turn lanes or deceleration/acceleration lanes,
- desired presence of islands,
- pedestrian facilities, and
- desired turning vehicle speed.

The *Urban Intersection Design Guide*, Chapter 3, Section 3 <link> provides information on turning radius.

Background

An exploration of the impacts of the above factors will be undertaken to show their influence on the design of a major intersection. The primary variables used in the evaluation are shown in Table 3-16.

Table 3-16. Design Variables Used in Example.

Variable	Range of Values
Design Vehicle	WB-50 [WB-15]
Intersection Angle	90 deg (no skew) 75 deg (15-deg skew)
Corner Radii	100 ft [30 m] 60 ft [18 m] Simple curve radius with taper 50 ft [15 m] 30 ft [9 m]

This application uses turning templates to approximate the wheel paths of the vehicle. Simulation software such as AutoTURN or IGIDS can be used to better customize the turning path to the geometry present at an intersection.

The variables in Table 3-16 will be used to show their influence on the design of the intersection, including influences on turning vehicle speed, pedestrian facilities, and the desirability of including corner islands.

The selection of the design vehicle for an intersection should be made after consideration of the vehicle mix that is projected to use the facility. For more information regarding design vehicle selection, see *Roadway Design Manual* Chapter 7, Section 7 <link>. When determining the appropriate design vehicle at a specific location, it is advisable to check the Statewide Traffic and Recording System (STARS) to determine the types of vehicles (i.e., vehicle classification) actually present at a given location. The data are available from the Transportation Planning and Programming Division (TPP).

The WB-50 [WB-15] truck is used in this application, although other design vehicles are appropriate in other circumstances. Passenger cars would be capable of traversing the designs created for a WB-50 [WB-15] truck; therefore their turning paths are not included on the figures for clarity. It is noted that the choice of a WB-50 [WB-15] truck as a design vehicle is not appropriate for many locations, and its use as a design vehicle may undesirably affect intersection designs with regard to pedestrian facilities (i.e., crossing distance and vehicle turning speed) and the layout of the resulting intersection design (i.e., island use may not be an option and a large poorly defined paved area may result).

The designs shown share the following dimensions:

- Outside, or curb, lanes: 12 ft [3.7 m]
- Curb offset: 2 ft [0.6 m]
- Inside lanes (if present): 12 ft [3.7 m]

Proposed Designs

100-ft [30 m] Radius, WB-50 Design Vehicle. Figure 3-35 shows a design using a 100-ft [30 m] radius with the turning template of a WB-50 [WB-15] truck. The truck is able to turn right without infringing on other lanes of the original or receiving roadway. It is apparent that the use of a large simple radius, while effective at allowing the truck to turn without infringing on other lanes, results in a very large, poorly defined intersection area. The turning path shown in Figure 3-35 shows that the radius could be reduced while still allowing the design vehicle to complete the right turn.

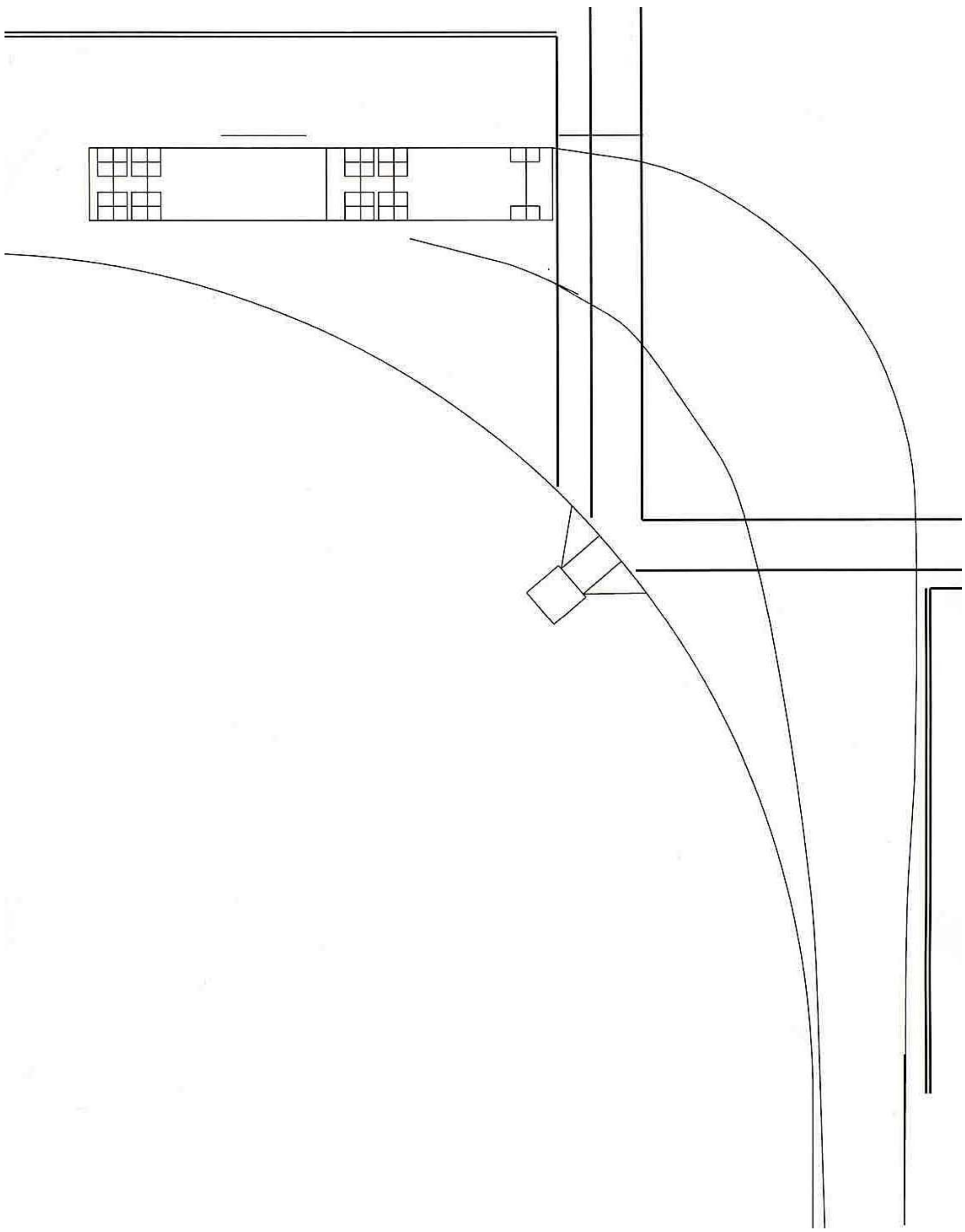


Figure 3-35. WB-50 [WB-15] Truck on 100-ft [30 m] Radius Curve.

Figure 3-36 shows the design as modified by the inclusion of an island. The use of the island provides better definition for the intersection, by channelizing the traffic. The islands also provide refuge for pedestrians and locations for traffic control devices. The island is shown with a cut-through pedestrian path rather than curb ramps because the island is too small to allow the necessary 5 ft by 5 ft [1.5 m by 1.5 m] landing area at the top of the ramps.

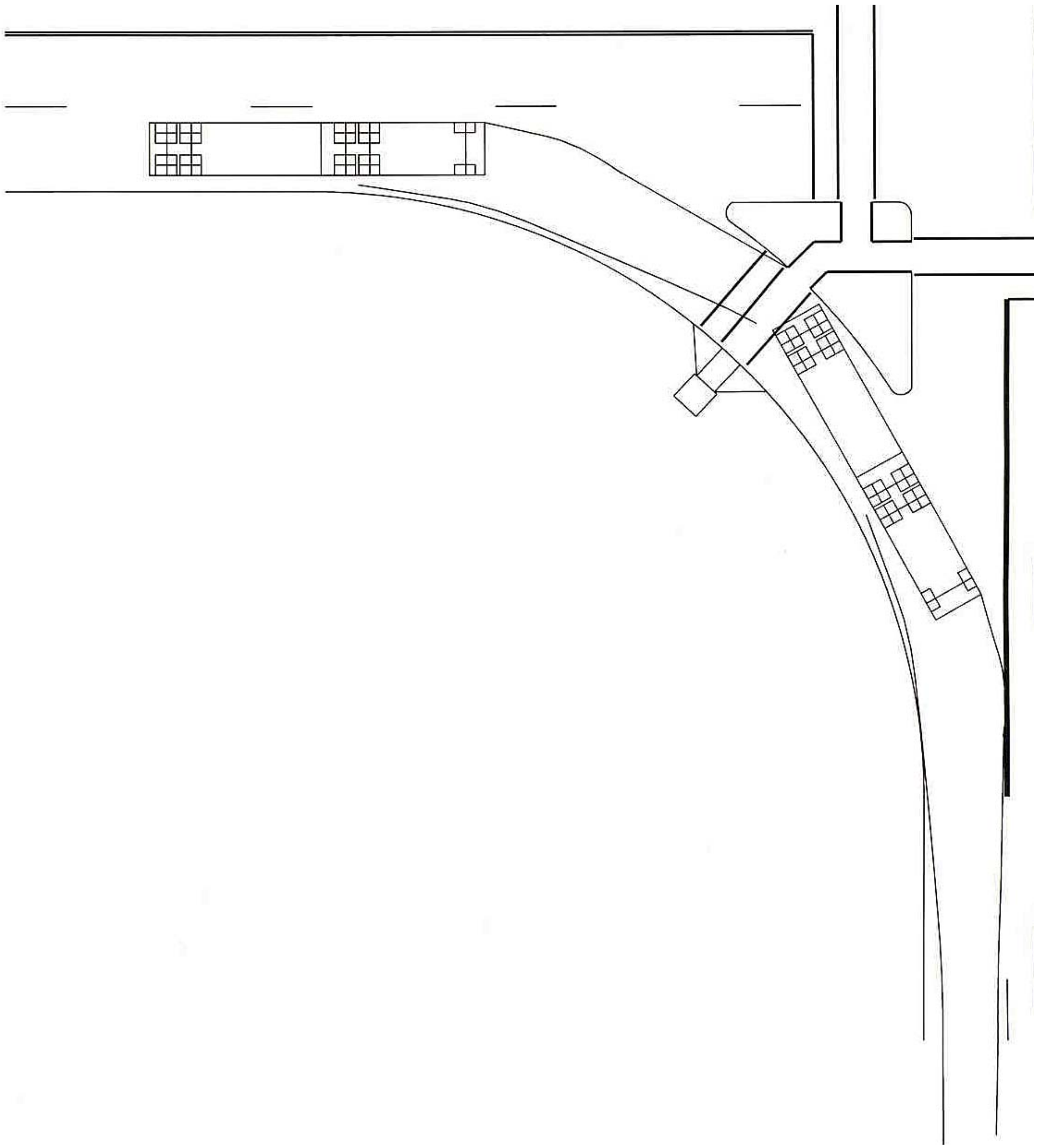


Figure 3-36. WB-50 [WB-15] Truck on 100-ft [30 m] Radius Curve with Island.

Passenger car turning speeds on the roadways shown in Figure 3-35 and Figure 3-36 can be predicted using equations developed as part of TxDOT Project 4365-4[2] and are included in the *Urban Intersection Design Guide*, Chapter 3, Section 3 <link>. These equations include consideration of channelization present, and corner radius, length of right-turn lane, and width of right-turn lane. In this example, the corner radius is 100 ft [30 m], the width of the right-turn lane is 12 ft [3.7 m], and an island is assumed to be built. The length of the right-turn lane has yet to be determined for this example, and so the average length used to generate the original equations (193 ft [59 m]) was used to predict the speed of the passenger car at the beginning and near the middle of the right turn.

Speed at the beginning of the turn:

$$V85BT = 17.50 - 1.00\ Chan + 0.10\ CR - 0.006\ Len + 0.13\ Wid$$

$$V85BT = 17.50 - 1.00\ (0) + 0.10\ (100) - 0.006\ (193) + 0.13\ (12)$$

$$V85BT = 27 \text{ mph } [43 \text{ km/h}]$$

Where:

V85BT = 85th percentile free-flow speed near the beginning of the right turn (mph)
Chan = channelization present at site, Chan = 0 for islands and 1 for lines
CR = corner radius (ft)
Len = length of right-turn lane (ft)
Wid = width of right-turn lane at start of right turn (ft)

[2] Fitzpatrick, K., and W.H. Schneider IV. *Turn Speeds in Right-Turn Lanes*. FHWA/TX-05/4365-4. September 2004.

Speed near the middle of the turn:

V85MT = 13.03 + 0.23 Chan + 0.06 CR – 0.01 Len + 0.40 Wid
V85MT = 13.03 + 0.23 (0) + 0.06 (100) – 0.01 (193) + 0.40 (12)
V85MT = 22 mph [35 km/h]

Where:

V85MT = 85th percentile free-flow speed near the middle of the right turn (mph)
Chan = channelization present at site, Chan = 0 for islands and 1 for lines
CR = corner radius (ft)
Len = length of right turn lane (ft)
Wid = width of right-turn lane at start of right turn (ft)

For this example, the 85th percentile speed of turning vehicles at the start of the turn would be 27 mph [43 km/h] slowing to 22 mph [35 km/h] near the middle of the turn.

Although recommended limits for vehicle turning speeds in various environments have not been established, it has been found that survival rates of pedestrians struck by motor vehicles are much higher if vehicle speeds are reduced.[3] Eighty percent of pedestrians are killed when struck by motor vehicles traveling 35 to 45 mph [56 to 72 km/h]; only 5 percent are killed at speeds of 18 mph [29 km/h]. Stopping sight distance for 20 mph [32 km/h] is 115 ft [35 m].[1] Because of the potential for vehicle-pedestrian conflicts at an intersection, this stopping distance is relatively high. If the turning speed were reduced by one-fourth to 15 mph [24 km/h], the stopping sight distance would be 80 ft [24 m], a 30 percent reduction.

The use of corner islands would be desirable with respect to reducing crossing distances and providing refuge areas, although drivers of WB-50 [WB-15] trucks would have to exercise care to avoid over-running the curb. The turning speeds of passenger cars would remain an issue.

[3] *Killing Speed: Saving Lives*. Department of Transport, London, United Kingdom, 1992.

60-ft [18 m] Simple Curve Radius with Taper. Figure 3-37 shows a design using a 60-ft [18 m] simple radius with a 4-ft [1.2 m] taper and a 1:15 taper (see Figure 7-7 in the *Roadway Design Manual* <link> for an example of the pavement edge geometry for this type of design). As shown in the figure, this design more closely approximates the turning path of the WB-50 [WB-15] truck, reducing the amount of paved area in the intersection and the crossing distance for pedestrians. The turning speed would be reduced by the smaller corner radius (60 ft [18 m]), with a predicted midturn speed of 19 mph [31 km/h] and a beginning turn speed of 24 mph [39 km/h].

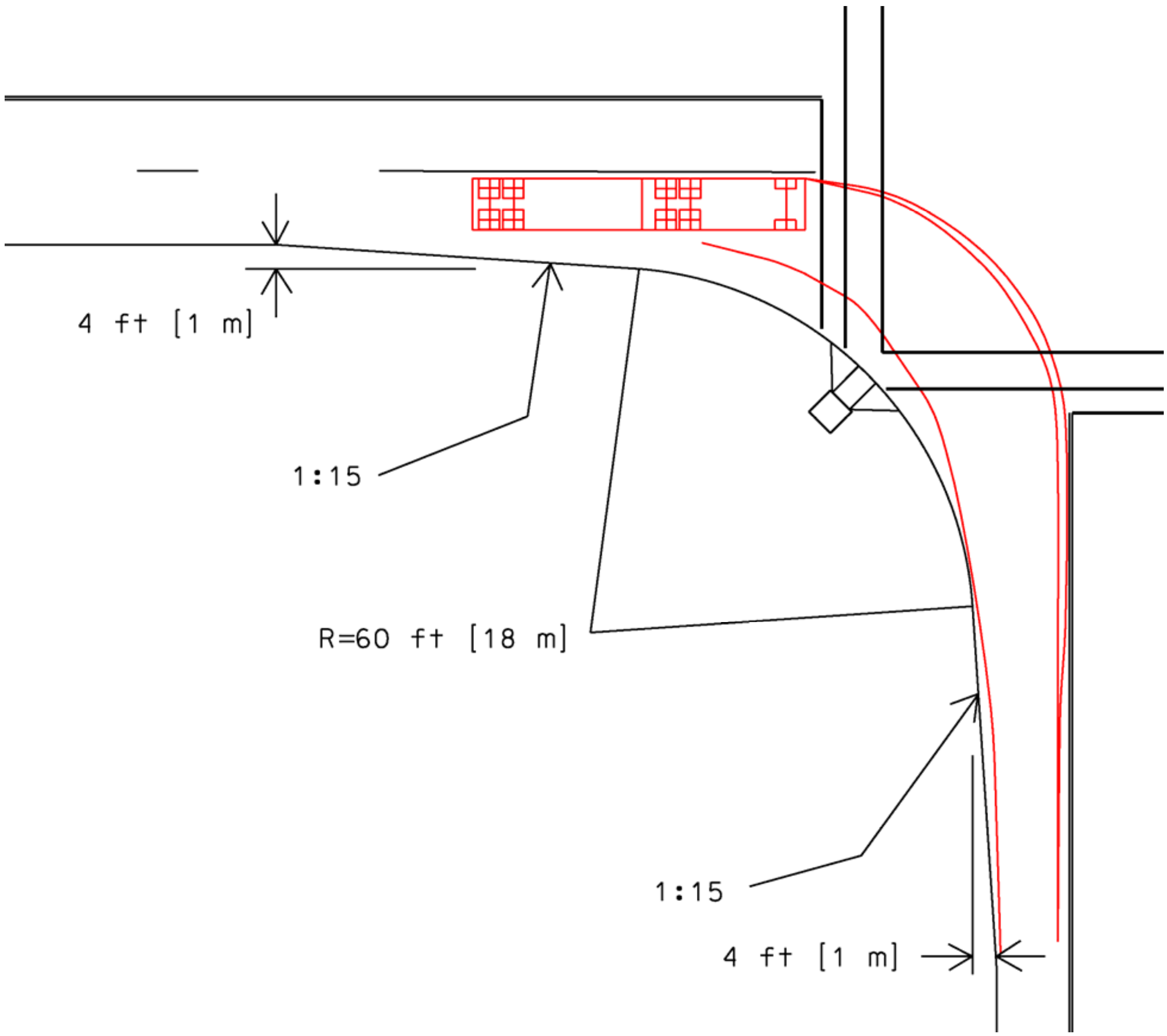


Figure 3-37. WB-50 [WB-15] on 60-ft [18 m] Radius with Taper.

50-ft [15 m] Radius, WB-50 [WB-15] Design Vehicle. Figure 3-38 shows a design using a 50-ft [15 m] radius with the turning template of a WB-50 [WB-15] truck. The truck is able to turn right only by infringing on the opposing lane of the receiving roadway by approximately 4 ft [1.2 m], yielding an unacceptable design.

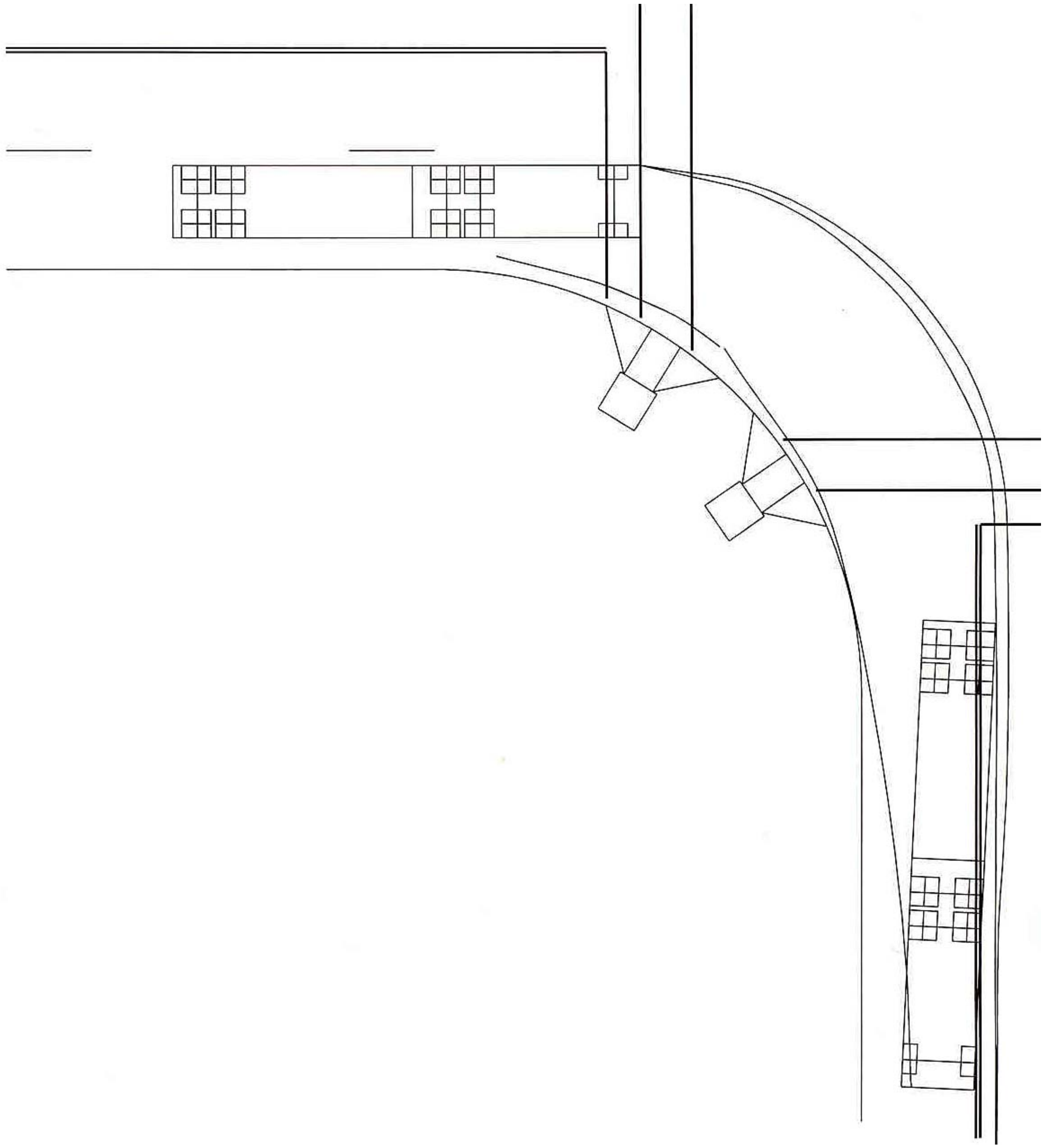


Figure 3-38. WB-50 [WB-15] on 50-ft [15 m] Radius.

If the receiving roadway has more than one lane per direction, trucks may normally encroach onto an adjacent lane traveling in the same direction with little impact on operations. Texas law states that vehicles turning right must stay as close as practicable to the right-hand curb or edge of the roadway. Figure 3-39 shows the truck turning into the inside lane of a four-lane cross street, an acceptable design.

The predicted turning speed of passenger vehicles near the middle on this right-turn radius is 18 mph [29 km/h]. The intersection area is still fairly large, and pedestrians would again have a long crossing distance.

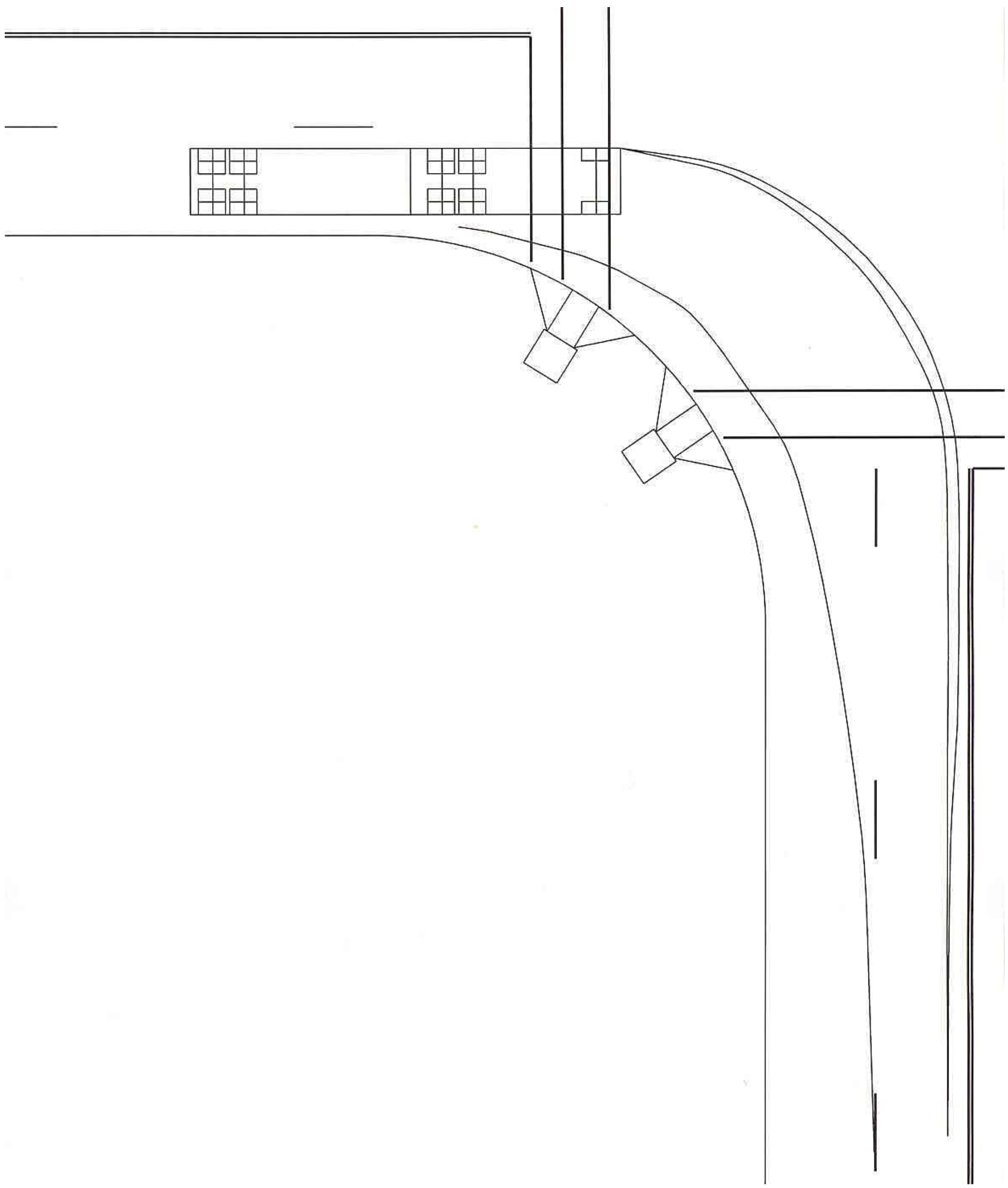


Figure 3-39. WB-50 [WB-15] on 50-ft [15 m] Radius Curve with Four-Lane Crossroad.

30-ft [9 m] Radius, WB-50 Design Vehicle. Figure 3-40 shows a design using a 30-ft [9 m] radius curve and a WB-50 [WB-15] truck. The truck is not able to negotiate the curve without entering the oncoming lane of the receiving roadway, therefore, the design is unacceptable.

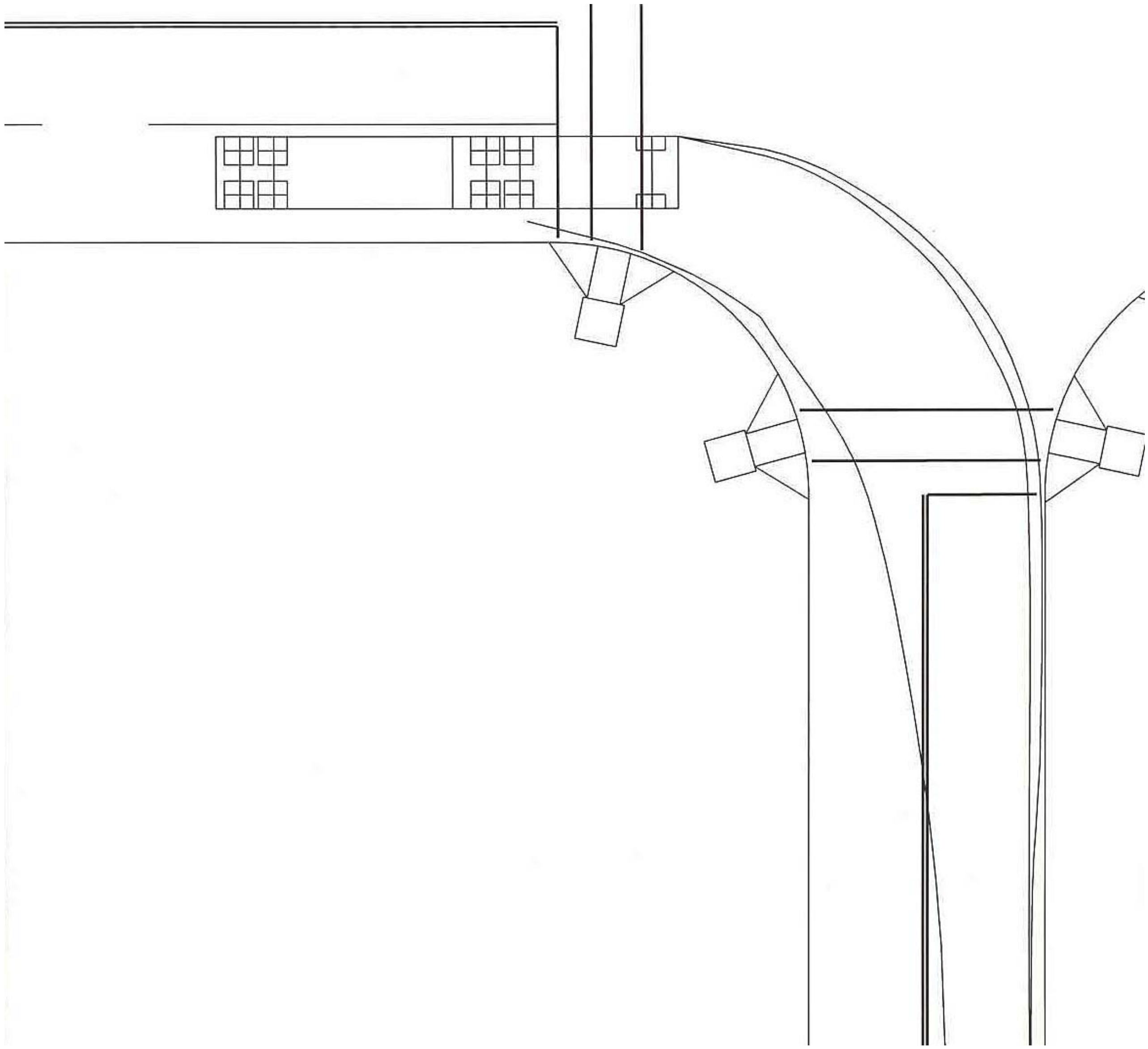


Figure 3-40. WB-50 [WB-15] on 30-ft [9 m] Radius Curve with Two-Lane Crossroad.

Figure 3-41 shows a design that also uses a 30-ft [9 m] radius curve and a WB-50 [WB-15] truck. Instead of a two-lane crossroad, a four-lane crossroad is shown. The truck is able to turn into the inner lane without encroaching into oncoming lanes although the turn would require optimal positioning by the driver.

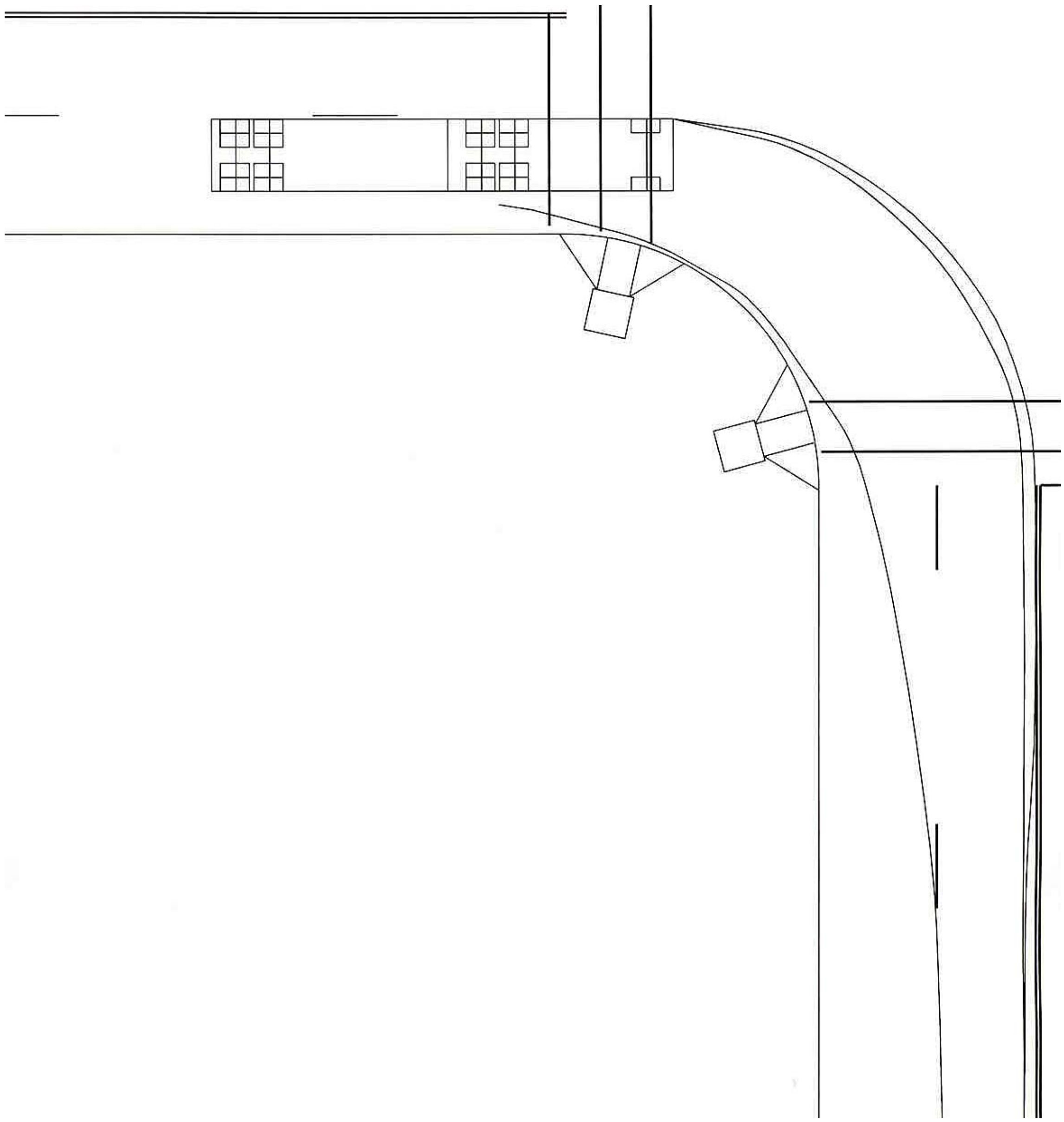


Figure 3-41. WB-50 [WB-15] on 30-ft [9 m] Radius Curve with Four-Lane Crossroad.

100-ft [30 m] Radius, WB-50 [WB-15] Design Vehicle, 15-deg Skew. Figure 3-42 shows a WB-50 [WB-15] truck turning template on a 100-ft [30 m] radius curve in an intersection with a 15-deg skew. Because of the skew, truck-turning capability must be reviewed at both corners A and B. Both corners are negotiable by the design vehicle; however, the intersection design is not optimal because of the large size of the intersection. Passenger car turning speeds would be predicted to be 22 mph [35 km/h], which is higher than desired for pedestrians. It is noted that the equations used to predict speed in this example are not sensitive to turning angle; it appears likely that the speeds on corner B could be even greater because the vehicles do not turn through 90 deg.

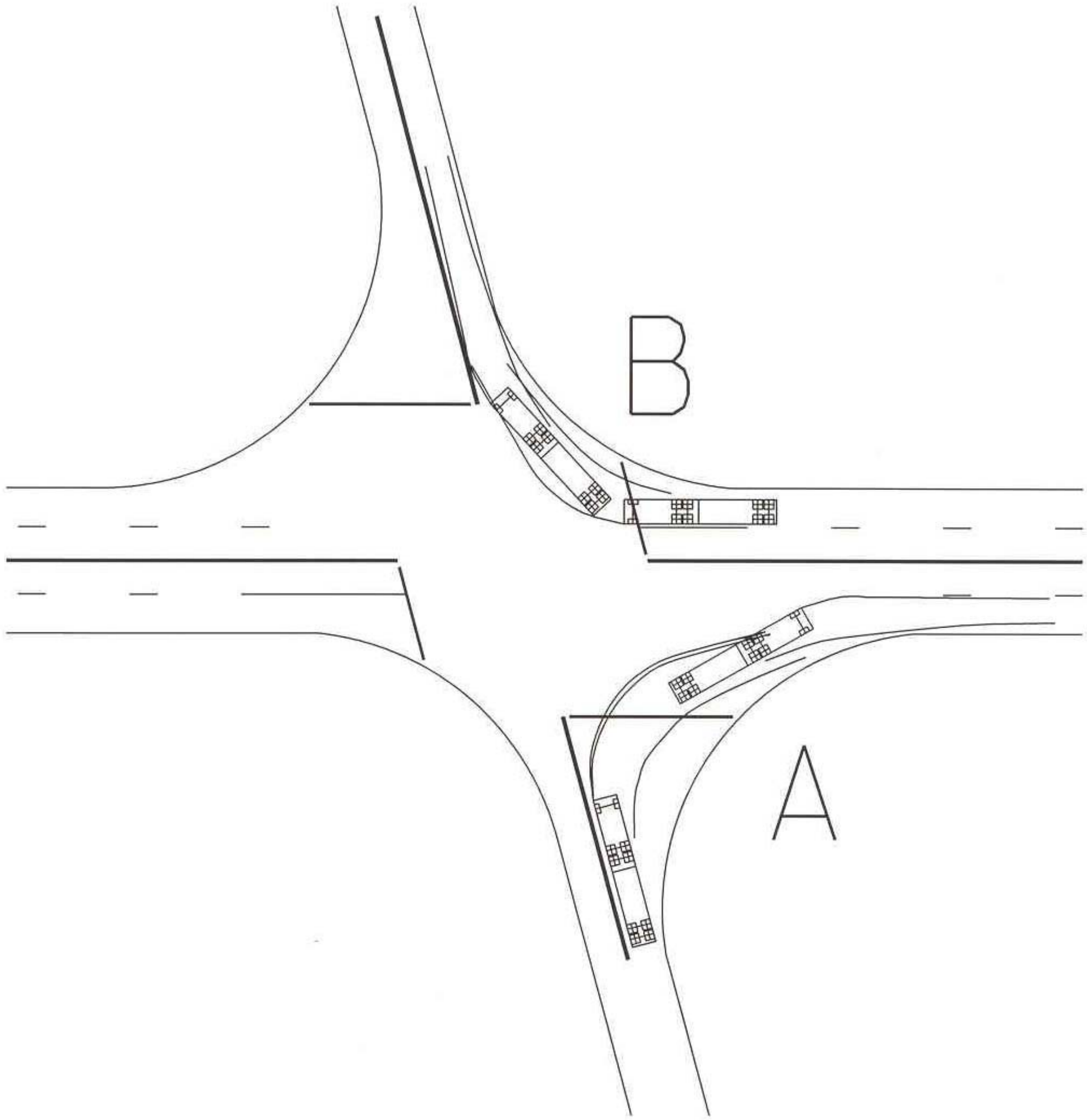


Figure 3-42. WB-50 [WB-15] on 100-ft [30 m] Radius Curve with 15-deg Skew (Pedestrian Elements Not Shown).

When crosswalk markings are added to the intersection in Figure 3-43, the design becomes more difficult to complete. According to the *TMUTCD*, stop lines must be placed no more than 30 ft [9 m] from the nearest edge of the intersecting roadway, and must be parallel to and 4 ft [1.2 m] in advance of any crosswalk markings present. Figure 3-43 shows the stop lines and crosswalk markings at the *TMUTCD* limits. As shown in corner A of the figure, it would not be possible to satisfactorily mark the crosswalk without infringing on those requirements.

Other possibilities for the intersection that could be investigated include the use of smaller curve radii on corner A, and the use of alternative corner designs such as a simple curve radius with taper or compound curves. The use of these alternative designs might alleviate the problems shown in Figure 3-43.

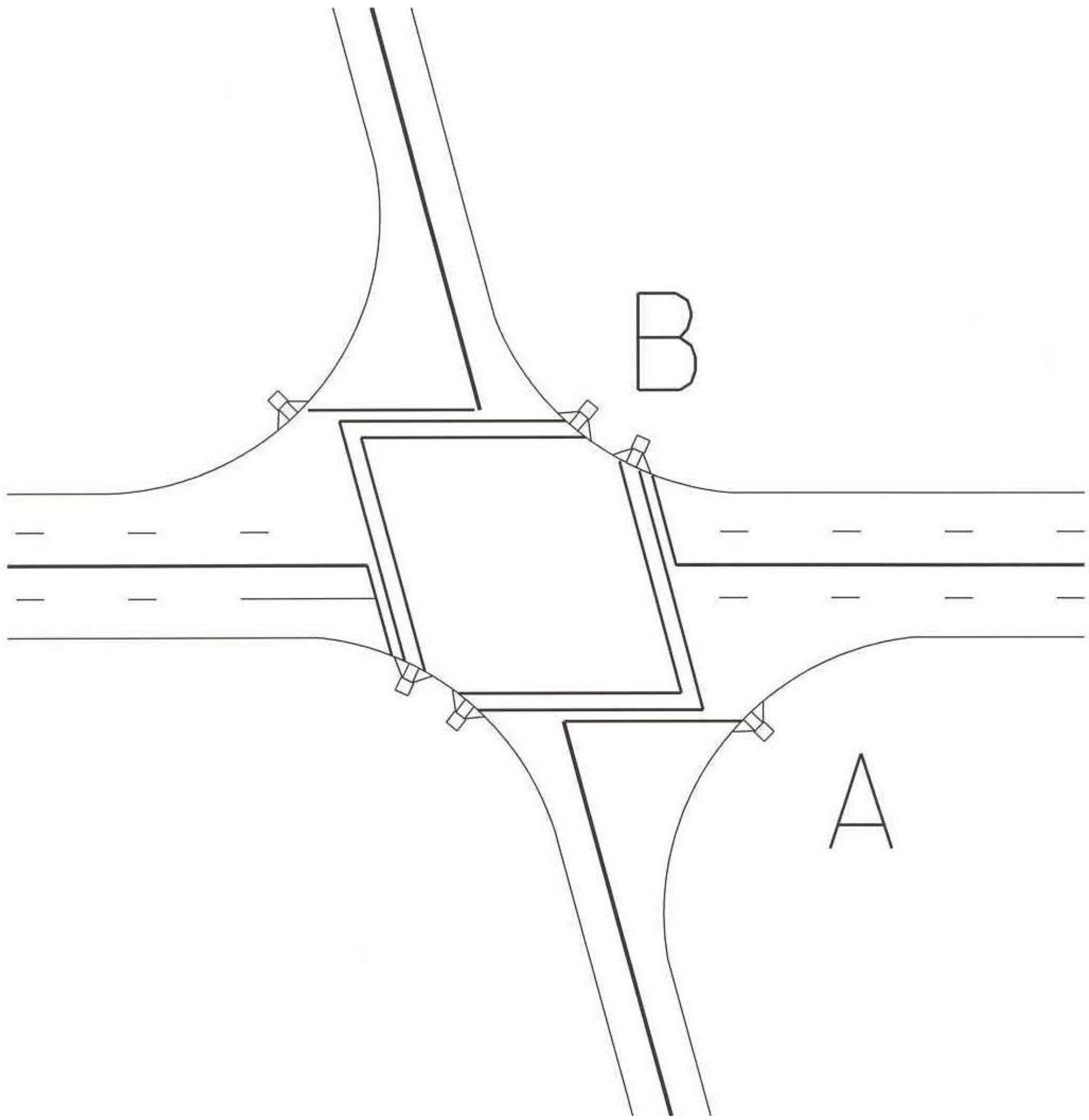


Figure 3-43. WB-50 [WB-15] on 100-ft [30 m] Radius Curve with 15-deg Skew for Pedestrian Elements and Stop Lines Shown.

Figure 3-44 shows a WB-50 [WB-15] truck turning template on the same skewed intersection, but islands have been added to provide channelization. The presence of the islands greatly reduces the area of the intersection and allows the appropriate placement of crosswalks and stop lines.

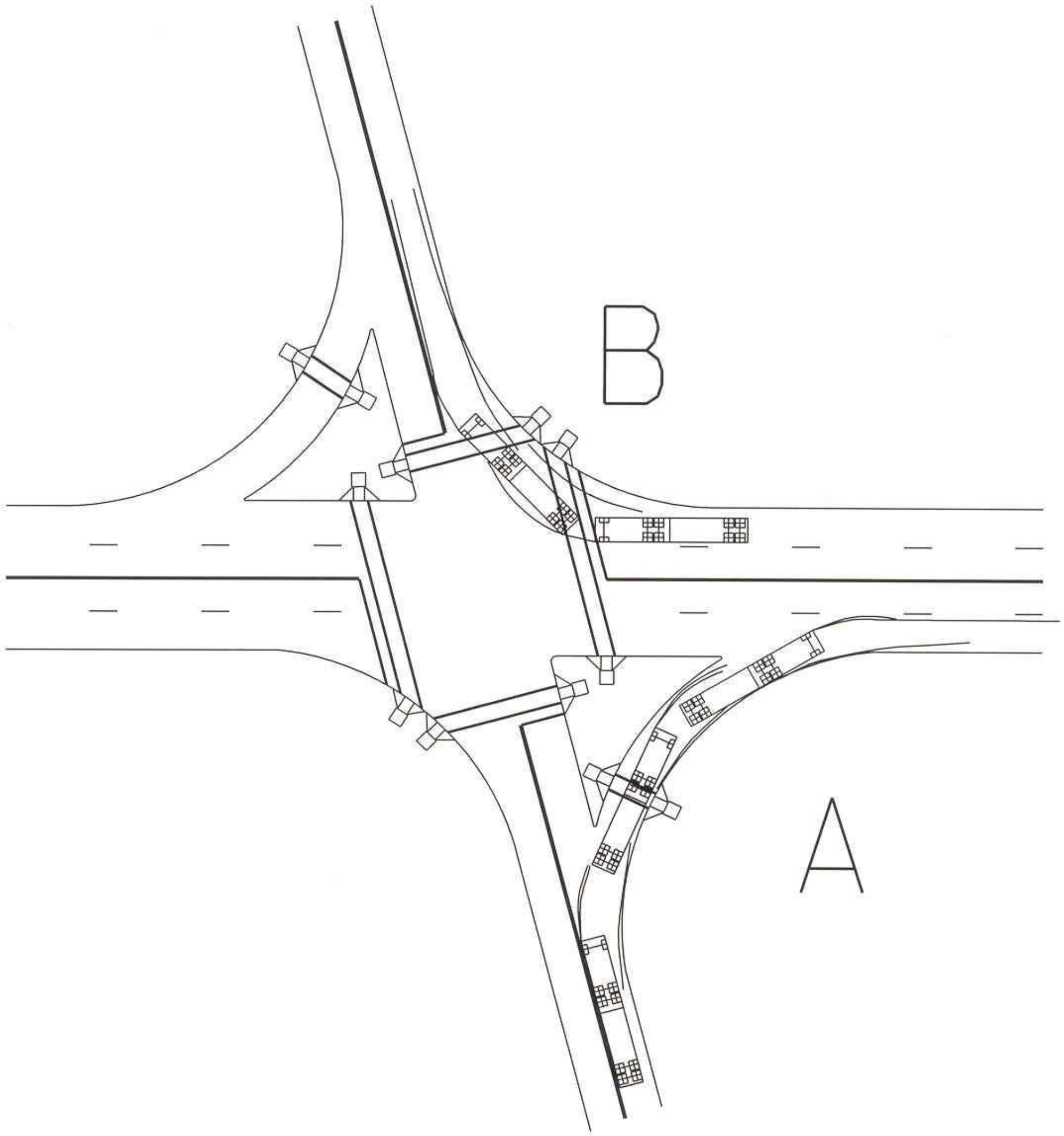


Figure 3-44. Use of Islands on 100-ft [30 m] Radius Curve with 15-deg Skew.

Chapter 4
Cross Section

Contents:

Application 1

Lane Drop After Intersection

Overview

Discussion on dropping a lane after an intersection is presented in the *Urban Intersection Design Guide*, Chapter 4, Section 1 <link>. Lanes may be dropped after intersections because of reduced volumes on succeeding roadway segments or because of the limits of a particular project. If an overall corridor is being provided with an increased number of through lanes but the construction is in segments, it may be more efficient to construct the end intersections using the "final" section and drop the additional lanes after the intersection. This can allow future construction projects to avoid performing work in the intersection.

Background

Problem. Birch Street is being expanded to three through lanes, with the project ending just east of its intersection with 13th Street. The project will end past 13th Street to avoid reconstructing the intersection in a future construction project planned for 5 years hence. The design of the lane drop should consider the required separation distance from the intersection and the appropriate taper.

Known Information. Figure 4-1 illustrates the current intersection layout. The following is known:

- The existing median openings are designed using a 40-ft [12.2 m] radius.
- The existing right-turn radii are 20 ft [6.1 m].
- The intersection is signalized with a cycle length of 90 sec.
- Truck percentage on Birch St. is 14 percent.
- The area is a newly designated industrial development.
- Pedestrians are relatively few in number.
- Projected traffic volumes in vph, design speeds, and roadway classifications are shown in Figure 4-1.

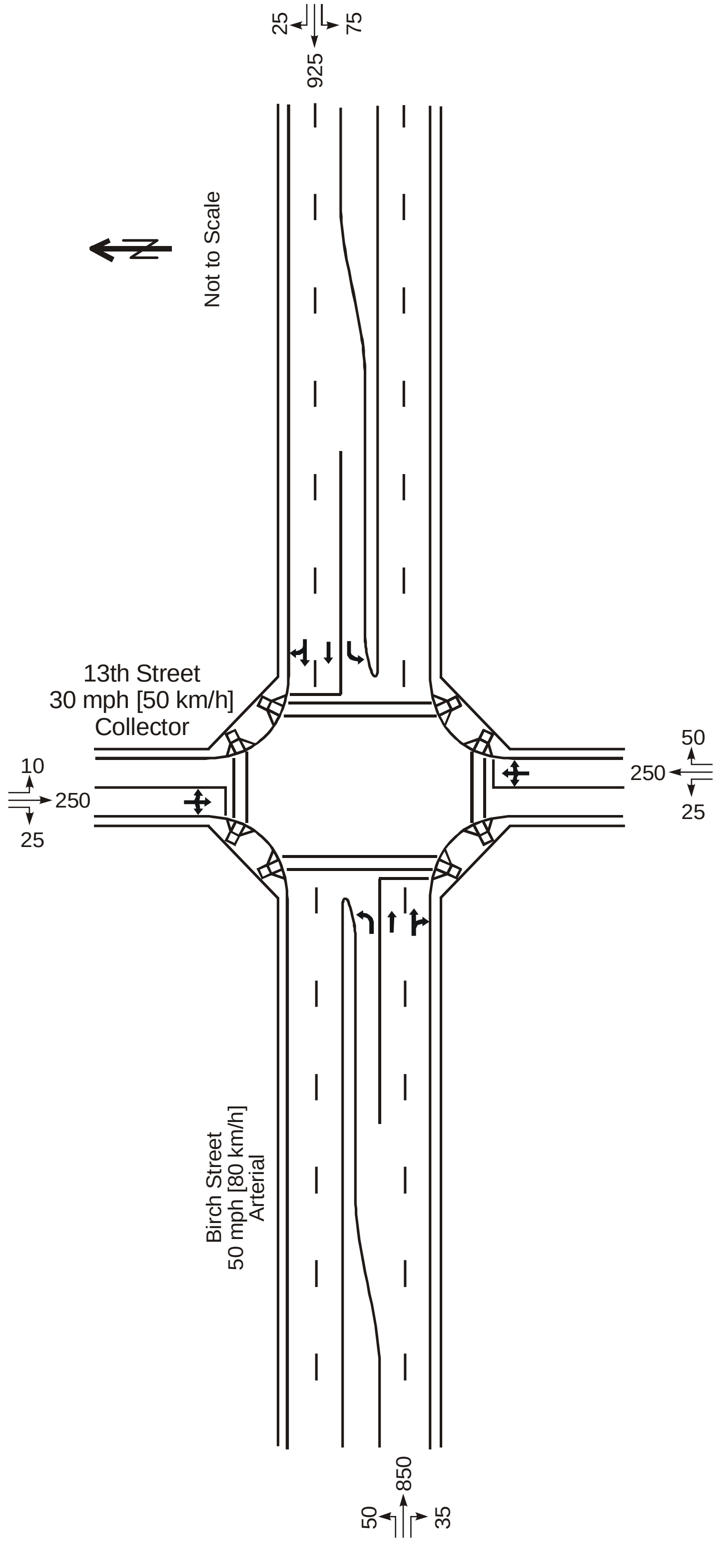
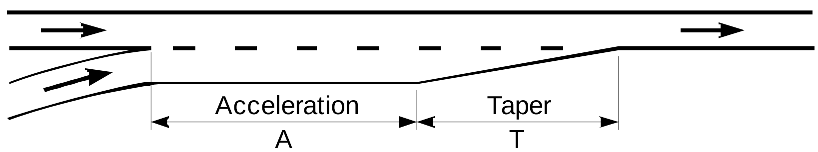


Figure 4-1. Existing Conditions.

Proposed Design. Following are the steps used to generate the proposed design for adding through lanes.

- *Step 1: Calculate taper length.*

The taper length on Birch Street is determined from Figure 3-10 of the *Roadway Design Manual* <link> (reproduced as Figure 4-2 in this document) and is determined by the design speed. From Figure 4-2, a taper length of 230 ft [70 m] is used with a design speed of 50 mph [80 km/h].



Highway Design Speed (mph)	Minimum Length of Taper T (ft)	Acceleration Length, A (ft) for Entrance Curve Design Speed (mph).								
		Stop Condition	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**
		AND INITIAL SPEED (mph)								
		0	**14**	**18**	**22**	**26**	**30**	**36**	**40**	**44**
30	150	180	140	–	–	–	–	–	–	–
35	165	280	220	160	–	–	–	–	–	–
40	180	360	300	270	210	120	–	–	–	–
45	200	560	490	440	380	280	160	–	–	–
50	**230**	**720**	**660**	**610**	**550**	**450**	**350**	**130**	**–**	**–**
55	250	960	900	810	780	670	550	320	150	–
60	265	1200	1140	1100	1020	910	800	550	420	180
65	285	1410	1350	1310	1220	1120	1000	770	600	370
70	300	1620	1560	1520	1420	1350	1230	1000	820	580
75	330	1790	1730	1630	1580	1510	1420	1160	1040	780

Note:
Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1300 ft.
Lengths of Right-Turn Acceleration Lanes (US Customary).

Figure 4-2. Length of Right-Turn Acceleration Lanes[1] (Reproduced from Roadway Design Manual Figure 3-10 <link>).

- *Step 2: Calculate acceleration length.*

The acceleration distance from a stop condition with a design speed of 50 mph [80 km/h] is provided also in Figure 3-10 of the *Roadway Design Manual* <link> (see Figure 4-2 in this document). From Figure 4-2, acceleration length is 720 ft [219 m].

- *Step 3: Calculate storage length for left-turn lanes.*

The required storage may be obtained using an acceptable traffic model such as the latest version of the HCM software (HCS), SYNCHRO, VISSIM, or other acceptable simulation models as suggested in the *Roadway Design Manual*. Alternatively, the required storage

[1] Texas Department of Transportation. *Roadway Design Manual*. Revised April 2002. http://manuals.dot.state.tx.us/dynaweb/coldesig/rdw.pdf. Accessed September 2003.

length can be estimated according to the *Roadway Design Manual's* Table 3-3 (see Table 4-1 of this document) or with the following *storage length* equation:

L = (V/N)(2)(S)

Where:

L = storage length, ft

V = left-turn volume per hour, vph

N = number of cycles/hour for the traffic signal,

2 = factor that provides for all left-turning vehicles on most cycles; a value of 1.8 may be acceptable on collector streets;

S = queue storage length per vehicle, ft.

Table 4-1. Lengths of Single Left-Turn Lanes on Urban Streets[1]
(From *Roadway Design Manual* Table 3-3).

(US Customary)						
Speed (mph)	**Deceleration Length[A,B] (ft)**	**Taper Length (ft)**	**Storage Length[A]**			
			Signalized		**Non-Signalized**	
			Calculated	**Minimum[D]**	**Calculated[E]**	**Minimum[D]**
30	160	50	See footnote C	100	See footnote E	100
35	215	50	See footnote C	100	See footnote E	100
40	275	50	See footnote C	100	See footnote E	100
45	345	100	See footnote C	100	See footnote E	100
50	***425***	***100***	***See footnote C***	***100***	***See footnote E***	***100***
55	510	100	See footnote C	100	See footnote E	100
Metric						
Speed (km/h)	**Deceleration Length[A,B] (m)**	**Taper Length (m)**	**Storage Length**			
			Signalized		**Non-Signalized**	
			Calculated	**Minimum[D]**	**Calculated[E]**	**Minimum[D]**
50	50	15	See footnote C	30	See footnote E	30
60	65	15	See footnote C	30	See footnote E	30
70	85	30	See footnote C	30	See footnote E	30
80	***105***	***30***	***See footnote C***	***30***	***See footnote E***	***30***
90	130	30	See footnote C	30	See footnote E	30

[A]The minimum length of a left-turn lane is the sum of the deceleration length plus queue storage. In order to determine the design length, the deceleration plus storage length must be calculated for peak and off-peak periods; the longest total length will be the minimum design length.

[B]See Deceleration Length discussion immediately following Table 3-3.

[C] ***See Storage Length Calculations discussion immediately following Table 3-3A.***

[D] ***The minimum storage length shall apply when: 1) the required queue storage length calculated is less than the minimum length, or 2) there is no rational method for estimating the left-turn volume.***

[E]The calculated queue storage at unsignalized location using a traffic model or simulation model or by the following:

L=(V/30)(2)(S)

Where: (V/30) is the left-turn volume in a two-minute interval and other terms are as defined in the Storage Length Calculations discussion immediately following Table 3-3A.

The storage length of vehicle, S, is determined by the percentage of trucks. The *Roadway Design Manual* provides the following:

% of Trucks	*S, ft [m]*
<5	25 [7.6]
5 to 9	30 [9.1]
10 to 14	**35 [10.7]**
15 to 19	40 [12.2]

Because the percent trucks on Birch Street is 14 percent, S is determined to be 35 ft [10.7 m]. The number of cycles per hour is determined by the cycle length used at the intersection. There are 40, 90-sec cycles in one hour (3600 sec/90 sec = 40 cycles).

Eastbound Birch Street Through Lanes. Because the through (and right-turn) traffic will now be distributed over three lanes, for the eastbound direction the traffic, volume per lane is given by:

$V = (V_{Thru} + V_{Rt})/NL$

Where:

V = traffic volume per hour per lane, vphpl

V_{Thru} = through traffic volume, vphpl

V_{Rt} = right-turn traffic volume, vphpl

NL = number of lanes

Substituting in the equation:

$V = (850 + 35)/3$

$V = 295$ vphpl

Substituting in the storage equation to obtain the expected queue length for the through lanes (and the through/right-turn lane):

$L = (295 \text{ vph}/40)(2)(35)$

$L = 516$ ft [157 m]

Westbound Birch Street Through Lanes. Traffic volume per lane for westbound Birch Street is given by:

$V = (925 + 25)/3$

$V = 320$ vphpl

Substituting in the *storage length* equation to obtain the expected queue length for the through lanes (and the through/right-turn lane):

$L = (320 \text{ vph}/40)(2)(35)$

L = 560 ft [171 m]

- *Step 4: Calculate storage length for left-turn lane.*

The storage length of the left-turn lane is also determined by the *storage length* equation:

L = (V/N)(2)(S)

Eastbound Birch Street Left-Turn Lane. From the traffic volumes in Figure 4-1, the left-turn volume is 50 vph. Substituting in the equation:

L = (50/40)(2)(35)

L = 88 ft [27 m]

This length is less than the minimum queue length of 100 ft [30 m] shown in Table 4-1 (which is a reproduction of Table 3-3 of the *Roadway Design Manual <link>)*, and less than the storage length determined for the through lanes (516 ft [157 m]). Therefore, the left-turn storage length should be increased. By increasing the storage length to match that of the through lanes, the through-lane queue should not block the entrance to the left-turn lane.

Westbound Birch Street Left-Turn Lane. From the traffic volumes in Figure 4-1, the left-turn volume is 75 vph. Substituting in the equation:

L = (75/40)(2)(35)

L = 131 ft [40 m]

This length is greater than the minimum queue length of 100 ft [30 m], but less than the storage length for the through lanes (560 ft [171 m]). Therefore, the left-turn storage length should be increased to match that of the through lanes.

- *Step 5: Determine taper and deceleration length for left-turn lane.*

The taper length for the left-turn lanes on Birch Street is shown in Table 4-1 as 100 ft [30 m] using the design speed of 50 mph [80 km/h]. The deceleration length is also provided in Table 4-1; the length given is 425 ft [130 m].

- *Step 6: Provide dimensions for lane lengths.*

The proposed design for the addition of the through lane on each of the Birch Street approaches along with the left-turn lane is shown in Figure 4-3.

- *Step 7: Select right-turn radius and median-turn radius.*

The radii used in the intersection design shown in Figure 4-3 were selected with regard to the location and traffic mix found at the site. Because the site is in an industrial area with high truck volumes and low pedestrian volumes, relatively large radii were used. A WB-50 [WB-15] design vehicle was used to select the radii for both the right-turn and median turns.

Right-Turn Radius. The 40-ft [12.2 m] right-turn radii were used to accommodate the larger vehicles found at the site, based on recommendations in the *Roadway Design Manual,* Chapter 7, Section 7 <insert link to page 7-26 of RDM> for urban intersections with frequent turns by combination trucks. Islands were not used because of their potential for impeding these vehicles. The use of radii of this size may result in higher turning speeds for vehicles. This is undesirable for areas with larger numbers of pedestrians but acceptable for an industrial area with few pedestrians. The turning path of a WB-50 [WB-15] truck with a right-turn radius of 40 ft [12.2 m] is shown in Figure 4-4.

Median-Turn Radius. A 60-ft [18 m] median-turn radius was selected to accommodate the WB-50 [WB-15] truck without increasing the size of the intersection more than necessary. A turn template is overlaid on the intersection design in Figure 4-4 to show the minimum path of the design vehicle used.

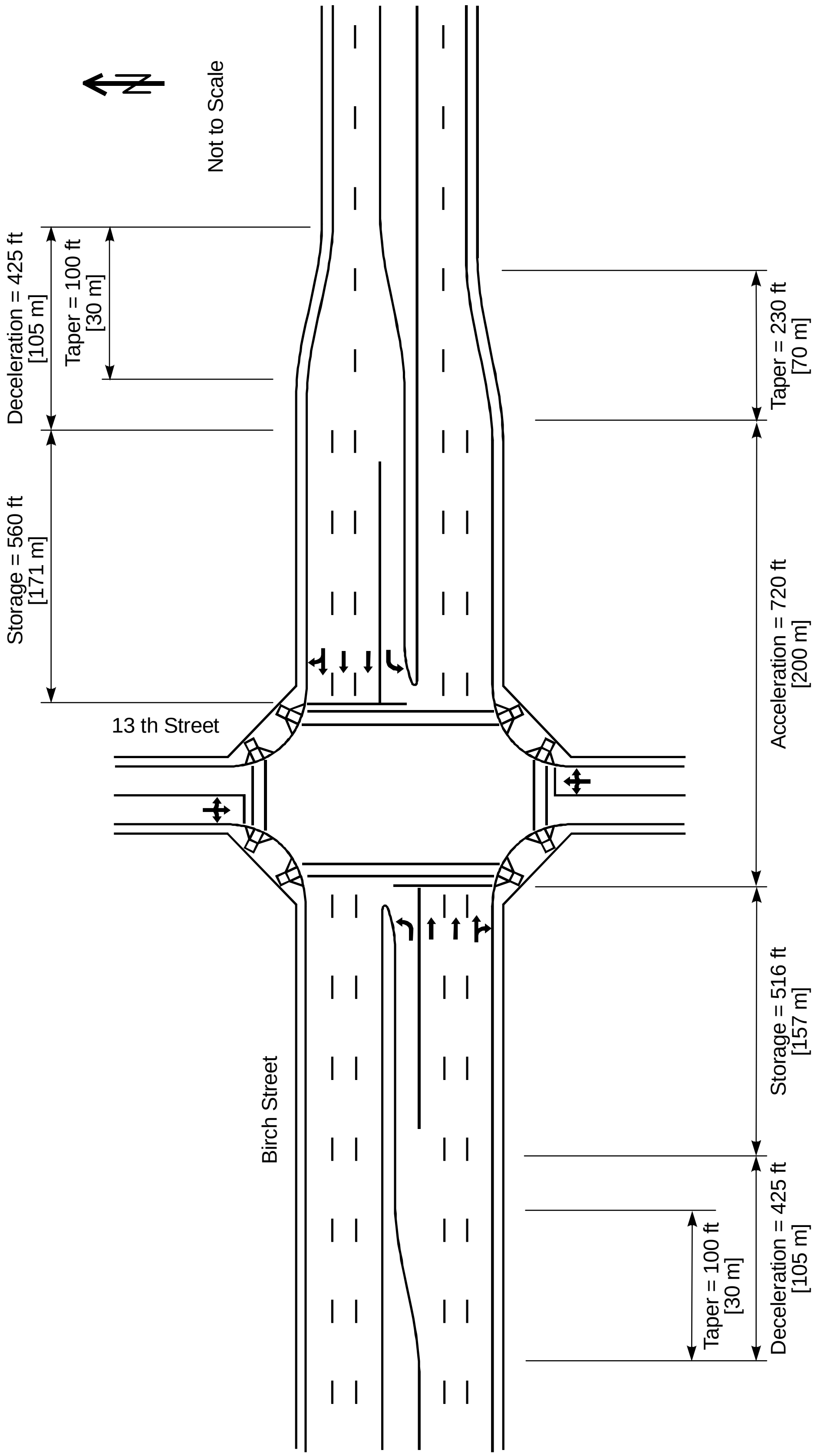


Figure 4-3. Proposed Design.

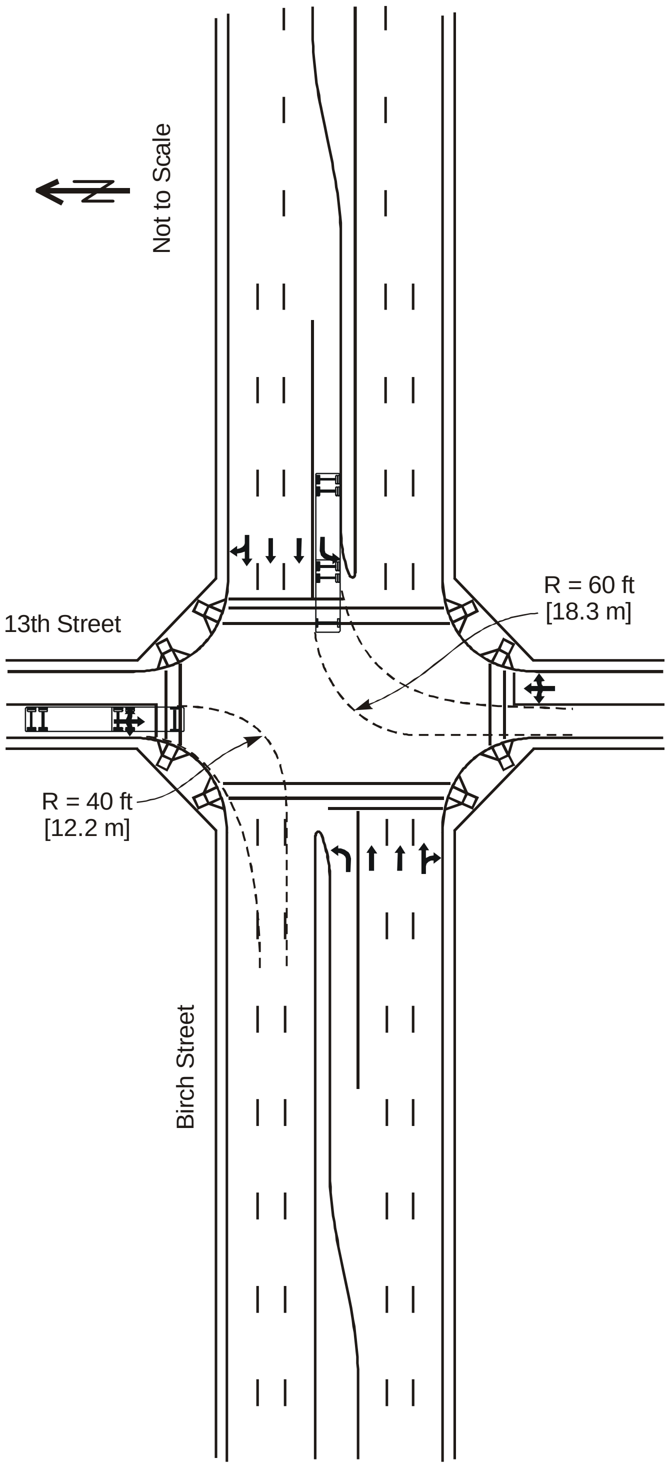


Figure 4-4. Design Overlaid with WB-50 [WB-15] Truck Template.

Application 2
Reallocation of Cross Section

Overview

The following application discusses an approach of reallocating pavement to a different lane configuration. The *Urban Intersection Design Guide*, Chapter 4, Section 1 <link> provides discussion on through lanes.

Background

An old city with a population of approximately 200,000 has two minor arterial streets that consistently have higher-than-average accident rates. Washington Street is a four-lane, undivided street with a pavement width of 43 ft [13 m] (face-of-curb to face-of-curb) that predominately serves an older section of the city. Most adjacent development is commercial with some multifamily residential. There are several elementary schools, a junior high school, and two large public parks that are located either adjacent to or in close proximity to the roadway. Traffic volumes on the street are about 15,000 vehicles per day. The percentage of trucks on the street is very low. The speed limit on Washington Street is 35 mph [56 km/h].

Brock Avenue is a four-lane, undivided street with a pavement width of 40 ft [12 m] (face-of-curb to face-of-curb) that serves a variety of developments, including both commercial and industrial properties. This street has a higher percentage of large vehicles (trucks) compared to other arterial streets in the city. Traffic volumes on the street are about 13,000 vehicles per day. The speed limit on Brock Avenue is 35 mph [56 km/h].

The city engineering staff has reviewed the accident histories of both streets and determined that a large percentage of the accidents involved vehicles making unsafe left turns, or were rear-end collisions involving stopped or slow-moving vehicles attempting to turn left or right from the travel lanes. Review of turning movement counts and observations of traffic flow on both streets indicated that a large number of left and right turns were being made from both streets. The ideal solution to the accident problem for both arterials was to widen the streets and provide either a median with separated left-turn lanes or a continuous, center, left-turn lane. In addition, separate right-turn lanes were desired at specific locations along the streets.

The desired street improvements were quickly eliminated from consideration. Both roadways were developed many years ago with numerous driveways along the streets to serve the adjacent developments. The city's street system was not conducive to providing alternate routes to provide access to the developments from the rear. Hence, the possibilities of installing a median or prohibiting turns along both streets were quickly eliminated from consideration. In addition, many developments adjacent to the streets were positioned very close to the right of way. Existing rights of way outside of the curb lines on both streets were very narrow. Hence, the acquisition of additional right of way for roadway widening would be extremely difficult and expensive.

The city staff eventually determined that any roadway improvements would have to be made within the existing curb lines. Therefore, the number of potential operational improvements was few.

Issues Considered

The city staff recognized that because of the large number of left turns being made along the street, the inside travel lanes were being used essentially as left-turn lanes. Vehicles traveling in opposite directions that were stopped at intersections while attempting to make left turns were laterally offset from one another, which significantly affected the ability of drivers of those vehicles to observe other vehicles approaching from behind the opposing left-turning vehicles. The consistent interaction between through vehicles and turning vehicles (especially with the 10-ft-wide [3 m] travel lanes on Brock Avenue) reduced operational efficiency and available capacity. Ideally, providing separated left-turn lanes was the desired solution to the problem. Neither street had sufficient right of way or existing pavement width, however, to permit the addition of a median or left-turn lane and maintain two travel lanes in both directions.

The only alternative was to reduce the number of lanes on the streets to three lanes, providing a continuous center, left-turn lane and a single travel lane in each direction. This type of alternative, recently termed as a road diet, has been implemented in other cities with some success where conditions are conducive to lane reductions. Generally, four-lane streets can accommodate about 18,000 to 20,000 vehicles per day. These capacity volumes are reduced substantially if there are a large number of turning vehicles, especially left-turning vehicles. Two-lane streets can accommodate about 7000 to 10,000 vehicles per day. Again, these capacity volumes are reduced substantially if there are a large number of turning vehicles.

Providing three travel lanes appeared to be a realistic compromise considering the traffic volumes on the two streets. Both streets had traffic volumes much less than the capacity volumes of a typical four-lane street, so the three-lane cross section would not likely create a serious congestion problem. Obviously, a three-lane street would be inadequate to accommodate volumes greater than 20,000. The three-lane cross section would separate left-turning vehicles from through vehicles and provide drivers of those left-turning vehicles with better visibility conditions to see conflicting vehicles. Hence, the recommended three-lane cross section was considered to be a safer design that likely would reduce the accident rates.

One concern was that drivers may confuse the existing concrete joints for the lane lines. The city decided to include an overlay to cover the joint lines.

Designs Selected

As shown in Figure 4-5, the existing striping on Washington Street provides four travel lanes, each 10.75 ft [3.27 m] in width. The first alternative striping design, also shown in Figure 4-5, considered providing two 15.5-ft-wide [4.7 m] through lanes and a 12-ft-wide [3.7 m] continuous, center, left-turn lane. The wide through lanes created a concern that speeds would increase due to the wide travel lanes. Also, a single wide travel lane possibly would encourage occasional use as two narrow travel lanes or encourage on-street parking. Neither condition would be desirable. Hence, a wider center lane was considered in order to narrow the through lanes.

However, the city staff considered a second alternative, shown in Figure 4-5, that would include narrower travel lanes and bicycle lanes. The 11-ft-wide [3.4 m] travel lanes were considered sufficiently wide for the predominantly passenger-vehicle traffic on the street but sufficiently narrow as to encourage maintenance of the existing typical vehicle speeds. Right-turn movements from the 11-ft-wide [3.4 m] through lanes would benefit with the additional 5 ft [1.5 m] of pavement width from the bicycle lanes. The bicycle lanes would encourage bicycle use to and from the schools (especially the junior high school) and the recreational facilities. The 5-ft-wide [1.5 m] bicycle lanes were 1 ft [0.3 m] narrower than the city staff's preferred 6 ft [1.8 m] width; however, Williams Boulevard is constructed of portland cement concrete so there is no gutter seam. Therefore, the entire 5-ft-wide [1.5 m] bicycle lane would be available for use. The second alternative was preferred by the city staff, the city council, and local citizens and was selected for implementation.

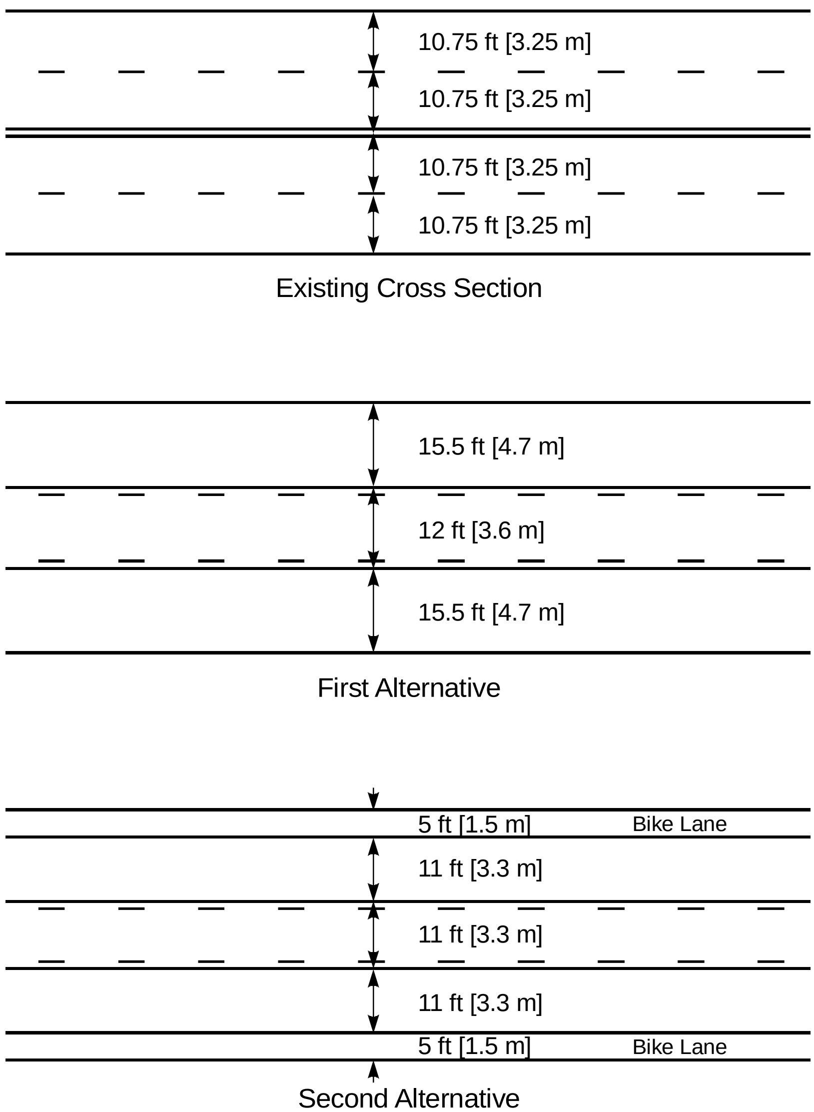


Figure 4-5. Existing and Proposed Alternatives for Williams Street.

As shown in Figure 4-6, the existing striping on Brock Avenue provided four, 10-ft-wide [3 m] travel lanes. The proposed restriping, also shown in Figure 4-6, provided 14-ft [4.3 m] travel lanes and a 12-ft-wide [3.7 m] center two-way left-turn lane. The outside lane widths were considered desirable to accommodate right turns by the numerous large vehicles that use the street. The width of the center lane was considered appropriate for all vehicle sizes. The street was too narrow to consider bicycle lanes; however; considering the minimal bicycle activity in the area, bicycle lanes were not desired. The recommended striping plan was selected for implementation.

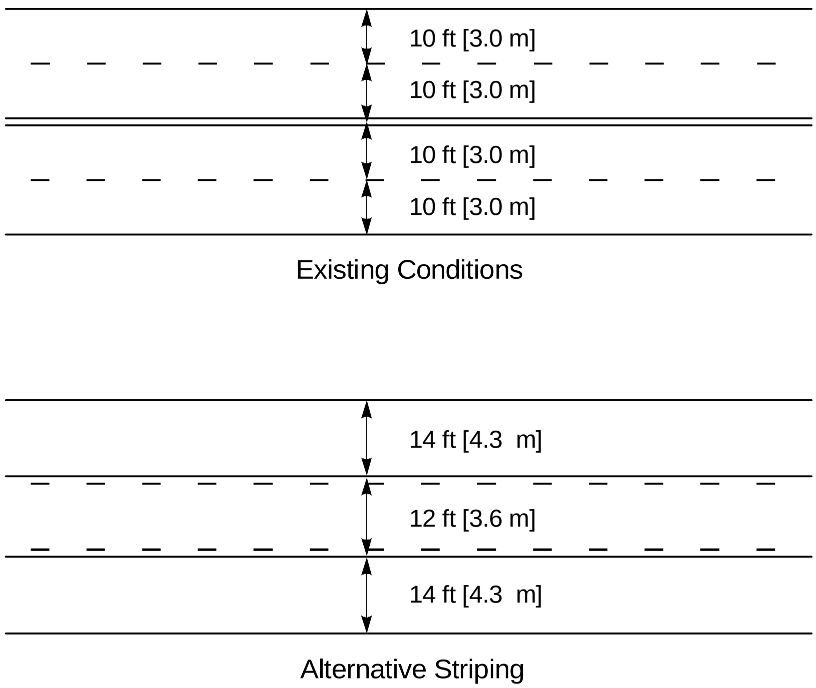


Figure 4-6. Existing and Proposed Alternatives for Brock Avenue.

Application 3
Inclusion of Left-Turn Lane

Overview

The following application explores information available on evaluating when to consider a left-turn lane. The *Urban Intersection Design Guide,* Chapter 4, Section 2 <link> provides information on left-turn lane design.

Effectiveness of Left-Turn Lanes

The inclusion of a left-turn lane can result in a reduction in crashes. A recent Federal Highway Administration (FHWA) study on 280 three- and four-leg intersections found the expected effectiveness on crash reduction as shown in Table 4-2.

Table 4-2. Expected Effectiveness of Left-Turn Lanes on Crash Reduction.

		Crash Reduction (%)	
Intersection Type	**Intersection Traffic Control**	**Left Turn Installed on One Approach**	**Left Turn Installed on Both Approaches**
Rural			
Three-leg intersection	Stop Sign	44	
	Traffic Signal	15	
Four-leg intersection	Stop Sign	28	48
	Traffic Signal	18	13
Urban			
Three-leg intersection	Stop Sign	33	
	Traffic Signal	7	
Four-leg intersection	Stop Sign	27	47
	Traffic Signal	10	19

When to Install a Left-Turn Lane

The decision to include a left-turn lane may be governed by a city's thoroughfare plan or guidelines based on function class of the intersecting roadways. For example, the intersection of two major arterials typically includes left-turn lanes (and in some cases dual left-turn lanes). Other factors considered include available sight distances and crash history. TxDOT's *Roadway Design Manual* <link> contains guidance for use in determining when to consider a left-turn lane on two-lane highways (see Table 4-3). A similar table is also present in the AASHTO *Green Book*. Figure 4-7 illustrates the following terms used in the table along with other terms needed for the procedure:

- ***Advancing Volume (V_A)*** **–** the total peak hourly volume of traffic on the major road approaching the intersection to include through, right- and left-turn volumes.
- ***Left-Turn Volume (V_L)*** **–** the portion of the advancing volume that turns left at the intersection.

- ***Percent Left Turns (PL)*** **–** the percentage of the advancing volume that turn left; equal to the left-turn volume divided by the advancing volume (PL = V_L/V_A).
- ***Straight Through Volume (V_S)*** **–** the portion of the advancing volume that travels straight through the intersection ($V_L + V_S = V_A$).
- ***Opposing Volume (V_O)*** **–** the total peak hourly volume of vehicles opposing the advancing volume.

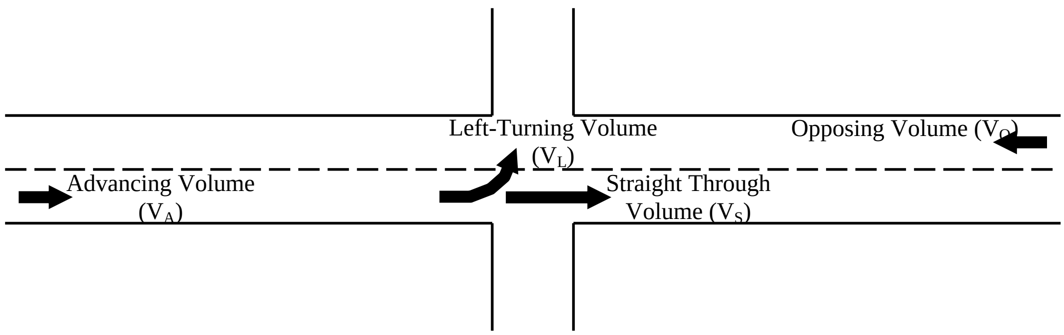


Figure 4-7. Volume Definitions.

Table 4-3. Guide for Left-Turn Lanes on Two-Lane Highways (Reproduction from TxDOT *Roadway Design Manual* Table 3-11 <link>).

Opposing Volume (vph)	Advancing Volume (vph)			
	5% Left Turns	**10% Left Turns**	**20% Left Turns**	**30% Left Turns**
40 mph [64 km/h] operating speed				
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
50 mph [80 km/h] operating speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	240
100	615	445	335	295
60 mph [100 km/h] operating speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

Should a Left-Turn Lane Be Installed?

Problem. In a rapidly developing area, citizens are concerned with the number of cars using the shoulder to pass slow-moving left-turning vehicles (see Figure 4-8). The area is currently considered rural; however, the anticipated development within the next 5 years will change the performance of the roadway. The state highway will provide more access and less mobility. It has already started evolving into a suburban high-speed arterial. The designers noted that the TxDOT *Access Management Manual* contains criteria on connection spacing for Other State Highways <link>. They decided to check both the *Access Manual* and the *Roadway Design Manual* as part of their traffic operations evaluation.

Figure 4-8. Example of Vehicle Using Shoulder to Pass Left-Turning Vehicle.

Known Information. The information known for this site includes:

- Peak hour turning movement counts are shown in Figure 4-9.
- The 85th percentile speed is 59 mph [95 km/h].
- Posted speed is 55 mph [89 km/h].
- Grades at the intersection are level.
- Lane widths are 12 ft [3.7 m].
- Shoulder width on the major road is 10 ft [3.0 m].
- Another left-turn bay is not within 425 ft [130 m] of the site.

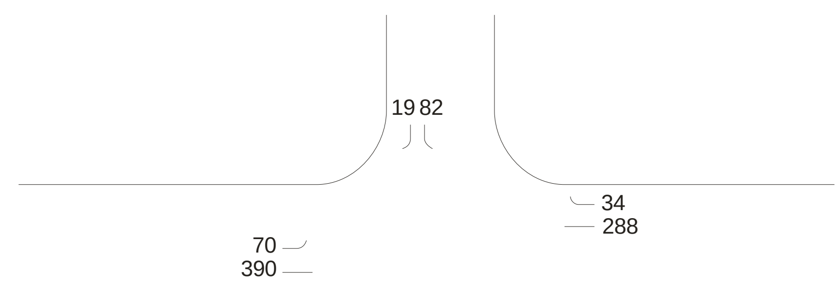


Figure 4-9. Peak Hour Turning Movement Count.

Solution. Following is the solution for this example:

- ♦ *Step 1: Determine the opposing, advancing, and percent left-turn volumes.*

Advancing volume consists of those vehicles moving toward the intersection on the same approach as the left-turning vehicles. For this example it is:

390 + 70 = 460 vph

Opposing volume consists of those vehicles that conflict with the left-turning vehicle. Therefore it should include both through and right-turning vehicles. For this example it is:

288 + 34 = 322 vph

The percent left-turn volume is determined based on the approaching volume. Therefore, it is:

70 vehicles turning left / 460 advancing vehicles = 0.152 or 15 percent

- ♦ *Step 2: Use table to determine if a left-turn lane should be considered.*

The volumes are used with the guidelines in Table 4-3, reproduced from Table 3-11 in the *Roadway Design Manual* <link> to determine if a left-turn lane should be considered. The operating speed of 59 mph [95 km/h] is near 60 mph [97 km/h]; therefore, the data for 60 mph [97 km/h] are used in this example. Because the exact volumes are not provided in the table, interpolation must be used. Table 4-4 shows the interpolated volumes for this situation.

Table 4-4. Interpolated Volumes.

Advancing Volume (vph)			
Opposing Volume	**10% Left Turns**	***15% Left Turns***	**20% Left Turns**
400	270	***235***	200
322	***293***	***256***	***220***
200	330	***290***	250
Original data are from TxDOT *Roadway Design Manual* Table 3-11 <link> for design speed of 60 mph [97 km/h]. ***Bold and italics values are the interpolated values.***			

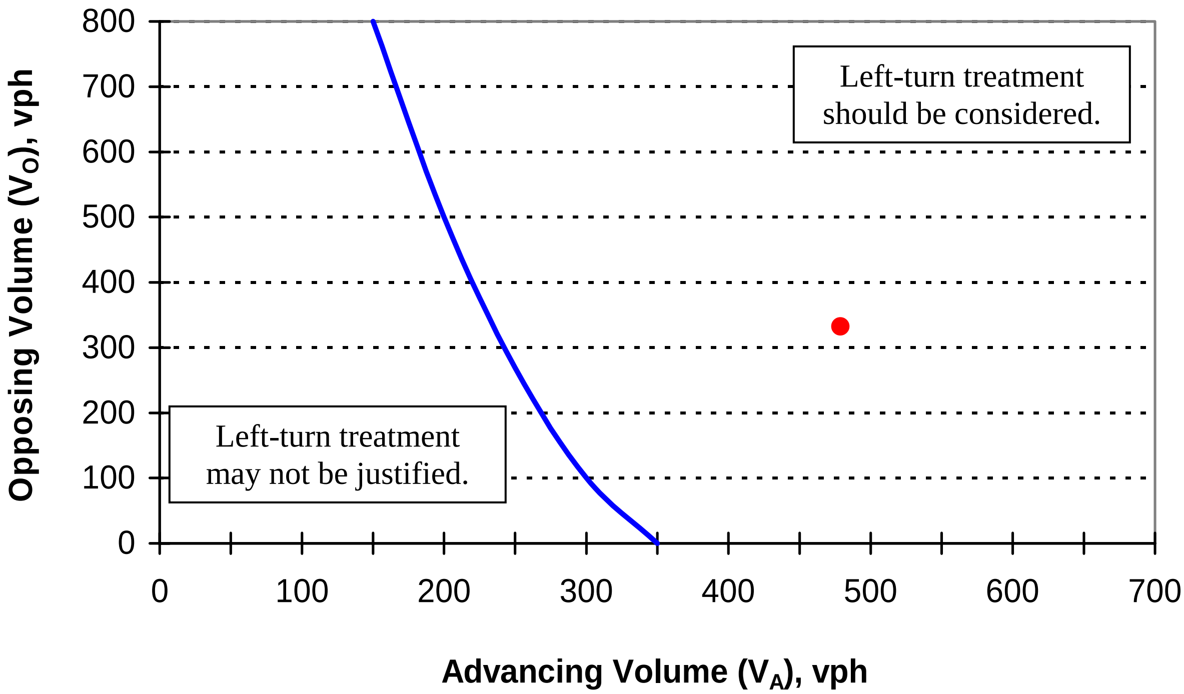
The advancing volume determined from the table is 256 vph. The number of vehicles advancing toward the left-turning vehicles as measured in the field was 460 vph. Because 460 vph exceeds 256 vph, a left-turn lane should be considered.

Another method for evaluating this situation is to use the spreadsheet included in NCHRP 457.[2] Table 4-5 and Figure 4-10 show the results from the evaluation.

[2] Bonneson, J., and M. Fontaine, *Engineering Study Guide for Evaluating Intersection Improvements*, NCHRP Report No. 457. Available Online. <http://trb.org/trb/publications/nchrp/esg.pdf> Accessed: August 2003. Transportation Research Board, Washington, D.C., 2001.

Table 4-5. NCHRP 457[2] Results for Application.

Input	
Variable	**Value**
85^{th} percentile speed, mph	60
Advancing volume (V_A), vph	460
Opposing volume (V_O), vph	322
Output	
Limiting advancing volume (V_A), vph	238
Guidance for determining the need for a major-road left-turn bay:	
Left-turn treatment should be considered.	



• = plot of Advancing Volume and Opposing Volume for the example intersection (see Table 4-5).

Figure 4-10. Results for Application Using Material from NCHRP 457.[2]

♦ *Step 3: Check TxDOT Access Management Manual.*

The *Access Management Manual*, Table 2-2 lists spacing criteria for Other State Highways <link>. With a posted speed in excess of 50 mph [81 km/h], the spacing distance is 425 ft [130 m]. Currently a turn bay for another intersection or driveway is not present within that distance.

A driveway is present approximately 300 ft [91 m] west of the intersection. The design team decided to move forward with the turn lane at the intersection and to inform the property owner that a median opening at the driveway will not be considered when the highway is widened.

Application 4
Left-Turn Lane

Overview

The presence of large numbers of left-turning vehicles can degrade the performance of an intersection. Higher traffic volumes and degraded intersection performance may justify the construction of a left-turn lane. The left-turn storage bay design length can be determined by the projected volumes of left-turn and through traffic volumes and the design speed. The *Urban Intersection Design Guide*, Chapter 4, Section 2 <link> provides design guidelines.

Background

The intersection of Diamond Boulevard and Douglas Street is in an urban area. Diamond Blvd. is a four-lane arterial with a raised median, while Douglas St. is a two-lane collector. Developments on each corner are:

- ♦ convenience stores on the northeast and northwest corners,
- ♦ a fast-food restaurant on the southwest corner, and
- ♦ a small shopping center on the southeast corner.

This intersection has a relatively high left-turn volume. The presence of the left-turning vehicles restricts the traffic flow through the intersection and limits the options available for signalization strategies that could reduce delay in the intersection.

Figure 4-11 illustrates the current intersection layout. The following is known:

- ♦ Diamond Boulevard is an urban arterial roadway.
- ♦ Douglas Street is an urban collector roadway.
- ♦ Design speed on Diamond Boulevard is 45 mph [72 km/h].
- ♦ The intersection is signalized.
- ♦ Cycle length of the traffic signal is 75 sec.
- ♦ Truck percentage is 8 percent.
- ♦ Projected design hour traffic volumes in vph are shown in Figure 4-11.
- ♦ Median width is 18 ft [5.5 m].
- ♦ Median width at crosswalk is 6 ft [1.8 m].

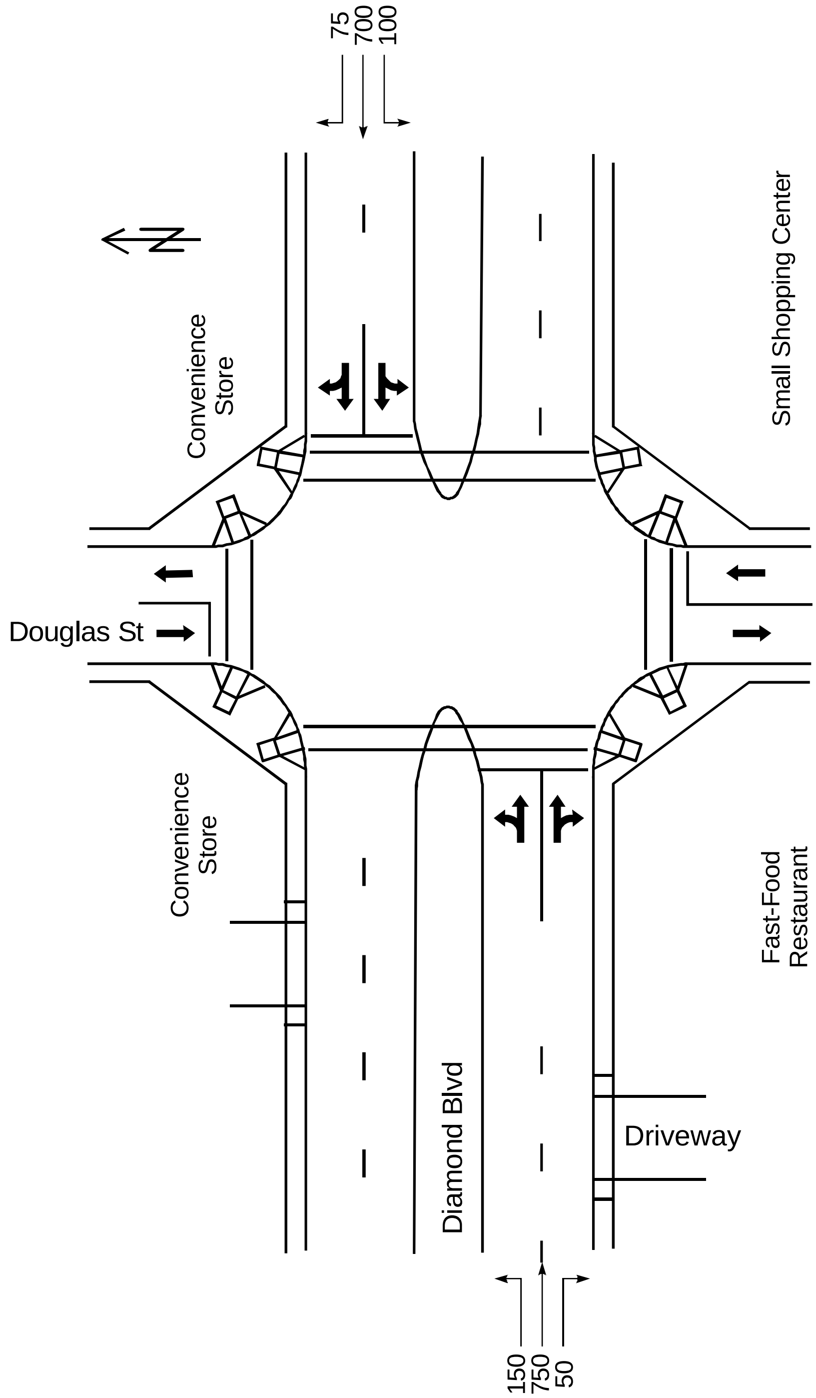


Figure 4-11. Existing Intersection Layout.

Issues Considered

Issues considered during an upgrade to the site include the following:

- ♦ Provide upgraded traffic signal hardware including:
  - longer mast-arms,
  - replacement of foundations and signal poles as necessary, and

  - left-turn signal head.

- Provide left-turn speed change lanes that:
  - accommodate the queues in the left-turn lanes, and
  - consider the length of the queues in the through lanes so they do not block the entrance to the left-turn bays.

Proposed Design

The design of the left-turn lane includes both deceleration and storage. The design of the storage available for left-turning vehicles depends on the queues projected to develop in the turn lanes and the adjacent through lanes. The through lane queues should be estimated because of the possibility of their blocking the entrance to the turn lanes. Other characteristics of the design include the width of the median and the width of the turn lane.

- *Step 1: Determine left-turn storage length.*

The left-turn storage length is determined according to Table 3-3 of the *Roadway Design Manual* (reproduced in Table 4-6).

The required storage length is a calculated length based on the queue (with a minimum of 100 ft [30 m]). The calculated length may be obtained using an acceptable traffic model such as the latest version of the HCM software (HCS), SYNCHRO, VISSIM, or other acceptable simulation models, as suggested in the *Roadway Design Manual*. However, if those techniques have not been employed, then the queue may be estimated by the following *storage length* equation:

$L = (V/N)(2)(S)$

Where:

L = storage length, ft

V = left-turn volume per hour, vph

N = number of cycles/hour for the traffic signal,

2 = factor that provides for all left-turning vehicles on most cycles; a value of 1.8 may be acceptable on collector streets,

S = queue storage length per vehicle, ft.

Table 4-6. Lengths of Single Left-Turn Lanes on Urban Streets (Reproduction of *Roadway Design Manual* Table 3-3 <link>)

(US Customary)						
Speed (mph)	**Deceleration Length[2] (ft)**	**Taper Length (ft)**	**Storage Length**			
			Signalized		**Non-Signalized**	
			Calculated	**Minimum[4]**	**Calculated[5]**	**Minimum[4]**
30	160	50	See footnote 3	100	See footnote 5	100
35	215	50	See footnote 3	100	See footnote 5	100
40	275	50	See footnote 3	100	See footnote 5	100
45	***345***	***100***	***See footnote 3***	***100***	***See footnote 5***	***100***
50	425	100	See footnote 3	100	See footnote 5	100
55	510	100	See footnote 3	100	See footnote 5	100
Metric						
Speed (km/h)	**Deceleration Length[2] (m)**	**Taper Length (m)**	**Storage Length**			
			Signalized		**Non-Signalized**	
			Calculated	**Minimum[4]**	**Calculated[5]**	**Minimum[4]**
50	50	15	See footnote 3	30	See footnote 5	30
60	65	15	See footnote 3	30	See footnote 5	30
70	85	30	See footnote 3	30	See footnote 5	30
80	105	30	See footnote 3	30	See footnote 5	30
90	130	30	See footnote 3	30	See footnote 5	30

[1]The minimum length of a left-turn lane is the sum of the deceleration length plus queue storage. In order to determine the design length, the deceleration plus storage length must be calculated for peak and off-peak periods; the longest total length will be the minimum design length.

[2]See Deceleration Length discussion immediately following Table 3-3.

[3] ***See Storage Length Calculations discussion immediately following Table 3-3A.***

[4] ***The minimum storage length shall apply when: 1) the required queue storage length calculated is less than the minimum length, or 2) there is no rational method for estimating the left-turn volume.***

[5]The calculated queue storage at unsignalized location using a traffic model or simulation model or by the following:

L = (V/30)(2)(S)

Where: (V/30) is the left-turn volume in a two-minute interval and other terms are as defined in the Storage Length Calculations discussion immediately following Table 3-3A.

The storage length of vehicles, S, is determined by the percentage of trucks. The *Roadway Design Manual* provides the following:

% of Trucks	*S, ft [m]*
<5	25 [7.6]
5 to 9	***30 [9.1]***
10 to 14	35 [10.7]
15 to 19	40 [12.2]

Because the percent trucks on Diamond Blvd. is 8, S is 30 ft [9.1 m]. The number of cycles per hour is determined by the cycle length used at the intersection, 75 sec, or 48 cycles per hour.

Substituting in the equation for left-turning vehicles approaching from the east on Diamond Blvd.:

L = (150 vph/48)(2)(30)

L = 188 ft [57 m]

Substituting in the equation for left-turning vehicles approaching from the west on Diamond Blvd.:

L = (100 vph/48)(2)(30)

L = 125 ft [38 m]

- *Step 2: Check through lane queue.*

The queue lengths should be compared to the estimated through-lane queue, to see if that queue will extend back far enough to block vehicles from entering the left-turn lane. The same technique is used to estimate the through-lane queue. The volume used in the equation is the number of through vehicles per lane for eastbound and westbound through traffic on Diamond Blvd., 800 and 775 vph, respectively. The volumes are split evenly between the two through lanes available in each direction.

Substituting in the equation for through vehicles approaching from the east on Diamond Blvd.:

L = (400 vph/48)(2)(30)

L = 500 ft [152 m]

Substituting in the equation for through vehicles approaching from the west on Diamond Blvd.:

L = (388 vph/48)(2)(30)

L = 485 ft [148 m]

Because the through-lane queue is estimated to be longer than the left-turn lane queue, its length is used for the design of the left-turn lane. The left-turn lane design is shown in Figure 4-12. This design obviously occupies a considerable length of roadway, and may exceed the block spacing in some locations. Practical constraints such as this may necessitate the installation of turn bays that are shorter than those otherwise desired.

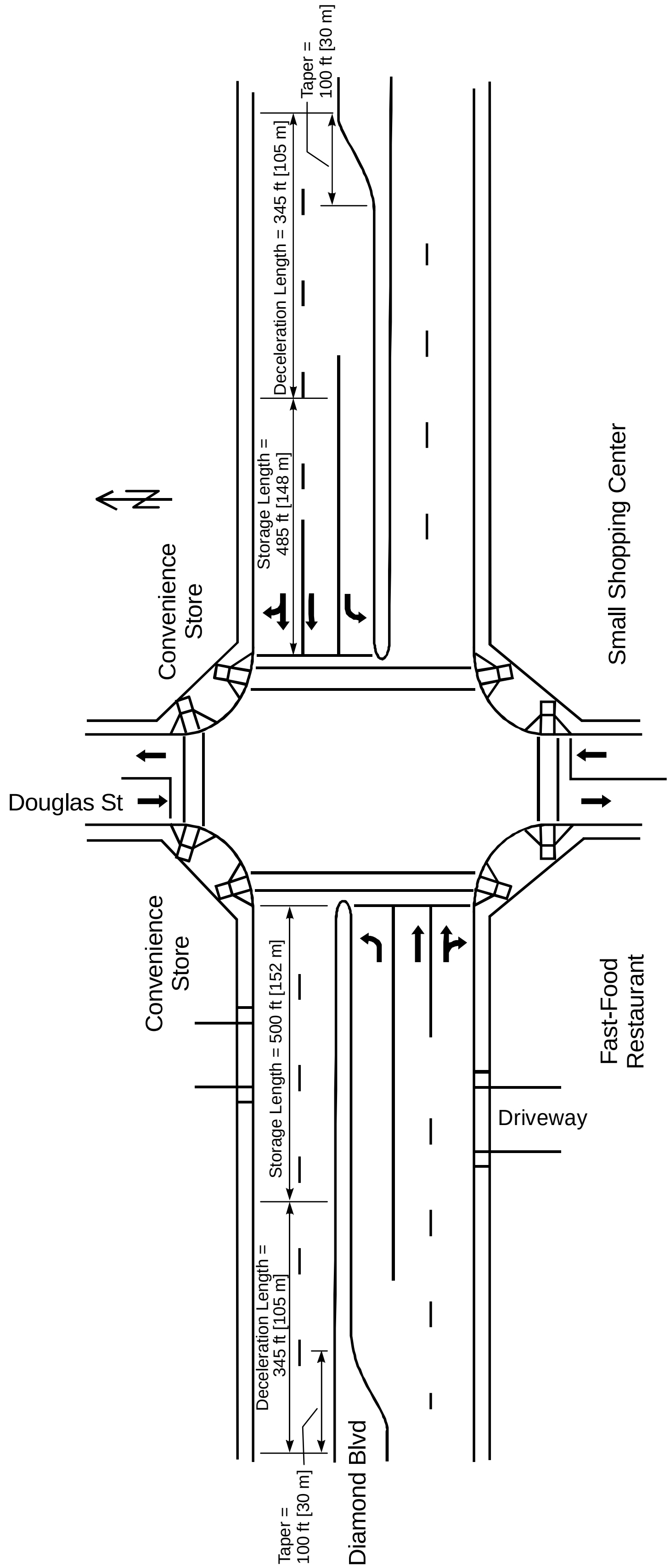


Figure 4-12. Proposed Intersection Layout.

- *Step 3: Determine left-turn deceleration and taper length.*

As shown in Table 4-6, the deceleration length and taper length are provided as 345 ft [105 m] and 100 ft [30 m], respectively.

- *Step 4: Determine median and turn lane width.*

The width of the left-turn lane was selected from Table 3-1 of the *Roadway Design Manual.* The range allowed is 11 to 12 ft [3.4 to 3.7 m] desirable and 10 ft [3 m] minimum; a 12-ft [3.7 m] lane width was selected. The width of the median prior to the inclusion of a turn lane was 18 ft [5.5 m], greater than the minimum width of 16 ft required for the design of a single left-turn lane as discussed in the *Roadway Design Manual*. The use of a 12-ft [3.7 m] lane allows the retention of a 6-ft [1.8 m] median adjacent to the turn lane, meeting pedestrian refuge width requirements (see *Roadway Design Manual*, Chapter 4, Section 5 <link>).

- *Step 5: Relocate crosswalk and update traffic signals.*

The addition of the left-turn lanes requires the end of the median lines to be moved back to allow the same turning radius as used previously (50 ft [15 m]). A bullet nose shape was used for the median end to minimize the distance the nose was set back from the intersection. For further information, see Chapter 4, Section 5, Median End Treatment Design <link> and Figure 4-24 <link> of that chapter of the *Roadway Design Manual*.

The crosswalk across Diamond Blvd. was placed in approximately the same location as the previous design; no refuge area is provided in the median. If the crosswalks were moved far enough back to provide pedestrian refuge areas within the median, the stop lines would be too far back from the intersection to meet *TMUTCD* requirements (stop lines should be placed 4 ft [1.2 m] prior to crosswalks, and should be no more than 30 ft [9 m] from the face of the curb on the intersecting roadway). The pedestrian curb ramps were relocated to match the new crosswalk locations.

The traffic signal will be updated to provide a signal head with a left-turn indication. The additional mast-arm length will require a larger signal pole and pole foundation.

Application 5
Offset Left-Turn Lanes

Overview

Left-turn lanes are used to provide space for the deceleration and storage of turning vehicles.[1] They may be used to improve safety and/or operations at intersections. However, vehicles in opposing left-turn lanes can limit each other's views of conflicting traffic. Benefits of offset left-turn lanes include:

- better visibility of opposing through traffic,
- decreased possibility of conflict between opposing left-turn movements within the intersection, and
- more left-turn vehicles served in a given period of time (particularly at signalized intersections).

Guidelines[3] were developed for offsetting opposing left-turn lanes at 90-deg intersections on level, tangent sections of divided roadways with 12-ft (3.7 m) lanes <link to Table 4-3 of the *Guide*>. The guidelines presented in Table 4-3 of the *Urban Intersection Design Guide* would typically involve reconstructing the left-turn lanes. Increasing the width of the lane line between the left-turn lane and the adjacent through lanes can also improve the sight distance by encouraging the driver to position the vehicle closer to the median. For new location and full reconstruction projects, wider offsets are suggested with provisions for pedestrian refuge.

Example

The view of oncoming vehicles available to left-turning vehicles can be improved by reallocating the existing median to provide an offset left-turn lane.

Problem. An existing signalized intersection with a 6-ft [1.8 m] median adjacent to the left-turn bay is shown in Figure 4-13; the full median width is 18 ft [5.5 m]. The figure shows the visual blocking caused by a vehicle present in the opposing left-turn lane. As shown, oncoming vehicles are blocked from view of left-turning vehicles. An accident pattern of left-turning vehicles turning in front of oncoming traffic has become apparent, with a presumed cause of impaired sight distance.

[3] Tarawneh, M.S., and P.T. McCoy. Guidelines for Offsetting Opposing Left-Turn Lanes on Divided Roadways. In *Transportation Research Record 1579*, TRB, National Research Council, Washington, D.C., 1997, pp. 43-52.

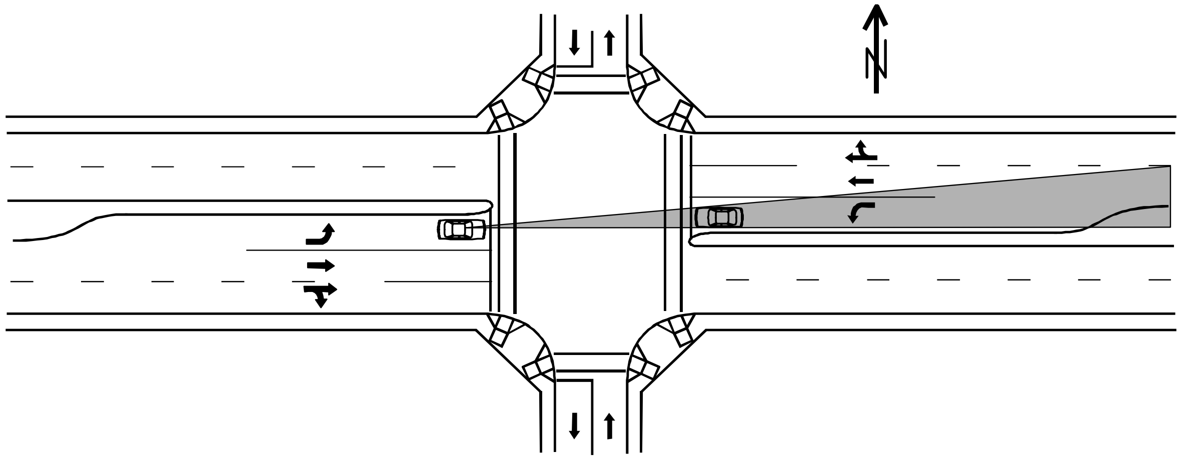


Figure 4-13. Existing Intersection Showing Visual Blocking.

The questions to be answered are as follows:

1. How wide should the left-turn lane line be?
2. How wide should the median be?
3. What improvement in sight distance can be expected from the wider lane line?

Solution. Left-turn lanes should desirably be 12-ft wide [3.7 m], while medians on urban arterials should be at least 2 ft [0.6 m] to avoid recurring damage to the divider.[1] Although the reduced median width will not be adequate to provide a pedestrian refuge, the signal timing provided is such that pedestrians may cross the intersection without stopping in the median.

Figure 4-14 shows the suggested design of a 4-ft [1.2 m] white line and a 2-ft [0.6 m] divider. In the proposed design the left-turning drivers' vision is not impaired. Figure 4-15 shows that vision is not impaired for left-turning drivers even if the opposing vehicle is a city transit bus.

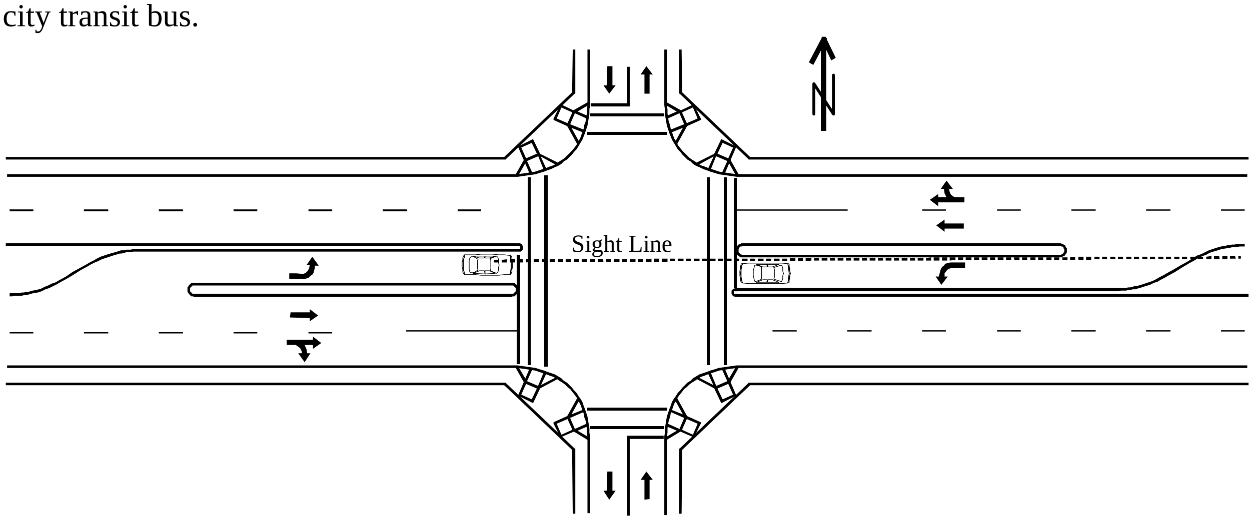


Figure 4-14. Proposed Design for Offset Left-Turn Lane.

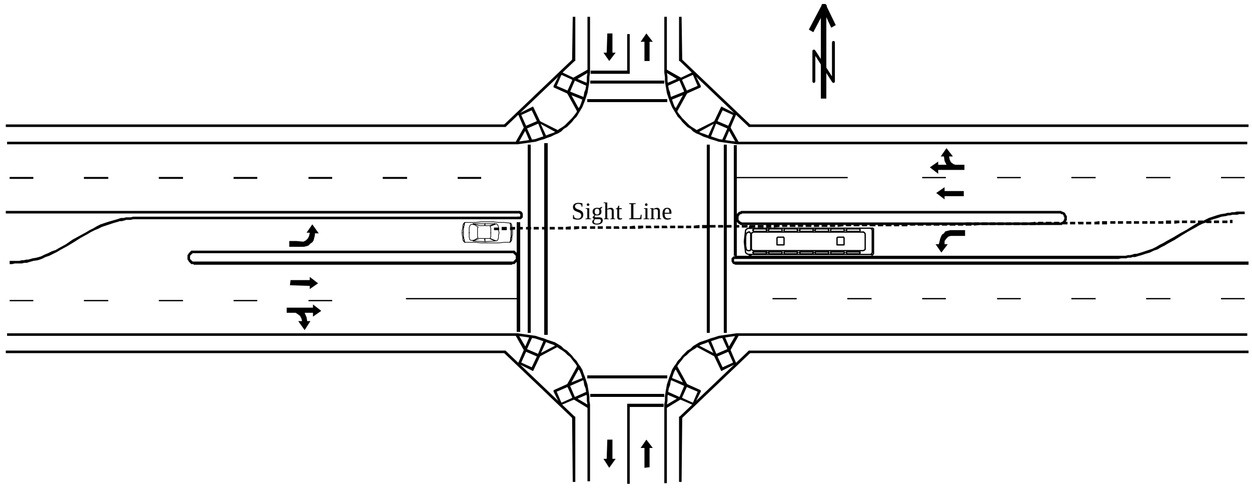


Figure 4-15. Proposed Design for Offset Left-Turn Lane with Bus.

Application 6
Adding Right-Turn Lane

Overview

Significant volumes of right-turning traffic can adversely affect the performance of an intersection. Higher turning volumes may warrant the addition of a right-turn lane to expedite turning movements and improve traffic signal operations. Right-turn lanes are discussed in the *Urban Intersection Design Guide*, Chapter 4, Section 3 <link>.

Background

An urban intersection has a substantial amount of right-turning traffic on a particular approach. Queues in the right-hand lane become lengthy during certain times of day, increasing delay and driver frustration.

The intersection of Jackson Road and Park Drive is in an urban area, near a large hospital and medical park. Jackson Road is a four-lane arterial with a raised median, while Park Drive is a two-lane collector. Developments on each corner are:

- a large parking area for the hospital on the southeast corner,
- a park area/green space adjacent to part of the medical park on the northeast corner,
- a fast-food restaurant on the northwest corner, and
- a free-standing pharmacy/variety store on the southwest corner.

The southwest corner is the location of the problem: eastbound right-turning vehicles on Jackson Road share the right-hand lane with eastbound through vehicles, causing delays. This southwest corner contains a driveway to the pharmacy.

Figure 4-16 illustrates the current intersection layout. The following is known:

- Jackson Road is an urban arterial roadway.
- Park Drive is an urban collector roadway.
- Design speed on Jackson Rd. is 45 mph [72 km/h].
- The intersection is signalized with a traffic signal cycle length of 90 sec.
- Truck percentage is 4 percent.
- Projected approach volume eastbound on Jackson Rd. is 720 vph.
- Projected right-turn volume from Jackson Rd. eastbound to Park Dr. southbound is 160 vph.
- Median width is 16 ft [4.9 m].

- Median width at crosswalk:
  - West side: 4 ft [1.2 m] and
  - East side: 7 ft [2.1 m].

Issues Considered

Issues to consider during an upgrade to the site include the following:

- Acquire right of way.
- Consider need to move utility lines.
- Move signal poles.
- Tie into existing sidewalk, but ensure grades are appropriate for usage by disabled pedestrians.
- Accommodate current traffic flow during construction.
- Accommodate moving of pharmacy driveway during construction.

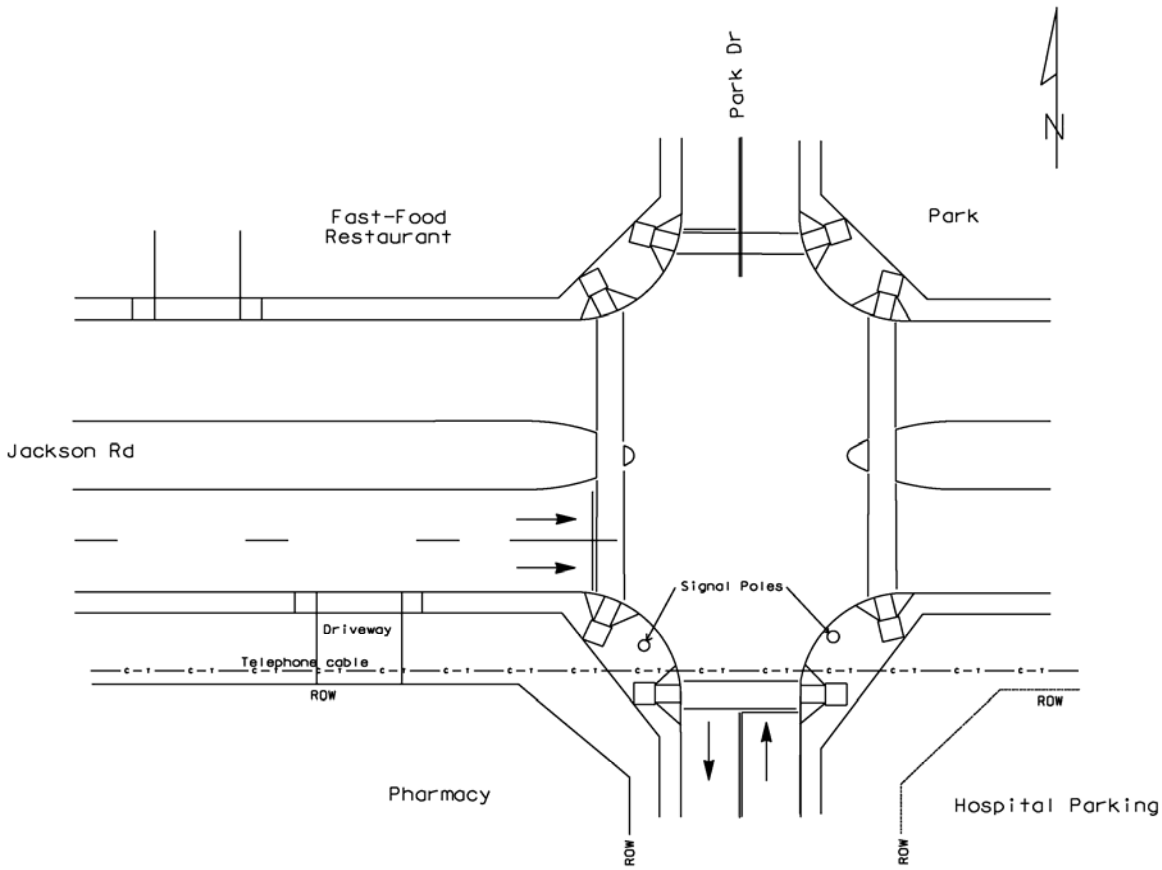


Figure 4-16. Existing Intersection Layout.

A range of options are available to enhance the operation of the intersection. The designer has chosen to focus on:

- Add a right-turn lane to the eastbound approach of Jackson Road to better distribute traffic approaching the intersection.

Other issues that were considered in the design include:

- Relocate the sidewalk to accommodate the new lane.
- Relocate pharmacy driveway to reduce conflicts from exiting vehicles.
- Relocate traffic signal poles to avoid the utility line, curb ramps, and the increased intersection area.
- Ensure that adequate width is provided for pedestrian refuge in the median.

Proposed Design

- *Step 1: Determine right-turn bay design.*

The right-turn bay design is determined according to Table 3-3 of the *Roadway Design Manual* (reproduced as Table 4-7). As shown in Table 4-6, the deceleration length and taper length are 345 ft [105 m] and 100 ft [30 m], respectively.

Table 4-7. Lengths of Single Left-Turn Lanes on Urban Streets Used in Right-Turn Bay Example (From *Roadway Design Manual*, Table 3-3).

(US Customary)						
Speed (mph)	**Deceleration Length[A,B] (ft)**	**Taper Length (ft)**	**Storage Length**			
			Signalized		**Non-Signalized**	
			Calculated	**Minimum[D]**	**Calculated[E]**	**Minimum[D]**
30	160	50	See footnote C	100	See footnote E	100
35	215	50	See footnote C	100	See footnote E	100
40	275	50	See footnote C	100	See footnote E	100
45	***345***	***100***	***See footnote C***	***100***	***See footnote E***	***100***
50	425	100	See footnote C	100	See footnote E	100
55	510	100	See footnote C	100	See footnote E	100
Metric						
Speed (km/h)	**Deceleration Length[A,B] (m)**	**Taper Length (m)**	**Storage Length**			
			Signalized		**Non-Signalized**	
			Calculated	**Minimum[D]**	**Calculated[E]**	**Minimum[D]**
50	50	15	See footnote C	30	See footnote E	30
60	65	15	See footnote C	30	See footnote E	30
70	***85***	***30***	***See footnote C***	***30***	***See footnote E***	***30***
80	105	30	See footnote C	30	See footnote E	30
90	130	30	See footnote C	30	See footnote E	30

[A]The minimum length of a left-turn lane is the sum of the deceleration length plus queue storage. In order to determine the design length, the deceleration plus storage length must be calculated for peak and off-peak periods, the longest total length will be the minimum design length.

[B]See Deceleration Length discussion immediately following Table 3-3.

[C]***See Storage Length Calculations discussion immediately following Table 3-3A.***

[D]***The minimum storage length shall apply when: 1) the required queue storage length calculated is less than the minimum length, or 2) there is no rational method for estimating the left-turn volume.***

[E]The calculated queue storage at unsignalized location using a traffic model or simulation model or by the following:

L = (V/30)(2)(S)

Where: (V/30) is the left-turn volume in a two-minute interval and other terms are as defined in the Storage Length Calculations discussion immediately following Table 3-3A.

The required storage length is based on the anticipated queue with 100 ft [30 m] being the minimum (see Table 4-6). Because right turns on red are prohibited, the queue should be determined. The required storage may be obtained using an acceptable traffic model such as the latest version of the HCM software (HCS), CORSIM, SYNCHRO, VISSIM, or other acceptable simulation models, as suggested in the *Roadway Design Manual*. However, if those techniques have not been used, then the queue may be estimated by the following equation:

L = (V/N)(2)(S)

Where:

L = storage length, ft

V = turning volume per hour, vph

N = number of cycles/hour for the traffic signal

2 = factor that provides for all left-turning vehicles on most cycles; a value of 1.8 may be acceptable on collector streets,

S = queue storage length per vehicle, ft.

The storage length of vehicle, S, is determined by the percentage of trucks. The *Roadway Design Manual*[1] provides the following:

% of Trucks	*S, ft [m]*
<5	**25 [7.6]**
5 to 9	30 [9.1]
10 to 14	35 [10.7]
15 to 19	40 [12.2]

Because the percent trucks on Jackson Rd. is 4, S is determined to be 25 ft [7.6 m]. The number of cycles per hour is determined by the cycle length used at the intersection, 90 sec, or 40 cycles per hour. Substituting in the equation:

L = (160 vph/40)(2)(25)

L = 200 ft [61 m]

- *Step 2: Check through lane queue.*

The queue length of 200 ft [61 m] should be compared to the estimated through-lane queue, to see if that queue will extend far enough to block vehicles from entering the right-turn lane. The same technique is used to estimate the through-lane queue.

The approach volume of 720 vph should be reduced by the number of vehicles turning right, 160, leaving 560 vehicles. Assuming that the remaining traffic is distributed evenly in the left lane (through and left-turning vehicles) and the right lane (through vehicles), this volume should be divided by two to determine the number of vehicles per lane of 280 vph. This volume of 280 vph is then used in the equation:

L = (280 vph/40)(2)(25)

L = 350 ft [107 m]

Because the through-lane queue is estimated to be longer than the right-turn lane queue, its length is used for the design of the right-turn lane.

The right-turn deceleration lane consists of a 345-ft [105 m] deceleration length and a 350-ft [107 m] storage length, for a total length of 695 ft [212 m]. Using a 100-ft [30 m] taper and 25-ft [7.6 m] radius, the full width length is 570 ft [174 m].

The lengths determined for the right-turn lane may not always be achievable because of factors such as the block length; it may not always be practical to accommodate the through-lane queue.

The downstream end of a right-turn lane is normally calculated to end at the face of the curb on the cross street as shown in Figure 3-4 of the *Roadway Design Manual* <link>. This design is shown in Figure 4-17. In this case, however, the city has requested that the overall length be calculated to end at the stop bar of the intersection so that the design does not depend on drivers stopping past the stop line and blocking the crosswalk. The completed right-turn lane design is shown in Figure 4-18.

The width of the right-turn lane was selected from Table 3-1 of the *Roadway Design Manual*. The range allowed is 11 to 12 ft [3.4 to 3.7 m] desirable and 10 ft [3 m] minimum; a 12-ft [3.7 m] lane width was selected.

- ♦ *Step 3: Relocate signal poles, sidewalks, and pharmacy driveway.*

As shown in Figure 4-17, the pharmacy driveway has been relocated onto Park Drive. This reduces the conflicts between the traffic queues at the traffic signal and vehicles entering the pharmacy driveway (see *Urban Intersection Design Guide*, Chapter 11, Section 3 <link>). This type of solution may not always be practical; TxDOT's *Access Management Manual* should be consulted for further guidance.

Although the telephone cable appears to be located in an acceptable location in Figure 4-17, its location should be confirmed in the field to determine any need for increased burial depth or other modification to the line.

The traffic signal poles have been relocated to avoid conflicts with the curb ramps and to relocate them for the widened roadway. This will result in the use of longer traffic signal mast arms, larger support poles, and larger pole foundations (see *Urban Intersection Design Guide*, Chapter 8, Section 3 <link>).

The width of the median at the west side of the intersection was originally 4 ft [1.2 m]. This is less than the preferred 6 ft [1.8 m] or minimum 5 ft [1.5 m] for pedestrian refuge (see *Urban Intersection Design Guide*, Chapter 4, Section 5 <link>) so the crosswalk and ramps were relocated, and 6 ft [1.8 m] was provided. The east side was satisfactory without modification.

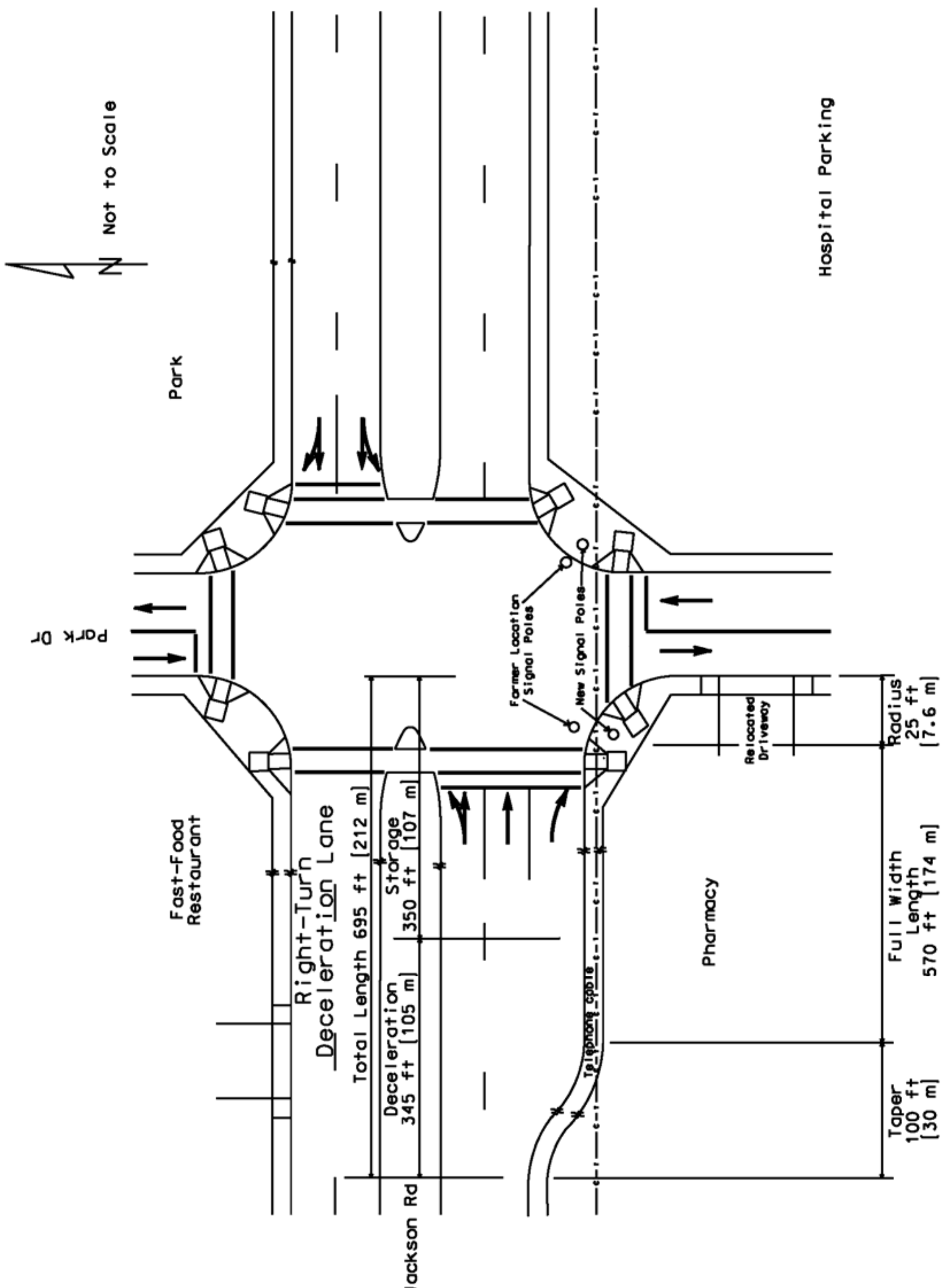


Figure 4-17. Design of Right-Turn Lane; Right-Turn Lane Design Ending at Cross-Street Curb.

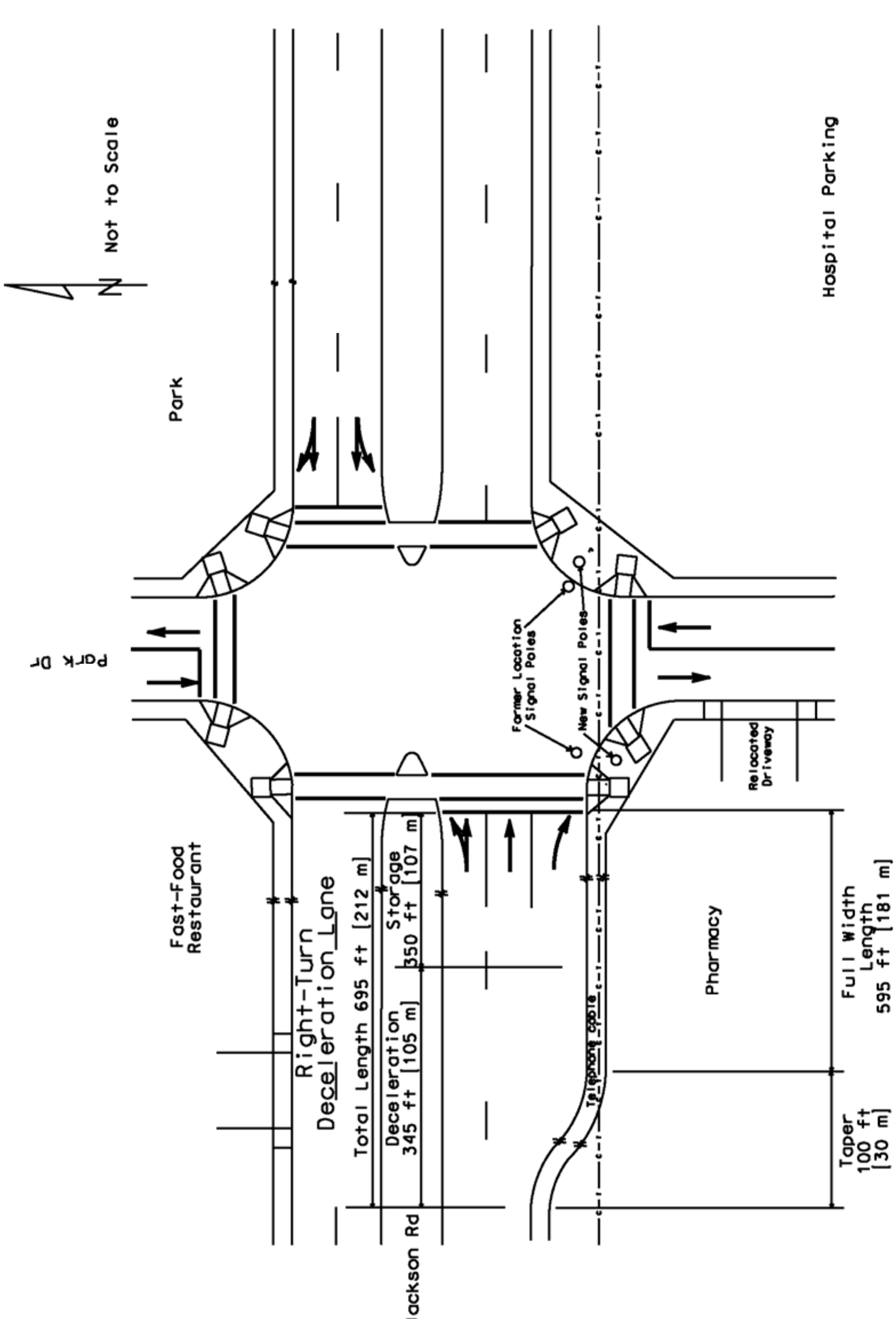


Figure 4-18. Design of Right-Turn Lane; Right-Turn Lane Design Ending at Stop Bar.

Application 7

Auxiliary Lane Improvements

Overview

The following application presents a situation where a right-turn lane is being added. The *Urban Intersection Design Guide,* Chapter 4, Section 3 <link> provides information on right-turn lane design.

Background

An intersection of two arterials exists within a developed area of a city of approximately 100,000. (See Figure 4-19.) Pine Road is a 62-ft-wide [19 m], five-lane arterial with a continuous center, two-way left-turn lane. A traffic signal exists at the intersection of Pine Road and David Boulevard. At this intersection, Pine Road has left-turn lanes on both approaches. The daily traffic volume on Pine Road is about 17,000.

David Boulevard is an old city street that originated as a collector street between the city's downtown area and its original residential developments, but it slowly evolved into an arterial street as traffic volumes increased and as the street continued to be extended from its original design length. Several years ago, David Boulevard was widened east of Pine Road from its original 40-ft-wide [12 m] cross section to a 55-ft-wide [17 m] cross section consisting of two travel lanes in each direction and a continuous center two-way left-turn lane. Right-of-way constraints limited the width of the widening project. West of Pine Road, David Boulevard maintained its 55-ft-wide [17 m] cross section for a distance of 150 ft [46 m], and then tapered back to its original 40-ft-wide [12 m] cross section.

Currently, David Boulevard has a daily traffic volume of about 15,000 vehicles. Extensive redevelopment northwest of the intersection of David Boulevard and Pine Road is expected within the next two to 4 years, partially due to an existing medical complex expanding its facilities and an existing junior college campus growing from an enrollment of about 6000 to 12,000. In order to accommodate the expected growth in traffic volumes, city officials plan to widen David Boulevard west of Pine Road to provide additional roadway capacity, make additional operational efficiency improvements, improve transit operations, and provide for expected increases in pedestrian traffic.

In anticipation of the future widening of David Boulevard, the city obtained right-of-way dedications from new developments as they occurred adjacent to the north and south side of the street. In addition, because Bobcat Drive is the only entrance to the junior college from David Boulevard, the city obtained additional right of way from the landowner to eventually construct a right-turn lane on the westbound David Boulevard approach to Bobcat Drive. Bobcat Drive also provides access to part of the medical complex as well.

The city is prepared to design the David Boulevard widening project and is attempting to determine the most cost-effective design that will incorporate as much available right of way as possible and maximize operational efficiency.

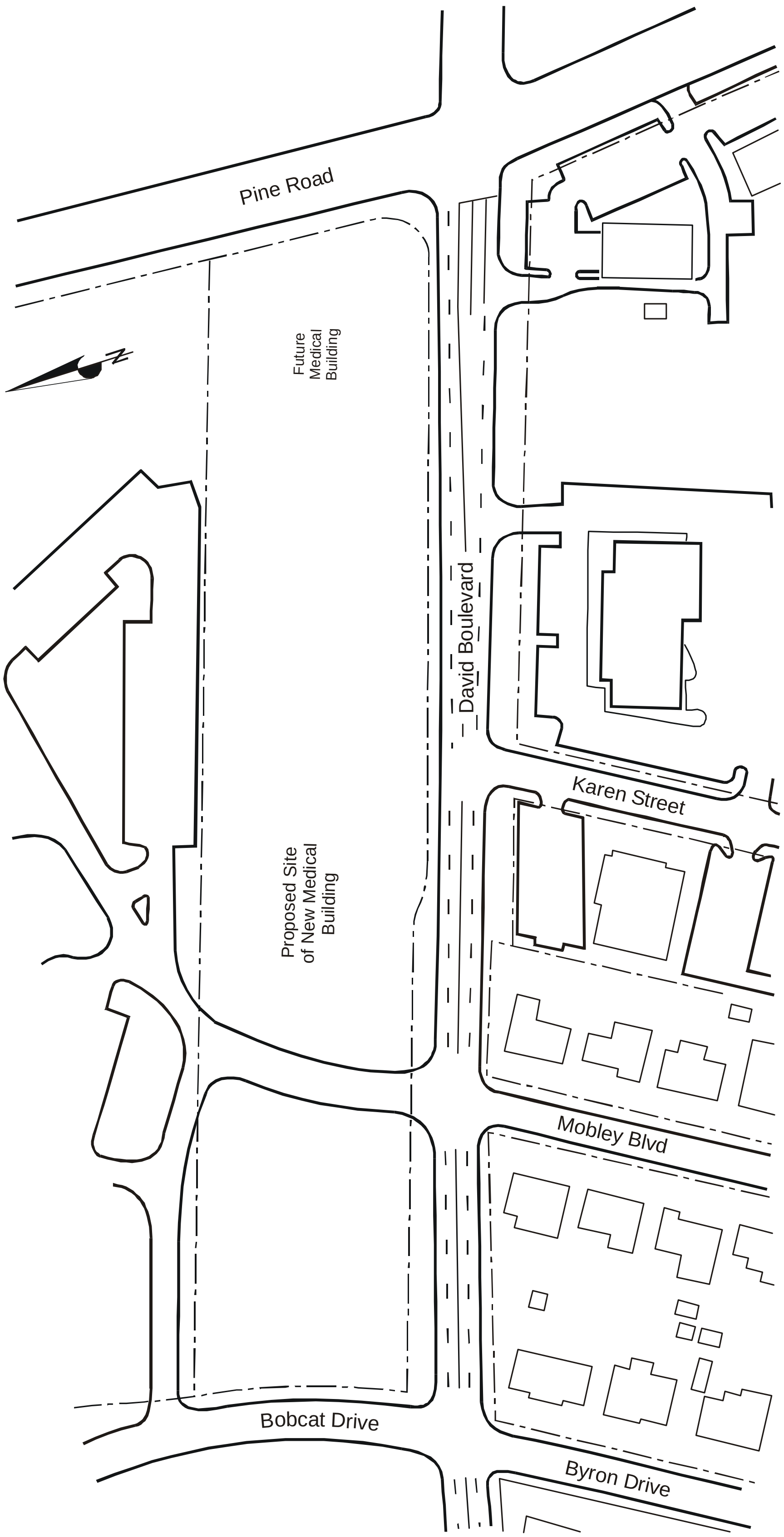


Figure 4-19. Existing Conditions.

Issues Considered

As shown in Figure 4-19, the properties south of David Boulevard and west of Karen Street are primarily residential properties that have rights of way very close to the existing curb lines. Existing houses also are located relatively close to David Boulevard. Obtaining right of way from these residential properties will be difficult, and purchasing these homes would be costly. Also, an overhead power line is located along the south side of David Boulevard that would have to be removed or relocated if the arterial is widened along that side of the street. Hence, widening David Boulevard on the south side would likely be very costly.

The property along the north side of David Boulevard between Pine Road and Bobcat Drive is undeveloped; however, two tracts have planned medical buildings and access to those properties has already been promised, as illustrated in Figure 4-19. There is little doubt that future development along the north side of David Boulevard between these two streets will attract a substantial number of vehicles. In contrast, the existing properties along the south side of David Boulevard are expected to generate only a minimal amount of traffic.

Ultimately, a traffic signal will be installed at the intersection of David Boulevard and Bobcat Drive. This signal will be justified due to the large number of vehicles entering and exiting Bobcat Drive to and from the college campus and the medical complex. The roadway intersecting David Boulevard on the south side across from Bobcat Drive, Byron Drive, is a low-volume, local street. Signalizing a low-volume, local street would encourage additional traffic onto that residential street.

The city's decision to obtain right of way to provide a right-turn lane on the westbound David Boulevard approach to Bobcat Drive was commendable. However, as shown in Figure 4-20, the right of way would provide a right-turn lane that would begin a short distance east of the northwest hospital entrance. Therefore, it would not serve the entrance to the hospital. The city made the decision that the right-turn lane should be extended further to the east. Again, the additional extension of the right-turn lane to serve the hospital would not serve the entrance to the future medical building on the corner of the intersection of Pine Road and David Boulevard. The logical conclusion is that a right-turn lane (or essentially an additional travel lane used primarily for right turns into and out of adjacent properties) would be desirable along David Boulevard from the intersection of Pine Road to Bobcat Drive.

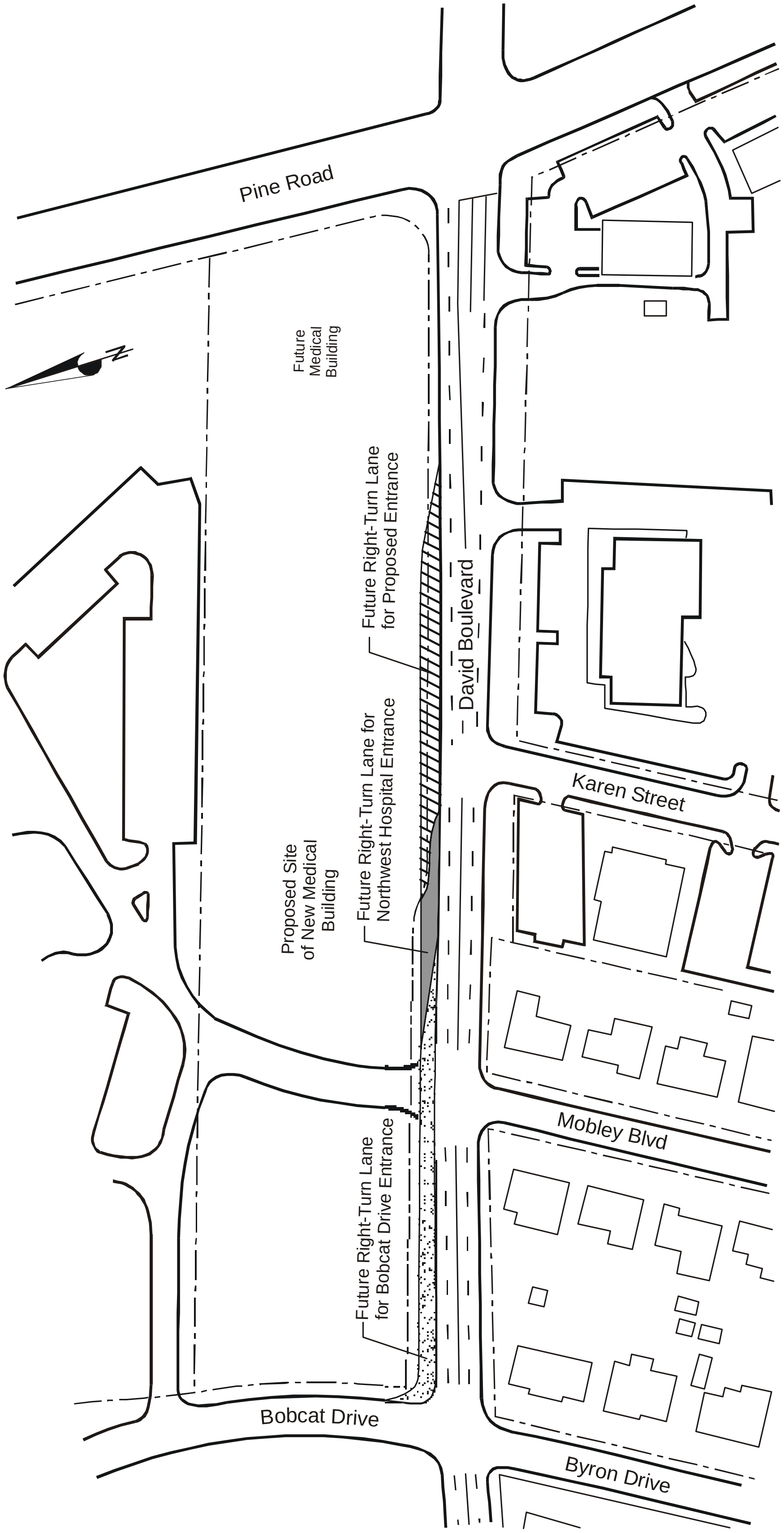


Figure 4-20. Turn-Lane Recommendations.

Design Selected

The widened section of David Boulevard west of Pine Road required design features for intersections of two arterials and for several intersections of an arterial with numerous local streets and driveways. While it is desirable to limit intersections of arterials with driveways and restrict intersections of arterials with local streets, real world conditions require compromises. The primary design goal was to maximize operational efficiency on David Boulevard. The widening of the roadway to a 55-ft-wide [17 m] cross section to provide two lanes in each direction and a continuous center, two-way left-turn lane was the primary design element to accomplish this objective. This selected cross section (which was limited to a 55-ft-wide [17 m] cross section because of minimal right of way availability) widened existing travel lanes and provided an additional center lane for left-turning vehicles. Both of these geometric improvements increased roadway capacity and operational efficiency. The widening was designed to occur solely on the north side of David Boulevard because right of way could be most easily obtained from undeveloped land, and the proposed widened section was continued along the north side of David Boulevard west of Bobcat Drive for another 0.5 mi [0.8 km]. (See Figure 4-21.)

Because future development would generate high volumes of vehicles turning right from westbound David Boulevard into the developments (and right-turn exits onto David Boulevard), a right-turn lane was designed along the north side of the widened David Boulevard. This right-turn lane provided numerous operational benefits for David Boulevard. Vehicles could use the right-turn lane to decelerate before turning into the several driveways along David Boulevard and to accelerate after turning from the driveways onto David Boulevard. Because these diverging and merging movements could be made at higher speeds, there would be less interference with the westbound “through” vehicles on David Boulevard, which would maintain operational efficiency for through traffic.

The auxiliary right-turn lane provides storage areas for right-turning vehicles regardless of turning demands. Without this lane, large numbers of right-turning vehicles accessing one or more of the driveways would slow down or block the outside traffic lane, which would significantly affect operational efficiency on both David Boulevard westbound travel lanes. Large, slow-moving vehicles making turns from travel lanes have much more negative impact on traffic flow efficiency. By providing long return radii at the driveways, larger vehicles could use the right-turn lane without interfering with traffic on the two through lanes.

The auxiliary right-turn lane also provided the opportunity for construction of a continuous right turn from southbound Pine Road onto westbound David Boulevard. The continuous right turn from southbound Pine Road onto westbound David Boulevard also will increase the traffic signal operational efficiency at the intersection of the two streets. The continuous right turn allows these right-turning vehicles to merge into the westbound through lanes of David Boulevard with greater efficiency as well.

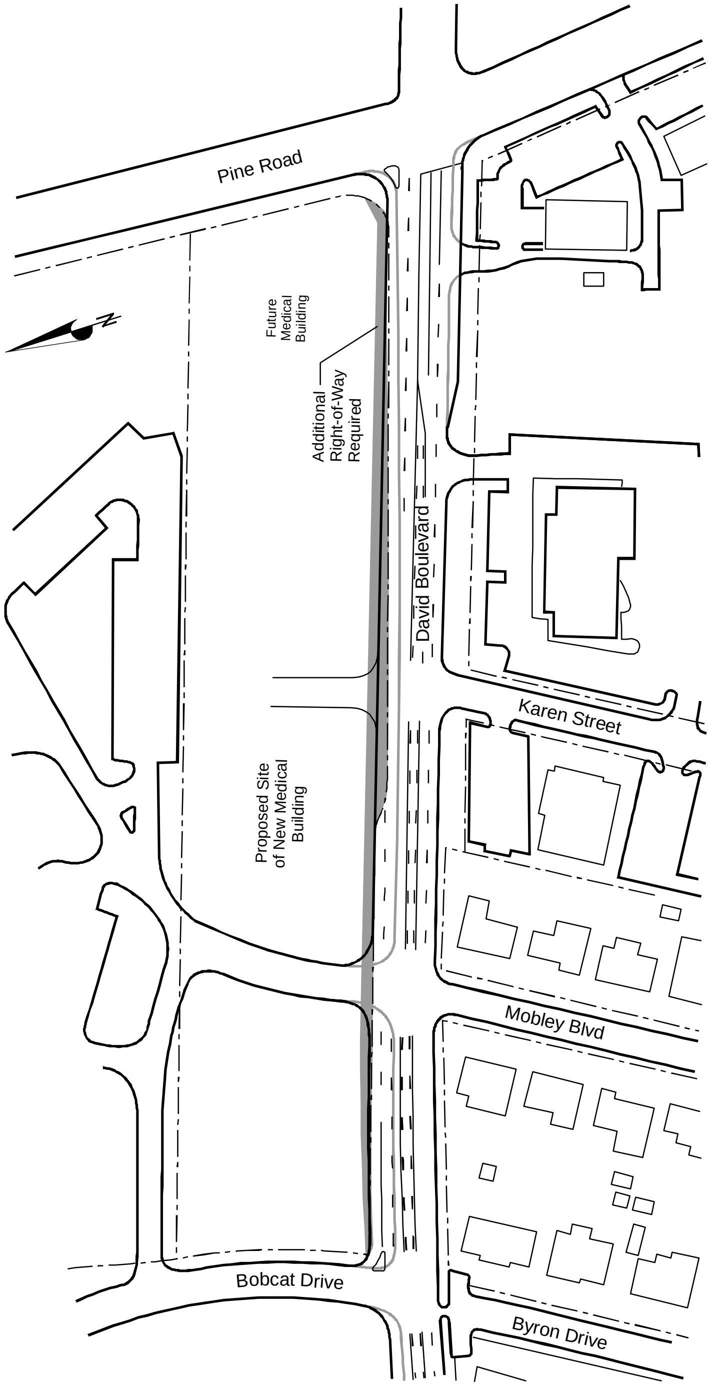


Figure 4-21. Selected Design.

Because the right-turn lane would be used primarily by vehicles entering or leaving the properties along David Boulevard, a bus stop could be located along the lane an adequate distance from Pine Road. The additional travel lane would provide westbound buses with the opportunity to pull into the lane to decelerate to its stop and then accelerate before returning to the westbound through travel lanes. A bus stop for eastbound buses could be established at the intersection of David Boulevard and Bobcat Drive. Pedestrian signals at the intersection will provide assistance for transit riders who desire to cross David Boulevard near the bus stop. Bus transit operations are expected to benefit from these improvements and have minimal effect on David Boulevard traffic flow conditions.

There was some discussion in the design phase of the project about constructing islands along the right-turn lane to force vehicles to turn right into the driveways. These islands could be painted or raised. The decision concerning painted islands was quickly resolved. Painted islands would have little or no effect as a method of forcing vehicles to turn right. Motorists would determine quickly that the pavement continues past the painted islands and would travel across the island if they considered this action convenient. Construction of raised islands would, however, force the right turns to be made. However, the raised islands would interfere with bus operations, minimize lengths available for vehicle acceleration and deceleration, reduce storage capacity for right-turning vehicles, and reduce overall operational capacity. Hence, it was determined that the only raised island that would be constructed would be at the end of the deceleration lane at the Bobcat Drive intersection.

The disadvantages of the auxiliary right-turn lane included additional cost for construction and right of way. However, the disadvantages associated with the additional costs for construction and right of way were determined to be minimal and more than offset by the improvement to operational efficiency. A negative result from the roadway widening project was the need to increase pedestrian crossing times. The crossing distance for pedestrians increased from 40 ft to 55 ft, with the amount of additional time required for the crossing being relatively small. The pedestrian crossings at the signalized intersections of David Boulevard with Pine Road and Bobcat Drive will have pedestrian signals to increase pedestrian safety. Hence, provisions were made to accommodate pedestrians along the corridor.

In regards to the pedestrian crossing concerns, it was readily apparent that anticipated developments along the south side of David Boulevard likely would not generate a substantial amount of pedestrian crossings within the street's five-lane cross section. Should the land use on the south side of David Boulevard change or if an increase in pedestrian crossings between Pine and Bobcat is observed, an additional marked pedestrian crossing could be installed at Karen Street or Mobley Boulevard.

Accommodation of bus transit was considered to determine how transit could be incorporated into the project's design. The junior college had a small bus shuttle operation that transported students from the interior of the campus to nearby parking lots and apartment complexes. Experience with the shuttle bus system indicated that few students rode the city transit buses to and from the campus. The city also had a good paratransit system for older residents and individuals with disabilities. Hence, many of the patients traveling to the medical complex (and the rehabilitation center) would be able to receive door-to-door service. It was anticipated that bus service to the area would be desirable but ridership levels in city transit buses likely would remain moderate at best.

Because additional right of way was available along the south side of David Boulevard near the intersection of Pine Road, a right-turn lane also was designed for the eastbound David Boulevard approach to Pine Road. The additional right of way presented other geometric options for consideration. One option was to widen the intersection approach and provide a median storage area for pedestrians. This option was not selected primarily because greater overall operational improvements were anticipated with the additional right-turn lane.

The installation of traffic signals at the intersection of David Boulevard and Bobcat Drive would create an interesting operational problem for Byron Drive, the local street located opposite of Bobcat Drive. It is unusual for a local street to intersect an arterial street. It is also unusual for an intersection of an arterial street and a local street to be signalized. If Byron Drive remains accessible from David Boulevard, traffic volumes on Byron Drive likely will increase as neighborhood traffic would be attracted to the signalized intersection. Hence, Byron Drive would begin to operate more like a collector street than a local street. Byron Drive does not have the cross section or pavement structure necessary to accommodate higher traffic volumes; therefore, additional treatment of the intersection of Byron Drive and David Boulevard was necessary.

The two options considered for intersection treatment were turn restrictions or street closure. The intersection could be redesigned to allow right turns into and out of Byron Drive. This treatment would reduce the amount of accessibility to Byron Drive, but the right turns would affect the intersectional traffic flow. Closure of Byron Drive would be the preferred treatment, because it would restrict all vehicular traffic from accessing David Boulevard via Byron Drive.

The street closure could be designed as a circular cul-de-sac, or it could be constructed as shown in Figure 4-21. The closure could be designed to permit bicycle and pedestrian access to David Boulevard. The closure of Byron Drive would allow the street to function as a local street, as it was intended. Also, traffic volumes on Byron Drive would remain low and residents along the street would not have to contend with increased traffic volumes, which would exist if the street was not closed.

The street closure also would permit the intersection of David Boulevard and Bobcat Drive to function as a T-intersection, which can operate more efficiently than a signalized four-legged intersection. Also, T-intersections have fewer conflict points created by turning vehicles. Fewer conflict points result in less vehicle interaction and fewer accidents. Hence, the closure of Byron Drive would result in more operational efficiency and increased safety.

Application 8
Island Offsets

Overview

The design of corner islands at intersections is presented in the *Urban Intersection Design Guide,* Chapter 4, Section 5 <link>. The designs have offsets to the curb lines of the roadway depending on their characteristics.

Background

This example will review the effects of corner radius and pedestrian facilities on the design of corner islands. A turning roadway width of 14 ft [4.3 m] is used in each of the three cases examined.

Issues Considered

The design of corner islands depends on a number of issues, including:

- corner radius or more complex curvature (see *Urban Intersection Design Guide,* Chapter 3, Section 3, Turning Radius <link>, and *Urban Intersection Design Applications,* Chapter 3, Application 9 <link>;
- island size (see *Urban Intersection Design Guide,* Chapter 4, Section 4, Channelization <link>, and Section 5, Island and Median Design <link>);
- design vehicle (see *Urban Intersection Design Guide,* Chapter 2, Section 1, Motorized Vehicles <link>; and
- pedestrian facility characteristics (see *Urban Intersection Design Guide,* Chapter 7, Sections 1 and 2 <link>, and *Urban Intersection Design Guide,* Chapter 4, Section 5, Island and Median Design).

Figure 4-16 of the *Urban Intersection Design Guide* <link> provides details regarding curb offsets on urban streets. Some island dimensions depend upon its classification as “large,” “intermediate,” or “small.” The *Green Book* provides the following guidance on island classification:

- Small: area of approximately 50 ft^2 [5 m^2] normally or 100 ft^2 [9 m^2] preferable
- Large: side dimension of at least 100 ft [30 m]

Proposed Design, 100-ft [30 m] Turning Radius

- *Step 1: Establish island outer boundary and size.*

As shown in Figure 4-22, the island was first sketched using the corner radii and offsets indicated in Figure 4-16 of the *Urban Intersection Design Guide* <insert link to *Guide* Figure 4-16>.

The initial estimates of its size indicated either a large or intermediate classification, so the island was designed in a manner consistent with those requirements shown in Figure 4-16 of the *Urban Intersection Design Guide*. As shown, the island has the following characteristics:

- Area: 584 ft^2 [54 m^2]
- Edge dimensions: approximately 43 ft [13 m] by 35 ft [11 m] by 35 ft [11 m]

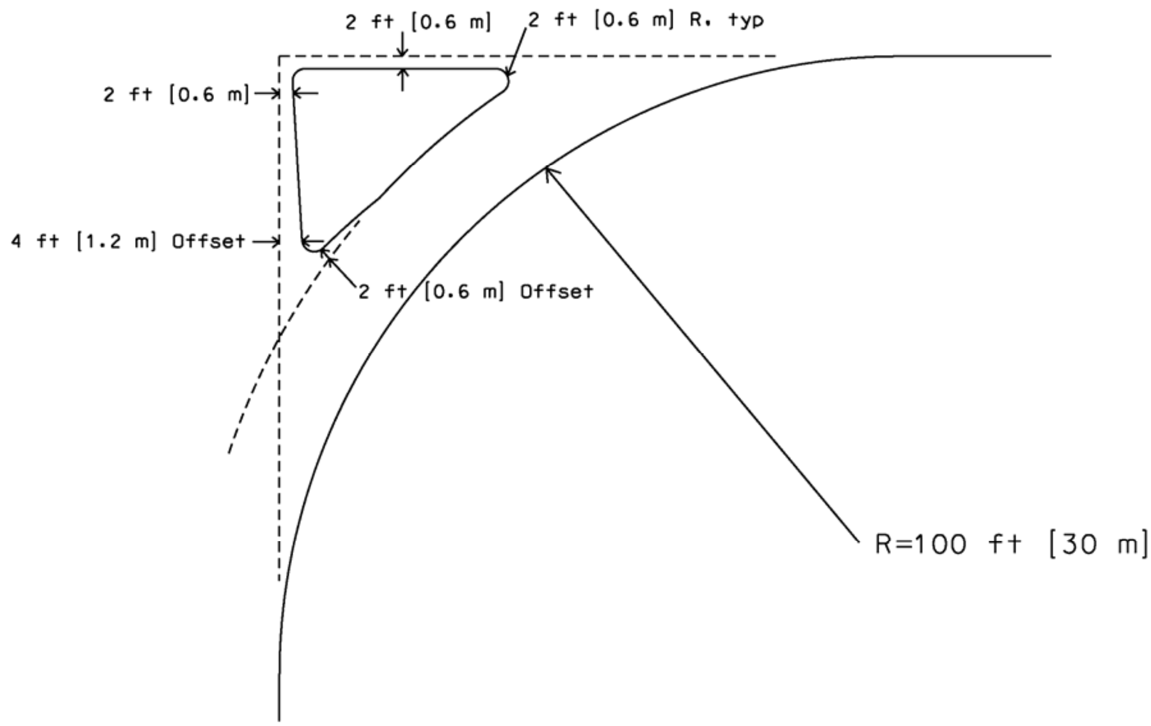


Figure 4-22. 100-ft [30 m] Turning Radius and Island.

Reviewing the classification guidelines from the *Green Book*,[4] the area is considerably larger than the minimum island size (584 ft^2 [54 m^2] compared to a preferred minimum of 100 ft^2 [9 m^2]), indicating either a “large” or “intermediate” island. To be classified as a large island, the island’s edge dimensions must be greater than 100 ft [30 m]. The edge dimensions for the island are less than the requirement, so it is classified as an intermediate island.

The design as initially developed is consistent with the intermediate island dimensions and offsets shown in Figure 4-16 of the *Urban Intersection Design Guide* <insert link to *Guide* Figure 4-16>. The island will be curbed to provide delineation for the traffic movements at the intersection. This is acceptable because it exceeds the preferred minimum area of 100 ft^2 [9 m^2] for a curbed island.[1]

The island is offset from the projected through lane face of curb by 2 to 3 ft [0.6 to 0.9 m]; the nose of the island on the approach end is offset an additional amount for a total 4 to 6 ft [1.2 to 1.8 m] offset. The nose of the island is also offset from the right-turn traffic by 2 to 3

[4] American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets.* AASHTO, Washington, D.C., 2001.

ft [0.6 to 0.9 m]. The radii used for the corners of the islands may be 2 to 3 ft [0.6 to 0.9 m] on the island noses and 2 to 5 ft [0.6 to 1.5 m] on the 90-deg corner. Figure 4-22 shows the offsets selected for this particular design.

♦ *Step 2: Design pedestrian crossing.*

Pedestrian crossings of curbed islands can use curb ramps to rise to the surface of an island if sufficient width is available to accommodate the curb ramps and their associated landing areas. The curb ramps are 4 ft [1.2 m] wide, while the landing areas are 5 ft by 5 ft [1.5 m by 1.5 m]. If the available island width is inadequate for curb ramps, 5-ft [1.5 m] cuts may be provided through the island to allow passage for pedestrians.

As shown in Figure 4-23, space is adequate to provide curb ramps and a landing on the island. The curb ramps should be aligned perpendicular to the curb and end within the provided crosswalks. The curb ramps are each shown with a 5 ft by 5 ft [1.5 m by 1.5 m] landing area blocked out at the top of the curb ramp; the curb ramps would actually be constructed to the top of a uniformly surfaced island. Overlapping the landing areas in this manner is permissible.

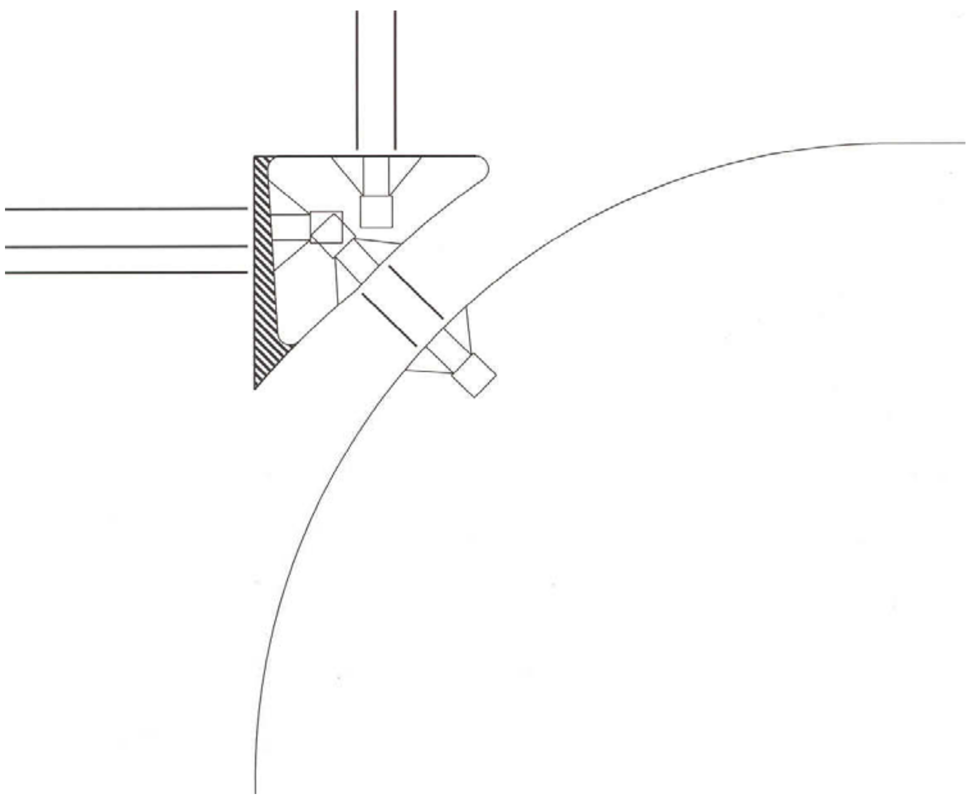


Figure 4-23. 100-ft [30 m] Turning Radius Island with Pedestrian Elements.

Figure 4-23 also includes the pavement markings used at the intersection, and includes crosswalk and stop line markings. For further information regarding crosswalks see the *Urban Intersection Design Guide*, Chapter 7, Section 2 <link>. Transverse markings are also provided on the through side approaching the nose of the island, as shown in Figure 4-16 of the *Urban Intersection Design Guide* “Pedestrian and Bicyclist Accommodation” <link>.

Proposed Design, 60-ft [18 m] Turning Radius

♦ *Step 1: Establish island outer boundary and size.*

As shown in Figure 4-24, the island was first sketched using the corner radii and offsets indicated in Figure 4-16 of the *Urban Intersection Design Guide* <link>. Some of the dimensions for the island depend upon its classification as "large," "intermediate," or "small." The initial estimates of its size indicated that it probably met requirements for a small classification, so the island was designed in a manner consistent with those requirements. As shown, the island has the following characteristics:

- Area: 140 ft^2 [13 m^2]
- Edge dimensions: approximately 19 ft [6 m] by 13 ft [4 m] by 13 ft [4 m]

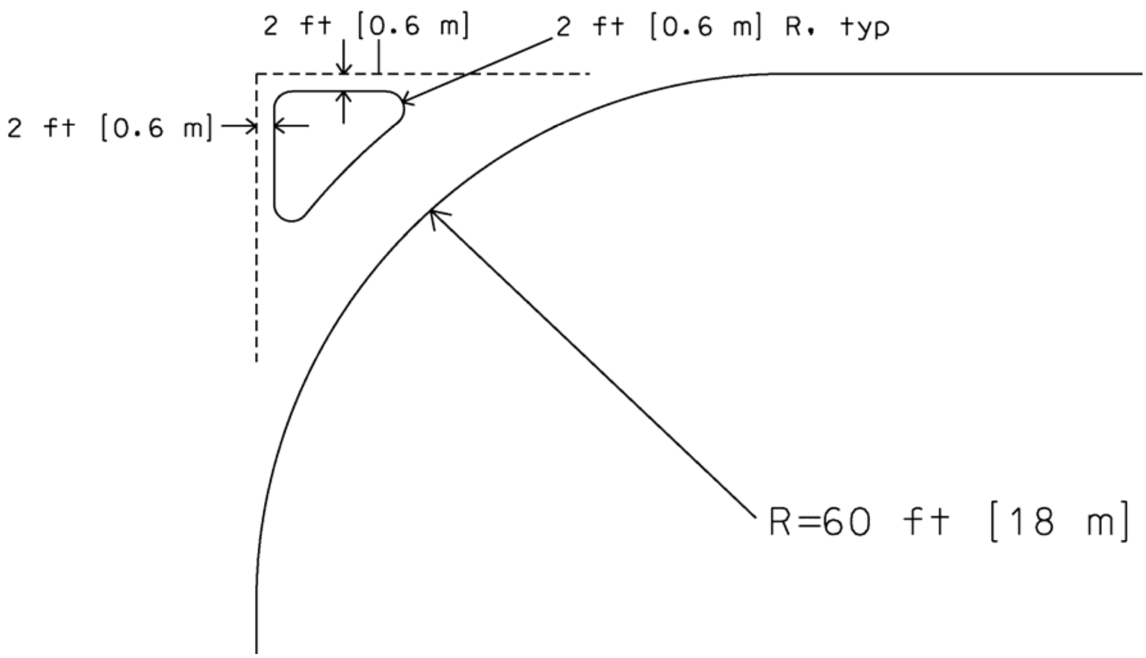


Figure 4-24. 60-ft [18 m] Turning Radius and Island.

Reviewing the classification guidelines from the *Green Book*, the area is near the minimum island size (140 ft^2 [13 m^2] compared to the preferred minimum 100 ft^2 [9 m^2]) and its outer dimensions are considerably less than 100 ft [30 m]. The island is thus classified as small.

The design as initially developed is consistent with the small island dimensions and offsets shown in Figure 4-16 of the *Urban Intersection Design Guide*. This size is acceptable because it exceeds the minimum area of 50 ft^2 [5 m^2] for a curbed island.[1] The island will be curbed to provide delineation for the traffic movements at the intersection, with 2-ft [0.6 m] offsets from the through traffic lanes and 2-ft [0.6 m] radii on the corners.

♦ *Step 2: Design pedestrian crossing.*

The small island size is not sufficient to provide room for the pedestrian curb ramps and their landing areas, so 5-ft [1.5 m] cuts were provided through the island to allow passage. The cuts are aligned with the crosswalks as shown in Figure 4-25.

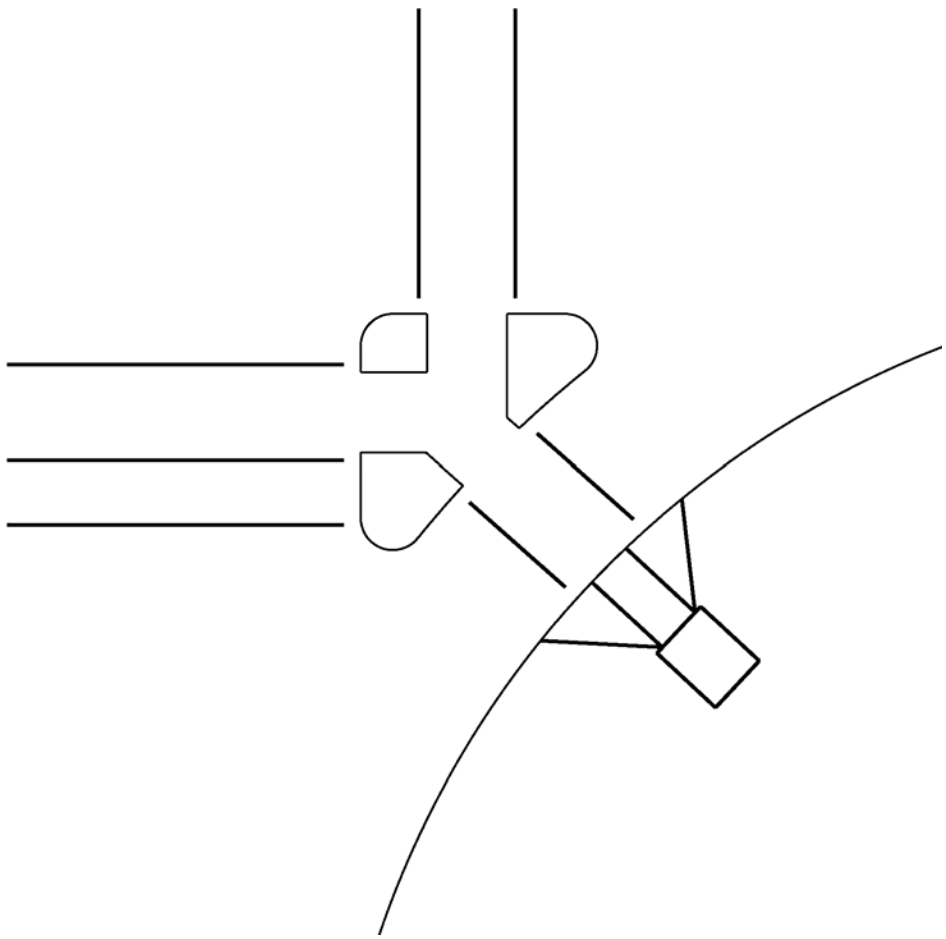


Figure 4-25. 60-ft [18 m] Turning Radius Island with Pedestrian Elements.

Proposed Design, 60-ft [18 m] Simple Curve Turning Radius with Taper

♦ *Step 1: Establish island outer boundary and size.*

As shown in Figure 4-26, the island was first sketched using the corner radii and offsets indicated in Figure 4-16 of the *Urban Intersection Design Guide* <link>. The initial estimates of its size indicated that it probably met requirements for a small classification, so the island was designed in a manner consistent with those requirements. As shown, the island has the following characteristics:

- Area: 180 ft^2 [17 m^2]
- Edge dimensions: approximately 22 ft [7 m] by 16 ft [5 m] by 16 ft [5 m]

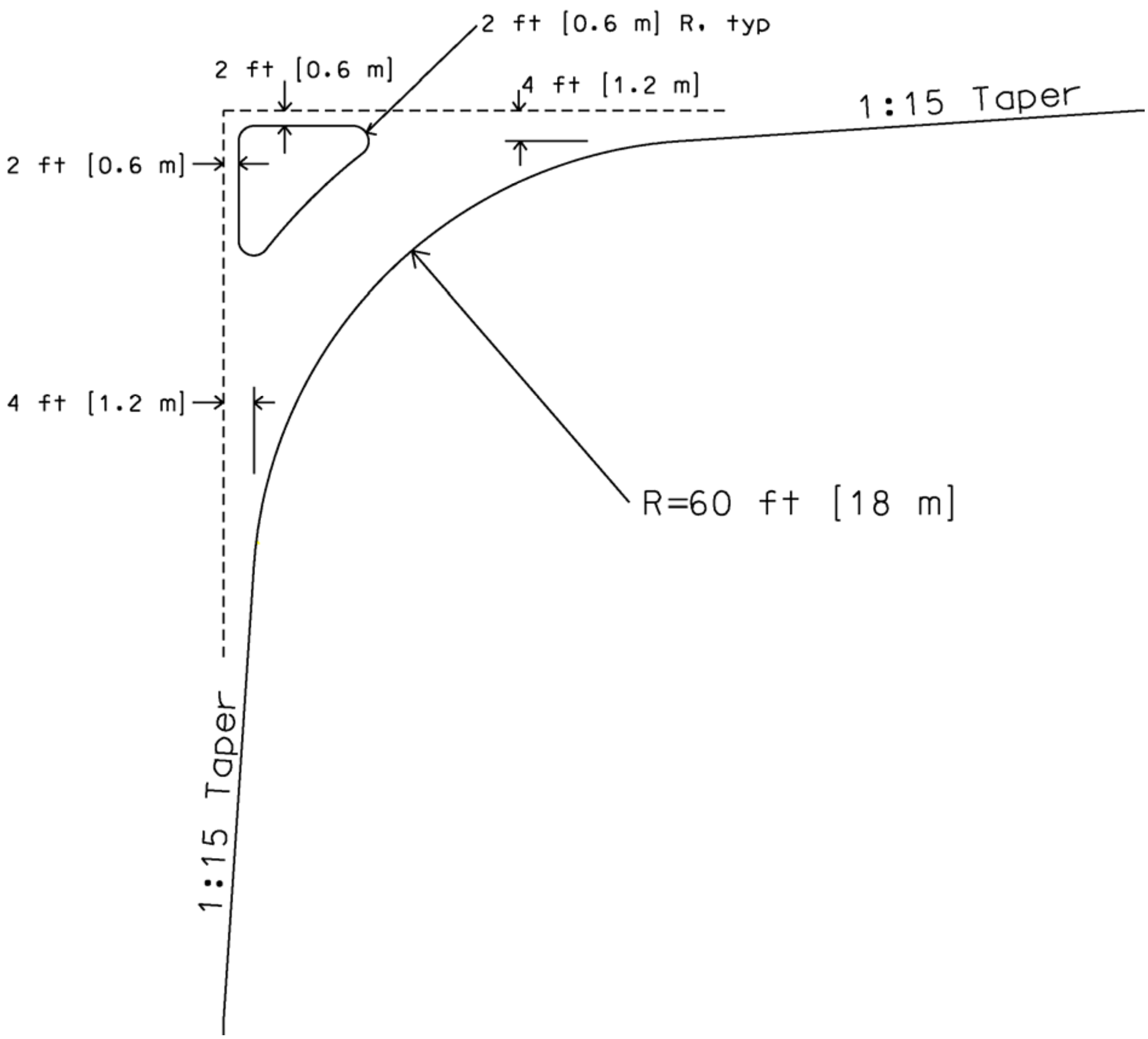


Figure 4-26. 60-ft [18 m] Simple Curve Turning Radius with Taper and Island.

Reviewing the classification guidelines from the *Green Book*,[4] the area is near the minimum island size (180 ft^2 [17 m^2] compared to a preferred minimum of 100 ft^2 [9 m^2]) and its outer dimensions are considerably less than 100 ft [30 m]. The island is thus classified as small.

The design as initially developed is consistent with the small island dimensions and offsets shown in Figure 4-16 of the *Urban Intersection Design Guide*. This size is acceptable because it exceeds the preferred minimum area of 100 ft^2 [9 m^2] for a curbed island.[1] The island will be curbed to provide delineation for the traffic movements at the intersection, with 2-ft [0.6 m] offsets from the through traffic lanes and 2-ft [0.6 m] radii on the corners.

♦ *Step 2: Design pedestrian crossing.*

The small island size is not sufficient to provide room for the pedestrian curb ramps and their landing areas, so 5-ft [1.5 m] cuts were provided through the island to allow passage. The cuts are aligned with the crosswalks as shown in Figure 4-27.

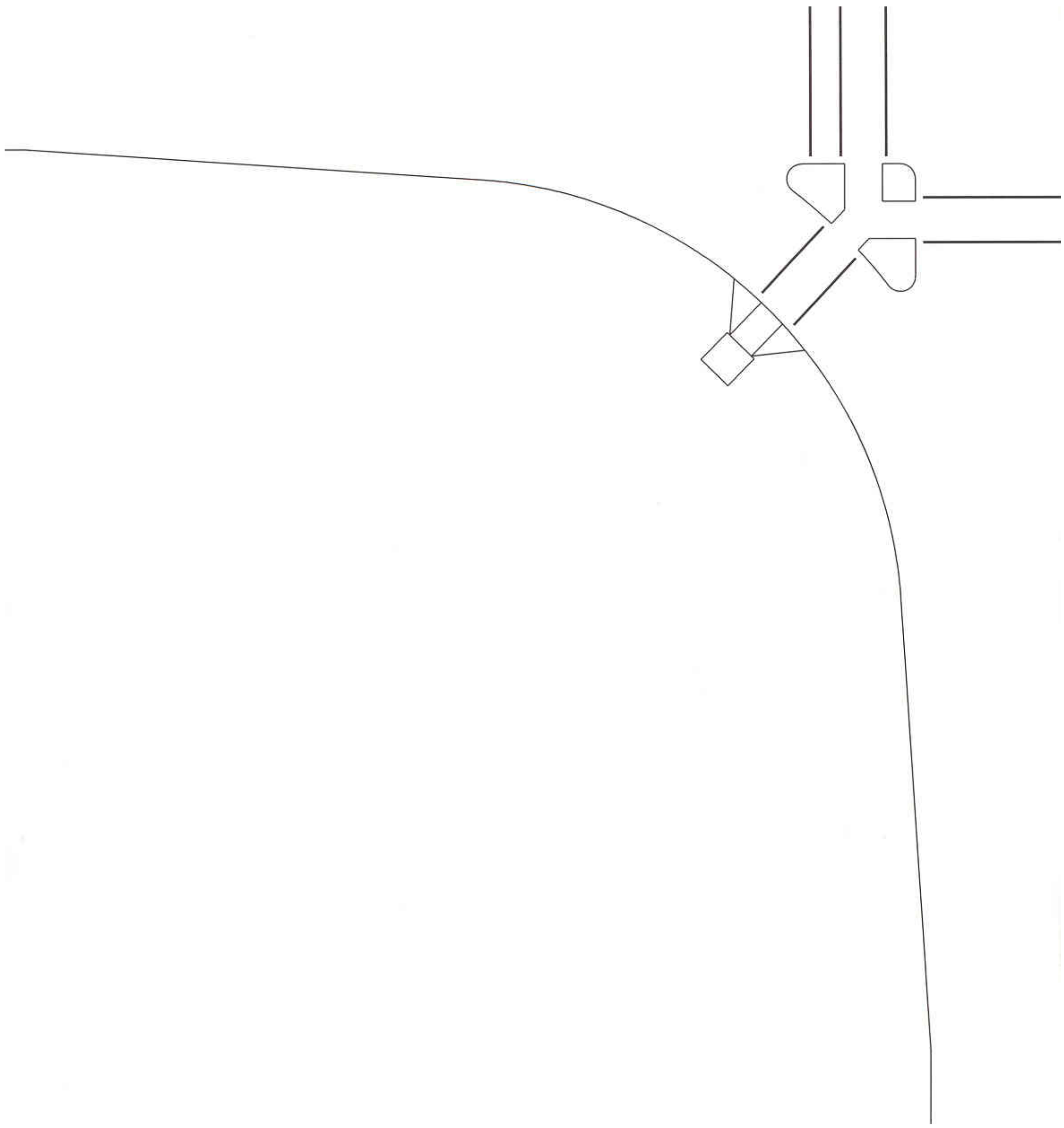


Figure 4-27. 60-ft [18 m] Simple Curve Turning Radius Island with Taper with Pedestrian Elements.

Application 9

Median Design for Large Vehicles

Overview

The following application presents a situation where a design vehicle impacts design decisions. The *Urban Intersection Design Guide,* Chapter 4, Section 5 <link> provides information on island and median design.

Background

A city of over 400,000 is planning the widening of an existing arterial street, Morgan Avenue, that is 33 ft [10 m] in width from edge of pavement to edge of pavement and has no curbs, to a completely curbed-and-guttered cross section having four lanes and a raised median. The city plans to construct the arterial with a basic cross section of two, 28-ft-wide [9 m] roadway sections (measured face-of-curb to face-of-curb) with an 18-ft-wide [5 m] median. Some of the intersections along the arterial will have more traffic volumes than others, some will have more turning movements than others, some will have more large truck traffic than others, and some will have more pedestrian use than others. However, each redesigned intersection will have to be designed to accommodate pedestrians. The city prefers to design each intersection as required to accommodate the unique conditions for each intersection, but remain essentially consistent with the preferred cross section. Right of way is more than sufficient for the widening project, but the city prefers to keep the cost of roadway construction as low as possible.

Morgan Avenue currently intersects Stanton Drive, a four-lane divided arterial, at an acute angle (see Figure 4-28). It will not be practical to realign either roadway to change the intersecting angle. The proposed new intersection is expected to accommodate a moderate to high amount of traffic with twice the average percentage of buses found at the other intersections. Bus stops currently are located on the near corners of each approach leg. Pedestrian use is expected to be moderate but high during peak hours of operation. The area near the intersection is relatively flat, and there are no obstructions to sight distance at and in the vicinity of the intersection. Furthermore, the intersection's accident history does not indicate any unusual operational or safety problems; however, there have been complaints about the large number of city transit buses interfering with or blocking vehicular traffic at the intersection. The intersection is signalized.

The intersection of Morgan Avenue and Stanton Drive has some unique conditions that are dissimilar to the other intersections along Morgan Avenue. Hence, this intersection generated more interest and was selected for special design considerations. A concern specifically mentioned by the transit agency was in regard to the ability of the city's buses being able to comfortably turn through the intersection.

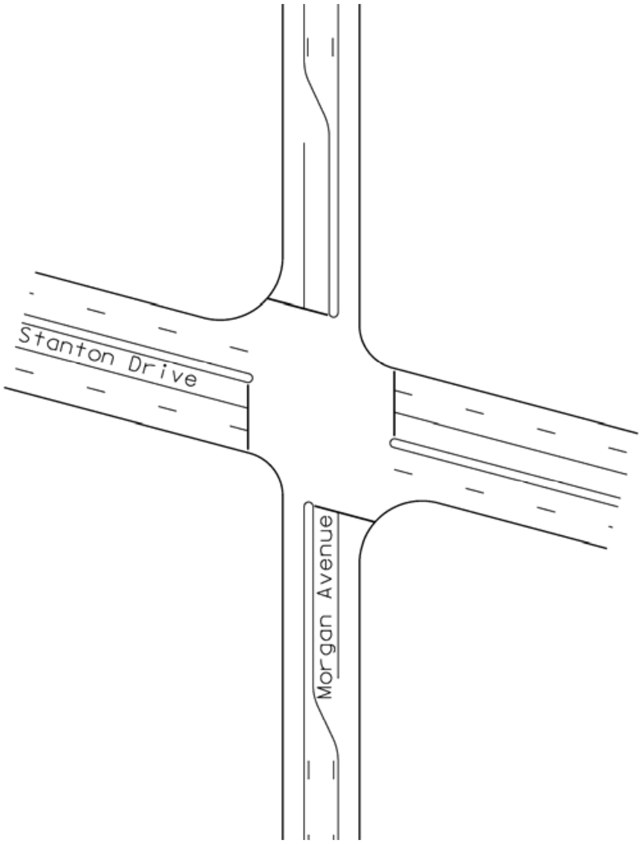


Figure 4-28. Existing Conditions at Morgan Avenue and Stanton Drive.

Issues Considered

In urban areas, intersections on divided roadways need to be designed with operational efficiency in mind but able to accommodate all travel modes. Operational efficiency can be maximized by keeping the intersectional area as small as possible, which minimizes both vehicle clearance times and pedestrian crossing times. Hence, confining the size of the intersection is preferred in urban areas.

Selecting a design for an urban intersection includes the selection of a design vehicle. When designing for a large vehicle, intersections become larger to accommodate the large turning radii required by large vehicles. This larger size is detrimental to efficient operation. At the same time, designing a more confined intersection affects the ability of larger vehicles to make turns at the intersection. The design vehicle chosen for this intersection design is a city transit bus.

The design selected for the intersection of Morgan Avenue and Stanton Drive requires consideration of large vehicle operation (which means a large intersectional area), consideration of pedestrian crossings (which benefit from a small intersectional area), and an attempt to provide operational efficiency for the moderate to high amount of traffic (which can best be accommodated with a small intersectional area). The decision that must be made is whether to design for large vehicles and accommodate pedestrians and the moderate to large traffic volumes, or design for the pedestrians and moderate to large traffic volumes and accommodate the larger vehicles. Obviously, the final design must be a compromise that addresses each of these design considerations.

Proposed Design to Accommodate Turning Buses

There is no absolute right or wrong design decision for this intersection. Because sight distance is not an issue and accidents have not been a problem, then the question that must be answered is whether the intersection should be designed to be slightly more advantageous to operational efficiency and pedestrian movements, or slightly more advantageous for large vehicle operations. Also, the cost considerations were a factor. It is more cost-efficient to build a smaller intersectional area rather than a large intersectional area. The design selected for the median, as shown in Figure 4-29, incorporated aspects of operational efficiency for large vehicles, including buses and pedestrian movements. The intersection was designed to accommodate the turns of buses, and special bus stop locations were provided on each of the downstream sides of the intersection to allow buses to exit the travel lanes to board passengers. An 8 ft by 5 ft [2.4 m by 1.5 m] bus boarding area has been added at each bus stop to comply with ADAAG requirements.[5] Chapter 5, Section 6 <link> of the *Urban Intersection Design Guide* should be consulted for further information regarding bus stop design.

The design of the median was the primary design concern for the intersection at this stage; hence the signal pole and inlet locations are not shown for this example. With the acute angle of intersection, the median had to be “cut back” to accommodate the large vehicle turning maneuvers. Therefore, the resulting design included a mountable median that could be used by the larger vehicle if needed. The mountable median was 10 ft [3 m] in length and was constructed with cast-in-place concrete. The sides of the mountable median were 2 inches [5.1 cm] high at the edges adjacent to the travel lanes, and the median was sloped gradually to its center where its height was 4 inches [10.2 cm]. The design did not permit any vegetative growth in this section of the median, and it allowed the occasional large vehicle to complete its left turn by mounting the median area (Figure 4-30 illustrates the turning path of the design vehicle as it crosses over the mountable median).

[5] U.S. Access Board, *Americans with Disabilities Act Accessibility Guidelines* (ADAAG). September 2002. http://www.access-board.gov/adaag/html

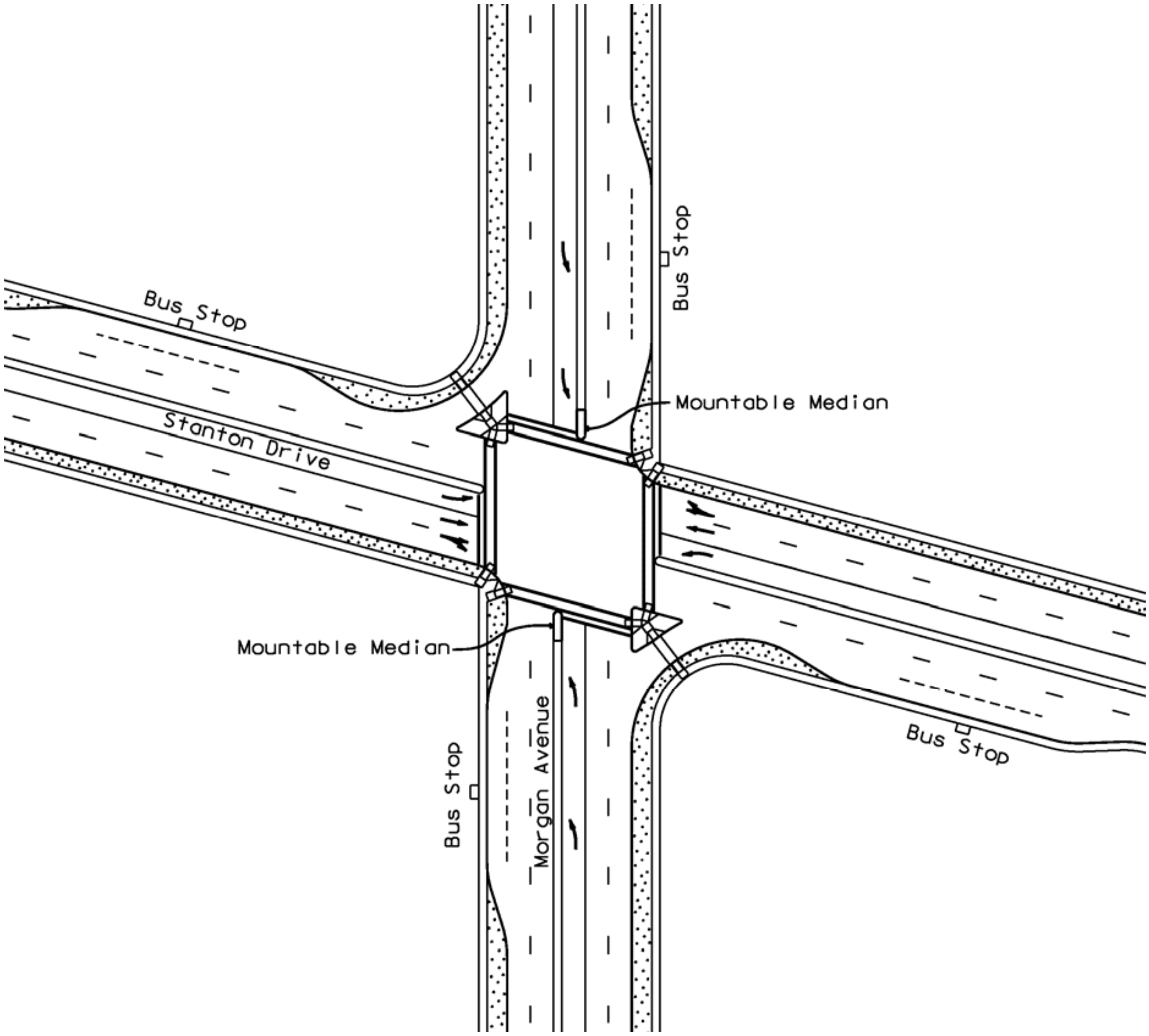


Figure 4-29. Proposed Intersection Design for Morgan Avenue and Stanton Drive.

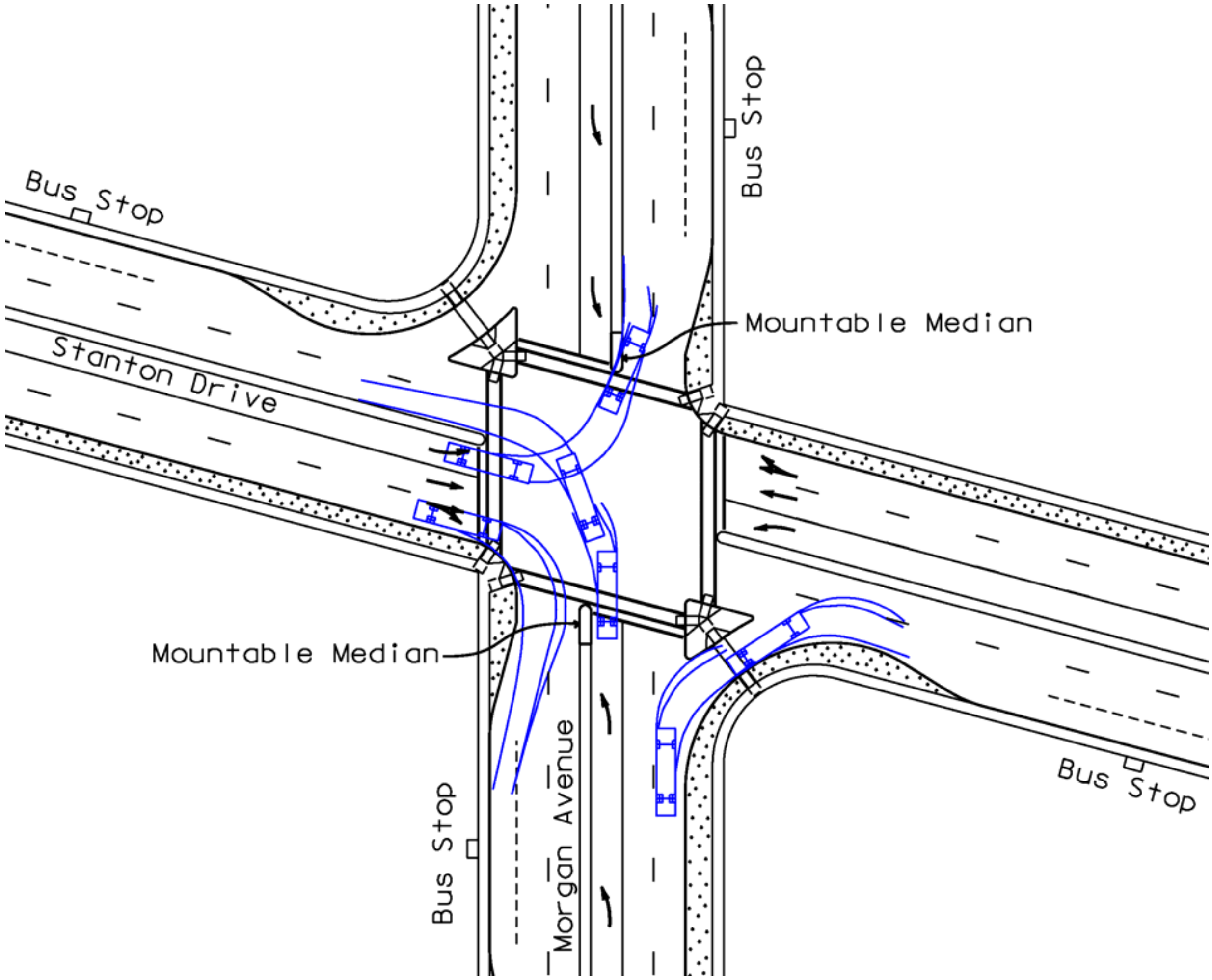


Figure 4-30. Proposed Intersection Design with City Transit Bus Turning Template.

Application 10

Temporary and Ultimate Medians and Outside Curbing

Overview

This application provides a review of some of the issues related to median design in a situation that considers the staged development of a roadway and its median. The *Urban Intersection Design Guide*, Chapter 4, Section 5 <link> provides information on island and median design.

Background

An intersection of a two-lane state highway with a four-lane divided major collector street that serves residential subdivisions currently exists on the suburban edge of a city of approximately 250,000. The state highway (called Stagecoach Road) has the typical rural design, consisting of 12-ft-wide [3.7 m] travel lanes, paved shoulders, no curbs, and open ditches to accommodate drainage. The collector street (Pin Oak Drive) has curbs and gutters that end on the approaches to Stagecoach Road. At the intersection, Stagecoach Road has separate left-turn lanes on both approaches. Traffic signals already exist at the intersection. Figure 4-31 shows the existing condition.

Major improvements (widening project) have been planned for Stagecoach Road and at its intersection with Pin Oak Drive. Stagecoach Road will be widened to a four-lane, divided cross section with curbs and gutters. Left-turn lanes will be provided at the intersection of Pin Oak Drive, and also to both Pin Oak Drive approaches to the intersection. Sidewalks will be provided on both sides of both roadways. The right of way for Stagecoach Road provides room for an additional lane for both directions (or a six-lane, divided roadway); however, the need for more than four lanes has not been justified for construction in the immediate future. Stagecoach Road is expected, however, to eventually be widened to six lanes due to its designation as a principal arterial and the number of lanes and the traffic volume on other, more heavily urbanized segments of the roadway.

The design of the intersection improvements also was affected by the positioning of the sidewalks and traffic signal hardware. A primary consideration in the design involved allowing for the future expansion of Stagecoach Road to six lanes.

Issues Considered

The initial design of the four-lane divided section of Stagecoach Road obviously will be altered when it is widened to its ultimate six-lane divided cross section. The initial design consideration was whether to design the initial cross section (1) with the ultimate median in place and then widen the roadway to the outside (which would include removing the curbing on the outside of the roadway), or (2) with the ultimate outside curbing in place, providing initially a wide median, and then adding the additional lanes in the inside or median area.

The design engineer recognized that more compact intersections operated more efficiently because less time is required for both vehicles and pedestrians to travel through the intersections. Hence, a narrow median would be preferred over a wider median from an operational perspective. A wider median (initially) obviously would increase intersection clearance times.

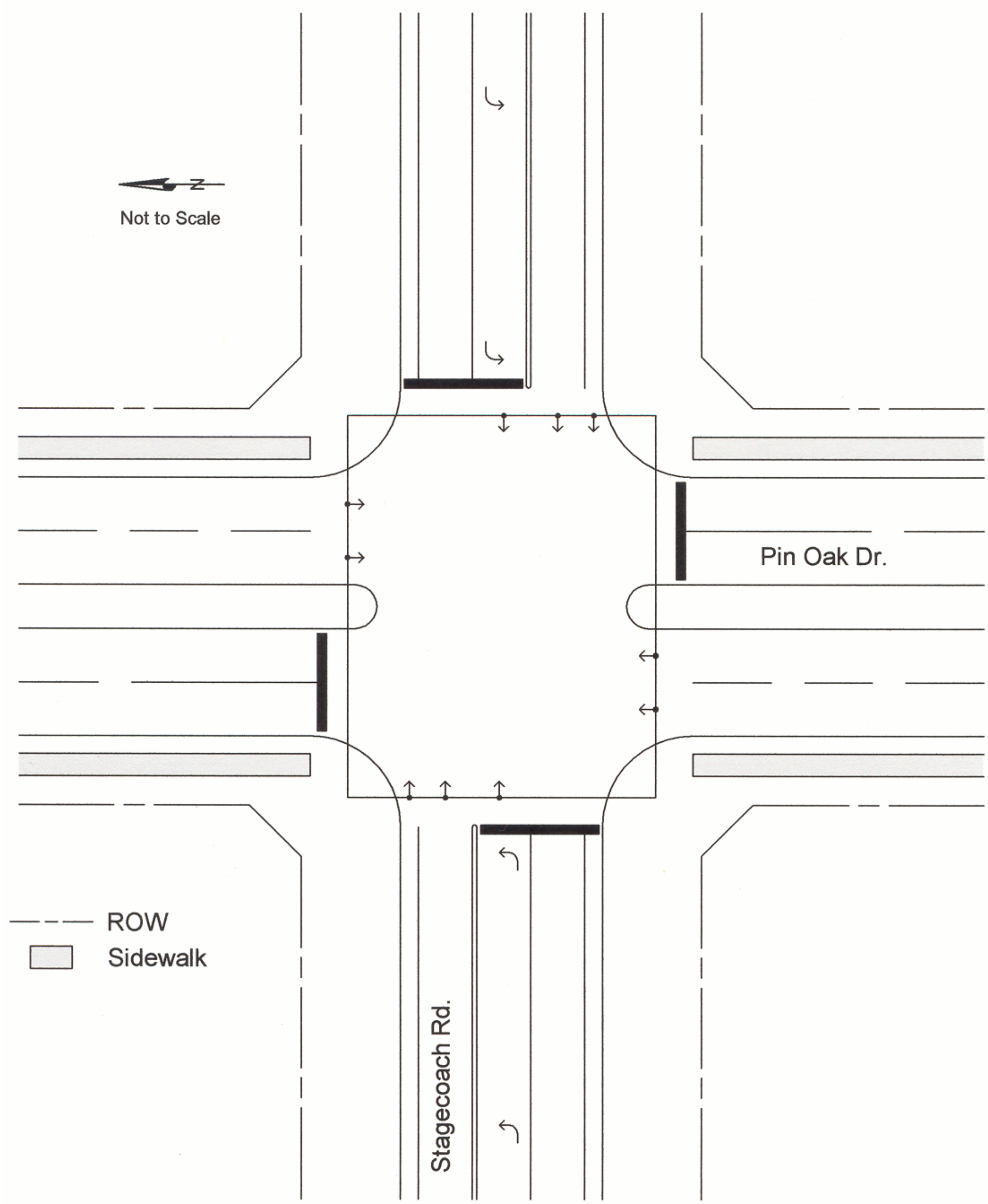


Figure 4-31. Existing Conditions at Stagecoach Road and Pin Oak Drive.

The design engineer recognized that construction of the ultimate median and temporary outside curbing would require more unique drainage design (to accommodate the ultimate cross section). Construction of the ultimate outside curbing initially would permit a more typical construction of the ultimate drainage facilities, minimizing the need to relocate storm sewer inlets and lines. In addition, by constructing the outside curbing initially and maintaining a wide median, the existing roadway could remain in place and accommodate traffic during the widening project. If the ultimate narrower median was constructed initially, then maintaining traffic during construction would be more difficult.

The design engineer considered how the future construction project (that would take place when the six-lane cross section was constructed) would be affected by the initial design. If the ultimate outside curbs were constructed initially, then the future construction project could be staged totally within the median area with less disruption of traffic. If the ultimate median is constructed initially, then the future construction project would require activity on both sides of the roadway with more disruption of traffic.

The design engineer also recognized that construction of the ultimate outside curbs would allow installation of utilities in the right of way that would not have to be disturbed when the ultimate cross section is constructed. Furthermore, if the ultimate outside curbs were constructed initially, then development (and the associated access points) that occurs along the roadway would not be affected significantly when the future widening project occurs.

Construction of the ultimate median and temporary outside curbs would be less costly than the construction of the ultimate outside curbs with a temporary wide median. However, because the ultimate cross section was considered to be viable in the future and sufficient right of way was available, the benefits associated with building the ultimate outside curbs initially was considered greater than the disadvantages (including additional costs) associated with constructing the ultimate median initially.

Design Selected

The interim and final designs selected are shown in Figure 4-32 and Figure 4-33, respectively.

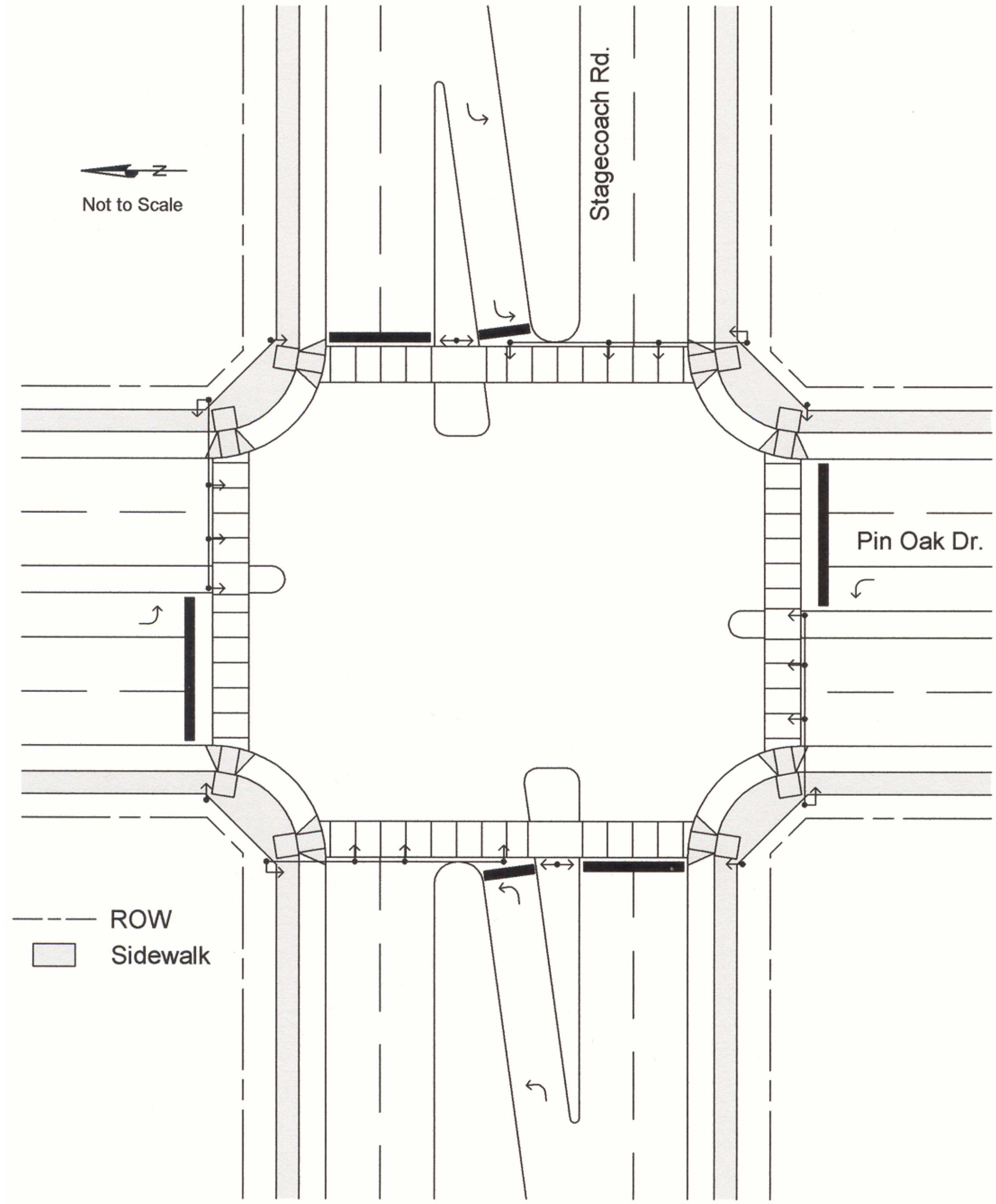


Figure 4-32. Selected Interim Design at Stagecoach Road and Pin Oak Drive.

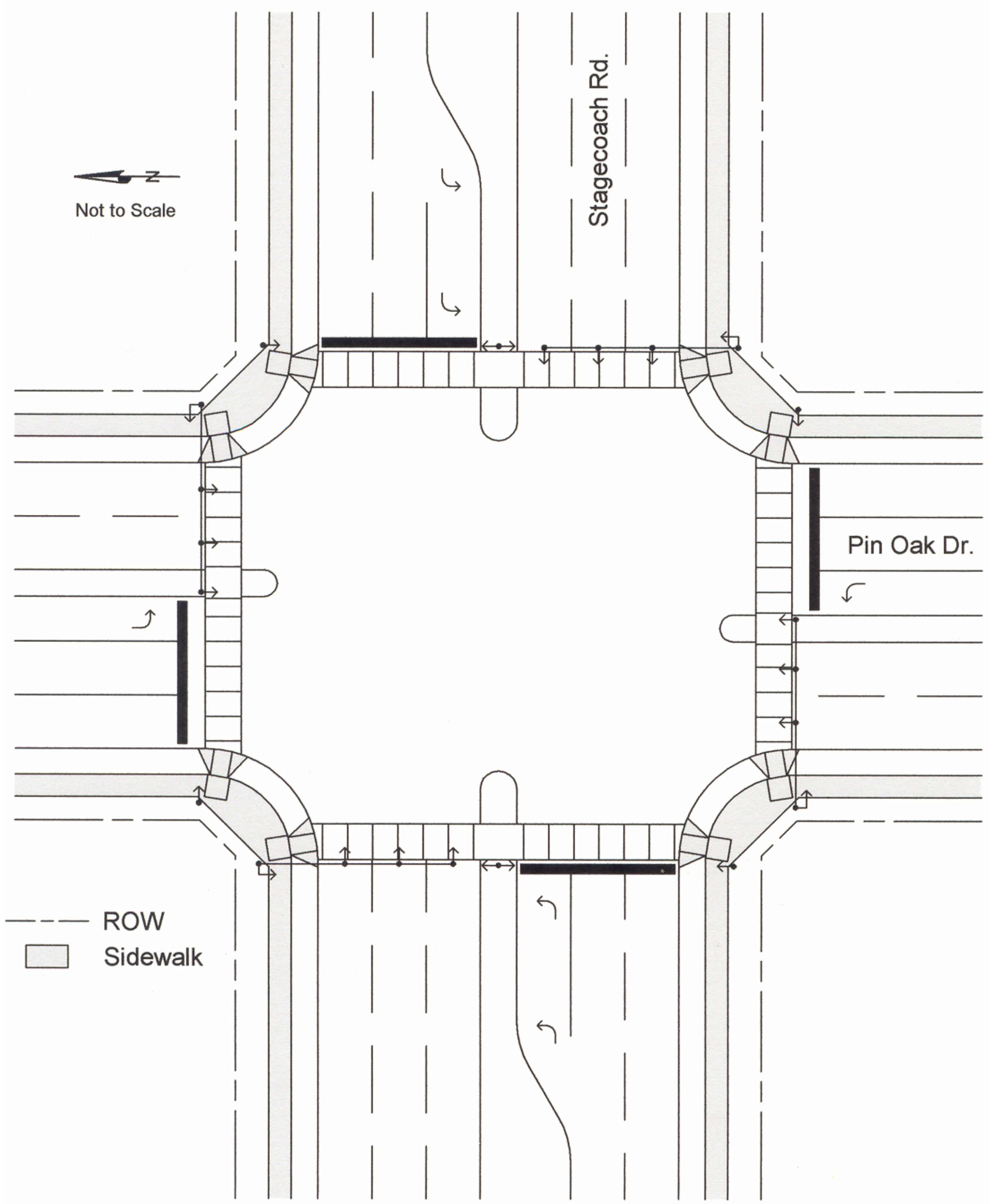


Figure 4-33. Final Design at Stage Coach Road and Pin Oak Drive.

The design engineer decided to build the four-lane, divided cross section with the ultimate outside curbs (see Figure 4-32). Once this decision was made, there were three more important design considerations associated with the intersection of Stagecoach Road with Pin Oak Drive. These three design issues included the design of the left-turn lanes on Stagecoach Road, the design of the crossings of the pedestrian crossing, and the design of the new traffic signal installation.

One of the disadvantages of designing a wide median is the increase in intersection clearance times for both vehicles and pedestrians. Hence, the design of the intersection should incorporate those design features that would reduce clearance times. Also, wide medians make the design of left turns more difficult because there is the potential for negative offsets of opposing left-turn lanes. Hence, a more unique design is required to provide positive left-turn lane offsets or at least minimize negative offsets. Once the position of the left-turn lanes and pedestrian crossings were established, the final design for the traffic signal installation and the intersection itself was determined.

In the ultimate six-lane configuration of Stagecoach Road (see Figure 4-33) the installation of the additional lanes will eliminate the offset left-turn lanes present in the selected design shown in Figure 4-32.

As shown in Figure 4-32, the intersection of Stagecoach Road and Pin Oak Drive had the following design characteristics:

- The four-lane cross section of Stagecoach Road had a 42-ft-wide [12.8 m] raised median.
- Tapered left-turn lanes were constructed on the Stagecoach lane approaches to create positive offsets.
- Left-turn lanes were added in the existing medians of both Pin Oak Drive approaches.
- Pedestrian crosswalk markings were included at the intersection.
- Median islands were used to provide pedestrian refuge areas.
- Traffic signals were installed in the median islands for the left-turning vehicles on the Stagecoach Road approaches.
- Pedestrian signals were installed on the same median signal poles so that pedestrian crossings could be timed so that pedestrians would cross half of the roadway at a time to minimize disruption of traffic on Stagecoach Road.
- Long curb return radii were included on all four corners to expedite right-turning vehicles.
- Sidewalks were offset 6 ft [1.8 m] from the edge of the streets in order to accommodate slopes for curb ramps.

Chapter 5
Roadside

Contents:

Application 1

Redevelopment Near an Intersection

Overview

The following application discusses sidewalk considerations along with other concerns during a redevelopment. The *Urban Intersection Design Guide*, Chapter 5, Section 1 <link> presents information on sidewalks; Chapter 5, Section 4 <link> presents information on street furniture and fixtures; and Chapter 5, Section 6 <link> provides information on bus stops.

Background

An intersection of two 40-ft-wide [12.2 m] streets exists near the central business district of a very large city. Bluebonnet Drive (the east/west street) is striped for four narrow travel lanes and is the major street at the intersection. Its average daily traffic volume is about 9000. Dawn Avenue (the north/south street) is striped for two travel lanes with permitted parking. Stop signs are installed on both Dawn Avenue approaches to Bluebonnet Drive. The average daily traffic on Dawn Avenue is about 5000. (See Figure 5-1.)

The city has had significant redevelopment within and near its central business district, including the area near the Bluebonnet Drive/Dawn Avenue intersection. The general area is beginning to develop into an upscale neighborhood as old homes are being either renovated or replaced with expensive larger homes. New commercial developments, especially along Bluebonnet Drive, have generated additional traffic volumes. Off-street parking areas have been constructed to serve these businesses, and pedestrian volumes also have increased. The city has recognized the potential of the area as a vibrant residential and commercial development and has responded to numerous requests by residents and business owners to make major street improvements in the area.

At the present time, sidewalks are located adjacent to Bluebonnet Drive but are not constructed with curb ramps. Dawn Avenue has no sidewalks. Power poles are located along the north side of Bluebonnet Drive and are located within the sidewalk itself. Curb return radii are relatively short by modern standards (about 20 ft [6.1 m]). Existing street hardware also includes fire hydrants and street lights. The city desires to install traffic signals at the intersection (which have been warranted), construct sidewalks, improve intersection lighting, and provide aesthetic streetscaping. Two major constraints exist: additional right of way will be difficult to obtain (only 10 ft [3 m] exists behind the back of the curb) and the overhead electric power lines and poles cannot be replaced for at least 3 years. The city's bus transit system operates a route along Bluebonnet Drive, and bus stops are made on the near sides of both approaches to Dawn Avenue.

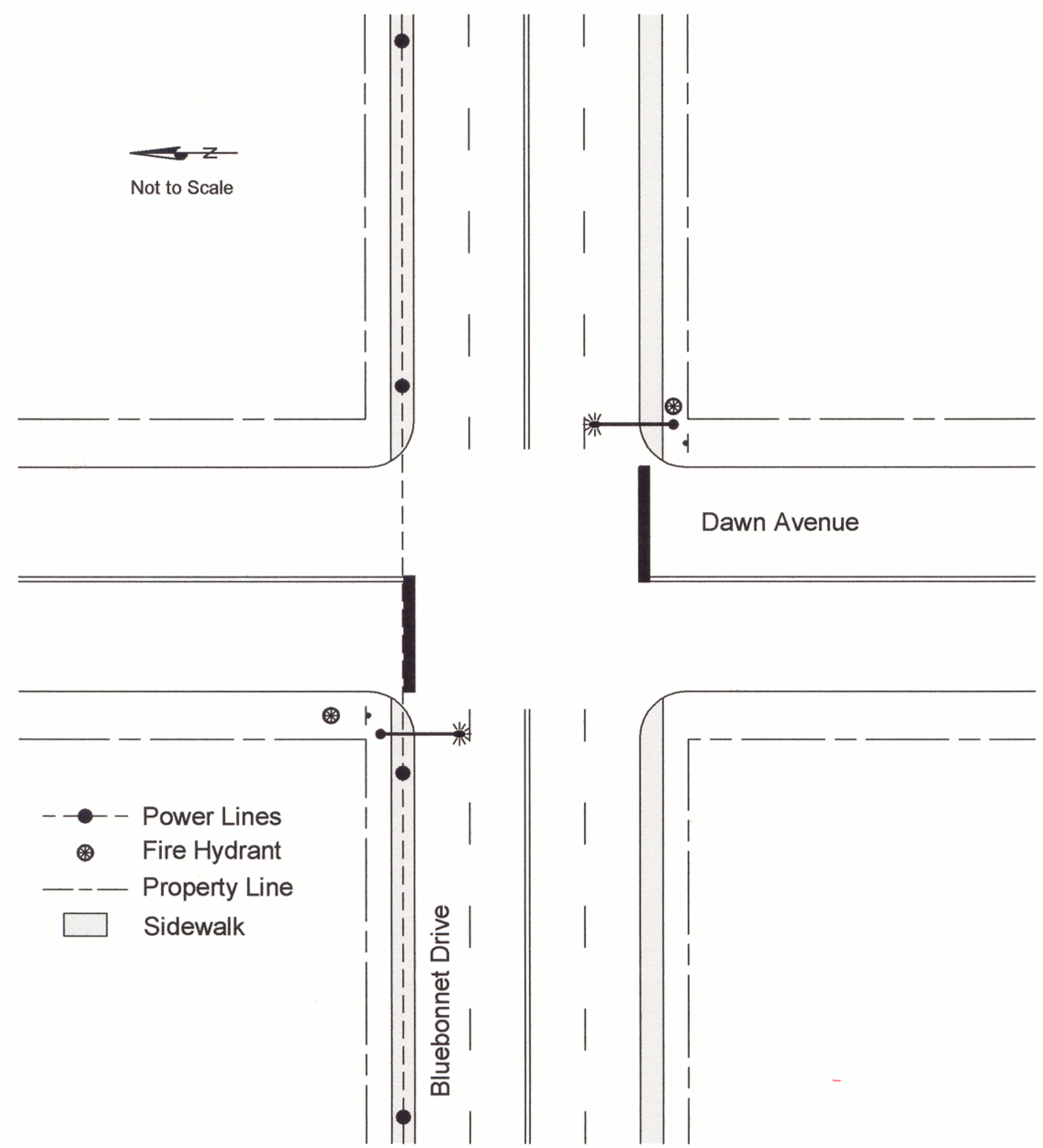


Figure 5-1. Existing Conditions at Bluebonnet Drive and Dawn Avenue.

Issues Considered

Traffic counts at the intersection revealed a relatively high volume of left turns, especially on the Bluebonnet Drive approaches. Hence, because the inside lanes were essentially being used as left-turn lanes, it was determined to provide separated left-turn lanes with no offset. Because of the numerous driveways that exist along Bluebonnet Drive that serve the off-street parking areas, a continuous center, two-way left-turn lane was incorporated into the new striping of Bluebonnet Drive. The operational capacity of a four-lane, undivided street versus a three-lane street (with the continuous two-way left-turn lane) is about the same if there are numerous left-turning vehicles. Converting an undivided four-lane street to a three-lane street will help reduce rear-end accidents, and will increase travel lane widths or provide additional space for bicycle or pedestrian facilities. The wider lanes allow more efficient right turns because more room is available for maneuvering.

All of these advantages were appealing to the city, especially to the manager of the bus transit system. The lane used by the buses could be increased in width from 10 to 14 ft [3 to 4.3 m].

Because of the recent increases in traffic volumes on Dawn Avenue (and expected additional future increases as well), the city elected to remove the parking on Dawn Avenue. Removal of parking allowed the street to be striped for three travel lanes, matching the striping used on Bluebonnet Drive.

The existing sidewalk on Bluebonnet was in a state of disrepair, and the presence of wood power poles in the sidewalk on the north side of the street seriously affected the usefulness of the sidewalk. Because the power poles were planned for removal sometime after 3 years, it was decided to construct the new sidewalk away from the power pole locations. It was determined that a 6-ft-wide [1.8 m] sidewalk would be constructed on both sides of Bluebonnet Drive adjacent to its right-of-way lines. The remainder of the right-of-way area (between the new sidewalk and the curb) would be lined with inlaid bricks. The bricks could be removed for streetscape hardware installation, and new bricks could be placed where the power poles exist after poles are removed.

Sidewalks along Dawn Avenue were planned to be 5 ft [1.5 m] in width and located 1 ft [0.3 m] from the right-of-way line. Due to the required lengths for ramps and size of the landings, additional right of way had to be obtained near the corners of the intersection. Signal poles will be located adjacent to these landings so that pedestrian push buttons could be installed adjacent to the landings. Luminaires are planned for installation on the four signal poles to increase intersectional lighting.

In a confined area, the positioning of a signal cabinet may be difficult. Because of the location of the signal poles near the landings, there was insufficient space for a pole-mounted cabinet. A ground-mounted cabinet has several desirable location design features:

- The cabinet should be located such that the cabinet door opens away from the roadway.
- The cabinet should be positioned so that the technician can stand on a firm foundation (preferably concrete) while working inside the cabinet.
- The cabinet should be located away from the street, if possible, but close to the intersection.
- When facing the controller, the technician should be able to view the signal heads without having to “turn around.”
- The cabinet should not be placed in a sidewalk.

After studying the geometry and right-of-way restrictions at the intersection of Bluebonnet Drive and Dawn Avenue, the most logical location for the controller was on the southeast corner.

With a protective/permissive signal operation, only two signal heads per approach would be needed. Pedestrian signals were included in the design, and crosswalks were provided. Additional pedestal poles were installed at the intersection for pedestrian push buttons.

Design Selected

The final design of the intersection (see Figure 5-2) included the following:

- Make changes to existing right of way or existing street and curbs, except for additional right of way at intersection corners and curb reconstruction due to installation of curb ramps.
- Install sidewalks 6 ft [1.8 m] in width along Bluebonnet Drive adjacent to the edge of the right-of-way lines, and install sidewalks 5 ft [1.5 m] in width along Dawn Avenue, offset 1 ft [0.3 m] from the back of the right-of-way line.
- Install inlaid bricks along the north and south sides of Bluebonnet Road at a width of 4 ft [1.2 m], placed between the edge of the sidewalk and the curb.
- Stripe both streets for three lanes, each with a center, continuous two-way left-turn lane. The two-way left-turn lane was discontinued in advance of the intersection so that separate left-turn lanes would be provided on each intersection approach.
- Install traffic signals (overhead on mast arms) and pedestrian signals, with pedestrian push buttons on all four signal poles and on four pedestal poles.
- Remove existing street lights and poles and install luminaires (with mast arms) on all four signal poles.
- Relocate a fire hydrant on the southeast corner into the inlaid brick area.
- Relocate a fire hydrant on the northwest corner of the intersection into the grassy area.
- Install the signal controller in the inlaid brick area on the southeast corner of the intersection, positioning the controller so that the door opens toward the sidewalk.
- Beneath the new sidewalks, the city planned to install several conduits for electrical, telephone, cable, and any other utility that may be positioned within the right of way. On-ground connections (boxes) can be provided at specific locations within the inlaid brick areas (along Bluebonnet Drive) or within the grassy areas (along Dawn Avenue).
- The city also planned to place plants and trees (in containers), and benches on top of the inlaid bricks at various sites. Periodic inspections will be required to ensure that vegetation does not protrude into the pedestrian envelope or create undesirable sight obstructions.
- Provide pad for bus stop landing on nearside approaches of Bluebonnet Drive.

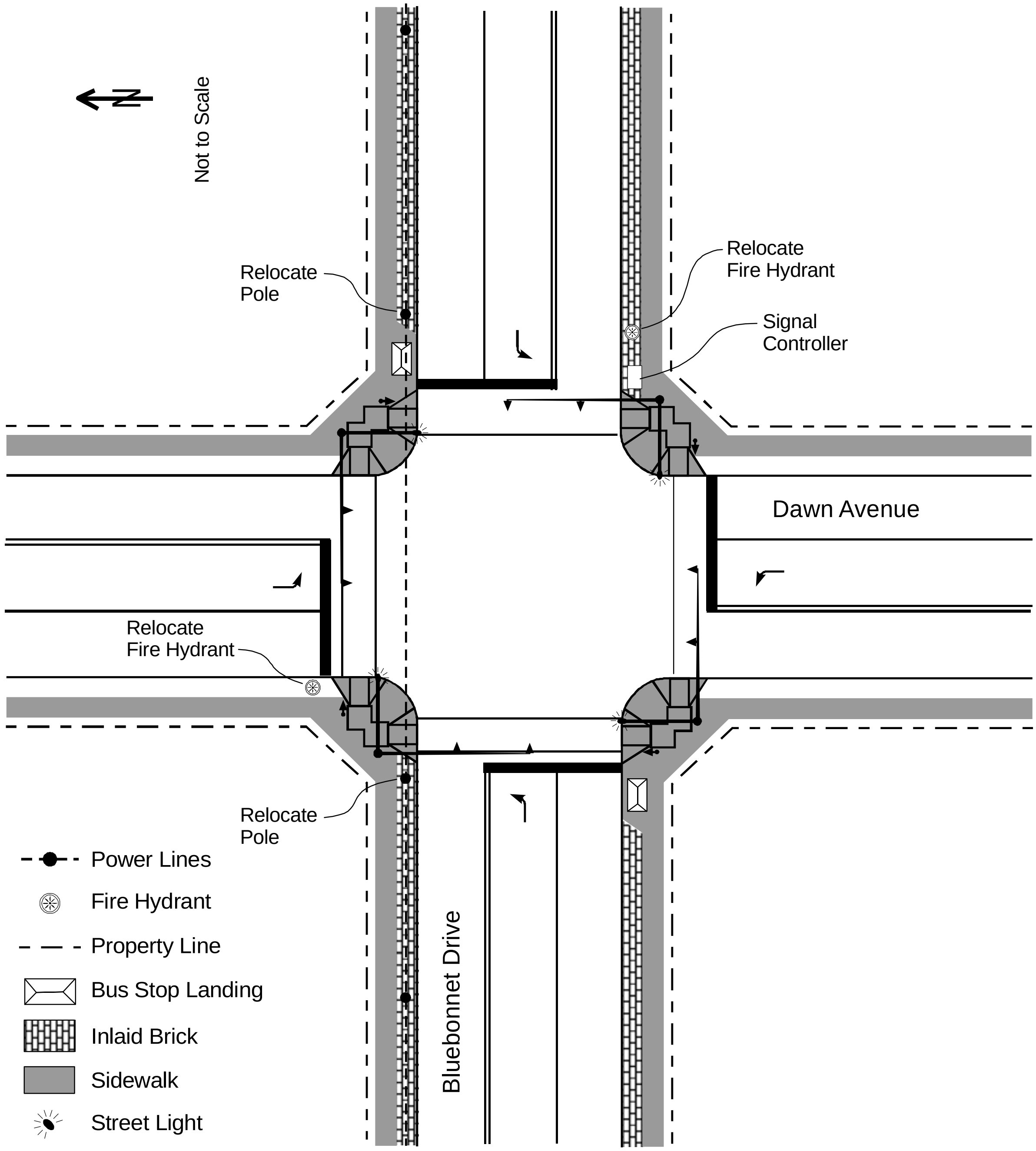


Figure 5-2. Proposed Design for Bluebonnet Drive and Dawn Avenue.

Application 2
Addition of Bus Bay

Overview

The operation of bus stops can significantly affect the performance of an intersection. A stopped bus at a nearside location can prevent vehicles from proceeding through the intersection. At these locations a bus bay can expedite turning movements and improve traffic signal operations; however, disadvantages include longer bus travel times and potential conflicts as the bus reenters the traffic. Bus bays are discussed in Chapter 5, Section 6 <link> of the *Urban Intersection Design Guide.*

Background

An urban intersection has a substantial amount of right-turning traffic on a particular approach. Queues in the right-turn lane are frequently blocked by stopped buses at a bus stop on the nearside of the intersection.

The intersection of Colgate Road and Fordham Drive is in an urban area, near a large hospital and medical park. Colgate Road is a four-lane arterial with a raised median, while Fordham Drive is a two-lane collector. The bus stop is located in the right-turn lane on eastbound Colgate Road. Developments on each corner are:

- a large parking area for the retirement home on the southeast corner,
- convenience stores on the northeast and northwest corners, and
- a small grocery store on the southwest corner.

The southwest corner is the location of the problem: eastbound right-turning vehicles in the right-turn lane are occasionally stopped by buses at the bus stop. When buses stop to board and alight riders, traffic in the right lane is stopped, causing increased delay.

Figure 5-3 illustrates the current intersection layout. The following is known:

- Colgate Road is an urban arterial roadway.
- Fordham Drive is an urban collector roadway.
- Design speed on Colgate Rd. is 45 mph [72 km/h].
- The intersection is signalized.

Issues Considered

Issues to consider during an upgrade to the site include the following:

- Move signal poles to accommodate redesign.
- Move the bus bench and waiting area during and after construction.

- Tie into existing sidewalk, but ensure grades are appropriate for usage by disabled pedestrians.
- Accommodate current traffic flow during construction.

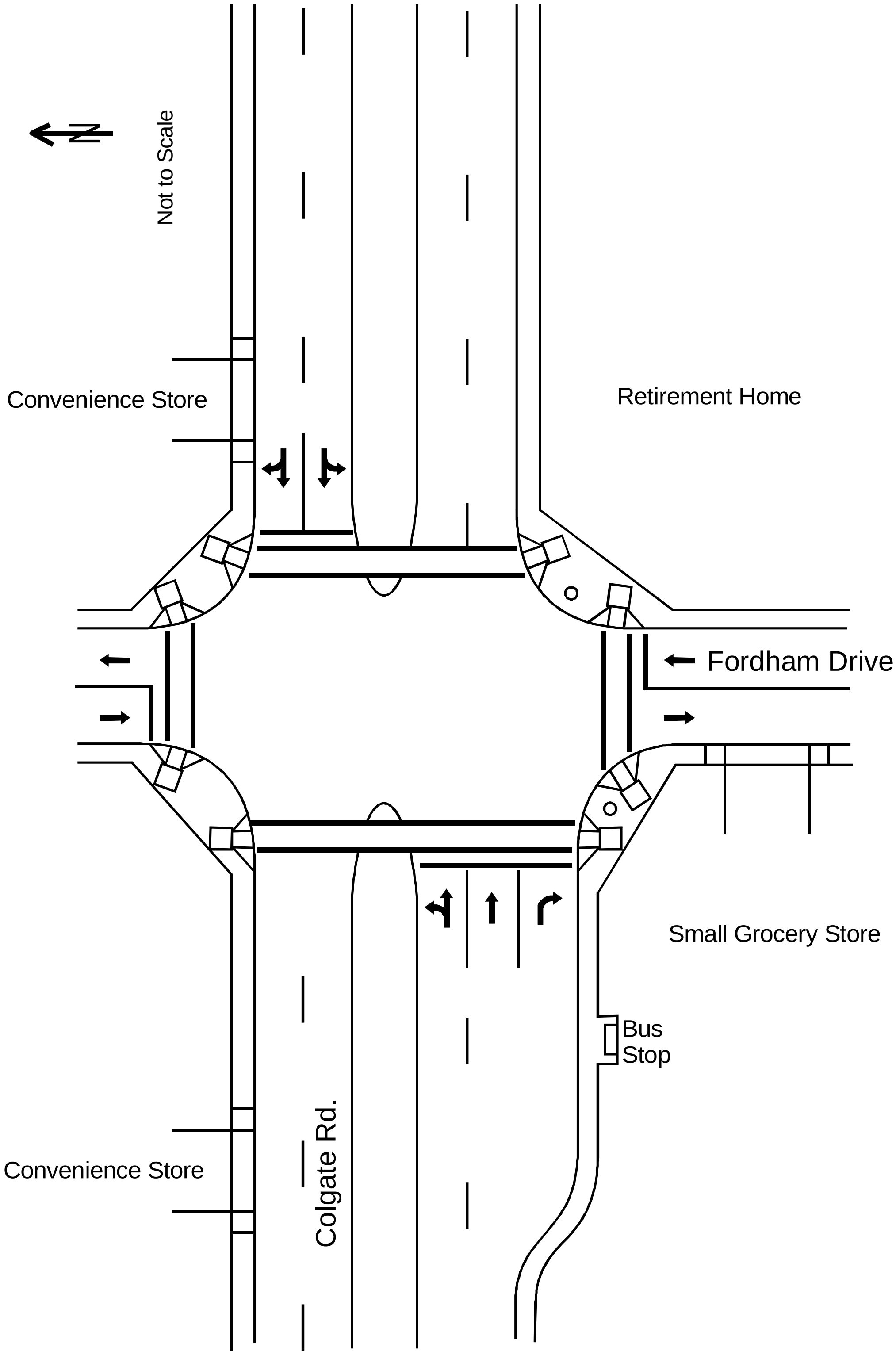


Figure 5-3. Existing Intersection Layout for Colgate Road and Fordham Drive.

In consultation with the local transit agency, the bus stop will be relocated to the far side of the intersection. This move will minimize delay to drivers while the bus is stopped and reduce conflicts with turning vehicles. It will also provide more direct access to users from the retirement home. A partial open bus bay will be used to provide a protected area away from moving vehicles for the bus and its patrons.

Other issues that were considered in the design include:

- ♦ Relocate the sidewalk to accommodate the new lane.
- ♦ Relocate a traffic signal pole due to the addition of the partial open bus bay.

Proposed Design

- ♦ *Step 1: Determine bus stop design.*

The operation of the bus stop presents a number of challenges at the intersection due to conflicts with right-turning vehicles and the inability of the bus to stop out of the travel lane while patrons board and alight. The original design, with the bus stop on the west side of the intersection, has the bus blocking other vehicles while it is stopped at the bus stop. Because the bus stop serves a hospital and the surrounding medical community the potential for both increased numbers of bus patrons and older or disabled bus riders is relatively high, potentially taking longer for bus stop operations. Because of these concerns, a partial open bus bay design was selected for the far side of the intersection, shown in Figure 5-4. This location eliminates the conflicts between the bus and turning and through vehicles, reduces the use of the bus bay as an acceleration lane by right-turning vehicles from Fordham Drive, and provides a sheltered area for bus patrons to board and alight.

The use of the partial open bus bay allows the bus to decelerate as it crosses the intersection, and shortens the crossing time for pedestrians through the provision of the curb extension, enhancing signal operations (see *Urban Intersection Design Guide*, Chapter 5, Section 5 <link>). The partial open bus bay also prevents right-turning traffic from south Fordham Drive from using the bus bay as an acceleration lane.

A disadvantage of using a bus bay is that bus drivers may have problems re-entering the traffic stream. The presence of the signal-controlled intersection, however, will provide gaps that the driver can use when exiting the stop.

An 8 ft by 5 ft [2.4 m by 1.5 m] bus boarding area has been added at the bus stop to comply with ADAAG requirements. Chapter 5, Section 6 <link> of the *Urban Intersection Design Guide* should be consulted for further information regarding bus stop design.

- ♦ *Step 2: Relocate signal pole and sidewalks.*

A traffic signal pole has been relocated to remove it from the roadway. This relocation will result in the use of a longer traffic signal mast arm, larger support pole, and larger pole foundations (see *Urban Intersection Design Guide*, Chapter 8, Section 3 <link>).

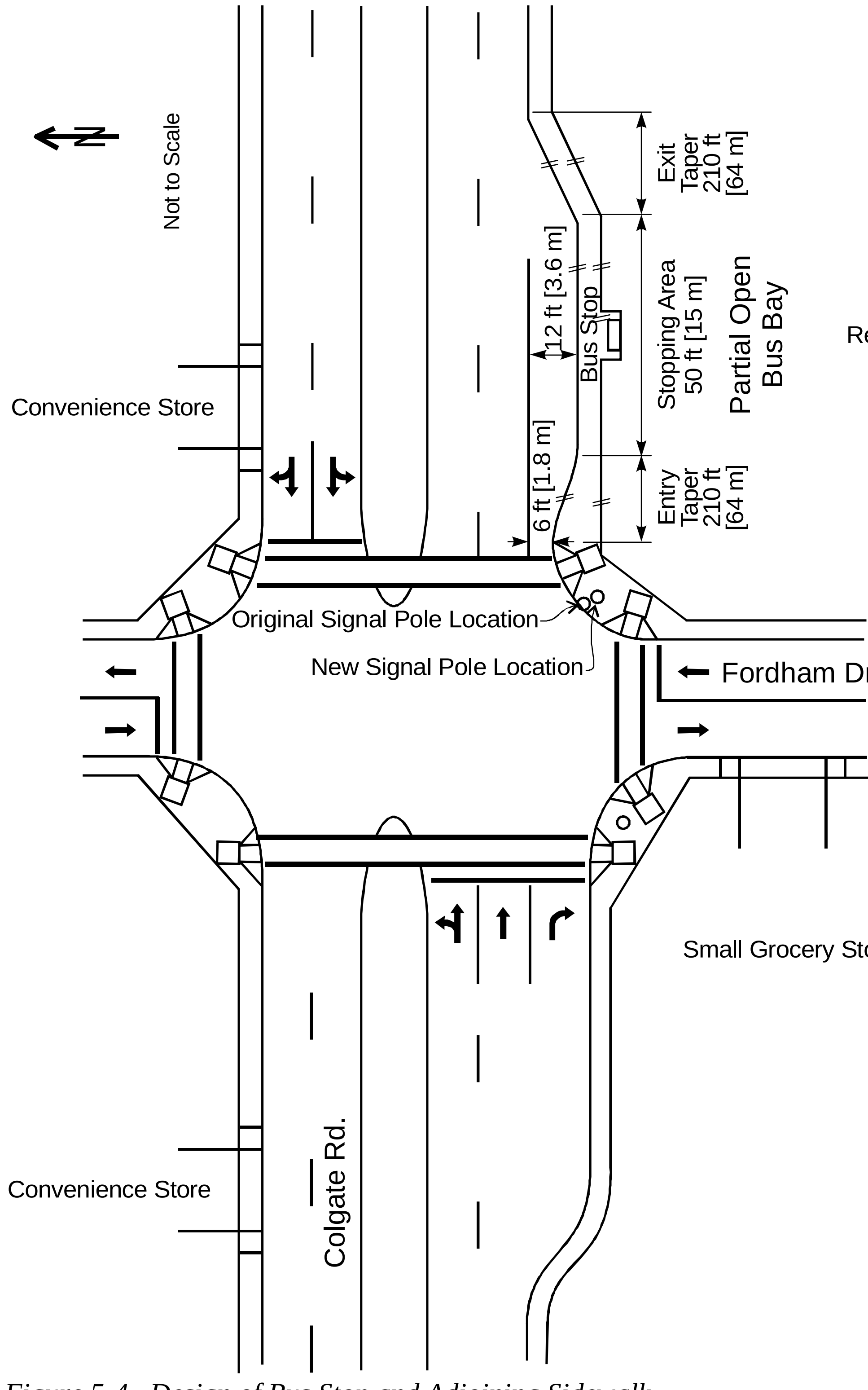


Figure 5-4. Design of Bus Stop and Adjoining Sidewalk.

Chapter 6
Drainage

Contents:

Application 1

Warped Profile and Cross Section

Overview

The following application discusses sidewalk considerations along with other concerns during a redevelopment. The *Urban Intersection Design Guide*, Chapter 6, Section 3 <link> presents information on drainage and roadway profiles.

Background

The intersection of Drake Avenue (an arterial roadway) and Fir Street (a local roadway) is in a small city. The roadways, shown in Figure 6-1, have the following characteristics:

- Drake Avenue
  - four 12-ft [3.7 m] travel lanes;
  - 14-ft [4.2 m] two-way left-turn lane;
  - curb and gutter cross section with 2-ft [0.6 m] curb offsets;
  - 2 percent cross slope; and
  - 40 mph [64 km/h] design speed.
- Fir Street
  - two 11-ft [3.4 m] travel lanes,
  - curb and gutter cross section with 1-ft [0.3 m] curb offsets,
  - 2 percent cross slope, and
  - 30 mph [48 km/h] design speed.

The intersection is being designed as part of a construction project that is reconstructing Drake Avenue and includes a storm drain system. Stop signs are installed on Fir Street at both approaches to Drake Avenue.

Pedestrians are frequently observed in the area of the intersection. Crosswalks are present across Fir Street but not across Drake Avenue. Pedestrians are encouraged instead to cross Drake Avenue at a nearby signalized intersection with pedestrian indications and crosswalks.

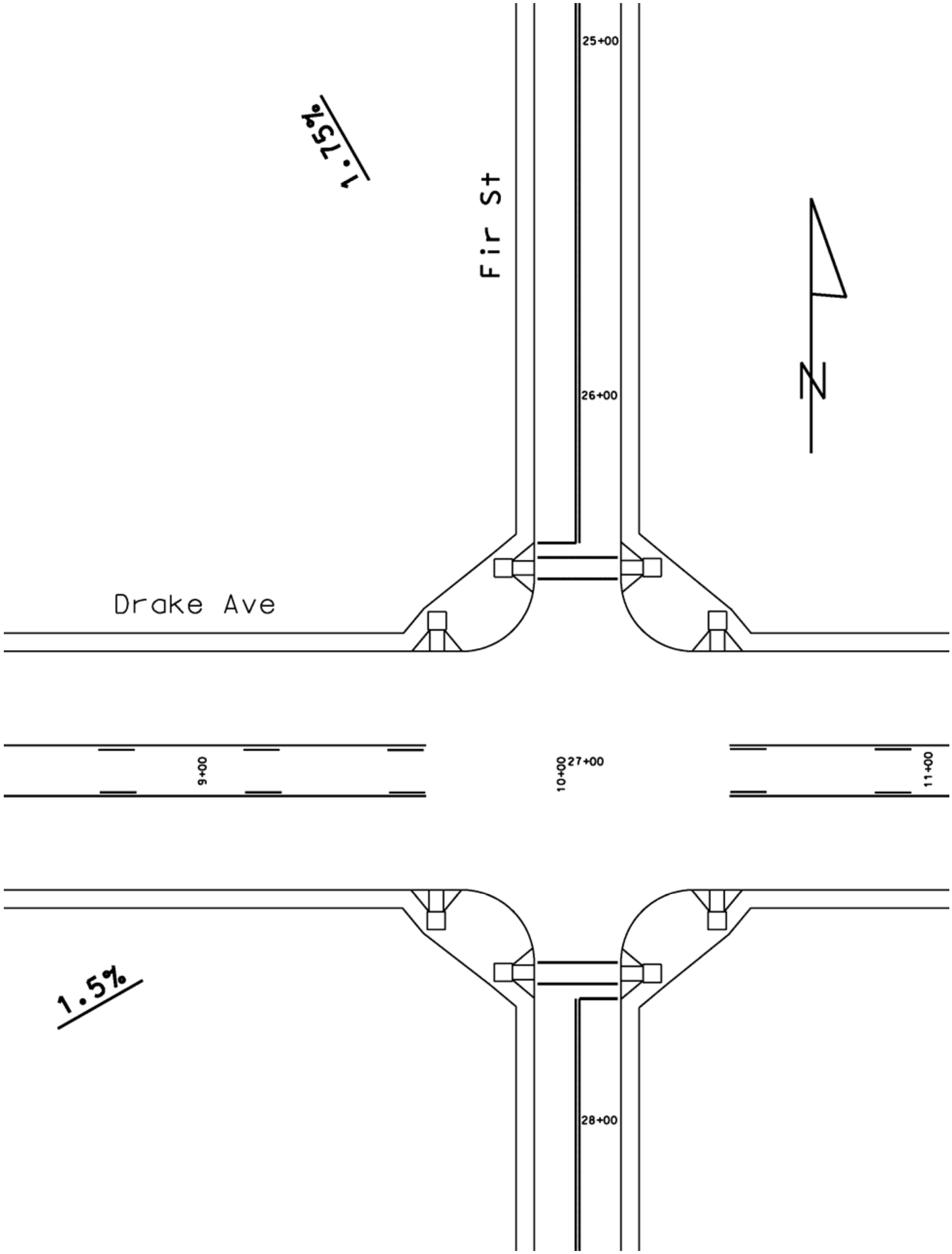


Figure 6-1. Drake Avenue and Fir Street.

Issues Considered

A principal concern at the intersection is the vertical profile present. The intersection is expected to remain stop-controlled on Fir Street. The speeds of vehicles crossing or turning onto Drake Avenue will be relatively low because of the stop condition. Drainage at the intersection has been a problem in the past, with water from Fir Street ponding at the intersection with occasional overflows crossing Drake Avenue. The present vertical alignment of the intersection was achieved by intersecting the centerline gradelines of the two roadways in a similar manner to that shown in Figure 6-7 of the *Urban Intersection Design Guide* <link>. The resulting roadway surface is uncomfortable to drive and results in water ponding in the corners of the intersection.

Another concern at the intersection is the design of the pedestrian elements. Pedestrians are common in the area, and pedestrian crossing points on both Fir Street and Drake Avenue frequently are flooded and have standing water present after storm events because of the vertical alignment.

Proposed Design

The design proposed for the intersection includes realigning the vertical profile on Fir Street and relocating the present storm drain inlets. The new vertical profile on Fir Street will help manage the stormwater runoff without allowing water to enter the intersection and impede the traffic on Drake Avenue. The relocated storm drain inlets will help keep water out of the intersection and help prevent water from ponding near the pedestrian curb ramps.

♦ *Step 1: Realignment of the vertical profile of Fir Street.*

The present vertical alignment on Fir Street north of Drake Avenue is shown in Figure 6-1. Because of the stop control on Fir Street, a new alignment of the general form shown in Figure 6-3 of the *Urban Intersection Design Guide* was selected <link>. The new alignment will insert two new vertical curves on Fir Street north of Drake Avenue to allow matching Fir Street's grades with the gutterline on Drake Avenue. Shown in Figure 6-2 and Figure 6-3, the alignment has a crest vertical curve at 23+49 [0+716] and a sag vertical curve at 25+37 [0+773] that allow Fir Street's alignment to match the cross slope on Drake Avenue.

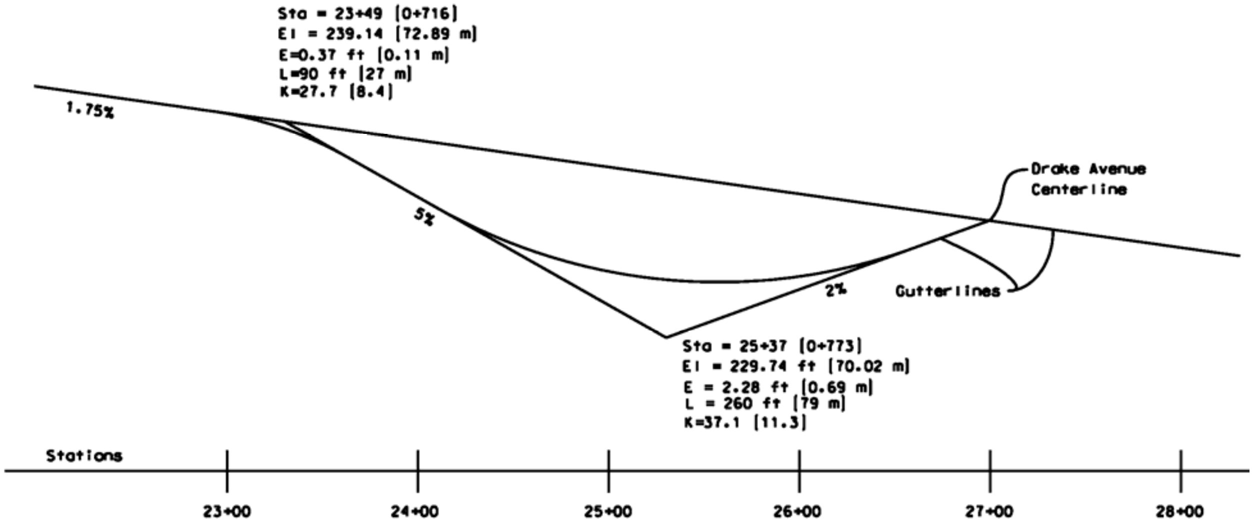


Figure 6-2. Vertical Profile of North Section of Fir Street.

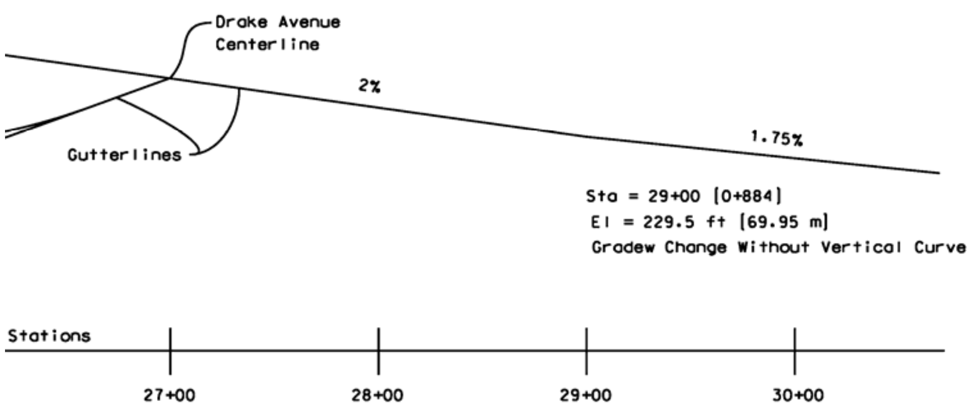


Figure 6-3. Vertical Profile of South Section of Fir Street.

The vertical curves on the north side of Drake Avenue were designed according to the design speed on Fir Street, 30 mph [48 km/h]. The K-factors for the curves were selected from Figures 2-6 and 2-9 [2-7 and 2-10 for metric units] of the *Roadway Design Manual* <link>. From the figures, minimum K-factors of 19 [11] and 37 [18] were determined for crest and sag curves, respectively.

The length of the crest vertical curve at 23+49 [0+716] was determined by the following equation:

$$L = K \times A$$

where:

L = length of the vertical curve, ft

A = algebraic differences in grade, percent

K = design control for the curve

Substituting in the equation:

$L = 19 \times 3.25 = 61\,\text{ft}$ [18.6 m]

The minimum length of a vertical curve is three times the design speed, however. In this case the minimum length is three times 30 mph [48 km/h], or 90 ft [27 ft].

The length of the sag vertical curve at 25+37 [0+773] was found in a similar manner:

$L = 37 \times 7 = 260\,\text{ft}$ [79 m]

The sag vertical curve length exceeded the minimum length, so 260 ft [79 m] was used in the design.

Proceeding to the south side of Drake Avenue, the gradeline was again matched to the crossfall on Drake Avenue. The alignment matched the existing street elevation at station 29+00 [0+884]. The grade change at the PI was very small, 0.25 percent. Because the grade change is less than 1 percent, no vertical curve is required at this location (see the *Roadway Design Manual* Chapter 2, Section 5, Grade Change Without Vertical Curve) <link>.

The addition of the sag vertical curve at 25+37 [0+773] will increase the right of way and require additional excavation because the new roadway gradeline will be below the present grade in the area of the vertical curve.

♦ *Step 2: Warp pavement cross section on Fir Street.*

The next step in the design was to match the cross section on Fir Street to the gutterline on Drake Street. To accomplish this, Fir Street's cross section was warped as shown in Figure 6-4. The rotation from the normal cross slope of a 2 percent crown to a constant 1.5 percent up to the east was accomplished over 90 ft [27 m], as shown in the figure. The transition section length was selected to match the rotation rate typically used for superelevated roadway sections (see the *Roadway Design Manual's* guidelines on transition length in its Chapter 2, Section 4 <link>). This criterion was used in the absence of firmly established guidance regarding the development of warped cross sections for drainage purposes.

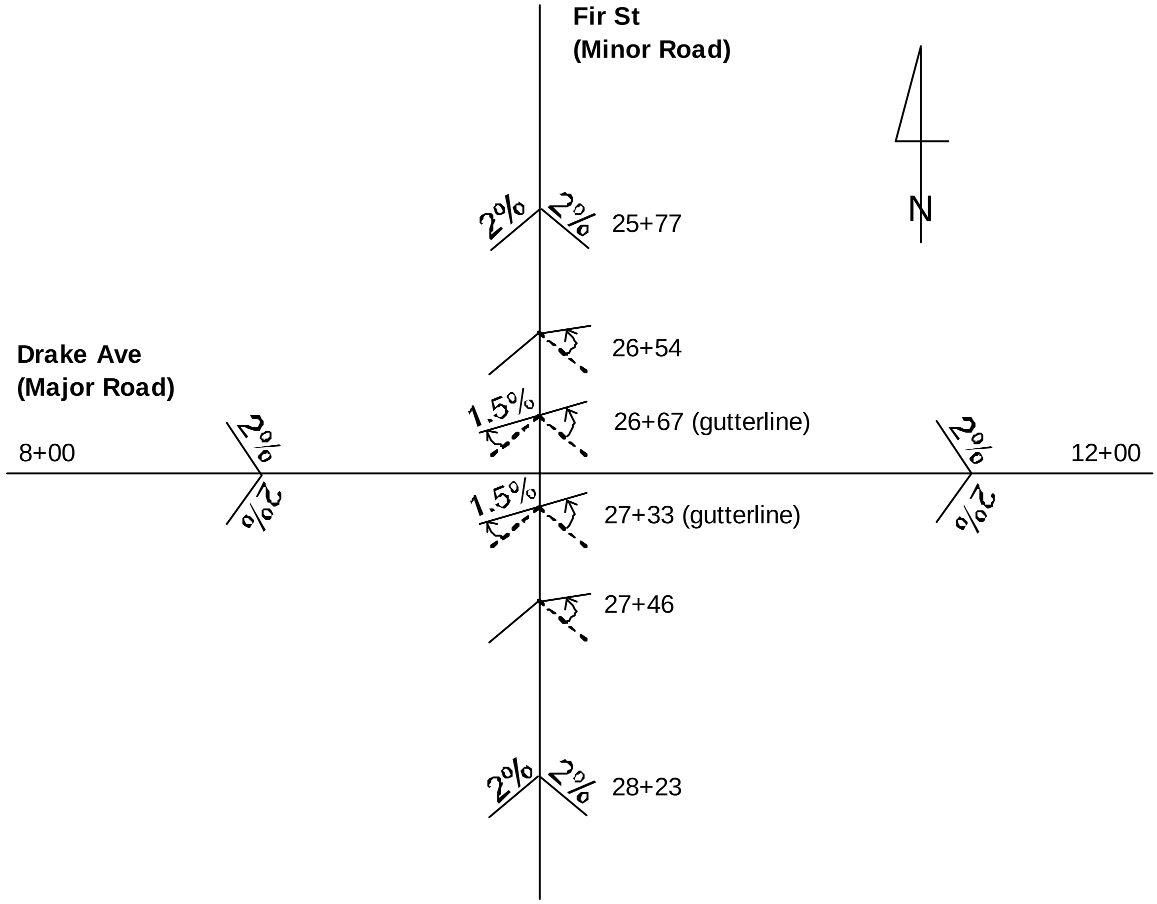


Figure 6-4. Fir Street Cross Section Transition.

- ♦ *Step 3: Develop intersection contour plot to review drainage.*

The next step in the design was to develop a contour plot of the intersection. Shown in Figure 6-5, the contour plot allowed the designer to determine where any low spots were located in the alignment so that the alignment could be adjusted if necessary to eliminate any undesirable "bird baths" or irregularities. Because of the way the pavement was warped and the presence of the vertical curves at the intersection, it is critical that the design be reviewed in this manner. The contour plots (shown at 0.5-ft [0.2 m] intervals for clarity in this example—normally, a smaller interval would be used for design) allow the determination of the locations for curb inlets in the low points on Fir Street.

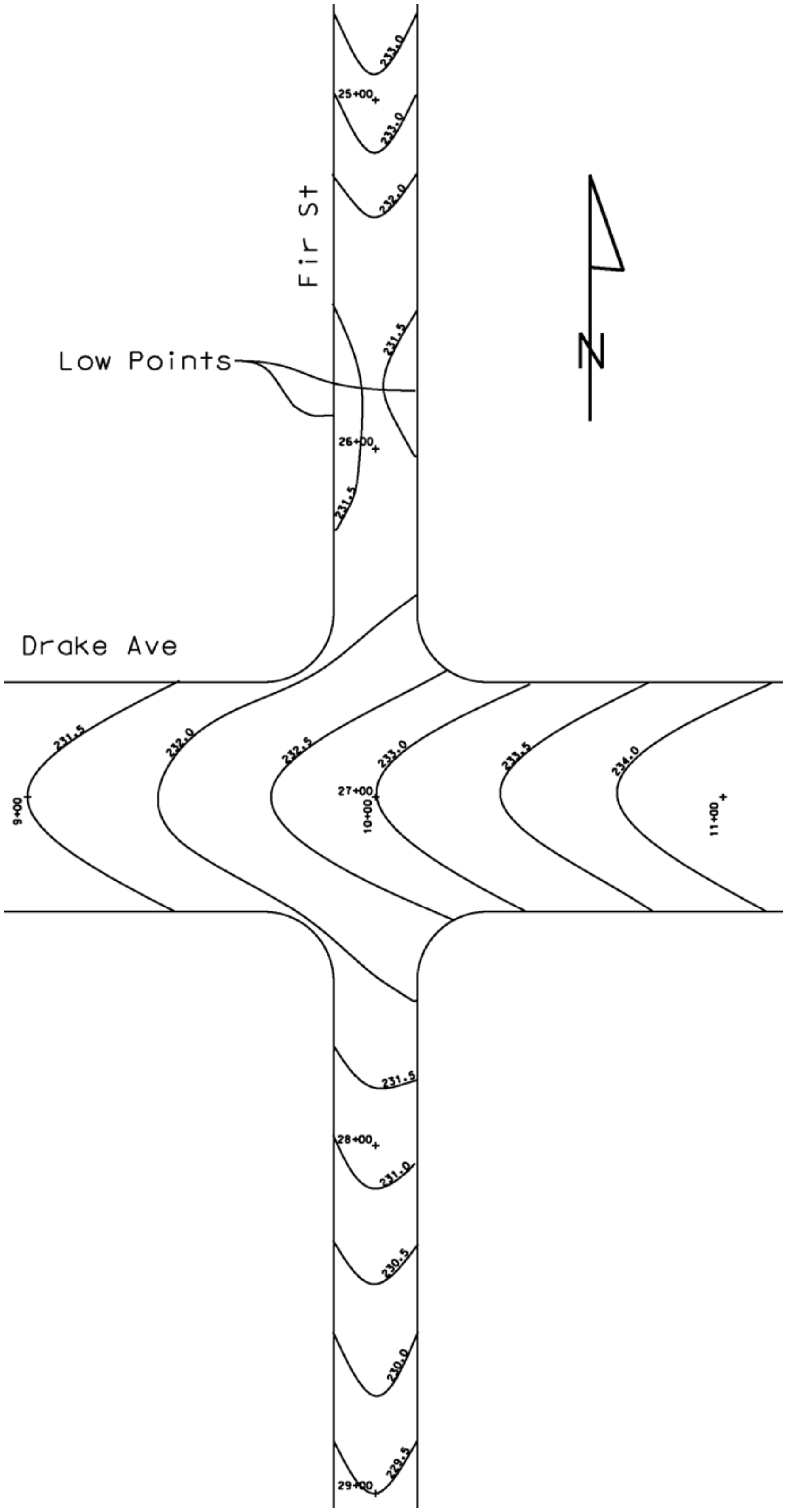


Figure 6-5. Contour Plot of Intersection.

- *Step 4: Locate curb ramps and remainder of curb inlets.*

The final step in the design was to locate the curb ramps and upstream curb inlets at the intersection.

Although a marked crosswalk across Drake Avenue is not present at this location, crossings may nevertheless occur. Accordingly, curb ramps are provided for pedestrians. Shown in Figure 6-6, the curb ramps are located within the crosswalks (or at the location a marked crosswalk would be placed, in the case of the Drake Avenue crossing point); further information about curb ramps and crosswalks can be found in the *Urban Intersection Design Guide* Chapter 7, Sections 1 and 2 <link>.

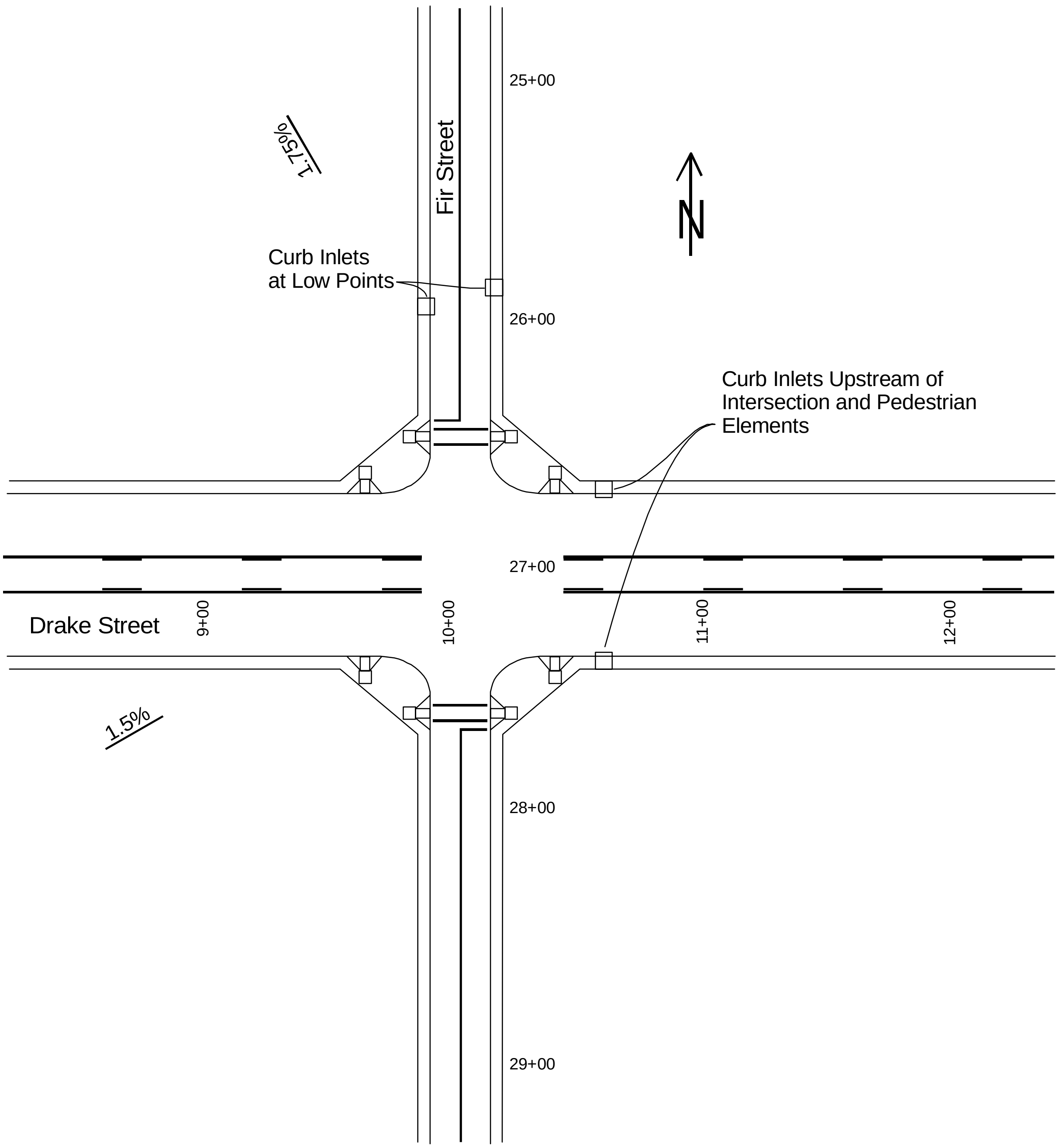


Figure 6-6. Final Intersection Layout.

Curb inlets were placed on the upstream side of the intersection on Drake Avenue. This prevents concentrations of water from entering the intersection. The curb inlets were placed upstream of the curb ramps, avoiding interference with pedestrian crossings and the construction of the ramps.

Chapter 7
Street Crossing

Contents:

Application 1

Suggestions for Making an Intersection Accessible

Background

The use of transportation facilities by disabled persons is increasingly an important concern in the design of those facilities. In this application an urban intersection, with moderate to heavy pedestrian traffic, is examined. It was built many years before current accessibility requirements were established. Information on street crossings including discussion on pedestrian considerations is in the *Urban Intersection Design Guide*, Chapter 7 <link>.

The intersection is shown by a sketch in Figure 7-1. Figure 7-2 shows the locations of the photographer and the direction the pictures were taken for the photos included in this application. Figure 7-3 and Figure 7-4 are overview photos of the intersection. The intersection is in an urban area, close to shopping, convention centers, and tourist attractions. The occupants of the corners immediately adjacent to the intersection are: a mall/hotel, a diner restaurant, a city public works building, and a parking garage. Main Street is a five-lane urban arterial running one way west. James Street is also five lanes wide at the intersection, with a combination of through and turning lanes.

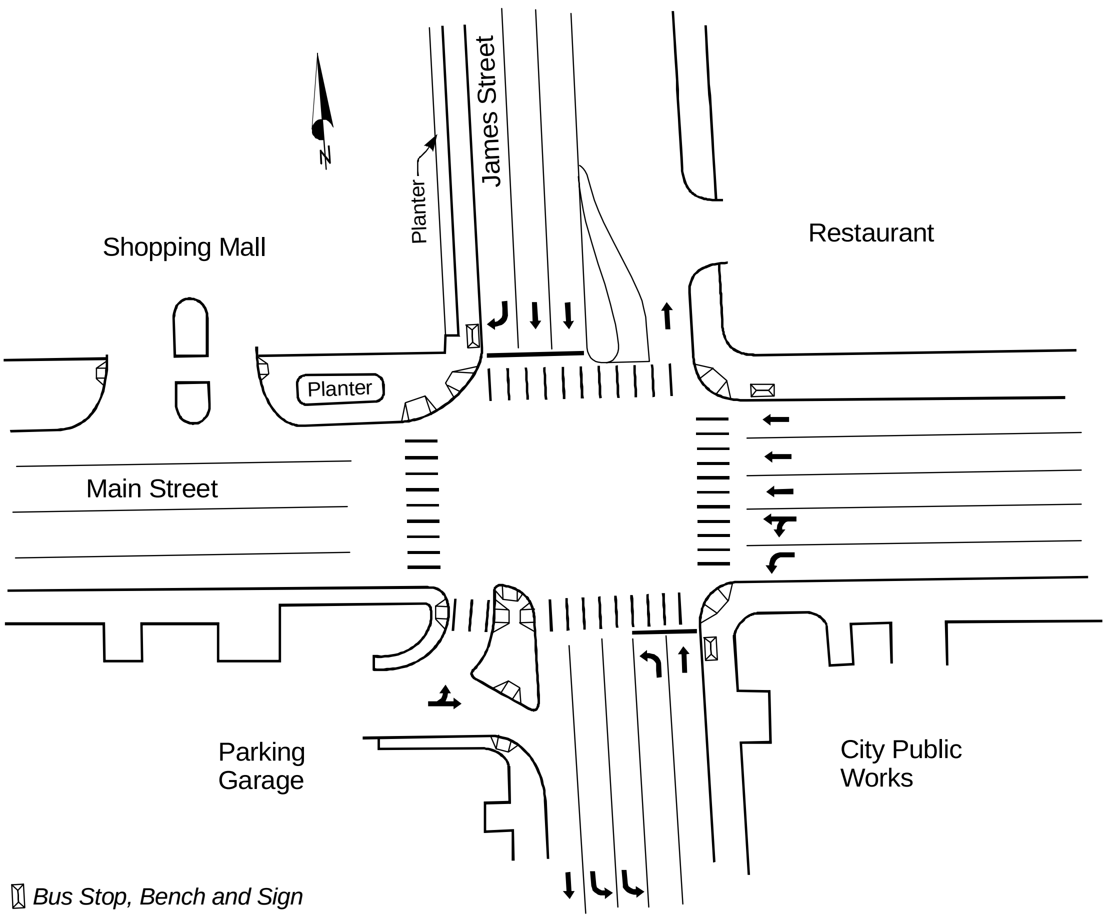


Figure 7-1. Plan Sketch of Intersection.

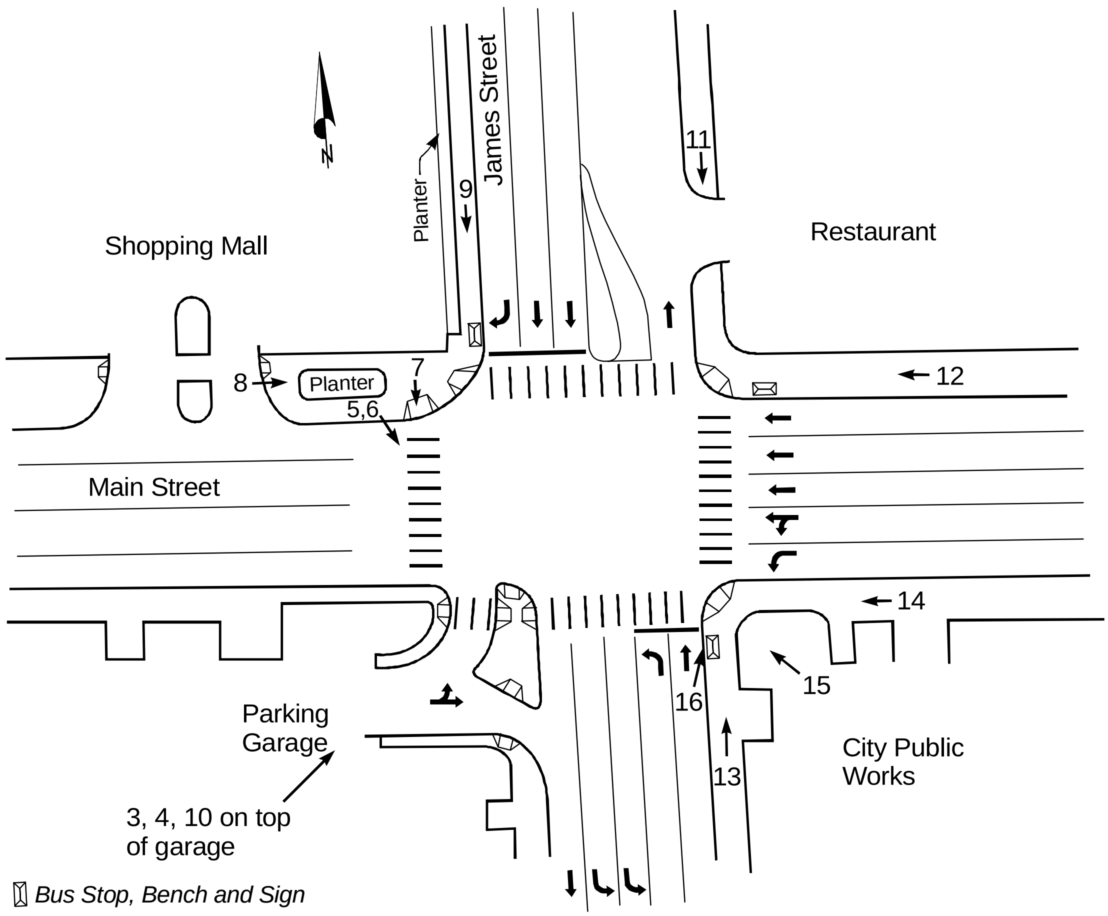


Figure 7-2. Location of Photographs (xx = figure number).

Pedestrian traffic at this intersection, particularly after special events, can be very high. All of the approaches have ADA compliance issues, particularly in the design of curb ramps. Grades and flares are too steep on most curb ramps and none have detectable warnings.

The exits to the parking garage on the southwest corner create two additional "crosswalks" in the approaches to the intersection (see Figure 7-5); these paths across the exits add complexity to negotiating the approach to the intersection on both sides of the garage adjacent to the intersection. During a period of observation, it was noted that a substantial number of pedestrians "created their own path" across that corner, causing increased potential for conflicts with vehicles.

The southwest quadrant has very high curbs, and not all curb ramps are aligned with crosswalk markings. The north-south crosswalk ends at a curb, and a light pole is on the sidewalk where a curb ramp landing should be. The paths of the two garage exits create an island on the corner (see Figure 7-6). This island is steep and not level, with no true landing area, and steep curb ramps. As shown in Figure 7-5, one curb ramp on the island leads out to the southbound right-turn lane.

Figure 7-3. Overhead View of Intersection Looking Northeast.

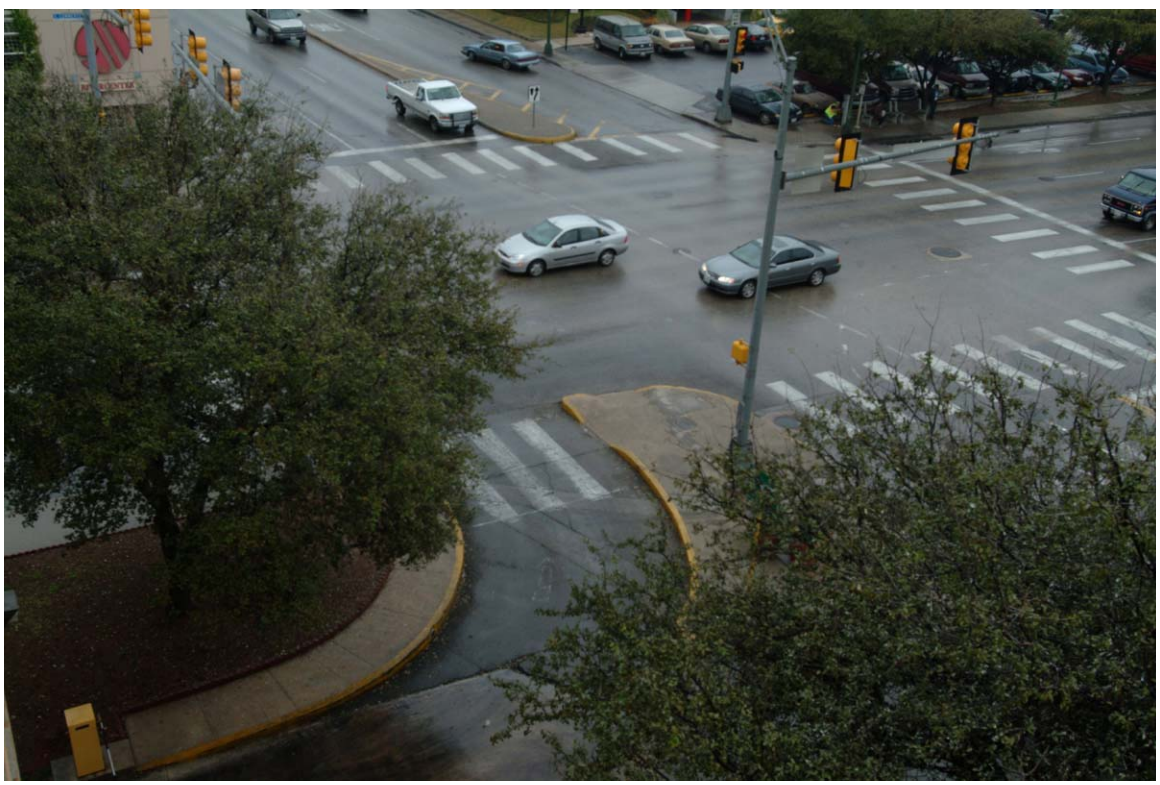


Figure 7-4. Overhead View, Showing Parking Garage Exit.

Figure 7-5. View of Exits from Parking Garage and Corner Island (Southwest Corner of Intersection).

Figure 7-6. Uneven Island and Ramp with No Crosswalk (on Southwest Corner of Intersection).

The design of the northwest corner appears the newest of the four corners (see Figure 7-7). The east-west approach sidewalk has a large planter that effectively divides the sidewalk into two smaller parts, as illustrated by Figure 7-8. Figure 7-9 shows that the north-south approach sidewalk has several street furniture, signs, and other obstructions.

Figure 7-7. View of Crosswalk across West Side of Intersection.

Figure 7-8. Planter Dividing Sidewalk on Northwest Corner of Intersection.

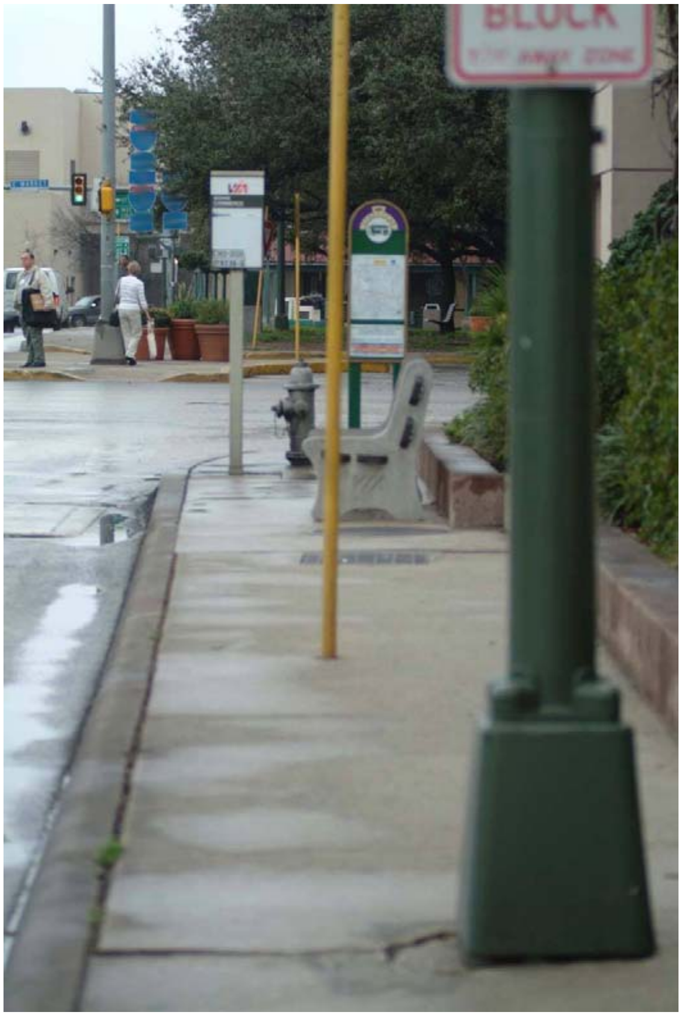


Figure 7-9. Street Furniture in Sidewalk Area Looking South on Northwest Corner of Intersection.

The restaurant in the northeast corner affects pedestrian movement in that area. A driveway is located close to the intersection. The parking area is adjacent to the sidewalk, and inappropriately parked cars can affect walking, especially in the restricted area where the signal pole partially blocks the sidewalk and restricts pedestrian storage (see Figure 7-10). Figure 7-11 illustrates a light pole in the middle of an already narrow approach sidewalk. The other approach sidewalk has good width, but has a cross slope that exceeds the 2 percent maximum allowed (Figure 7-12).

The sidewalks on the southeast quadrant are more open on the approaches, but have obstructions close to the intersection. Figure 7-13 illustrates that the approach sidewalk on James Street has several poles and low-height signs that protrude more than 4 inches [102 mm] between 27 inches [686 mm] and 80 inches [2032 mm] above the surface. The other sidewalk on Main Street has a utility cover in the pedestrian path (Figure 7-14). Figure 7-15 illustrates the multiple decorative paths in the vicinity of the corner. These paths could be disorienting to a visually impaired pedestrian. The substantial change in level and rough pavement at the curb ramp could also cause problems for some pedestrians (see Figure 7-16). However, one positive item is the bus bench, which has been set back from the sidewalk, as shown in Figure 7-13.

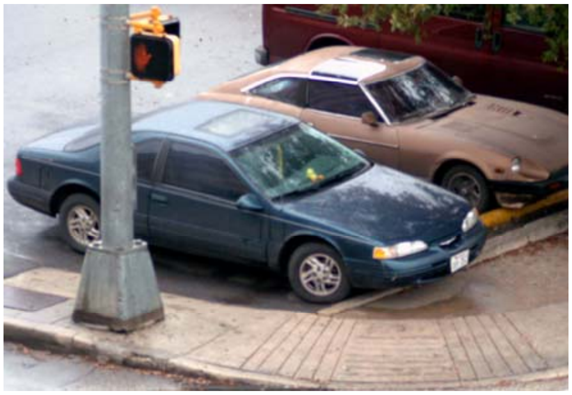


Figure 7-10. Parked Vehicle and Signal Pole Block Pedestrian Travel Near Northeast Corner of Intersection.

Figure 7-11. Light Pole in Narrow Sidewalk on Northeast Corner of Intersection Looking South.

Figure 7-12. Cross-Slope on Approach in Sidewalk on Northeast Corner of Intersection Looking West.

Figure 7-13. Poles and Signs in Sidewalk on Southeast Corner of Intersection Looking North.

Figure 7-14. Utility Cover in the Pedestrian Path at Southeast Corner.

Figure 7-15. Multiple Paths at Southeast Corner of Intersection.

Figure 7-16. Uneven Pavement and Change in Level at Southeast Corner of Intersection.

Issues Considered

Issues to consider during an upgrade to the site include the following:

- Acquisition of right of way could be costly and difficult to obtain.
- Redesigning the restaurant driveway or eliminating two parking spaces may require a lengthy negotiation with the restaurant owners to minimize perceived impacts on business.
- Closing and/or relocating garage exits will also require an agreement with the garage owners, who may be unwilling to alter their points of egress.

- A number of light poles, signs, and other obstructions would have to be relocated to accommodate compliant ramps, landings, and sidewalk widths.
- Reconstruction of curb ramps would cause significant disruption to currently existing pedestrian traffic. Coordination and planning would be necessary to minimize those impacts, particularly during times of special events.
- Improving sidewalk cross slopes also would impact the flow of pedestrian traffic. Again, coordination would be necessary to minimize disruption.

Suggested Designs

A review of this intersection resulted in the following general solutions to address ADA compliance issues:

- Provide landing areas of at least 5 ft by 5 ft [1.5 m by 1.5 m] at the top of ramps.
- Reconstruct curb ramps to compliant slopes.
- Install truncated dome detectable warnings (light and texture contrast).
- Relocate street furniture and other obstructions.
- Enforce no parking on sidewalks or add self-enforcing devices such as bollards.
- Install audible pedestrian signals.
- Increase pedestrian crossing times.
- Mill and overlay intersection to improve surface and eliminate lips at the bottom of ramps without increasing roadway crown (and therefore crosswalk grade). Ensure 2 percent maximum cross slope within crosswalk.
- Relocate signal poles and other supports as needed to eliminate obstructions.
- Place pedestrian push buttons near ramp landings.
- Increase width of sidewalks.

In addition, there are suggestions for improvement that are specific to each corner of the intersection.

SW Corner

- Line up a continuous crossing across James Street and garage exit driveway.
- Ideally, close existing garage exits and open a new exit at a greater distance from the intersection.
- Island improvements include the following:
  - On east-west crossing, install cut-through island with a flush 5-ft-wide [1.5 m] crossing. (Island is not wide enough to accommodate compliant ramp grades and a level landing.)
  - Install truncated domes on island cut-through.

- Eliminate northern ramp on island. (A visually impaired person may think this is the crossing location and wander into oncoming southbound traffic.) Block off this curb with the placement of decorative fencing or planters.
- Eliminate a sidewalk wrapping around from the corner toward the garage. Instead, construct a short sidewalk that lines up with the north-south crosswalk with ramps that are accessible to street level and garage surface level.
- Increase available right of way to provide compliant width on the east-west sidewalk, with appropriate landings on the sidewalk and corner island.
- Reduce height of curb along Main Street and install curb ramp at north-south crosswalk.

NW Corner

- ♦ Remove short retaining wall planter from north-south approach to increase available sidewalk width for pedestrians, particularly at bus stop (for wheelchair lift and bus bench).
- ♦ Install audible pedestrian signal to mitigate noise from underground exhaust vent.
- ♦ Reduce turning radius at corner to accommodate two compliant perpendicular ramps.
- ♦ Remove or reduce large planter from east-west sidewalk, as it occupies significant pedestrian space and would be disorienting to a visually impaired pedestrian.
- ♦ Reconstruct ramps with compliant slopes, landings, and truncated domes. If compliant perpendicular ramps cannot be accommodated, install parallel ramps.

NE Corner

- ♦ Acquire additional right of way on corner to provide landing area and storage.
- ♦ Reconstruct restaurant driveway to provide correct cross slope on the sidewalk that crosses the driveway.
- ♦ Relocate newspaper boxes and other street furniture.
- ♦ Improve delineation of parking spaces and eliminate the two stalls closest to the street to prevent cars from parking on the sidewalk.
- ♦ Reconstruct ramps with compliant slopes, landings, and truncated domes. Construct one perpendicular ramp for each crossing.
- ♦ Move crosswalk across Main Street back about 7 ft [2.1 m].

SE Corner

- ♦ Acquire right of way/easement to increase space available for compliant ramps and platoons waiting to cross.
- ♦ Remove corner radius since no northbound traffic can turn right.
- ♦ Reconstruct ramps with compliant slopes, landings, and truncated domes. Construct one perpendicular ramp for each crossing.
- ♦ Reduce height of curb along Main Street.

Figure 7-17 shows some of the suggested improvements for the intersection.

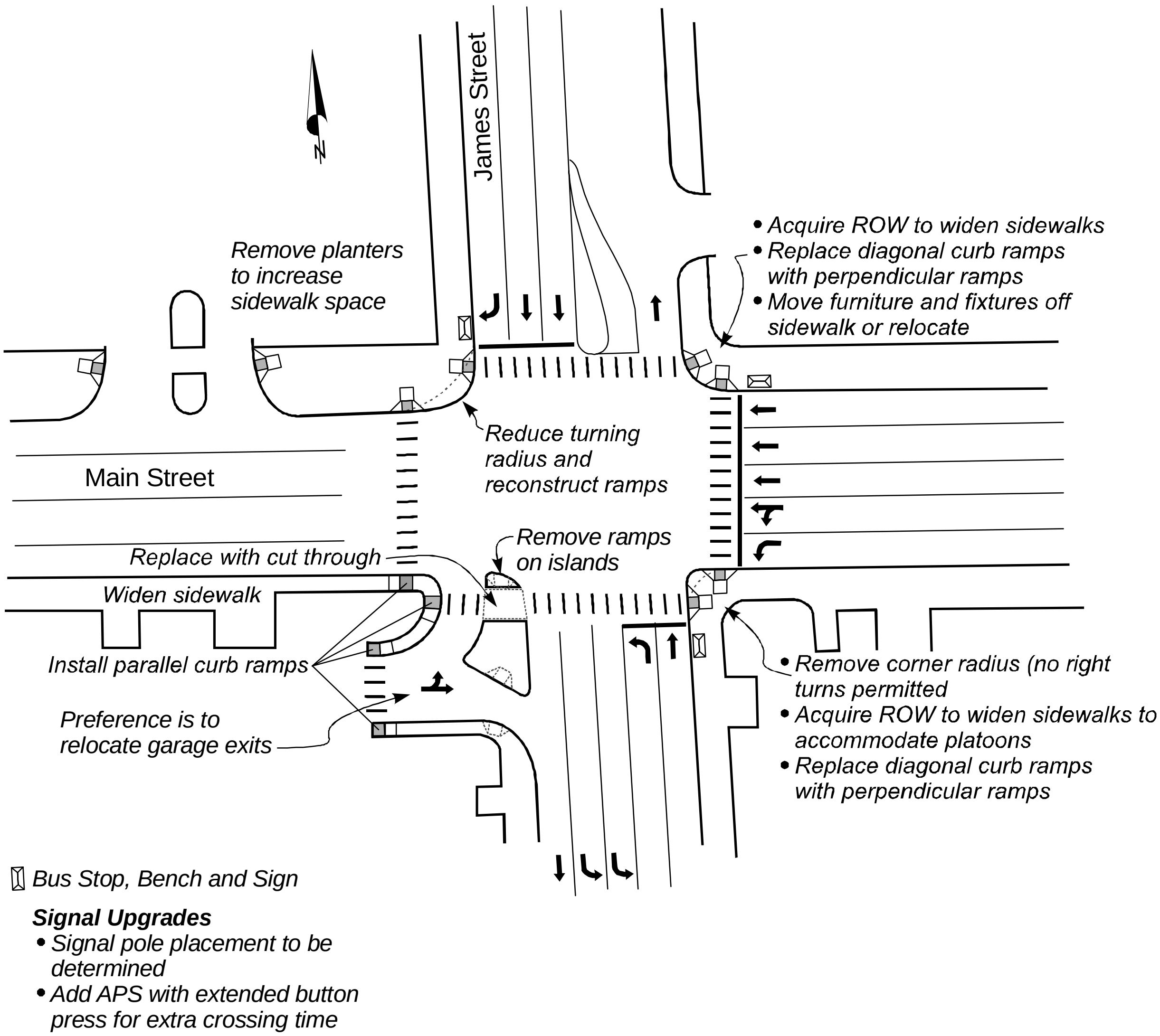


Figure 7-17. Improvement Suggestions for Intersection.

Application 2
Pedestrian and Bicyclist Accommodation

Overview

The following application discusses considerations in accommodating pedestrians at an intersection. The *Urban Intersection Design Guide*, Chapter 7, Section 2 <link> provides additional information on crosswalks.

Background

An unsignalized intersection of an arterial and a major driveway entrance exists between two arterial/arterial intersections within a developed area of a city of approximately 200,000. (See Figure 7-18.) Crockett Parkway is a 58-ft-wide [17.7 m], five-lane arterial with curb and gutter that accommodates approximately 30,000 vehicles per day, with a 45-mph [72.5 km/h] speed limit. Apple Lane is a 40-ft-wide [12.2 m] driveway that provides access to a large office and commercial development located to the south of Crockett Parkway. Apple Lane is not the primary development entrance/exit, but it is the only entrance to the development along Crockett Parkway. Arterial/arterial intersections exist approximately 0.25 mi [0.4 km] to the west and approximately 0.25 mi [0.4 km] to the east of Apple Lane. The city provides good vehicular progression between the two arterial/arterial intersections. Due to traffic flow patterns, there are high volumes of left turns from westbound Crockett Parkway onto Apple Lane and high volumes of right turns from northbound Apple Lane onto Crockett Parkway. The volume of right turns from eastbound Crockett Parkway onto Apple Lane and the volume of left turns from northbound Apple Lane onto Crockett Parkway are minimal.

The city anticipates expansion of the office and commercial development and an increase of traffic volumes on Crockett Parkway. Existing plans are to widen Crockett Parkway to six lanes with a raised median. Anticipating the widening project, the multiple owners of the office and commercial development informed the city that they would be willing to provide land along the south side of Crockett Parkway for the widening project in exchange for two additional project features. First, the owners wish to signalize the intersection of Crockett Parkway and Apple Lane. Second, the owners would like to provide a crossing for pedestrians and bicycles at the intersection. Numerous requests have been submitted to both the city and the owners of the office and commercial development from employees who reside in the area north of Crockett Parkway relatively close to Apple Lane. If the intersection (and a crossing) is made accessible to these residents, many would prefer to either walk or ride bicycles to work. The owners would like to accommodate these employees because fewer parking places would be needed by the employees, the exercise would be good for the employees, and the environment would benefit from the reduction of automobile traffic.

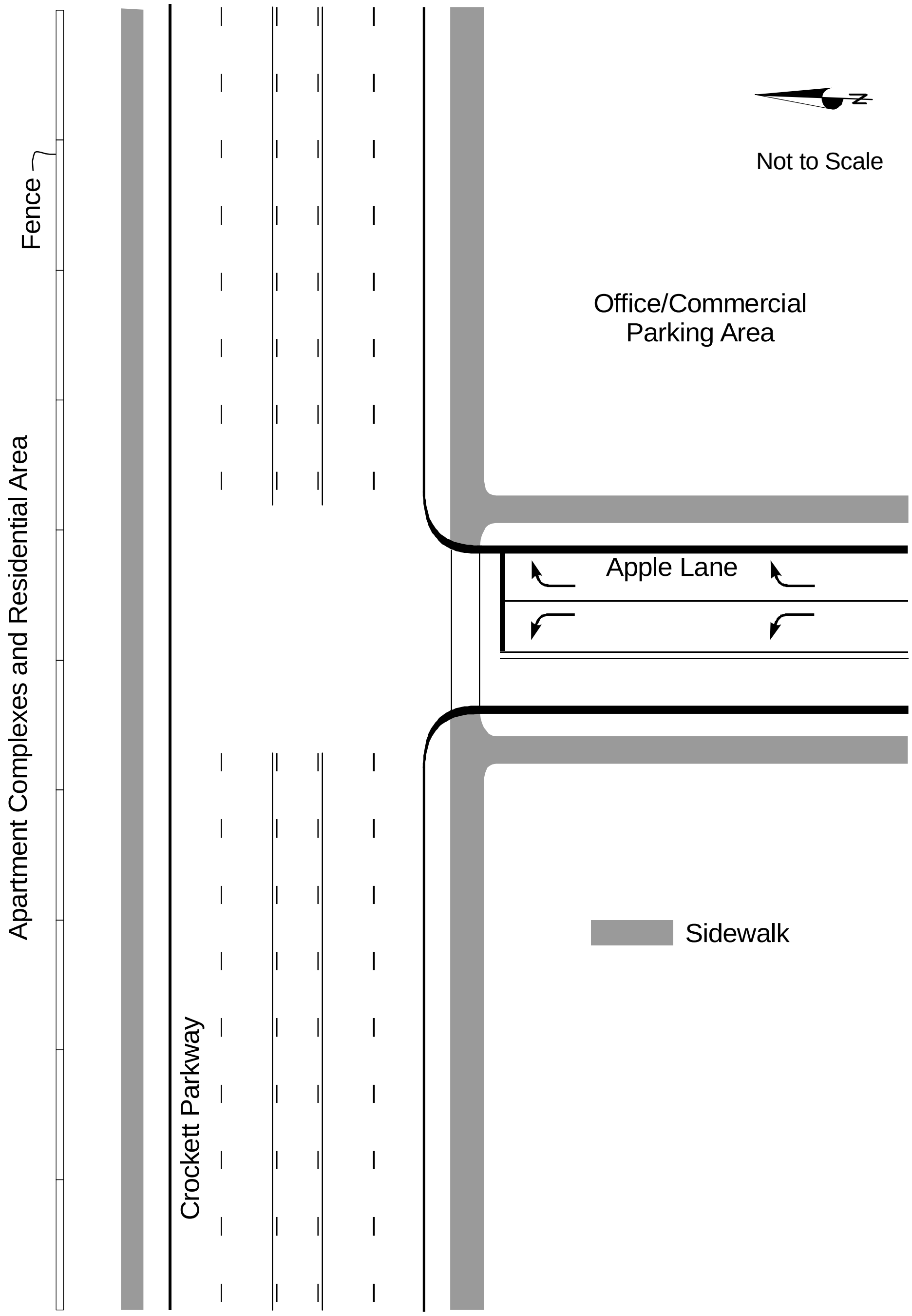


Figure 7-18. Existing Conditions.

Issues Considered

The city has attempted to encourage bicycle traffic by developing bicycle lanes, routes, and paths throughout the city, so city officials were willing to accommodate the owners of the office and commercial development. The city also recognized that the installation of a pedestrian and bicycle crossing of a wide, high-speed arterial (Crockett Parkway) would be difficult (and of questionable safety) without a traffic signal installation. In addition, allowing left turns from westbound Crockett Parkway onto Apple Lane across three high-speed travel lanes also caused some concern. Considering the amount of traffic and operating speeds on Crockett Parkway, the city recognized that the volume of traffic making left turns from northbound Apple Lane and from westbound Crockett Parkway would warrant signal installation. Because of the location of the Apple Lane intersection between two signalized intersections that were about 0.5 mi [0.8 km] apart, it was assumed that signal progression could be maintained between the two arterial/arterial intersections (and through the Apple Lane intersection) as long as the signal operation at the Apple Lane intersection could be designed to minimize disruption of through traffic.

Design Selected

The city planned to widen Crockett Parkway along the south side of the roadway, providing a six-lane arterial with a 17-ft-wide [5.2 m] raised median. As shown in Figure 7-19, the pedestrian/bicycle crossing was incorporated together on the west side of the intersection. The owners of the office/commercial development initiated plans to construct a separate bicycle path to connect the intersection and west-side crossing to existing bicycle lanes and paths within the complex. The crossing was planned to be 14 ft [4.3 m] in width to provide ample space for pedestrians and bicyclists to use the crossing simultaneously. Access northward from the intersection into the apartments and residential area could be provided with minimal problems. The west-side crossing location was preferred because the crossings could be made simultaneously with westbound Crockett Parkway left turns. Also, the crossing would not interfere with high-volume right-turning traffic from northbound Apple Lane. Because of the T-intersection, the median on the west side would not require a left-turn lane and it could remain 17 ft [5.2 m] in width, which provided ample room for storing bicycles and pedestrians and sufficient width to provide curb ramps.

Traffic signals operate with maximum efficiency if “green” time on the major street is maximized. Hence, “green” time should be provided for Crockett Parkway as much as possible. Two specific design features were incorporated into the intersection design to minimize “green” time for Apple Lane traffic and the pedestrian and bicycle crossings. First, the median on the west side of the intersection was equipped with pedestrian signals and pedestrian push buttons. The signals were planned to be programmed to provide crossing Crockett Parkway in two stages. Pedestrians and bicyclists would be required to store in the wide median area and wait for “green” time to cross, which would be provided only when signal progression would allow the disruption. Second, the northbound right-turn lane on Apple Lane was designed to operate as a “free” right-turn lane, and not be controlled by the traffic signal operation. An island was designed to separate the two approach lanes on the northbound Apple Lane approach.

A pedestrian crossing also was designed across Apple Lane on the south side of the intersection. The crossing extended across the free right-turn lane into the island, and then from the island to the southwest corner of the intersection.

Pedestrian signals and push buttons were designed for installation in the island and on the southwest corner of the intersection.

High-visibility, ladder-style crosswalk markings were selected for installation on the pavement to emphasize both the shared pedestrian/bicycle crossing on Crockett Parkway and the pedestrian crossing on Apple Lane.

After finalizing the initial design, the design engineers were informed by the city's legal department that the absence of a marked pedestrian crosswalk on the east side of the intersection did not mean that a "legal" crossing did not exist. Hence, it was determined that a crosswalk should be established on the east side of the intersection as well. The initial design of the intersection was modified, as shown in Figure 7-20, to incorporate the necessary changes.

First, the island on the southeast corner of the intersection was redesigned to incorporate the crosswalk. A curb ramp was designed for the northeast corner of the intersection to connect the crosswalk to the sidewalk that would be located on the north side of Crockett Parkway. Pedestrian signals and push buttons were designed to be installed on the planned traffic signal pole installation to be located on the northeast corner of the intersection.

Pedestrian signals also were designed to be installed on the island to be located in the southeast corner of the intersection. However, in order to separate the two pedestrian signals that will be installed in the island, the traffic signal pole had to be relocated eastward from its originally planned location. An additional pedestal pole was designed to be installed on the island to provide support for one of the pedestrian signal and push button installations.

Providing signal "green" time for crossing Crockett Parkway on the east side of the intersection would likely create some disruption for through-movement traffic on Crockett Parkway. Therefore, it was decided to install pedestrian signals and push buttons in the median island. The operation of the traffic signals at the intersection was planned to be programmed to call for "green" time for crossings at the east-side crosswalk only when "called" by pedestrian push buttons, and provided when it would result in minimal disruption to through traffic on Crockett Parkway.

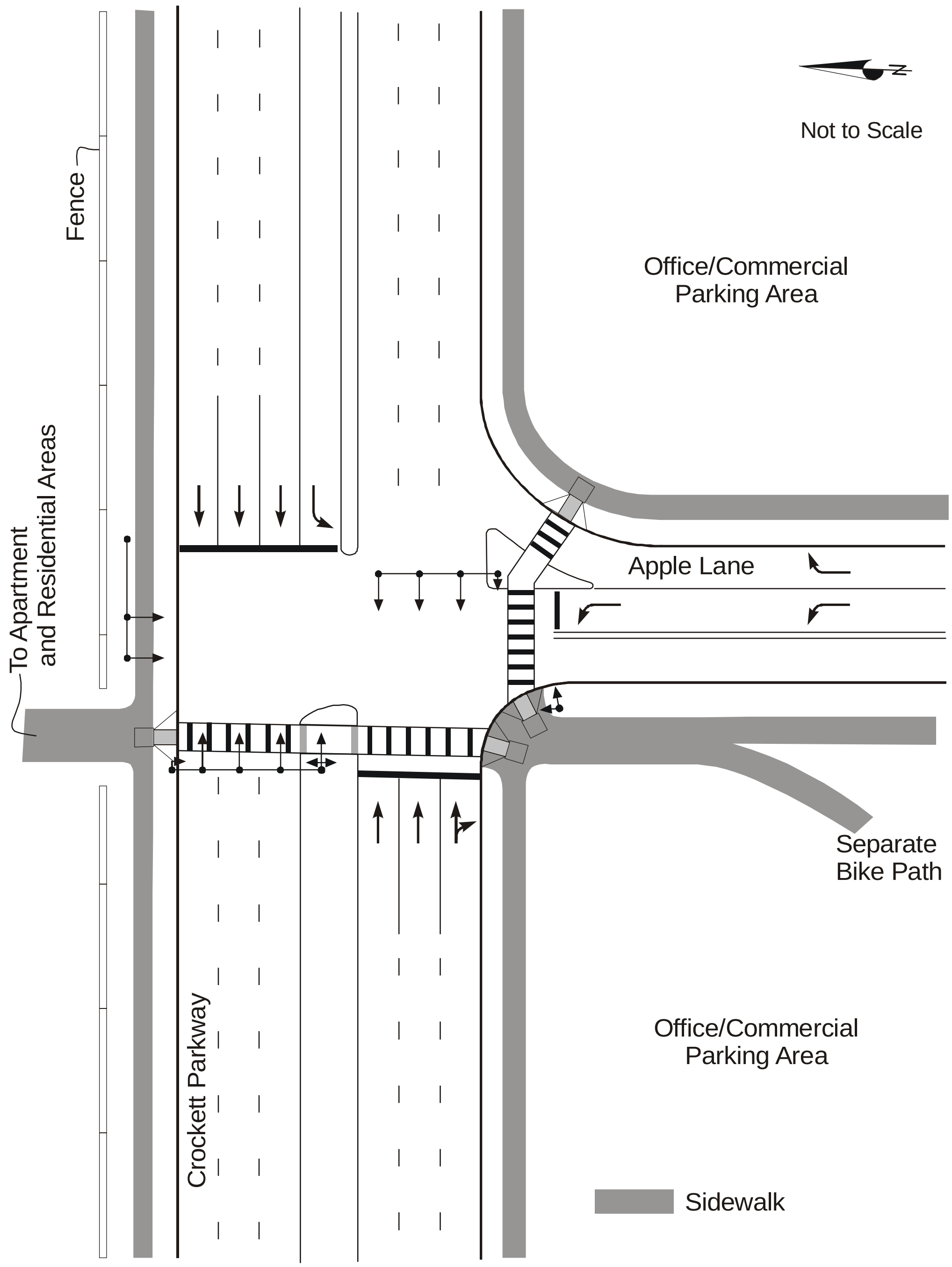


Figure 7-19. Initial Proposed Design.

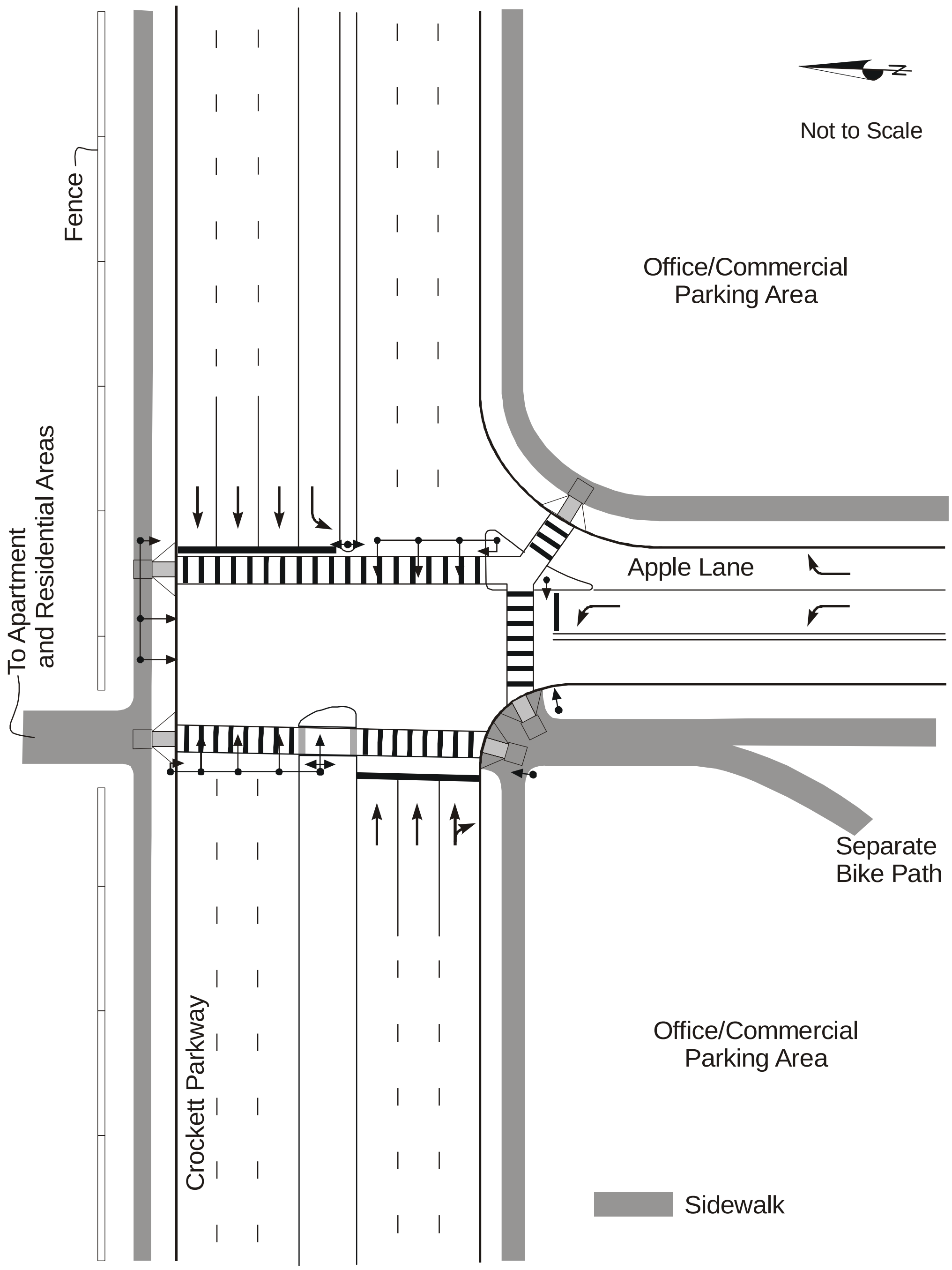


Figure 7-20. Final Proposed Design.

Application 3
Alternative Treatments for Major Street Crossings

Overview

The following are examples of treatments used at major street crossings. These treatments should be judiciously employed because they could lose effectiveness if overused. Chapter 7, Section 2, of the *Urban Intersection Design Guide* <link> presents additional information on crosswalks.

Curb Extension

Curb extensions extend a sidewalk across a parking lane to the edge of the travel lane. They also are called pedestrian bulbs and nubs. Additional information about this type of device is contained in Chapter 5, Section 5 of the *Urban Intersection Design Guide* <link>.

Refuge Medians and Islands

Medians and islands help pedestrians cross streets by providing a refuge area. Additional information is provided in Chapter 4, Section 6 of the *Urban Intersection Design Guide* <link>.

High-Visibility Markings

To heighten driver awareness at uncontrolled crosswalks, high-visibility markings with ladder or "Zebra"-style crosswalk pavement markings have been used. Figure 7-21 is an example of a high-visibility marking. Some agencies use diagonal markings as an alternative treatment. Advantages of the treatment are that it improves the visibility of the crossing from the driver's perspective and for pedestrians with low vision. Disadvantages include increased cost of installation and maintenance (with paint).

Figure 7-21. Example of High-Visibility Markings.

A study of two experimental and two controlled crosswalk locations was conducted in Clearwater, Florida. The devices installed at the experimental sites included overhead illuminated crosswalk signs, high-visibility crosswalk markings, and standard crossing signs. Significant differences were found in driver daytime yielding behavior between the experimental and control locations. Drivers were 30 to 40 percent more likely to yield at the experimental locations during daylight, while there was a small (8 percent) but insignificant increase in driver nighttime yielding behavior. There was also a large increase (35 percent) in the percentage of pedestrians using the crosswalks at the experimental locations. This suggests that pedestrians may feel that a highly visible crosswalk may provide an additional margin of safety and that they went out of their way to use them. Although pedestrians may feel safer, there is no evidence that they are overconfident or overly aggressive in the high-visibility crosswalk.[1]

Crosswalk Signs and Pavement Markings

Signs and other pavement markings have been used in crosswalk areas. A study of three crosswalks with experimental signs reading "LOOK FOR TURNING VEHICLES" and the painted message "WATCH TURNING VEHICLES" reported positive results. (Note: These are experimental signs and are not included in the current version of the *TMUTCD*. If a district wants to use these types of signs, then a request for experimentation to the TxDOT Traffic Operations Division in accordance with the *TMUTCD* is required in order to install the treatment.) The experiment was conducted at three intersections that had a large number of pedestrians and a high daily traffic volume. At the first site, the signs were installed first, and the pavement markings were added later. At the second site, the pavement markings were installed first and the signs were added later. At the third site, the signs and pavement markings were installed at the same time.

All three methods decreased the average number of conflicts for every 100 pedestrians. The number of pedestrians looking for turning vehicles increased for all vehicle types. In the follow-up studies, both the average number of conflicts and the percentage of pedestrians looking for turning vehicles were as good as or had slightly decreased when compared to the results immediately following installation. In either case, the follow-up study[2] showed a significant safety benefit over the before condition.

Figure 7-22 shows an example of a "LOOK BOTH WAYS" pavement marking used to remind pedestrians to look both ways. Again, this marking would be considered experimental in Texas.

[1] Nitzburg, M., and R.L. Knoblauch. "An Evaluation of High-Visibility Crosswalk Treatments-Clearwater, Florida." Report No. FHWA-RD-00-105. August 2001.
[2] Retting, R.A., R. Van Houten, L. Malenfant, J. Van Houten, and C.M. Farmer. "Special Signs and Pavement Markings Improve Pedestrian Safety." *ITE Journal*, Vol. 66, No. 12. December 1996, pp. 28-35.

Figure 7-22. Look Both Ways Pavement Markings.

Advance Placement of Stop and Yield Lines

Advance placement of stop and yield limit lines at uncontrolled crossings is used to encourage drivers to stop a greater distance from the marked crosswalk. Figure 7-23 shows advance yield lines at a midblock crossing. The Yield for Pedestrian sign being used in Shoreline, Washington, is shown in Figure 7-24. Typical applications are locations where drivers are stopping too close to the crossing, especially on multilane approaches. The treatment encourages drivers to stop well in advance of the crosswalk. This helps reduce the potential for pedestrian-related collisions that occur on streets with multiple lanes of traffic when one driver stops to let a pedestrian cross in the crosswalk and the pedestrian is struck by a trailing vehicle in the adjacent lane (see Figure 7-25). With a limit line in advance of the crossing, the trailing vehicle driver is better able to see pedestrians in the crosswalk.

Figure 7-23. Example of Advance Yield Line.

Figure 7-24. Example of Yield for Pedestrian Sign in Washington.

The additional cost for installing and maintaining stop and yield limit lines at a large number of crossings could be significant. If the lines are too far back, visually impaired pedestrians may not hear the sound cues that tell them vehicles have stopped to allow them to cross the street. The lines may reduce availability of on-street parking. Unpublished studies indicate that drivers are less likely to comply with 20-ft [6.1 m] advance stop lines, but compliance is better with 5-ft [1.5 m] advance stop lines. Studies on the effectiveness of stop and yield limit lines in advance of crosswalks on multilane streets found this treatment to be effective; however, the number of sites studied was limited.[3,4]

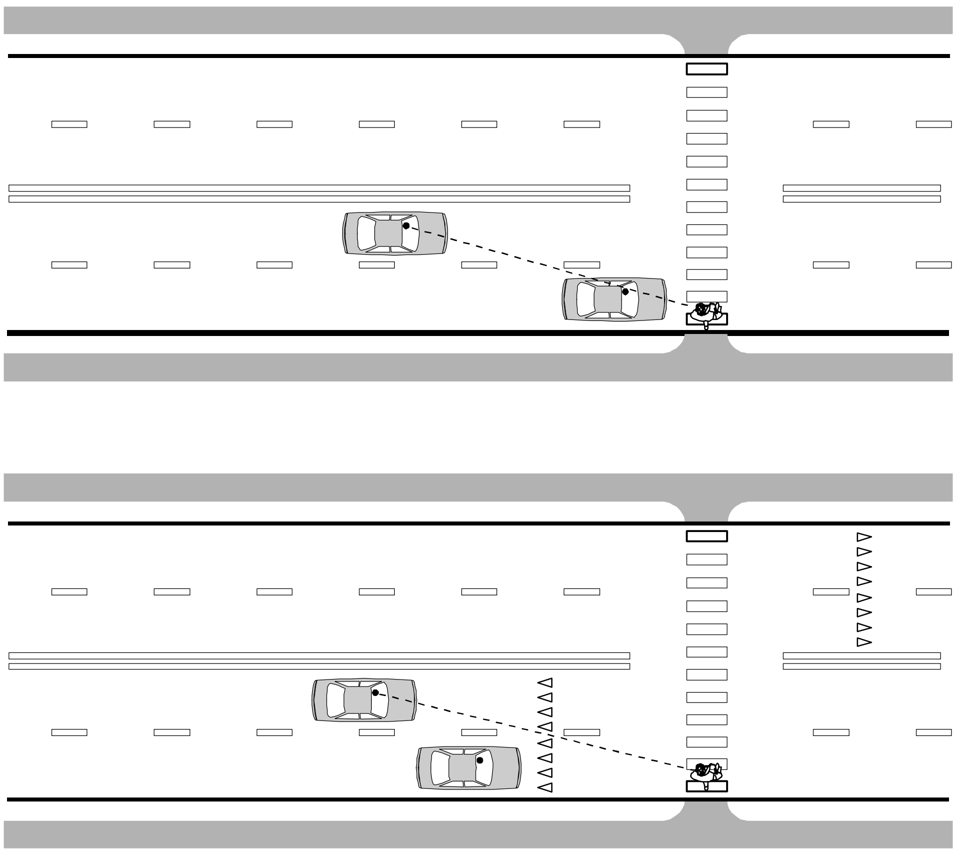


Figure 7-25. Example of Increased Visibility to Pedestrians from Advance Yield Line.

Overhead Signs

Overhead signs at uncontrolled crossings are used to improve visibility of the signs, for example, at locations where the visibility of ground-mounted signs would be limited for drivers in the inner lanes of multilane facilities or where on-street parking might obscure visibility of the signs. The warning signs are installed using span wire or mast arms.

[3] Van Houten, R. "The Effects of Advance STOP Lines and Signs Promote Pedestrian Safety in a Crosswalk on a Multi-lane Highway." *Journal of Applied Behavior Analysis,* 21:245-251, 1988.

[4] Van Houten, R., J.E.L. Malenfant, and D. McCusker. "Advance Yield Markings Reduce Motor Vehicle/Pedestrian Conflicts at Multi-lane Crosswalks with an Uncontrolled Approach." Center for Education and Research in Safety, Halifax Regional Municipality. Presented at the Transportation Research Board Annual Meeting, Washington, D.C., 2000.

Figure 7-26 is an example of an overhead sign.

Figure 7-26. Example of Overhead Pedestrian Sign in Kirkland, Washington.

Pedestrian Railings

Pedestrian railings are used to channelize pedestrians to the safest designated crossing points. Railings typically need to be 4 ft [1.2 m] high to be effective. The cost could be higher if aesthetic enhancements are important. They also are used where there is a need to discourage pedestrians from crossing at locations where complex turning and weaving movements increase the potential for collisions. Figure 7-27 illustrates the use of railings in a median to discourage crossings. Railings need to be highly visible and include a rail that is less than 27 inches [686 mm] above the curb height for detection by pedestrians who are visually impaired and travel with the aid of a white cane.

Disadvantages of railings include:

- may diminish the aesthetic quality of the street environment;
- make pedestrian movement more circuitous, which may encourage pedestrians to walk in the street to circumvent the effectiveness of the railings;
- costs increase if the railing needs to be replaced due to a vehicle striking it;
- considered by some to be anti-pedestrian; and
- can become obstacles to accessing the sidewalk from the street for pedestrians who ignore the railings.

Figure 7-27. Example of a Pedestrian Railing in a Median in Santa Monica, California.

In-Roadway Warning Lights

In-roadway warning lights are used to increase drivers' attentiveness when approaching marked crosswalks occupied by pedestrians at uncontrolled locations. Figure 7-28 shows an installation. Both sides of the crosswalk are lined with durable encased raised pavement markers. Most of the treatments use amber Light Emitting Diode (LED) strobe lighting in the raised pavement markers to alert drivers that they are approaching an occupied crosswalk. However, a few agencies have installed markers without LED strobe lighting. If the markers have lights on both sides, they may be installed on one side of the crosswalk. The LEDs in the raised pavement markers are activated either by push buttons or by automatic detection bollards using infrared sensors that cover the entrance to the crosswalk. Some applications include LED strobe lighting in the pedestrian crossing signs. In-roadway warning lights have been evaluated in numerous studies with varying results. It appears that the effectiveness of this treatment varies widely depending upon the characteristics of the site and existing motorist and pedestrian behavior.

Figure 7-28. Example of In-Roadway Warning Lights.

The ITE Traffic Engineering Council Technical Committee TENC-98-03[5] developed an informational report on in-pavement flashing markers. The report documents the history of how this treatment was developed, the initial test site parameters, signal head illumination and alignment, illumination and flash rate, and various activation methods. The *TMUTCD* provides information on the use of in-roadway warning lights at crosswalks in Section 4L.[6]

An advantage is increased driver awareness of pedestrians in the crossing, especially if the markers and pedestrian warning signs include amber strobe lighting that is activated when a pedestrian is present.

Disadvantages include the following:

- capital cost for installation;
- markers need to be reinstalled when road is resurfaced or undergoes utility repairs;
- may reduce the impact on drivers at crosswalks without this treatment;
- may diminish its effectiveness over time;
- tend to be seen only by the first vehicle in the platoon when there is heavy traffic in the other direction that restricts drivers' views of the entire crossing;
- tend to become shiny with use;
- low sun angles cause markers to be not as apparent to drivers;
- very directional, depending on the manufacturers' specifications;

[5] Traffic Engineering Council Technical Committee TENC-98-03. *In-Pavement Flashing Lights at Crosswalks*. Washington, D.C., ITE. February 2001.
[6] *Texas Manual on Uniform Traffic Control Devices for Streets and Highways*. Texas Department of Transportation. 2003. http://www.dot.state.tx.us/TRF/mutcd.htm. Accessed February 4, 2003.

- rapid degradation of the raised markers, including fogging, caused by harsh on-street conditions;
- lack of long-range visibility;
- could create slipping hazard for bicyclists when wet; and
- some brands of markers more susceptible to snowplow damage than others.

Flashing Beacons

Flashing beacons are used to increase driver attentiveness when approaching marked crosswalks at uncontrolled locations. Figure 7-29 shows a pedestrian crossing at a location with overhead flashing beacons. Flashing amber lights are installed on overhead signs, signs in advance of the crosswalk, or signs located at the entrance to the crosswalk on pedestal poles. Where pedestrian detection is used to activate beacons, it is necessary to install accessible pedestrian signals (APSs) so that visually impaired pedestrians can identify the crossing interval.

Figure 7-29. Example of Overhead Flashing Beacon.

The City of Los Angeles[7] studied two crosswalks across multilane major streets where overhead flashing beacons activated by microwave sensors had been installed. The results of the study indicated that the number of drivers yielding to pedestrians in the crosswalks increased by 10 to 14 percent after installation of the overhead beacons.

Automated Detection

Automated detection devices are used at some locations to activate flashing beacons, in-roadway warning lights, or other active warnings to alert drivers when pedestrians are present. Only activating the device when a pedestrian is present alerts drivers when a pedestrian is crossing the street rather than being a constant warning of a crossing location, thereby improving the effectiveness of the activated beacons. An advantage is that

[7] "Microwave Sensors Show Some Success in Detecting Pedestrians at Crosswalks in L.A." Urban Transportation Monitor, July 9, 1999.

pedestrians do not have to press the button to activate the warning devices. A disadvantage is that false calls have been reported by a number of agencies as a serious problem.

This type of treatment is based on the need to improve the credibility of warning signs and crosswalks. A recent study[8] recommended the following guidelines when using an automated detection system with in-roadway warning lights:

- ♦ If a bollard detection system is used, the bollards should be placed along the same line as each row of the in-roadway warning lights to avoid the possibility of a pedestrian entering a crosswalk between the row of lights but outside the bollards. Figure 7-30 shows a bollard detection system.
- ♦ Video detection was found to be superior to ultrasonic-based systems but still had false and missed activations. Infrared and microwave detection systems need further testing in conjunction with flashing crosswalks.
- ♦ A sign such as Yield for Pedestrians, preferably over the roadway, reminds drivers of their responsibilities. This sign could be retrofitted with lights that flash only in conjunction with the in-roadway warning lights. Alternatively, a beacon that flashes only with the in-roadway warning lights could be mounted below the standard pedestrian crosswalk sign.
- ♦ To improve pedestrian understanding of how the in-roadway warning lights work, custom-made signs directed at pedestrians could be placed on or near the bollards. (Note that if the selected treatment is not included in the *Texas Manual for Uniform Traffic Control Devices* or is not part of TxDOT standards, then a request for experimentation to the TxDOT Traffic Operations Division in accordance with the *TMUTCD* is required in order to install the treatment.) The suggested wording might include FLASHING CROSSWALK – WALK BETWEEN POSTS TO ACTIVATE – WATCH FOR CARS – CROSS ONLY WHEN IT IS SAFE TO DO SO. These messages would need to be audible to be of benefit to an unfamiliar pedestrian who is visually impaired.

[8] Huang, H., R. Hughes, C. Zegeer, and M. Nitzburg. “An Evaluation of the LightGuard™ Pedestrian Crosswalk Warning System.” Florida Department of Transportation, Safety Office, June 1999.

Figure 7-30. Example of Bollard Detection System.

Application 4

Alternative Treatments for Residential Street Crossings

Overview

Example treatments used at residential street crossings follow. Chapter 7, Section 2 of the *Urban Intersection Design Guide* <link> presents additional information on crosswalks.

Raised Crosswalk

A raised crosswalk is typically raised 6 inches [152 mm] above the roadway pavement to an elevation that matches the adjacent sidewalk. This treatment includes a flat area on the top that constitutes the crosswalk. This flat area may be made of asphalt, patterned concrete, or brick pavers. Where raised crosswalks meet the sidewalk, provision of a detectable warning surface with truncated domes is required to mark the street interface for pedestrians with visual impairments. Figure 7-31 shows an example of a raised crosswalk.

Figure 7-31. Example of Raised Crosswalk in Portland, Oregon.[9]

The objective of this treatment is to control traffic speeds approaching and then traversing the crosswalk to improve the safety of

pedestrians using the crosswalk. Advantages for this treatment include reducing traffic speeds at the crosswalk and providing an easier crossing for pedestrians. It also focuses the pedestrian crossing activity in the desired location. A disadvantage is the cost of

[9] Pedestrian and Bicycle Information Center. Image Library. http://www.pedbikeimages.org/index.cfm. Accessed November 25, 2003.

installation.[10] How the raised crosswalk affects drainage needs to be examined in the design phase to avoid undesirable ponding or flooding.

Drivers and passengers with disabilities have expressed concerns about the level of pain experienced by persons with spinal injuries when crossing vertical deflections in the roadway.

A recent study showed that the percentage of drivers yielding to pedestrians increased from an average of 15 percent before installation to 55 percent after the raised crosswalk was installed.[11] The 85th percentile speed declined from 31 mph [50 km/h] in the before period to 25 mph [40 km/h] in the after period. A report by ITE also evaluated the benefits of raised crosswalks and found speeds decreased from 37 to 35 mph [60 to 56 km/h] at one site and from 30 to 21 mph [48 to 34 km/h] at another site.[12]

Entry Treatments

Entrance treatments are used to create a sense of community or neighborhood identity. Entrance features may consist of textured and colored pavements, curb extensions, raised crosswalks or speed tables, landscaping, and entry signage at key entryways into neighborhoods or small towns. Entrance treatments create visual and/or audible cues to tell drivers that they are entering a local residential area or that the surrounding land uses are changing.

The advantage of entry treatments is that they reduce traffic speeds as vehicles enter the residential street. A disadvantage is the cost of installation. However, there are minimal cost implications if this design is incorporated as streets are being constructed.

Raised Intersections

Raised intersections are constructed as raised platforms 6 inches [152 mm] above the approaches to the intersection (see Figure 7-32). They frequently use asphalt and patterned concrete or brick pavers. The height matches the elevation of the adjacent sidewalk. Where raised intersections meet the sidewalk, provision of a detectable warning surface with truncated domes is required to mark the street interface for pedestrians with visual impairments. Accommodating the flow of stormwater near the raised intersection, especially when retrofitting a raised intersection, is needed to avoid ponding or flooding.

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[10] *Alternative Treatments for At-Grade Pedestrian Crossings*. An ITE Informational Report by Nazir Lalani and the ITE Pedestrian and Bicycle Task Force, Institute of Transportation Engineers, 2001.

[11] Watkins, K. "Cambridge's Traffic Calming Program – Pedestrians Are the Focus." Compendium of Papers, ITE Annual Meeting in Nashville, TN, 2000.

[12] Ewing, R. *Traffic Calming, State of the Practice*. Report No. FHWD-RD-135. Washington, D.C.: ITE under contract with US DOT, FHWA, 1999.

Figure 7-32. Raised Intersection in Portland.[9]

The objective is to slow vehicles as they approach the intersection and traverse the raised intersection, thereby improving the safety of pedestrians crossing at the intersection. Disadvantages include:

- cost of installation (however, it is minimal if the design is incorporated as streets are being constructed),
- vehicles can mount the sidewalk more easily, and
- pain experienced by those with spinal injuries (drivers and passengers with disabilities have expressed concerns about the level of pain experienced by persons with spinal injuries when crossing vertical deflections in the roadway).

Traffic Calming

Several documents are available with information on traffic calming measures used in residential areas including the following:

- ITE, Traffic Calming, State of the Practice.[12]
- ITE Web site (http://www.ite.org) in the section on traffic calming.
- TxDOT, Handbook of Speed Management Techniques.[13]

[13] Parham, A.H., and Fitzpatrick, K. *Handbook of Speed Management Techniques.* FHWA/TX-99/1770-2. Texas Transportation Institute. College Station, TX, 1998.

- ITE, Alternative Treatments for At-Grade Pedestrian Crossings.[10]

Application 5

Alternative Signal Control at Crossings

Overview

This section presents information on alternative signals that have been installed for pedestrian crossings. The discussions focus on types of signals generally not located at intersections or whose appearance or operations are significantly different from typical pedestrian crossings at signalized intersections. Chapter 7, Section 2 of the *Urban Intersection Design Guide* <link> presents additional information on crosswalks. Chapter 8 <link> provides general information on signals including pedestrian signals.

Midblock Signal

Examples of a midblock signal are shown in Figure 7-33 from University Way, Washington, and Figure 7-34 from Los Angeles, California. The city of Los Angeles has used midblock signals at 105 locations in the downtown and other retail areas. The treatment provides pedestrians an opportunity to cross midblock at a controlled crosswalk. The city used the pedestrian warrant contained in the California *Traffic Manual* to determine treatment locations, along with consideration of intense retail activity, high pedestrian volumes, midblock crossing demand, the presence of existing signals at the end of the subject block, and block length greater than 600 ft [183 m].[10]

Figure 7-33. Example of a Midblock Signal in Washington.

Figure 7-34. Example of a Midblock Signal in Los Angeles.

During the WALK interval, a steady red signal indication is displayed to drivers approaching the crosswalk. During the flashing DON'T WALK interval, drivers see a flashing red indication and, after stopping, they may proceed through the crosswalk area in front of them if it is not occupied by pedestrians. After the pedestrian clearance interval ends, the signal turns green to allow drivers to proceed. The flashing red minimizes the interruption to traffic progression. Vehicles remain stopped during the 4- to 7-second WALK interval. However, they are not required to wait the full 12 to 20 seconds that would be necessary if a steady red indication were displayed during the completion of the DON'T WALK clearance interval. A variation is to have drivers see a steady red indication during the DON'T WALK interval. Drivers may not proceed through the crosswalk area in front of them until the signal turns green. Signals remain green for drivers until a pedestrian reactivates the push button.

Split Midblock Signal

Tucson, Arizona, also uses midblock signals. They have an example where the pedestrian would cross the street in two stages:

- first to a median island and proceed along the median to a second signalized crossing point a short distance away, and
- then the pedestrian activates a second crossing button, and another crossing signal changes to red for the traffic, giving the pedestrian a WALK signal.

Figure 7-35 shows a close-up of the railing at a split midblock signal in Tucson. Figure 7-36 shows the split midblock signal treatment being used in Bellevue, Washington.

Figure 7-35. Example of a Split Midblock Signal in Tucson.

Figure 7-36. Split Midblock Signal Treatment in Bellevue.

The two crossings operate independently of each other, which allows signal operation to better fit into the major street traffic progression for each direction. This reduces the potential for:

- stops,

- delays,
- crashes, and
- environmental air-quality issues.

Intersection Pedestrian Signals (Half Signals)

Intersection pedestrian signals (also know as half signals) are used to provide signal control for a pedestrian crossing the major street while minimizing delay for major street traffic by retaining Stop sign control on the minor street. This treatment has been used at locations where there is heavy pedestrian demand to cross the major street, but the side street traffic on the minor approach is light. The lack of signal control on the side street does not attract more traffic to the street as conventional intersection signals would.

The cost of installation is significant. Drivers on side streets may be confused about right-of way assignment: the right of way relies on gaps in main street traffic to enter or cross the main street.

This treatment has been tested in several cities including Tucson, Arizona; Portland, Oregon; and Fairfax, Virginia. This device is not included in the current version of the *TMUTCD*. If a district wants to use this device, then a request for experimentation to the TxDOT Traffic Operations Division in accordance with the *TMUTCD* is required in order to install the treatment. Figure 7-37 shows an example of a half signal in Seattle.

Figure 7-37. Example of a Half Signal in Seattle, Washington.

Hawk Crossings

The objective of a Hawk (high-intensity activated crosswalk) crossing is to stop vehicles to allow pedestrians to cross while also allowing vehicles to proceed as soon as the pedestrians have passed. It is a combination of a beacon flasher and a traffic control signaling technique for marked crossings. The unit is normally off until activated by a pedestrian. The signal begins with a flashing yellow indication to warn approaching drivers, just like a school bus signal. The flashing yellow is then followed by a solid yellow indication advising the drivers of the requirement to prepare to stop. The signal is then changed to a solid red indication during the pedestrian interval, when drivers must stop at the crosswalk. The beacon signal then converts to an alternating flashing red, allowing drivers to proceed when safe.

This application provides a pedestrian crossing without signal control for the side street. This treatment is currently used in Tucson, Arizona, and their guidelines are summarized elsewhere.[14] An evaluation at one site found that motorists yielding to pedestrians increased from 31 to 93 percent. Figure 7-38 shows a Hawk signal. This device is not included in the current version of the *TMUTCD*. If a district wants to use this device, then a request for experimentation to the TxDOT Traffic Operations Division in accordance with the *TMUTCD* is required in order to install the treatment.

The advantages include:

- ♦ drivers are likely to stop for a form of traffic control resembling a traffic signal,
- ♦ minimized delay for major street traffic, and
- ♦ additional vehicular traffic is not attracted to the side street, which may be residential.

Disadvantages include:

- ♦ it may require driver education,
- ♦ drivers have a tendency to remain stopped when it is safe to proceed, and
- ♦ it may be confusing to have a dark signal display, which may convey a power outage to some drivers.

The Hawk is currently not included in the *MUTCD* or *TMUTCD* as a pedestrian crossing treatment. The device has similarities to an emergency-vehicle traffic control signal.

[14] Glock, J.W., R.B. Nassi, R.E. Hunt, and B.W. Fairfax. "Implementation of a Program to Reduce Pedestrian-Related Accidents and Facilitate Pedestrian Crossings." Paper submitted for the Pedestrian Project Award, Partnership for a Walkable America and ITE, August 2000.

Figure 7-38. Example of a Hawk Signal in Tucson, Arizona.

Application 6

Alternative Treatments for Signalized Intersections

Overview

Following are signalized intersection treatments that have the intention of making street crossings more pedestrian friendly. Some treatments, such as high-visibility markings, curb ramps, signs, refuge islands, and pavement legends, are also used at uncontrolled crossings and are discussed in Chapter 7, Application 3 <link>. Chapter 7 of the *Urban Intersection Design Guide* <link> presents additional information on crosswalks. Information on traffic signals for pedestrians is in the *Urban Intersection Design Guide*, Chapter 8, Section 5 <link>. As with all treatments, the *TMUTCD* should be consulted for guidance on selecting appropriate devices.

Treatments

The following treatments have been implemented to improve the pedestrian crossing at a signalized intersection.[10]

Leading Pedestrian Signal Intervals permit pedestrians to begin crossing several seconds before the release of potentially conflicting motor vehicles at signalized intersections. Equipment or new timing is installed at signalized intersections to release pedestrian traffic 3 seconds in advance of turning vehicles for signals with protected left-turn movements. The WALK indication or walking person symbol is displayed 3 seconds in advance of the green signal indication for vehicles. Accessible Pedestrian Signal would be needed to inform visually impaired pedestrians that the walk signal is shown.

Lagging Pedestrian Signal Intervals delay pedestrian crossing several seconds until after the release of potentially conflicting motor vehicles at signalized intersections.

Educational Signs for Pedestrian Signal Indications are used to improve the understanding of pedestrian signal indications at signalized intersections (see Figure 7-39). They are installed above pedestrian push buttons or integrated into the push button housing.

Figure 7-39. Examples of Educational Signs.

Advance Stop Lines at Signalized Intersections encourage drivers to stop a greater distance from the marked crosswalk. They reduce the potential for pedestrian-related collisions on four-lane streets that are caused when a driver stops to let a pedestrian cross who is then struck by a trailing vehicle in the adjacent lane (see Figure 7-25). Standard white stop lines are placed preferably 5 ft [1.5 m] to 20 ft [6 m] in advance of marked crosswalks at signalized intersections. The signal head visibility needs to be checked when moving the stop line upstream from the crosswalk.

Pedestrian Railings at Signalized Intersections channelize pedestrians to the safest designated crossing points. A typical description of the treatment is to use 4-ft-high [1.2 m] railings. The split midblock signal uses pedestrian railings as part of the treatment. Figure 7-35 and Figure 7-36 are photographs of installations.

Scramble Patterns at Signalized Intersections enable pedestrians to cross in all directions at an intersection, including diagonally rather than having to cross two legs of intersection. This reduces the distance pedestrians have to walk and reduces delays for pedestrians. There are three pedestrian indications at each corner, one each for the typical crosswalks and one for the diagonal crosswalk. During the time period when the diagonal crosswalk pedestrian indication permits pedestrians to cross, the vehicle indications display red on all approaches of the intersection.

Push Button Treatments at Signalized Intersections provide additional information to pedestrians. One example of this treatment incorporates a pedestrian acknowledgement device using a high-intensity LED indicator.

Automated Detection at Signalized Intersections detects pedestrians and/or eliminates unnecessary calls if the pedestrian leaves the area. This treatment is used also to extend pedestrian intervals if pedestrians are detected in the crosswalk. Pedestrians are detected at the curbside of and/or in a pedestrian crossing by means other than those requiring a physical response by pedestrians. Most applications use either infrared or microwave technology.

Wheelchair Detection at Signalized Intersections activates treatments specifically needed to assist pedestrians in wheelchairs to cross at a pedestrian crossing. In-pavement loop detectors are used to detect wheelchairs and to activate pedestrian crossing indications.

Accessible Pedestrian Signals at Signalized Intersections provide information to pedestrians who are visually impaired that is comparable to the visual information that is available. Under the *Draft Guidelines for Accessible Public Rights-of-Way,*[15] APS installation would be required at:

- new signalized intersections that have actuated pedestrian signals,
- intersections that lack the cues needed by people with visual disabilities, and
- intersections that are undergoing signal upgrades.

[15] U.S. Access Board. Draft Guidelines for Accessible Public Rights-of-Way. June 17, 2002. http://www.access-board.gov/rowdraft.htm. Accessed September 2003.

Additional information on accessible pedestrian signals is in Chapter 8, Section 5 of the *Guide* <link>.

Countdown Pedestrian Signal Indications at Signalized Intersections provide information to enable pedestrians to make better decisions about when to enter the crosswalk. Countdown signals are used in conjunction with conventional pedestrian signals to provide information to pedestrians regarding the amount of time remaining to safely cross the intersection. The countdown timer starts either at the beginning of the pedestrian phase or at the onset of the pedestrian clearance interval. At the end of the pedestrian clearance interval, the countdown device displays a zero and the DON'T WALK indication appears. Figure 7-40 shows examples of countdown devices in Florida.

Figure 7-40. Examples of a Countdown Indication.

Animated Eye Pedestrian Signal Displays at Signalized Intersections encourage pedestrians to look for turning vehicles traveling on an intersecting path by including a prompt as part of the pedestrian signal display. An animated eye display uses an LED pedestrian signal head and adds animated eyes that scan from side to side. The device uses a narrow (8-deg) field of view LED on a black background. The display is highly visible to pedestrians while limiting pedestrian signal displays to drivers. The blue LEDs display two blue eyes with blue eyeballs that appear to scan from left to right at the rate of one cycle per second.

Curb Extensions at Signalized Intersections reduce pedestrian exposure to vehicular traffic and the potential of being struck. The sidewalk extends across the parking lanes to the edge of the travel lanes to narrow the distance of the road that pedestrians cross. Curb extensions also improve the visibility of pedestrians waiting to cross by:

- bringing them closer to the center of the driver's cone of vision, and

- minimizing the impact of parked vehicles on driver's ability to see pedestrians waiting to cross.

Overhead Signs at Signalized Intersections make drivers more aware of the presence of pedestrians at specific locations. Overhead signs on mast arms alert drivers to the presence of pedestrians at signalized crossings. Figure 7-41 shows one style of a pedestrian crossing sign on a mast arm.

Figure 7-41. Example of Pedestrian Crossing Sign on Mast Arm in Tucson, Arizona.

Warnings of Turning Vehicles at Signalized Intersections encourage pedestrians to watch for through traffic and turning vehicles. Word legends, signs, and auditory devices are placed at each end of the crosswalk so that they are legible to pedestrians waiting to cross.

Turn Prohibitions at Signalized Intersections improve the pedestrian environment by reducing conflicts between turning vehicles and pedestrians crossing the street in the crosswalk at signalized intersections. Signs are placed prohibiting right turns on red at signalized intersections. (On one-way streets, this could involve prohibiting left turns on red.)

Perpendicular Crossings at Skewed Intersections shorten pedestrian crossing distances and reduce pedestrian clearance signal timing. Transverse pedestrian crossing markings are placed perpendicular to the road to be crossed instead of parallel to the skewed intersecting road. This treatment can be accompanied by textured or decorative pavements to further delineate the crossing location.

Application 7

Alternative Treatments for School-Related Crossings

Overview

The majority of motorists do not reduce vehicular speed in school zones unless they perceive a potential risk such as the presence of police or crossing guards, or they observe that children are present. Chapter 7, Section 2, of the *Urban Intersection Design Guide* <link> presents additional information on crosswalks.

Recommended Guidelines for School Trips and Operations

The Institute of Transportation Engineers developed a recommended practice on the selection of safe walking trip routes to school and traffic control measures called *School Trip Safety Program Guidelines*.[16] TxDOT developed materials that can assist with developing designs and selecting treatments at and near schools.[17]

Treatments

Following are example treatments used near schools identified in *Alternative Treatments for At-Grade Pedestrian Crossings*.[10] If a district wants to use experimental signs or pavement markings, then a request for experimentation to the TxDOT Traffic Operations Division in accordance with the *TMUTCD* is required in order to install the treatment.

Portable Signs are placed in the school crosswalk during school hours (see Figure 7-42). Some require drivers to stop when children are in the crosswalk. They also are used in advance of the crosswalk to notify drivers to slow to a specific speed (e.g., 15 mph [24 km/h]).

[16] *School Trip Safety Program Guidelines*. Recommended Practice, ITE Technical Committee 4A-1. Institute of Transportation Engineers. 1984.

[17] Cooner, S.A., K. Fitzpatrick, M.D. Wooldridge, and G.L. Ford. Traffic Operations and Safety at Schools: Recommended Guidelines. FHWA/TX-03/4286-2. Texas Transportation Institute. College Station, Texas, October 2003.

Figure 7-42. Portable Sign.

Overhead School Signs are installed on signal mast arms to warn drivers of the presence of school-age pedestrians crossing at the signal (see Figure 7-43). Overhead placement of the signs increases their visibility, but the overhead treatment is more difficult and costly to install and maintain. Wind loading on the span wire or mast arm is increased, which may necessitate a stronger and, therefore, more expensive design.

Figure 7-43. Example of Overhead School Signs.

Fluorescent Yellow-Green Signs with Flashing Beacons are installed at crosswalks used by school-age pedestrians to warn drivers of the presence of school-age pedestrians. Some applications use flashers that are active for a fixed, predetermined period of time. Other applications use sensors to activate flashers when pedestrians are detected or use pagers to activate the flashers from a remote location. The exact costs depend on whether sensors are included in the treatment. An advantage is that the combination of flashing beacon and fluorescent yellow-green signs has the potential to attract drivers' attention to the pedestrian crossing and the presence of school-age pedestrians. Disadvantages include that overuse of these types of devices may erode their effectiveness, and energy and ongoing maintenance costs can be significant.

Part-Time Street Closures can be used to create a pedestrian-only environment for part of the day. Gates can be used to close streets to traffic for part of the day during school hours, e.g., from 6:00 AM to 6:00 PM (see Figure 7-44). Several agencies have used part-time closures to deal with high levels of pedestrian activity near schools, to promote safety, and to deter gang violence. The treatment creates a unified school campus by creating a pedestrian mall on the section of the street that is closed to traffic. During closure hours, this may cause major detouring of traffic onto parallel streets, some of which could be residential. A study of a part-time closure of a street bisecting a high school campus in Ventura, California, showed that 6000 of the 8000 vehicles using the street on weekdays during the closure hours diverted to a parallel major street.[10] Volumes rose from 14,000 vehicles per day to over 20,000 on the parallel major street, two blocks away. The remaining 2000 vehicles used residential streets to bypass the closure, causing these streets to be closed part-time for several years until the residents felt that drivers had become used to using the parallel major street.

Figure 7-44. Example of Sign Used to Close Road During School Hours.

Portable Orange Barrels or large cones with reflector strips are placed in the school crosswalk during school hours to encourage drivers to slow down. Portable barrels in the school crosswalk attract drivers' attention to the crossing when children are crossing. Deploying the barrels each day takes time. Barrels are also prone to being hit or suffering weather damage, thereby requiring periodic replacement.

Chapter 8
Signals

Contents:

Application 1
Signal Visibility

Overview

The *Texas Manual on Uniform Traffic Control Devices* (*TMUTCD*) contains numerous standards and guidelines relative to the installation of traffic signals. The *Urban Intersection Design Guide*, Chapter 8 <link> includes discussions on intersection design considerations for signals. One of the design-related issues is adequate sight distance. A driver approaching a signalized intersection must be able to see the traffic signals at a specific distance from the appropriate stop position (or stop line if provided) so that the driver has adequate time and distance available to bring the vehicle to a stop prior to reaching the appropriate stop position. These sight distances vary according to operating speed. As operating speeds increase, the required sight distances increase.

Sight distance restrictions are caused by topographic conditions, vegetation (large trees), or man-made structures, like buildings or highway overpasses/underpasses. Most often, sight distance restrictions result from installing traffic signals at intersections that are located “on the other side of the hill” or “around the curve.”

Treatments

The *TMUTCD* suggests that a warning sign (SIGNAL AHEAD sign, W3-3) may be installed in advance of a signalized intersection when the recommended sight distance is not available. A warning beacon may be used as a supplement to draw attention to this sign. Also, a BE PREPARED TO STOP sign or a “Be Prepared to Stop When Flashing” sign may be installed as a supplement to the SIGNAL AHEAD sign if desired.

TxDOT has recently installed “Be Prepared to Stop When Flashing” signs at two locations; Waco and Brenham in Texas. Information about design, installation, and operation of these signs is available in the following documents.

- Messer, C.J., S.R. Sunkari, H.A. Charara, and R.T. Parker. *Design and Installation Guidelines for Advance Warning Systems for End-of-Green Phase at High Speed Traffic Signals*. Texas Transportation Research Report 4260-2, September 2003.
- Sunkari, S.R., C.J. Messer, H.A. Charara, and R.T. Parker. *Signal Technician’s Installation and Maintenance Manual for Advance Warning for End-of-Green Phase at High Speed Traffic Signals*. Texas Transportation Research Report 4260-3, September 2003.
- Messer, C.J., S.R. Sunkari, H.A. Charara, and R.T. Parker. *Development of Advance Warning Systems for End-of-Green Phase at High Speed Traffic Signals*. Texas Transportation Research Report 4260-4, September 2003.

Depending on the conditions that exist at and in advance of the signalized intersection that has the sight restriction, a supplemental traffic signal(s) may be installed to provide the motorist with an advance indication of the signals that control the approach. Such supplemental signals cannot be installed at all locations where sight restrictions exist.

The *TMUTCD* identifies specific requirements for traffic signal installations, including a minimum number of signal faces and positioning those signal faces within an appropriate cone of vision from the appropriate stopping location. Hence, most traffic signals are installed on the far side of the intersection and essentially “in line” with the approach lanes. When the far-side signals cannot be seen by motorists until they are close to the intersection, supplemental signals may be installed on the near side of the intersection to provide an advance notice of the signal indications that exist at the intersection. There is a limitation to where these signals should be installed. It would not be appropriate to install supplemental traffic signals a significant distance upstream of the intersectional approach. The most appropriate location for supplemental signals would be on one of the traffic signal poles installed at the intersection, on span wires supporting other signals at the intersection, or on other poles (utility or luminaire poles, for example) that exist at the intersection.

Because these supplemental signals are installed at locations that are not considered “typical,” caution must be exercised. Supplemental signals should not be installed if they may create possible confusion to motorists or pedestrians. Depending on where the supplemental signals are installed and “aimed,” motorists on conflicting approaches or pedestrians at the intersection may misinterpret the signal’s function and believe the signal indication is applicable to their desired movement. Hence, it is possible that a conflict may be created if the supplemental signals are not installed properly. This issue could be addressed by installing signals with louvered or programmable lenses so that the supplemental signal faces can be seen only by motorists approaching the intersection who are controlled by that signal’s indications.

It is not possible to provide any additional guidelines for the selection and installation of supplemental traffic signals. Obviously, supplemental signals should be considered at any intersection where sight distances to traffic signals are less than what is suggested by the *TMUTCD*. However, each potential location for supplemental signals is site specific and must be analyzed individually to determine if these signals can be installed without creating undesirable confusion.

Example Locations

Two signalized intersections having supplemental traffic signals were identified as example locations. The first example is located along U.S. Highway 290 east of Austin. The signalized intersection is located near the bottom of a crest vertical curve so approaching motorists on the high-speed U.S. Highway 290 approach had difficulty seeing the traffic signals installed at the intersection (located on span wires). An additional traffic signal was installed at a high elevation on the nearside, median steel strain pole supporting the signal span wires. This installation, shown in Figure 8-1, can be seen further upstream as needed in advance of the intersection.

The second example is located on Villa Maria Drive, an arterial street in Bryan, Texas. A horizontal curve is located in advance of the intersection, and an approaching driver cannot see the signals installed at the intersection, located around the curve to the right. The installation of a supplemental traffic signal, installed on the span wire located on the near side of the intersection and illustrated in Figure 8-2, provides the advance signal indication for the approaching motorist.

(A) Approach to Traffic Signal at Intersection of U.S. 290 and FM 3177 (Decker Lane).

(B) Traffic Signal More Visible Beyond Crest.
Figure 8-1. Traffic Signal Just Visible Beyond Crest of Hill.

Figure 8-2. Supplemental Signal Due to Horizontal Curve at Villa Maria and Cavitt Ave.

Application 2
Traffic Signal Design

Overview

The *Urban Intersection Design Guide,* Chapter 8, Section 3 <link> includes information on signal support systems.

Background

A city of about 150,000 planned the widening of a major arterial street, Lafayette Boulevard, from its four-lane, divided cross section to a six-lane, divided cross section. Several intersections along the arterial were signalized and would remain signalized. Each of these intersections required signal design modifications. The city engineering staff reviewed the various potential signal layouts that were available for consideration.

Basically, there were five possible signal layouts that were considered:

- span wire configuration with wooden poles and guy wires or with steel strain poles,
- steel poles with signals mounted overhead on cantilever arms,
- steel poles with signals pole-mounted,
- combination of steel poles with signals placed overhead on cantilever arms and side-mounted on poles, and
- signals placed overhead on an ornamental structure that spans over the travel lanes and has supports on both sides.

The city council had already established desired design features for signal hardware, which immediately eliminated any consideration for span wire configurations. While this type of design would be considered the most efficient from a construction cost viewpoint, the span wire layout was not considered aesthetically pleasing. The remaining four layouts were considered for the final traffic signal design.

The cross-sectional design of the six-lane, divided arterial street also was established. The arterial would have raised curbs and gutters, and a raised median. The travel lanes would be 14 ft [4.3 m] in width for the outside travel lanes, and 12 ft [3.7 m] in width for the center and left travel lanes. The median would be 18 ft [5.5 m] in width (face-of-curb to face-of-curb). Separated left-turn lanes, 12 ft [3.7 m] in width, would be provided within the median as necessary. City engineers determined that pedestrian volumes along and across Lafayette Boulevard were minimal. Therefore, it was decided that intersections along Lafayette Boulevard would be designed primarily for vehicular traffic, but pedestrians would be accommodated in the design features. To accommodate right turns at intersections, a minimum return radius of 35 ft [10.7 m] was established, although 50-ft [15.2 m] radii were preferred. A longer radius accommodates large trucks making right turns and provides space for larger islands that provide storage area for pedestrians.

The typical signal design used in the city consisted of signal poles with cantilever arms. In the downtown area of the city, some of the existing traffic signal installations have pole-mounted signals. The city staff decided that the signal design for the widened arterial ideally should remain consistent with existing installations, if possible. Also, pedestrians were to be accommodated in the design, although large pedestrian volumes were not expected at any of the arterial intersections.

Issues Considered

The geometric design requirements for the widened arterial street created very large intersections, which, in turn, created some difficulty with selecting the preferred design for the traffic signal installation. The wide and multiple travel lanes made the pole-mounted signal option inadequate because driver cone of vision requirements could not be provided unless the signal heads were mounted in the median. The city had removed older fixed signal pole locations in median areas years before as a safety benefit, and the city staff did not want to re-install signal poles in medians (except for breakaway pedestal poles for pedestrian signals). Therefore, pole-mounted signals alone would not be considered as an appropriate signal design layout.

The decision to place signals overhead in an ornamental structure spanning over the travel lanes was given special consideration. Such a design was selected for the downtown area signal installations in the city's master plan for a renovated downtown area. However, spanning over the proposed new Lafayette Boulevard was not considered applicable. Such a structure would be difficult to design considering the long span length, and its presence would make it difficult to accommodate the occasional oversized, high load. Spanning over half the roadway and having a support structure in the median would be feasible; however, the city did not want to have a rigid fixed object in the median area. Therefore, installing signals on an ornamental overhead structure was eliminated from consideration.

The only possible signal layout design remaining was steel poles with cantilever arms, with or without pole-mounted signals. The city staff had experience with cantilever arm lengths designs in the past and knew that keeping mast arm lengths at or less than 45 ft [13.7 m] was desirable. If longer mast arms were required, then signal poles needed to support those longer mast arms had to be larger (and more expensive) than normal, and the foundations for poles supporting longer mast arms were considerably larger, deeper, and more expensive. Hence, a preferred design was to keep mast arm lengths as short as possible.

Accommodating pedestrians at these signalized intersections also created some operational and design concerns. The wide, multiple travel lanes and long return radii (50 ft [15.3]) created lengthy walk distances for pedestrians. The initial signal pole layout at a proposed intersection (with Laddie Lane) to accommodate both signals for vehicular traffic and pedestrian signals and push buttons resulted in the need for very long mast arms, about 10 ft [3.0 m] longer than the desired 45-ft [13.7 m] minimum mast arm length. (See Figure 8-3.) Signal phase lengths would require relatively short clearance times for pedestrian clearances with this design, assuming that pedestrians would cross Lafayette Boulevard and store in the median.

The length of signal mast arms could be reduced by moving the signal poles further from the intersection so that they could be placed closer to the outside travel line. Pedestrian crossing times could be lessened by reducing the length of the crossings by moving the crosswalks further from the intersection. Providing these two changes, plus constructing two pedestrian ramps on each corner in line with the crosswalks, was another design option considered. However, these changes would have required motorists to stop their vehicles much further from the intersection, affected right-turn-on-red movements (because motorists would have to actually travel across the crosswalk to reach the intersection), and required additional time for signal clearances. In addition, the traffic signal heads would have been located beyond the 180-ft maximum distance from the stop line (on the Laddie Lane approaches) permitted by the *Texas Manual on Uniform Traffic Control Devices*. Hence, this alternative design was not selected.

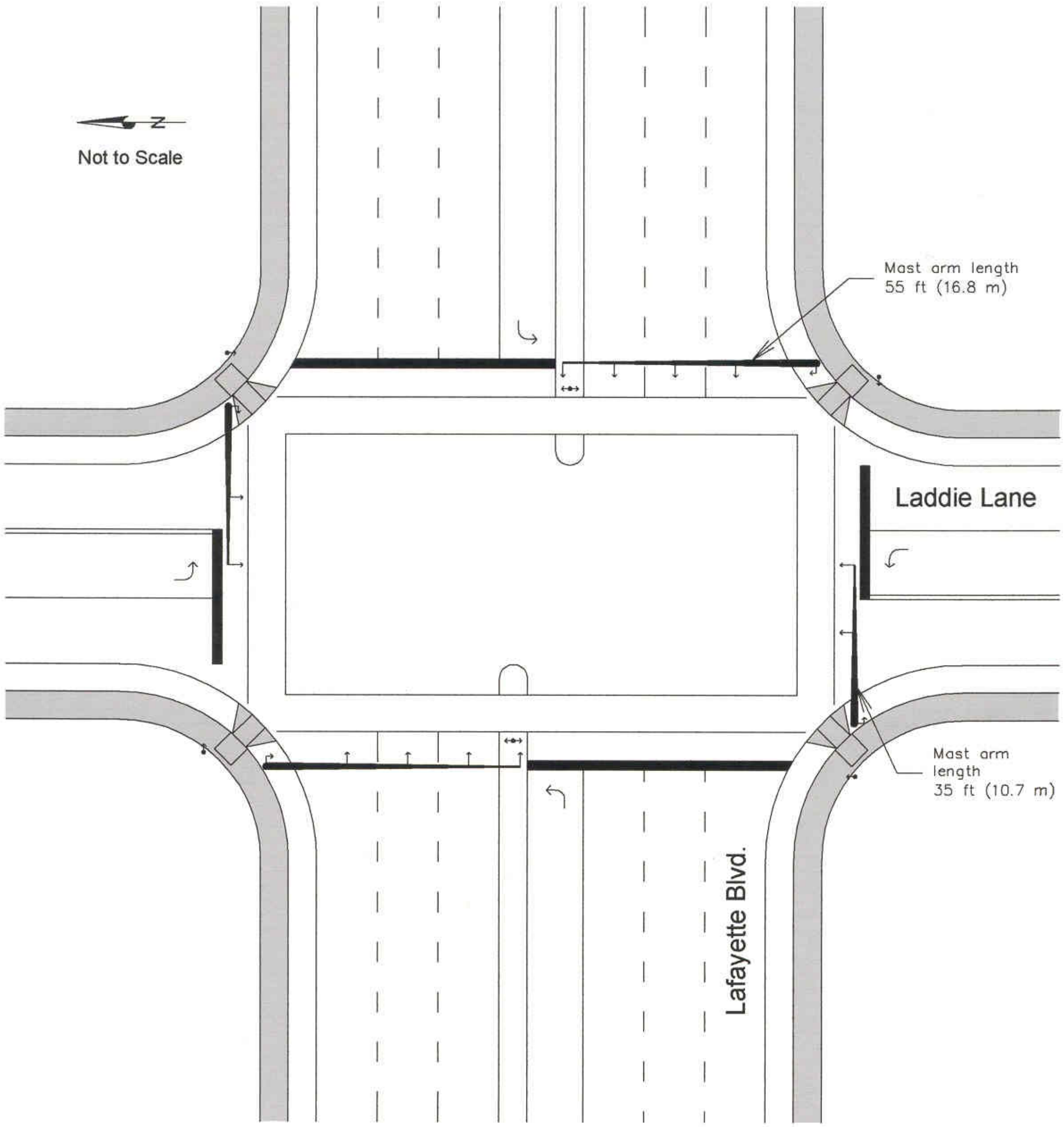

Figure 8-3. Initial Proposed Design.

A recommendation was made to alter the initial intersection design to include islands at the corners of the intersections. The addition of the islands, as shown in Figure 8-4, provides the opportunity to keep mast arm lengths below the 45-ft [13.7 m] maximum, and reduce pedestrian walk and clearance times. At the same time, the proposed design incorporated the desired lane widths and 50-ft [15.3] return radii. Channelizing islands had to be designed with a cut-through because there was insufficient space for curb ramps. Pedestrian push buttons had to be accessible from the cut-through islands, so additional pedestal poles were necessary for push-button installations. This design did not provide for pedestrian storage in the median. Hence, pedestrian phase walk times would need to be relatively long. However, because pedestrian volumes were expected to be low and pedestrian phases (with lengthy crossing times) would be provided only when pedestrians activated the phase, the decision to keep the design was considered appropriate.

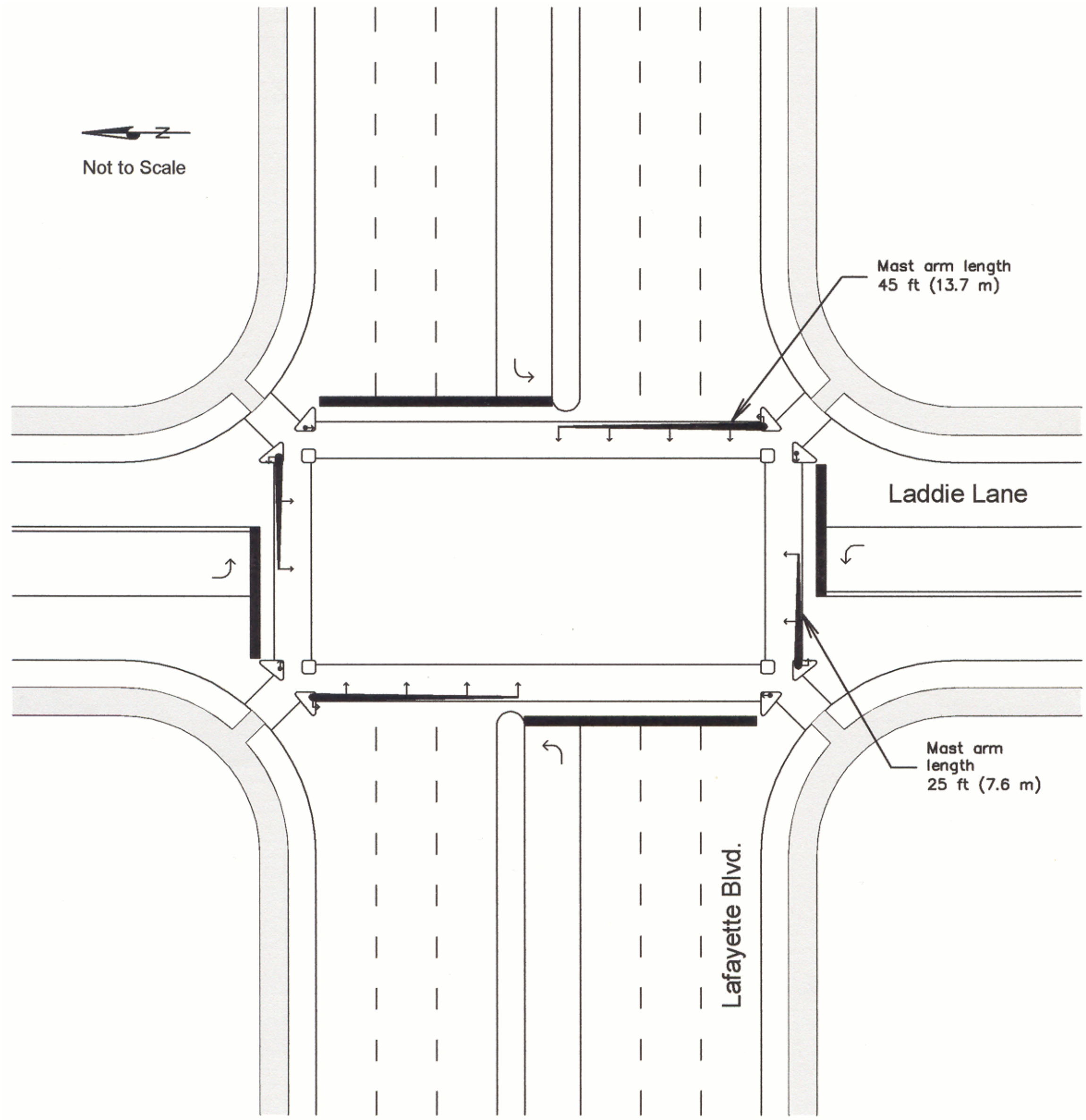

Figure 8-4. Final Proposed Design.

Design Selected

The intersection design incorporating the corner islands became the preferred geometric design for the signalized intersections along Lafayette Boulevard. It also was selected as the city's preferred intersection design along future six-lane, divided arterial streets. This design development process was unique. Typically, the geometric design of a roadway is determined initially and then traffic signals are designed to fit into the geometry of the roadway. In this example, the geometric design of the intersection was designed concurrently with the traffic signal design to provide an overall optimum, cost-effective design.

Application 3
Signal Support Considerations

Overview

The *Urban Intersection Design Guide,* Chapter 8, Section 3 <link> includes information on signal support systems.

Background

An intersection of two arterials is being designed and constructed near a major urban area. The initial design is shown in Figure 8-5. Traffic signals will not be warranted at the intersection initially because development in the area has not occurred and will not occur for several years. Because the intersection designer desires to prepare the intersection for eventual signalization, the designer has assumed that the city eventually will provide the following traffic signal features at the intersection:

- Signal heads will be installed on poles and mast arms.
- Pedestrian signals will be installed for all crossings with push button detectors.
- Vehicle detection will be provided by video imaging.
- Ground-mounted controllers will be used.
- The intersection will be illuminated when signalized.
- The city separates wiring for signals from wiring for luminaires.
- The city uses underground polyvinyl chloride (PVC) conduits to house wiring.

Issues Considered

Recognizing the need to accommodate these future traffic signal design features, the designer of the intersection incorporates the following geometric design features in the initial intersection design.

- The signal poles will be located on the corners of the intersection; therefore, long mast arms will be needed. Long mast arms require a large underground support structure (48 inches [1219 mm] in diameter), which requires much space. Hence, additional right of way is desired. Also, to minimize mast arm lengths, the poles should be located as close to the travel lanes as possible. This would require moving the sidewalk further from the roadway. Moving the sidewalk further from the roadway also makes it easier to accommodate the pedestrian landings and curb ramps. Therefore, the intersection was designed with sidewalks positioned 7 ft [2.1 m] from the curb.
- Pole locations could be located between the curb and the sidewalk at a sufficient distance from the travel way to satisfy clear zone requirements. The poles also could be placed where they may be accessible to pedestrians and used to house pedestrian push buttons.

- The separation between the sidewalk and the curb also provides sufficient space for the ground-mounted controller, and the sidewalk can be used by the signal technician, saving the cost of an additional concrete pad.
- The intersection was designed with very long curb return radii. Placement of pedestrian crosswalks away from the intersection and closer to the beginning of the curb returns, with curb ramps at the end of each crosswalk, was considered in the initial design. However, positioning sidewalks (and curb ramps) further from the intersection would have required placement of stop lines a considerable distance from the intersection, at locations close to or beyond the 180-ft [55 m] maximum permitted distance between stop lines and traffic signal heads. Motorists desire to stop relatively close to the intersecting street, especially when right turns on red are being made. Hence, in order to provide stop lines relatively close to intersecting streets and at appropriate distances from signal heads, pedestrian crosswalks were installed closer to the center of the intersection, and single curb ramps were installed at each corner of the intersection.
- To minimize disruption to the intersection when signals are to be installed, underground conduits were installed beneath all four approaches to the intersection as part of the original intersection construction. A 4-inch-diameter [102 mm] PVC conduit (for traffic and pedestrian signal wiring), a 2-inch-diameter [51 mm] PVC conduit (for luminaire wiring), and a 2+-inch-diameter [51 mm] PVC conduit (for coaxial cable used for video imaging) were designed for installation. These conduits were to be sealed and terminated in a small pull box on each corner of the intersection.

Design Selected

These design features, illustrated in Figure 8-5, will save the city money when it eventually constructs the signal installation, but more importantly, they will provide the space and geometry to incorporate an effective and efficient signal installation that will not require reconstruction of the intersection.

As shown in Figure 8-6, if separate right-turn lanes are provided on all approaches to the intersection, the pole locations could be moved to the islands. The resulting design does not help to shorten mast arm lengths, but it does locate signal faces closer to the stop lines. The signal poles can be used for pedestrian push button locations, but the buttons must be accessible. Otherwise, additional poles will be needed for pedestrian push buttons.

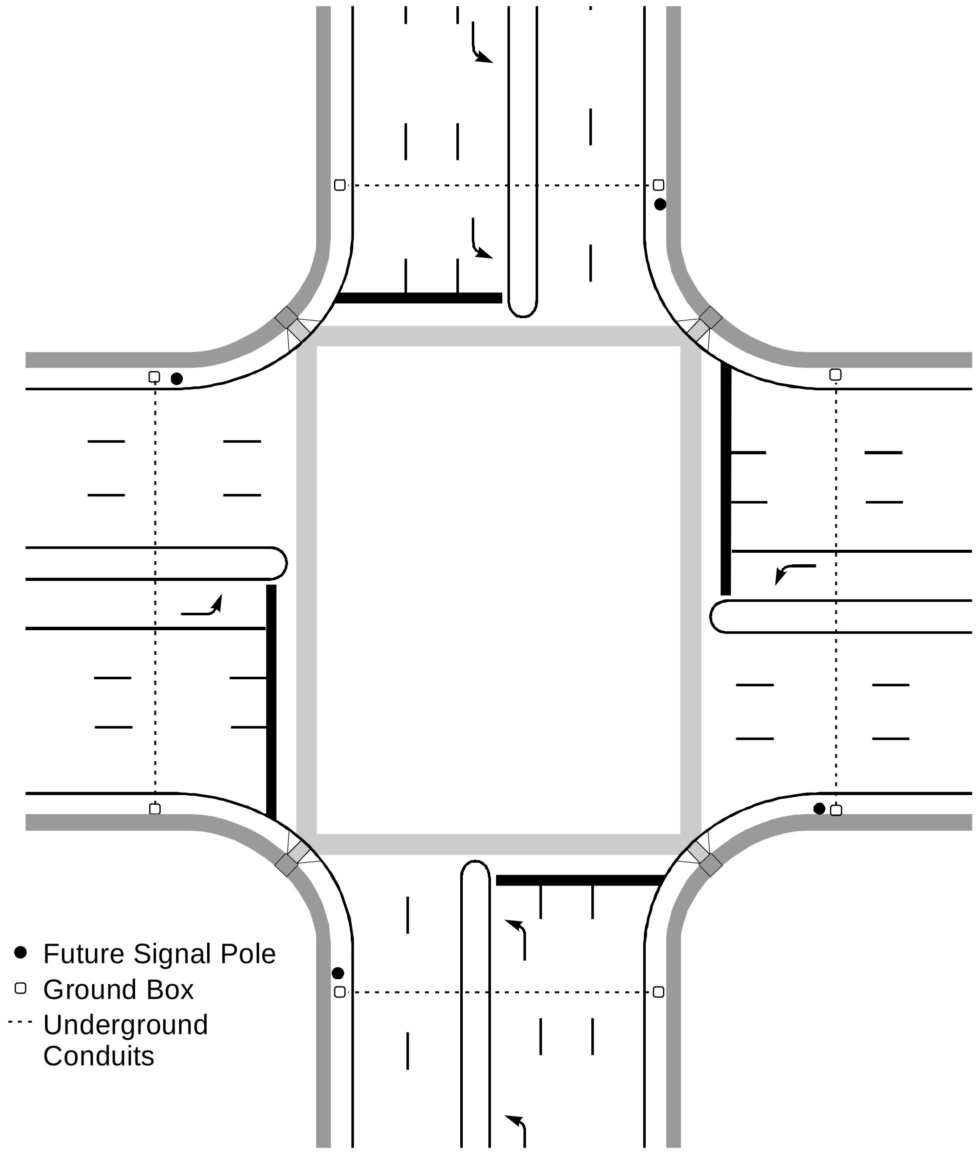

Figure 8-5. Intersection Design to Accommodate Traffic Signal Installation.

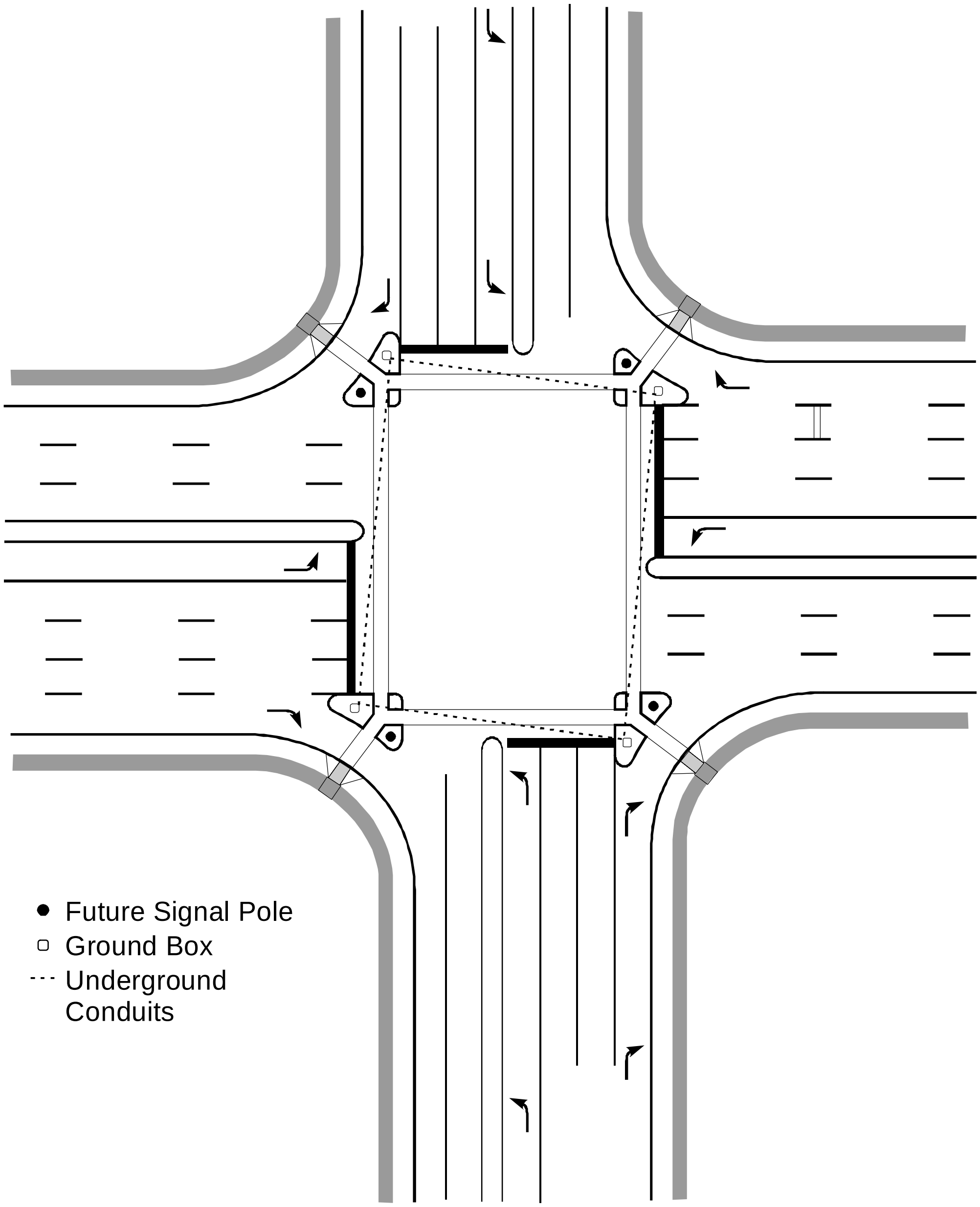

Figure 8-6. Alternate Intersection Design to Accommodate Future Traffic Signal Installation.

Chapter 9
Markings

Contents:

Application 1
Markings Checklist

Overview

The following presents a checklist on pavement markings. The *Urban Intersection Design Guide*, Chapter 9 <link> presents additional information on pavement markings.

Background

During an intersection design, several elements are competing for attention. Following is a checklist that can be used to review a design to assist in identifying whether markings were adequately considered within the design. The objective of the checklist is to assist with consideration of various pavement marking elements. It is not meant to be a comprehensive, point-by-point inspection on all pavement marking requirements at an intersection. Rather it is to serve as a reminder to check various elements of the intersection design.

Checklist

Checklist For Markings At An Urban Intersection

***Checklist scale**

Y = Yes or Acceptable N = No or Needs Improvement I = Irrelevant to Site

*Y	N	I	**MARKINGS**
			Are the required markings present at the intersection?
			Is an end of a school zone present? If so, are markings appropriate?
			Is a bus stop present? If so, are markings appropriate?
			Is a high occupancy vehicle (HOV) lane present? If so, are markings appropriate?
			Is a bicycle lane present? If so, are markings appropriate?
			Is a railroad crossing nearby? If so, are markings appropriate?
			Do parking spaces need to be delineated?
			Are markings needed for parking restrictions?
			Is retroreflective yellow needed on nose of median?
			Is retroreflective yellow or white needed on island?

Checklist For Markings At An Urban Intersection

Checklist scale

*Y = Yes or Acceptable N = No or Needs Improvement I = Irrelevant to Site

*Y	N	I	**MARKINGS (Continued)**
			Are old pavement markings adequately removed?
			Do markings conflict with signs?

*Y	N	I	**TREATMENTS**
			Is (are) left-turn lane(s) present? If so, are additional markings through the intersection needed?
			Are pavement arrows or the word ONLY needed on the pavement?
			Is advance information on lane assignments needed?
			Are crosswalk markings needed?
			Are high-visibility markings needed for the crosswalk?
			Can the crosswalk marking material (e.g., ladder) be spaced to avoid the wheel path?
			Is an advance yield or stop line needed?
			Are contrast markings needed?
			Are double white or yellow lines needed?
			Is hatching needed at the intersection?
			Are the markings for an offset left turn optimal?
			Do the markings need to be supplemented with raised pavement markers?

Checklist For Markings At An Urban Intersection

Checklist scale

*Y = Yes or Acceptable N = No or Needs Improvement I = Irrelevant to Site

*Y	N	I	**MATERIAL**
			Is the better material selected for the situation (raised markers versus paint versus combination)?
			Were aesthetics considered?
			Are bricks or other aesthetic treatments planned for the crossing location? If so, was the appropriate material selected?
			If a brick crosswalk or other aesthetic treatments are present are supplemental pavement markings needed to increase visibility?
			Will markings be adequately visible during snow, ice, or rain conditions?

*Y	N	I	**STOP BAR LOCATION**
			Is adequate sight distance present at the stop bar?
			Do the signals meet visibility requirements with selected stop bar location?
			Is the stop bar in the optimal location considering all users of the intersection?

Application 2

Traffic Control Devices for a Bicycle Lane

Overview

This application provides an example of traffic control devices for a bicycle lane. Information on bicycle lanes is included in *Urban Intersection Design Guide*, Chapter 9, Section 4 <link> and Chapter 4, Section 6 <link>.

Background

Bicycle lanes are useful for encouraging bicycle travel and separating bicycles from vehicles and pedestrians. However, problems can arise at intersections with conflicts between through-bicycles and right-turning vehicles.

An intersection that is being redesigned has two bicycle lanes as part of its new configuration. The bicycle lanes will interact with right-turn lanes that are also part of the new design. Minimizing conflicts between bicycle traffic and right-turning vehicles is important, as is allowing bicycles to travel safely through the intersection.

This signalized intersection is in a suburban fringe area, and contains a major four-lane arterial (First Street) and a minor four-lane arterial (Mara Street). First Street has a very wide raised approach median and double left-turn bays at the intersection, in addition to the right-turn lane. Mara Street is also four lanes with no median and a single left-turn bay at the intersection (see Figure 9-1). The two northern quadrants are vacant; the southwest corner contains a large professional building, while a free-standing pharmacy/variety store is on the southeast corner. The intersection is in somewhat of a valley, as each leg of the intersection rises as it departs. The west side of the intersection is substantially higher than the east side.

In the redesign of the intersection, right-turn lanes with a corner island will be included on First Street. In addition, a bicycle lane in each direction of First Street must be carried through the intersection in a manner that minimizes conflicts with right-turning vehicles prior to the intersection and crossing vehicles at the intersection.

Issues Considered

Issues to consider during an upgrade to the site include the following:

- acquisition of right of way,
- adequate signing and markings with a clear message of the expectations regarding the priority of bicycles over right-turning vehicles,
- adequate marking to carry the bicycle lane through a driveway near the transition taper for the right-turn lane, and
- retention of bicycle lane between through lanes and right-turn lanes.

Suggested Designs

- Construct the approach to the intersection such that right-turning vehicles must cross the bicycle lane to enter the right-turn lane, yielding the right of way to bicycles. The bicycle lane would then be located between the right-turn lane and the right-hand through lane on the intersection approach.
- Align the bicycle lane such that it is to the left of the island separating through traffic from right-turning traffic. This aligns their travel path with the bicycle lane on the other side of the intersection and keeps bicycles separated from pedestrians waiting on the island to cross the street.

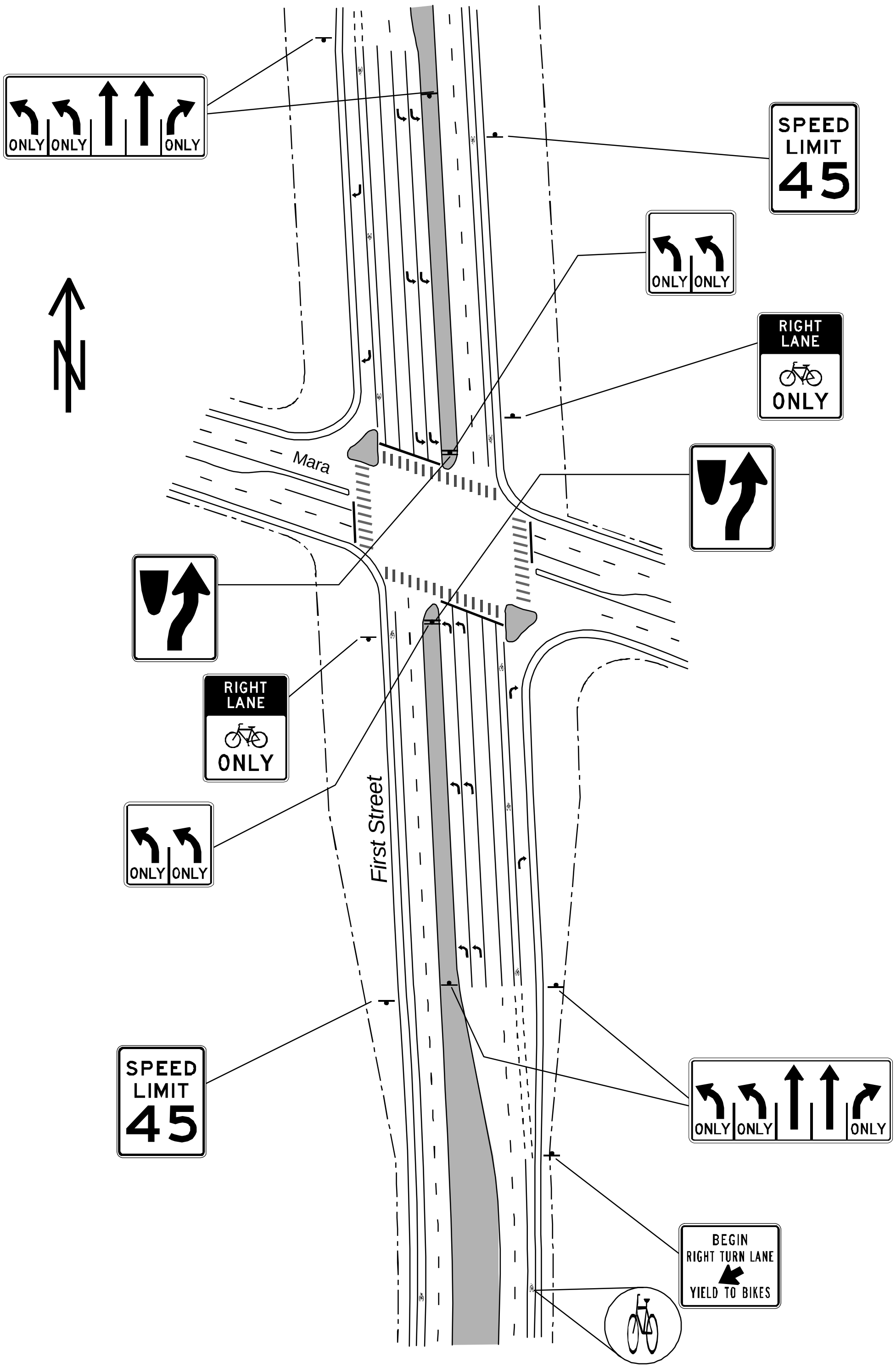

Figure 9-1. Sketch of Intersection with Bicycle Lane Carried through Right-Turn Lane (Note: Signals not shown on sketch).

Chapter 10
Checklist

Contents:

Application 1
Signs Checklist

Overview

The following presents a checklist on signs. The *Urban Intersection Design Guide*, Chapter 10 <link> presents additional information on signs.

Background

During an intersection design, several elements are competing for attention. Following is a checklist that can be used to review a design to assist in identifying whether signs were adequately considered within the design. The objective of the checklist is to assist with consideration of various sign elements. It is not meant to be a comprehensive, point-by-point inspection on all sign requirements at an intersection. Rather it is to serve as a reminder to check various elements of the intersection design.

Checklist

Checklist For Signs At An Urban Intersection

***Checklist scale**

Y = Yes or Acceptable N = No or Needs Improvement I = Irrelevant to Site

*Y	N	I	**SIGNS**
			Is there adequate space on the roadside or median to place signs?
			Were overhead guide or warning signs considered?
			Are required signs included (e.g., intersection control, regulatory, warning, guidance – including advance street name supplement)?
			Are/will signs be obstructed by vegetation?
			Do any city ordinances require additional/special signs?
			Are signs properly spaced from the intersection?
			Do signs obstruct visibility for traffic exiting adjacent property?
			Can any – existing or proposed – signs be eliminated? Consolidated?
			For existing signs and supports to be retained, is the sign condition adequate? Is the support adequate?

Checklist For Signs At An Urban Intersection

*__Checklist scale__

Y = Yes or Acceptable N = No or Needs Improvement I = Irrelevant to Site

*Y	N	I	**SIGNS (Continued)**
			Do signs obstruct each other?
			Are signs clear of pedestrian path (both width and height)?
			Are signs adequate for bicyclists? Are signs adequate for transit users (including signs for disabled users)?
			Are special purpose signs needed?
			Would the location benefit from using larger letters on the street-name signs (e.g., to accommodate the reduction in visual acuity associated with increasing age)?
			Do unusual geometrics create the need for special signs? Are advance signs necessary?
			Are any non-standard (non-*TMUTCD*) signs needed? If so, these signs will require approval.
			Are block numbers included on the street name signs?
			Do back-to-back signs have compatible shapes?
			Are median signs needed to improve visibility?
			Does median width require treatment as two intersections?
			Are lane assignment signs at correct location for drivers to make decision?
			Do signs agree with pavement markings?
			Should internally illuminated signs be considered?

Application 2

Traffic Control Devices for Dual Left-Turn Lanes

Overview

The inclusion of signs and markings at dual left-turn lanes provides drivers with additional guidance on how to maneuver through the intersection. Information on traffic control devices is included in the *Urban Intersection Design Guide,* Chapter 9 for Markings <link> and Chapter 10 for Signs <link>.

Example

Figure 10-1 illustrates typical signs and pavement markings for an intersection of two arterial streets with dual left-turn lanes and single right-turn lanes.

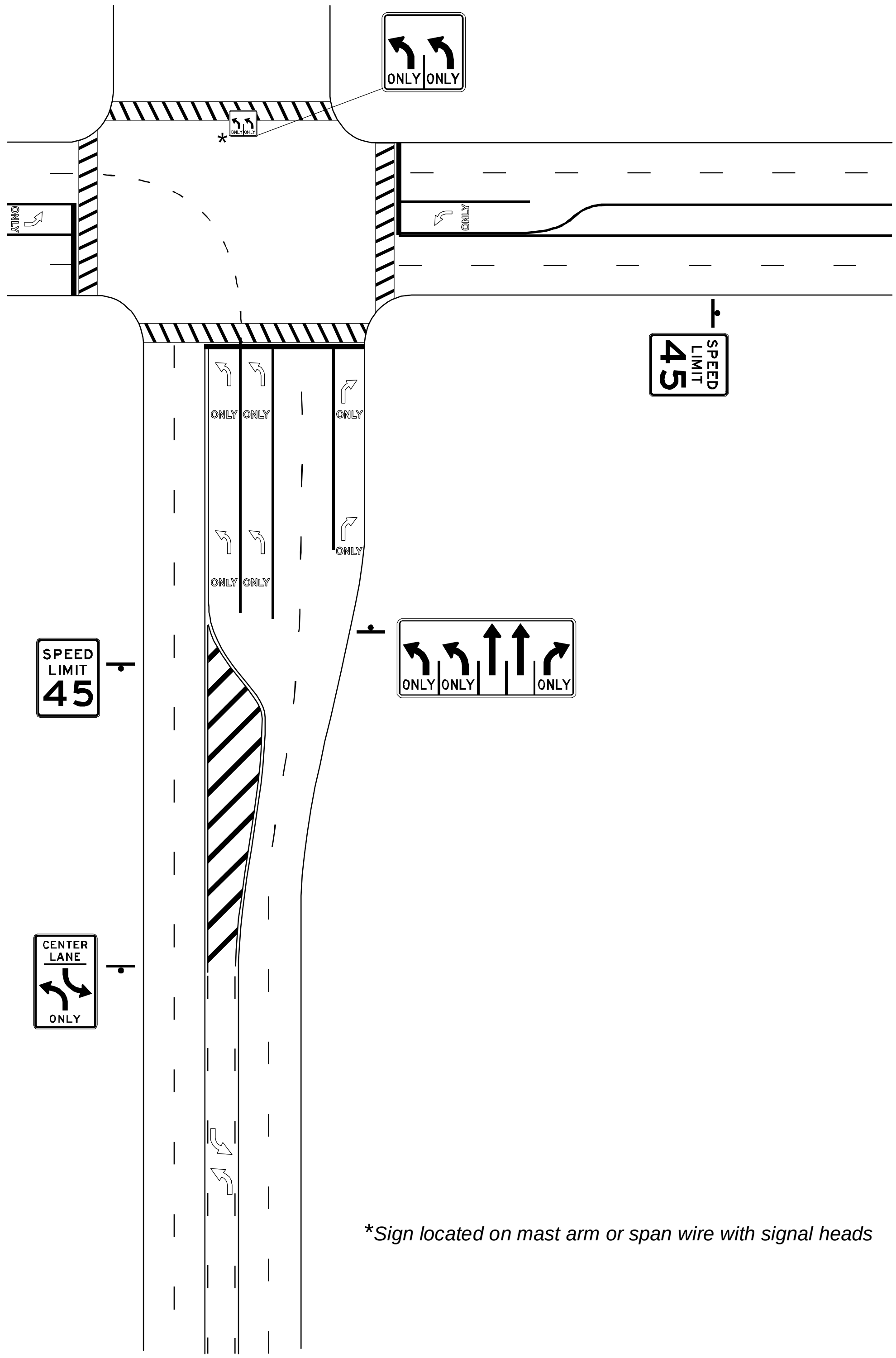

Figure 10-1. Example of Signs and Pavement Markings for Intersection with Dual Left-Turn Lanes and Single Right-Turn Lanes.

Chapter 11
Influences from Other Intersections

Contents:

Application 1

Realignment of Intersection

Overview

Two closely spaced T-intersections can present challenges because of a lack of space for left- or right-turn queuing. The close spacing of the two intersections presents a complex design problem that may not always be solvable without modifying the geometry of the intersection. The *Urban Intersection Design Guide*, Chapter 11, Section 1 <link> provides further information regarding closely spaced intersections.

Background

Problem. Two intersections on a major roadway, Austin Road, are very closely spaced. The intersections are not functioning well because inadequate space between the intersections is available for queuing. An additional challenge at the intersection is the small curb radii that are present. Transit buses occasionally use the intersection but have trouble negotiating the right turns at the intersection without encroaching on adjacent lanes.

Austin Road is an arterial with high volumes in a commercial area. Austin Road's intersections with Lilac Street and Oregon Street (collectors) are approximately 100 ft [30 m] apart. Combined with the large number of driveways in the area, there are several access points and turning maneuvers on this arterial. Lilac Street leads north into a residential subdivision, while Oregon Street leads south into more commercial development. The intersections are surrounded by fast-food restaurants on all sides except the southeast corner, which houses a grocery store. The lanes on Austin Road are narrow, and there are many commercial signs and other visual distractions. The curb radii at the intersections are very small (15 ft [5 m]), making right turns more difficult for larger vehicles.

It has been difficult to provide a satisfactory signal timing strategy for the intersections because of their close proximity. A potential solution for this issue could be to use one controller and treat the two intersections as a single intersection for signalization purposes. Inadequate storage is available between the signals for left-turning vehicles, however, and their close proximity prevents the efficient coordination of the traffic signals.

Known Information. The information known for this intersection includes:

- ♦ The intersections are controlled by traffic signals.
- ♦ Lilac St. and Oregon St. have stop control.
- ♦ The design speed on Austin Rd. is 45 mph [72 km/h].
- ♦ The design speed on Lilac St. and Oregon St. is 30 mph [48 km/h].

Issues Considered

Issues to consider during an upgrade to the site include the following:

- ♦ acquisition of right of way (the preferred realignment will require purchasing the corner of the grocery store parking lot, eliminating some of the parking spaces for the grocery store);
- ♦ relocation of utilities;
- ♦ accommodation of traffic during construction;
- ♦ accommodation of pedestrians during and after construction (provision of marked crosswalks added because of high likelihood of pedestrians in an area having a grocery store, fast food, and other retail outlets); and
- ♦ adequate signing and markings to guide motorists.

Proposed Design

An alignment based on minimum horizontal radius without superelevation is shown in Figure 11-1. Using a horizontal curve radius of 300 ft [91 m] based on the design speed of 30 mph [48 km/h] (see Table 2-5 of the *Roadway Design Manual* <link>), the design will eliminate approximately 32 parking spaces from the grocery store parking lot and require modifying its circulation pattern.

Further details of the design are shown in Figure 11-2. The intersection curb radii are designed at 50 ft [15 m], accommodating occasional large vehicles. The southwest corner curb radius is designed at 40 ft [12 m] because the deflection present in the realigned roadway allows a bus to negotiate the turn without encroachment. The increased curb radii used in the realigned intersection will improve the efficiency of right-turn maneuvers.

The realigned intersection will also reduce the complexity of the original two intersections and provide a single intersection that allows a better signal timing strategy.

The selected design has several trade-offs. The appropriateness of the choices made should be evaluated according to the characteristics of the site:

- ♦ The increased curb radii will:
  - provide better accommodation for large vehicles;
  - improve turning efficiencies;
  - increase pedestrian crossing time;
  - decrease signal timing efficiency somewhat because of the increased pedestrian crossing time for the larger intersection (increased from 11 to 13 sec to cross minor roadway, increased from 16.5 to 18.5 sec to cross major roadway);
  - require the use of longer traffic signal mast arms; and
  - decrease signal timing efficiency because of increased lost time due to the increased size of the intersection.
- ♦ The radii used for the reverse curves on Oregon St. will:

- minimize the amount of right of way (ROW) obtained for the project without requiring a design exception; and
- require the elimination of approximately 32 parking spaces from the grocery store parking lot.

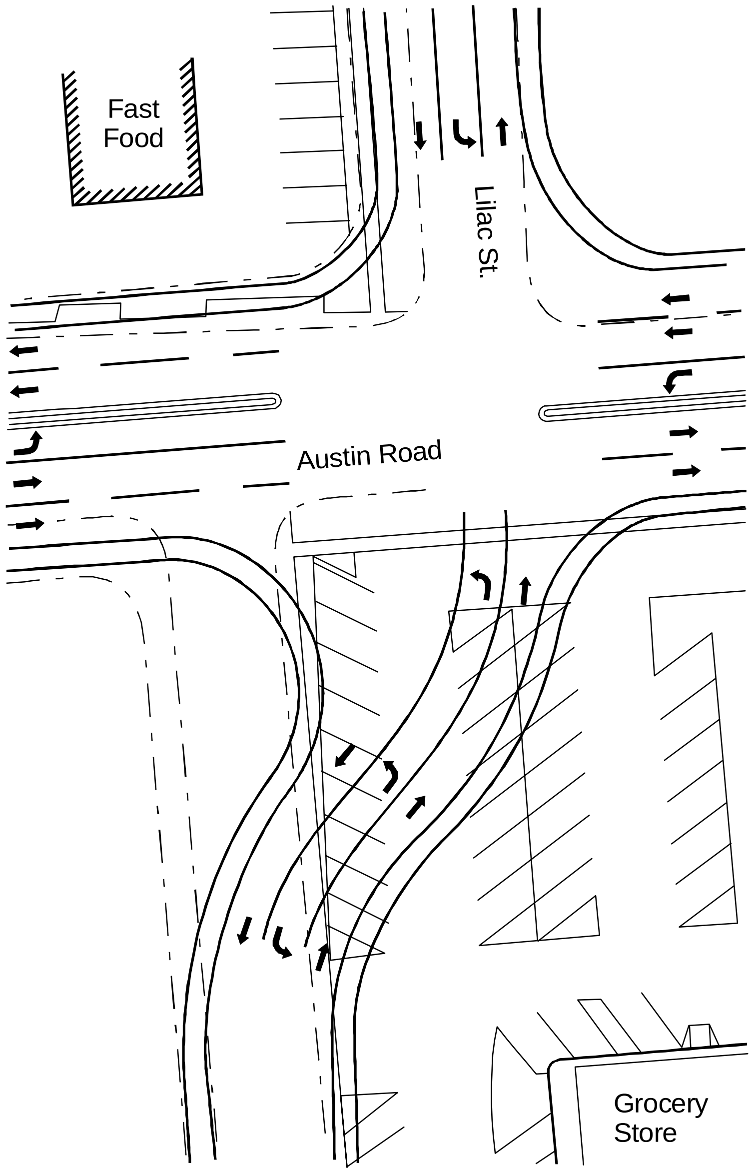

Figure 11-1. Sketch of Proposed Realignment Based on 30 mph [48 km/h] Design Speed.

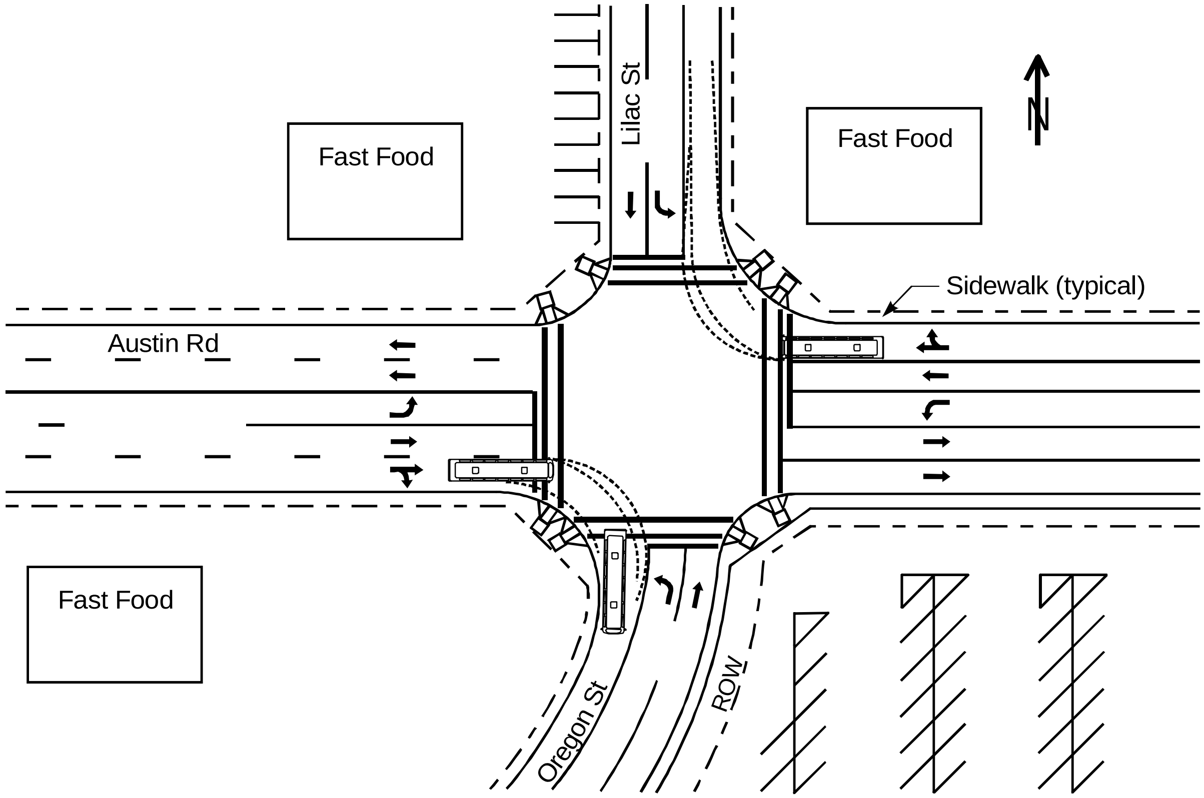

Figure 11-2. Intersection Details for Proposed Realignment.

Application 2

Control of Access to Driveways

Overview

The presence of driveways can adversely impact the operations of roadways. The introduction of additional conflict points near an intersection can overload drivers and result in an increased crash risk. Capacity can also be affected if drivers slow to enter driveways or avoid other drivers exiting driveways. The *Urban Intersection Design Guide,* Chapter 11, Section 3 <link> provides further information regarding driveways and their influence on intersections.

Background

A suburban intersection is in an area of heavy commercial development. The large numbers of driveways in close proximity to the intersection approaches creates traffic flow problems and increased potential for crashes with turning vehicles.

A signalized suburban intersection of two four-lane major arterials, White Parkway and Compass Boulevard, has a high level of commercial development on the approach streets. White Parkway and the northbound approach of Compass Blvd. have a continuous two-way left-turn lane up to the storage area for the intersection, where the TWLTL is converted to a left-turn bay. A large number of driveways open onto Compass Boulevard. Traffic volumes are high on both streets, and flow into and out of the commercial establishments is steady. A sketch of the unimproved intersection is shown in Figure 11-3.

Issues Considered

Issues to consider during an upgrade to the site include the following:

- accommodation of traffic flow during construction,
- negotiation with business owners to allay fears of decreased business because of change in access,
- adequate signing and markings to inform drivers of changes to cross section, and
- acquisition of right of way,

Proposed Design

- Construct a raised median on the approaches of Compass Boulevard. Extend the median approximately 300 ft [91 m] from the intersection, to discourage left-turning movements onto or off of Compass in the vicinity of the intersection. (See Figure 11-4 for a sketch of improvements.)

- ♦ Improve delineation with raised marker buttons on the centerline approach to the median.
- ♦ Improve radii on free-flow right-turn lanes on each approach to facilitate turning movements from street to street.

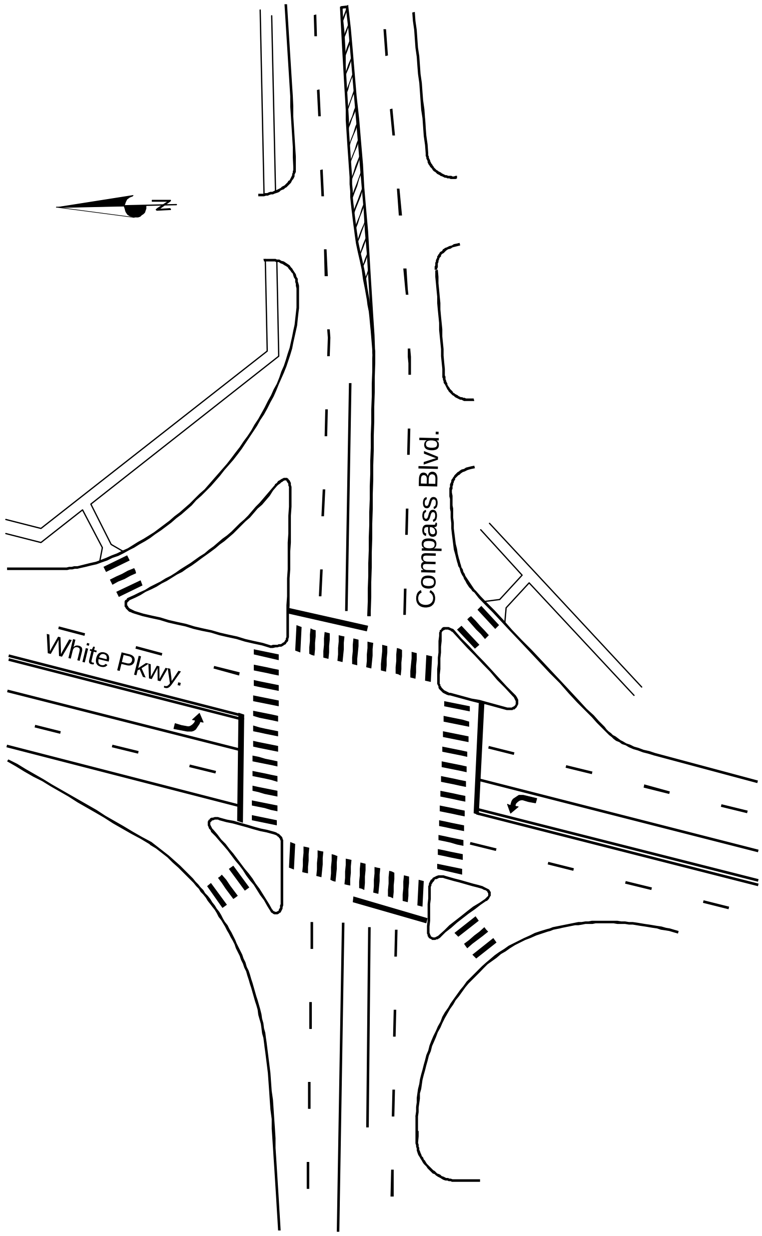

Figure 11-3. Sketch of Intersection Before Improvements.

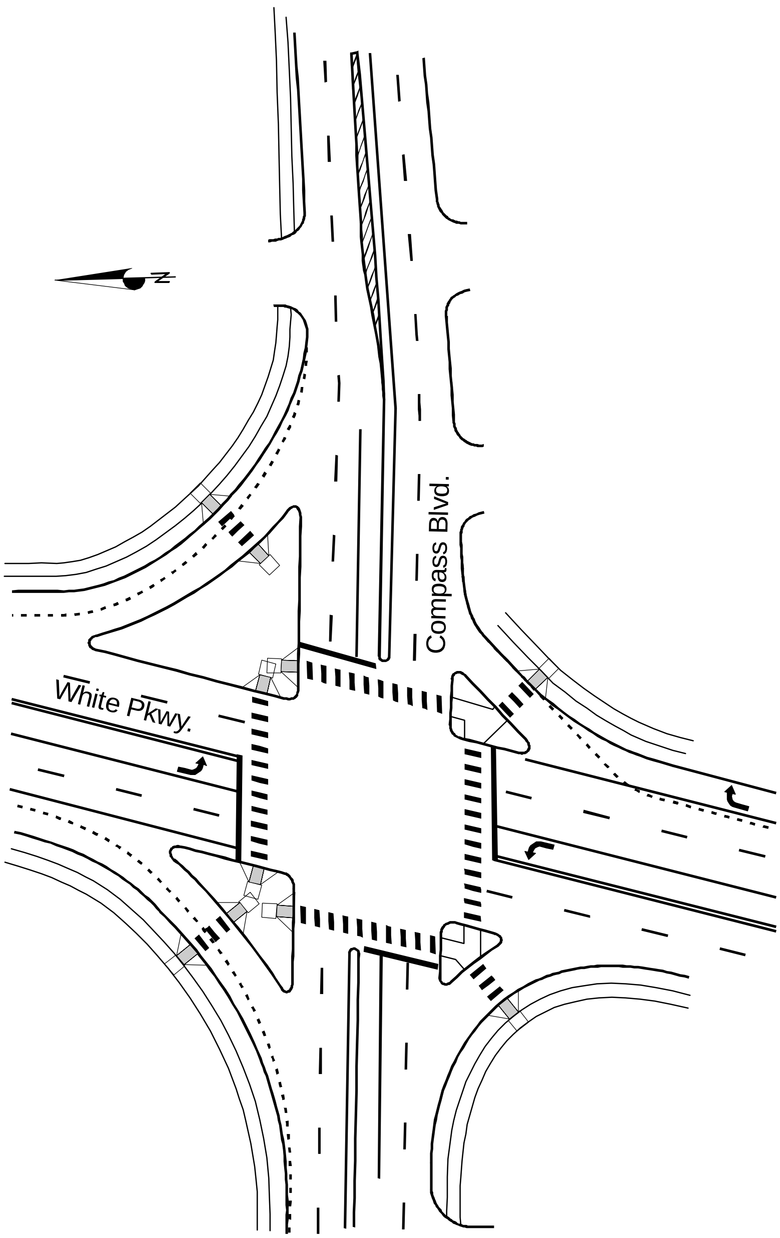

Figure 11-4. Sketch of Intersection with Improved Medians and Right-Turn Lanes.

Application 3
Turning Restrictions

Overview

The presence of an intersection near the end of a ramp can adversely impact operations due to drivers attempting to quickly cross other traffic in order to turn. The introduction of additional conflict points near an intersection can overload drivers and result in an increased crash risk. The *Urban Intersection Design Guide*, Chapter 11, Sections 1 <link> and 3 <link> provide further information regarding driveways and nearby intersections and their influence on intersections.

Background

Highway ramps near street intersections can experience extensive weaving and conflicts due to the many different movements in a small area. Restriction of some of the movements can improve safety and efficiency of operations.

The exit ramp from a major highway in a suburban area connects with the local street network in close proximity to a downstream intersection. A large number of drivers desire to turn left at that intersection and must cross other lanes of traffic to use the left-turn lane.

A stop-controlled exit ramp from Highway 713 intersects a major arterial (Concert Boulevard) approximately 400 ft [122 m] upstream of a signalized intersection with a minor arterial (Sam Houston Street) and shopping center entrance. (See Figure 11-5 for illustration.) A raised median on Concert Boulevard permits only right-turning traffic from the exit ramp. Many of the drivers on the exit ramp seek to enter the shopping center, which requires a left turn from eastbound Concert at the Sam Houston intersection. Drivers attempting this maneuver must immediately turn right from the ramp (as in Figure 11-6), cross two lanes of through traffic on Concert Boulevard, and enter the left-turn bay. All of these lane changes occur within 400 ft [122 m]. In addition to the increased potential for crashes, drivers who wish to make this maneuver must wait for an adequate gap to turn, which creates large queues and long delays on the ramp. Eliminating this movement would improve safety and efficiency at this location.

Issues Considered

Issues to consider during an upgrade to the site include the following:

- There is limited space for expansion at this intersection. The bridge columns limit the amount of median area available west of the intersection. Widening Concert Boulevard will also be difficult because of a stormwater viaduct on the south side of the street, and other entrance and exit ramps for Highway 713 on the north side of the street.
- Proper signing and markings are needed to acclimate drivers to the change in configuration. Signs will be necessary on the ramp to inform drivers that left turns to Sam Houston Street are not permitted from the ramp. Supplemental signing is needed

to show alternate entrances to the shopping area so as to provide guidance on when to exit from Highway 713.

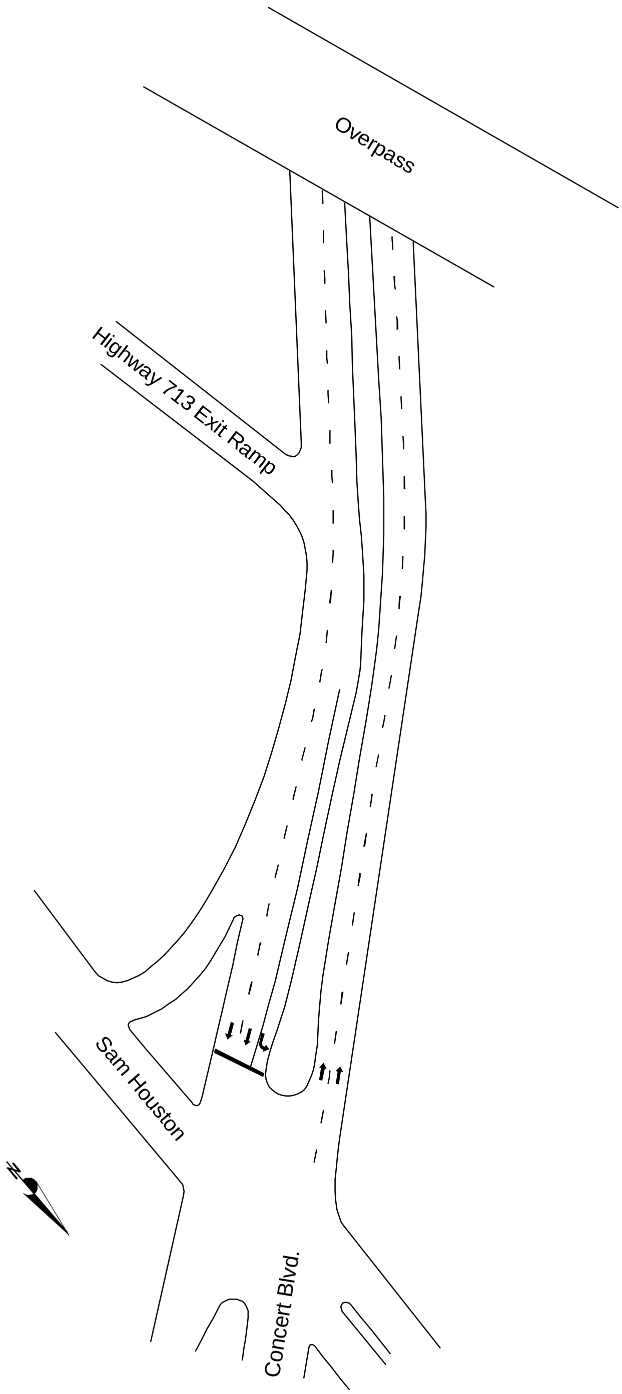

Figure 11-5. Current Conditions, Prior to Improvements.

Figure 11-6. Ramp Traffic Crossing Arterial to Make Left Turn.

Proposed Design

The suggested design is to separate the eastbound left-turn lane on Concert Boulevard with a modified median that prevents ramp traffic from entering the left-turn lane. A sketch of this new design is shown in Figure 11-7 and Figure 11-8. The design uses a raised median and turning restrictions to prevent vehicles exiting on the Highway 713 exit ramp from turning left into the shopping center on Concert Boulevard. Openings were necessary in the raised median to allow for adequate drainage.

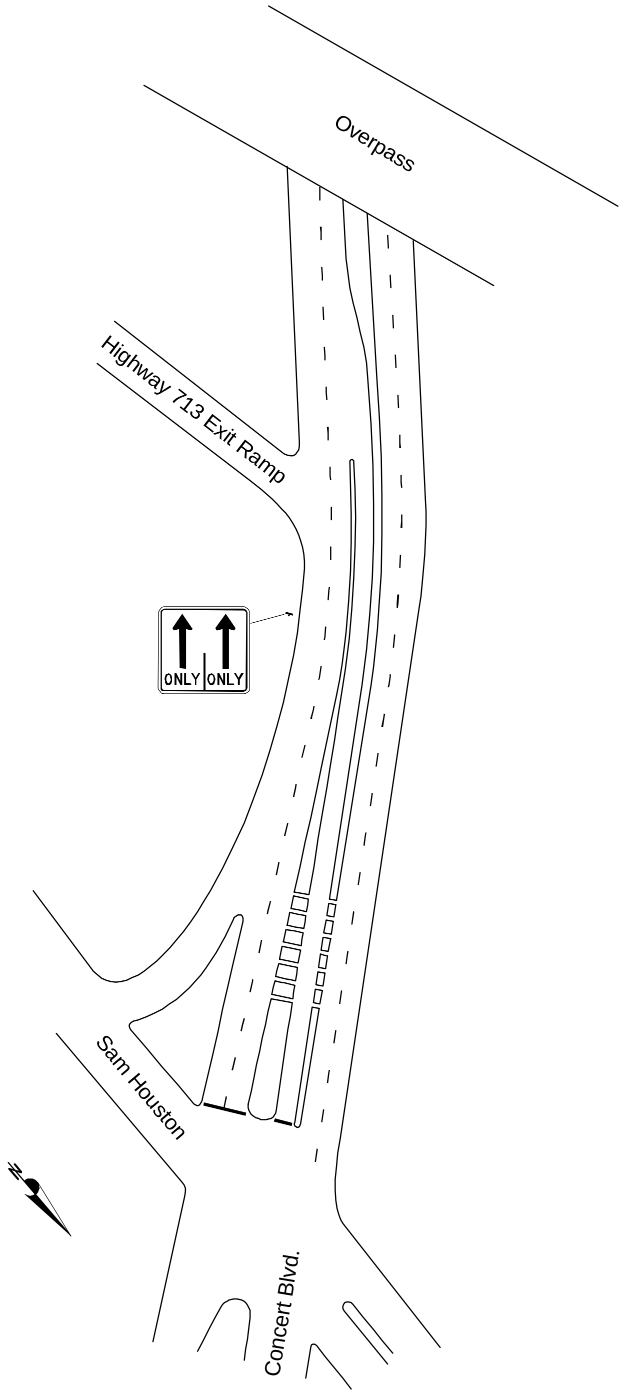

Figure 11-7. Suggested Improvement to Median.

Figure 11-8. Median Added to Limit Left Turns by Traffic from Ramp.