OREGON DEPARTMENT OF TRANSPORTATION TECHNICAL SERVICES

Roadway Engineering BULLETIN

SUBJECT	FINAL NUMBER	EFFECTIVE DATE	VALIDATION DATE	SUPERSEDES or RESCINDS
Modern Roundabout Design Chapter 8.6 Appendix P, Analysis For Roundabout Entrance and Exit Geometry	RD15-01(B) WEB LINK(S) http://www.oregon.gov/ODOT/HWY/TECHSERV/Pages/technicalguidance.aspx	2/1/2015		New
TOPIC/PROGRAM Highway Design Manual	APPROVED SIGNATURE Original signed by: Bob Pappe, P.E., PLS State Traffic/Roadway Engineer			

PURPOSE

When the Oregon Highway Design Manual (HDM) was revised in 2012, the section discussing modern roundabouts was temporarily removed from Chapter 8, Intersections in order to investigate current design practices and to create a fully revised discussion of modern roundabout design that reflects both current design practice and incorporates Oregon design policy.

GUIDANCE

This technical bulletin will add the revised section 8.6 Modern Roundabouts to the 2012 Oregon Highway Design Manual and includes Appendix P that discusses roundabout entrance and exit geometry.

Until the online HDM is updated, use the attached Chapter 8 and Appendix P.

CONTACT INFORMATION

Title: Senior Standards Engineer
Section: Traffic-Roadway
Phone: 503-986-3738
E-Mail: david.j.polly@odot.state.or.us

Title: Senior Urban Design Engineer
Section: Traffic-Roadway
Phone: 503-986-3561
E-Mail: Rich.Crossler-Laird@odot.state.or.us

Chapter

8

INTERSECTIONS

8.1 INTRODUCTION

This chapter covers the design standards, guidelines, and processes for designing road approaches, signalized and unsignalized at-grade intersections for State Highways. For information on general design considerations not fully covered in this chapter, or other parts of this manual, refer to AASHTO's "*A Policy on Geometric Design of Highways and Streets – 2011*," Chapters 9 and 10; "*Technology Sharing Report 80-204,*" Chapter 6; and/or the ODOT "*Modern Roundabouts For Oregon, Report 98-SRS-522*","*NCHRP Report 672, Roundabouts an Informational Guide*", second edition and those documents referenced in Section 8.6.

The Technical Services, Roadway Unit can provide design assistance in the areas of intersection design, channelizations, road approaches, roundabouts, large vehicle accommodation, and alternative mode accommodation. The Technical Services, Roadway Unit should be consulted about complex intersection designs that cannot meet the standards contained in this design manual.

Information on traffic volumes and requirements can be found in Sections 10.11 and 10.12 of this manual or further information can be obtained from Region Traffic Units and the Transportation Planning Analysis Unit of the Transportation Development Division of ODOT.

8.2 ROAD APPROACHES

8.2.1 GENERAL

The location and spacing of road approaches should be in conformance with the Access Management standards as described in the Oregon Highway Plan, Appendix C. The decision for placement and design of a road approach must be consistent with the function of the highway and optimize the safety and operational efficiency for vehicles as well as bicyclists and pedestrians. The road approach design must accommodate the turning movements of the appropriate design vehicle. All road approaches, public and private, require a construction permit from the appropriate District Maintenance Office. The District Manager and Regional Access Management Engineer and/or Access Management sub-team should be involved early in any road approach discussion and decisions.

Road approaches can be classified as either private or public. Private approaches connect private property with a state highway across the highway right of way. Public approaches are at-grade intersections of public roadway right of way with a state highway. The remaining part of this section will discuss the design requirements for private approaches. For public approach design, see Section 8.3, General Intersection Design.

8.2.2 DESIGN REQUIREMENTS FOR PRIVATE ROAD APPROACHES

Private approaches are connections to adjacent businesses, residences, or other private roadways. Generally, private approaches provide access to/from the highway and an adjacent property across the highway right of way. These approaches service all land use types including residential, commercial, and industrial. Typically, private approaches in urban areas will use a 'dust pan' style approach. This style drops the curb and possibly the sidewalk to highway grade to allow vehicular access. Standard Drawings RD725 through RD750 should be used when designing "dust pan" style private approach roads. For high volume driveways, a radius design style similar to that used by a public approach should be used. Refer to Table 8-1 to determine the style of approach to be used.

There are three general types of private road approaches. These are:

- Type A Non-curbed, ditch section highway with radius style approach.
- Type B Curbed highway section with "dust pan" style approach.
- Type C Curbed highway section with radius style approach.

Type C private approaches should be designed in accordance with Section 8.3, General Intersection Design. The design of Types A and B are described below.

The design of private road approaches is affected by many factors. The type of access, volume of vehicles, type of vehicles, grades, alignment, and adjacent land use all influence the design. The spacing of approach roads should be consistent with the spacing guidelines specified in the Oregon Highway Plan, Appendix C. The designer is encouraged to read the Access Management Policy contained in the OHP and Oregon Administrative Rule (OAR) 734, Division 51 for clarification of spacing guidelines and other guidance pertaining to access management.

1. All road approaches should be placed so that intersection sight distance is provided. The vehicle entering the traffic stream should have a view along the highway equal to the intersection sight distance for the design speed of the highway. At a minimum, stopping sight distance for the design speed of the highway must be provided at all approaches. For more information on intersection and stopping sight distances refer to AASHTO's "A Policy on Geometric Design of Highways and Streets - 2011" and Section 3.2.4 herein. Any proposed approach that cannot provide sight distance as required by Oregon Revised Statute (OAR) 734, Division 51 must obtain an approval from the Region Access Management Engineer (RAME). For more information related to access management deviations, see Section 2.6. Cut slopes may need to be widened and roadside vegetation removed in order to provide required sight distance.

2. Both public and private road approach grades should be designed so that drainage from the approach does not run on or across the traffic lane, shoulder areas, or sidewalk. In no case should the normal slope of the shoulder be altered. In urban areas where the drainage is along a curb and gutter, only the paved approach area to the right of way line may drain into the gutter. In the case of an approach below the street grade, a short vertical curve should be used to confine the drainage in the gutter line. In some instances inlets may be required on each side of the approach to collect runoff without ponding or to ensure that roadway drainage does not leave the right of way. The approach road should provide a flat landing area for vehicles entering the highway for at least 20 feet from the edge of the shoulder. A grade of two percent is desirable for these landings and four percent is the maximum. Approach grades steeper than four percent should be carefully evaluated by the Designer.

3. The maximum grade break between highway shoulder and approach is eight percent for Type A and B approaches. In addition, a 20 foot landing area should be provided. In some situations, the maximum break cannot be met. When this is the design condition, the designer should attempt to achieve a roadway-to-approach transition as smooth as possible. This may require using a short vertical curve.

4. The approach must accommodate the appropriate design vehicle. Generally, commercial accesses should be designed for at least a Single Unit (SU) truck design vehicle. Vehicles larger than an SU are not to be treated as the design vehicle unless 3 or more WB-40 or larger trucks are anticipated between 7:00AM and 7:00PM. Anytime the design vehicle is larger than a SU, the approach is to be designed as a radius style. When vehicles larger

than an SU are anticipated, but are not the design vehicle as described above, the approach must accommodate the larger vehicle. ('Accommodation' only refers to the physical ability to make the maneuver including encroaching on other lanes, whereas 'designed for' means that design elements do not require encroachment. A site visit and discussion with maintenance personnel along with information gathered from property and business owners will help determine the appropriate design for an approach. (See Figure 8-1 for more detail concerning "design for" and "accommodate for".)

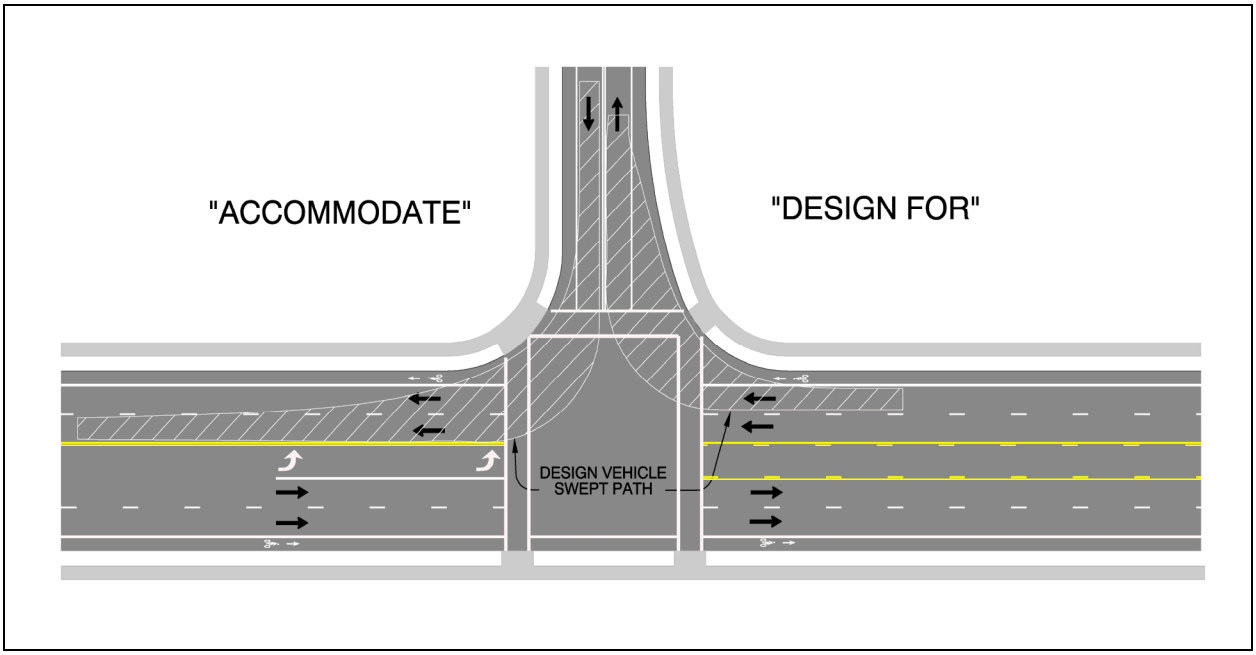


Figure 8-1: Accommodating And Designing For Vehicles

5. All approaches must be designed to aid in the longitudinal crossing of pedestrians. It is preferable to maintain sidewalks at a continuous grade. However, without a buffer strip or set back to provide a ramp down area to street grade, this is nearly impossible. Route continuity is also important to pedestrians. If a curbside sidewalk cannot be set back for a significant longitudinal distance, it is best to leave it curbside rather than break up the pedestrian continuity. For ADA compliance, sidewalk cross-slope must be maintained at 2 percent or less. To meet this requirement approaches may need to be designed with more than one slope to transition from roadway grade to final approach grade. Roadway standard drawings in the RD700 series provide information and various design options for curb, sidewalk, and driveway design at approaches.

6. All curbs and delineators used at approaches on highways without continuous curbs should be placed at the normal shoulder width from the edge of the traveled way to provide adequate shoulder adjacent to the approach.

7. Approaches on opposite sides of the highway should be located across from each other whenever possible. However, under high speed and high traffic volume conditions,

approaches may need to be separated to reduce the complexity and number of conflicts (see Figure 8-2). In addition to reduction in conflict points, separating approaches breaks the crossing maneuvers into distinct steps and isolates them reducing driver tasks and anxiety. When designing, the approaches need to be separated far enough that they operate independently outside their functional areas (see Figure 8-3). Although this situation is possible at some high volume private approaches, this treatment is generally only appropriate for public road approaches. Not all intersection locations are good candidates for separated approaches. The Technical Services, Roadway Engineering Unit and the Region Access Management Engineer should be contacted when considering separation of private approach roads. Major public roads with large volumes of through traffic should generally not be separated.

8. Approach roads should not be constructed within the functional area of an adjacent intersection. Refer to the Access Management Policies from the Oregon Highway Plan and OAR 734, Division 51 for more information on functional area (see Figure 8-3).

9. Where a private approach serves a high volume of traffic, additional design and/or traffic controls may need to be incorporated into the design. High volume approaches often will require channelization along the highway. Refer to Section 8.3 for details on left and right turn lanes. In some instances, the approach may require a traffic signal in order to operate safely and efficiently. The designer should work with the Region Access Management Engineer to determine solutions for high volume private approaches and potential private approaches opposite signalized intersections. Private approaches are not allowed directly opposite interchange ramp terminals.

   NOTE: All traffic signals must be approved by the State Traffic-Roadway Engineer prior to installation. Generally, only public road approaches should be considered for signalization. Avoid signalizing private approaches.

10. Type A approaches need to be designed to minimize the pedestrian longitudinal distance. This may require the design to incorporate a two-centered curve rather than a single radius when accommodating design vehicles larger than a Single Unit (SU) truck.

11. The approach design and corresponding site circulation plan should specify the entry/exit throat distance. This throat distance is critical in order to provide an efficient and functional connection between the highway and adjacent property. Throat lengths are critical for commercial and industrial type land use approaches. The Transportation Planning Analysis Unit or the Region Access Management Engineer can assist with determining the appropriate throat distance. See Figure 8-4.

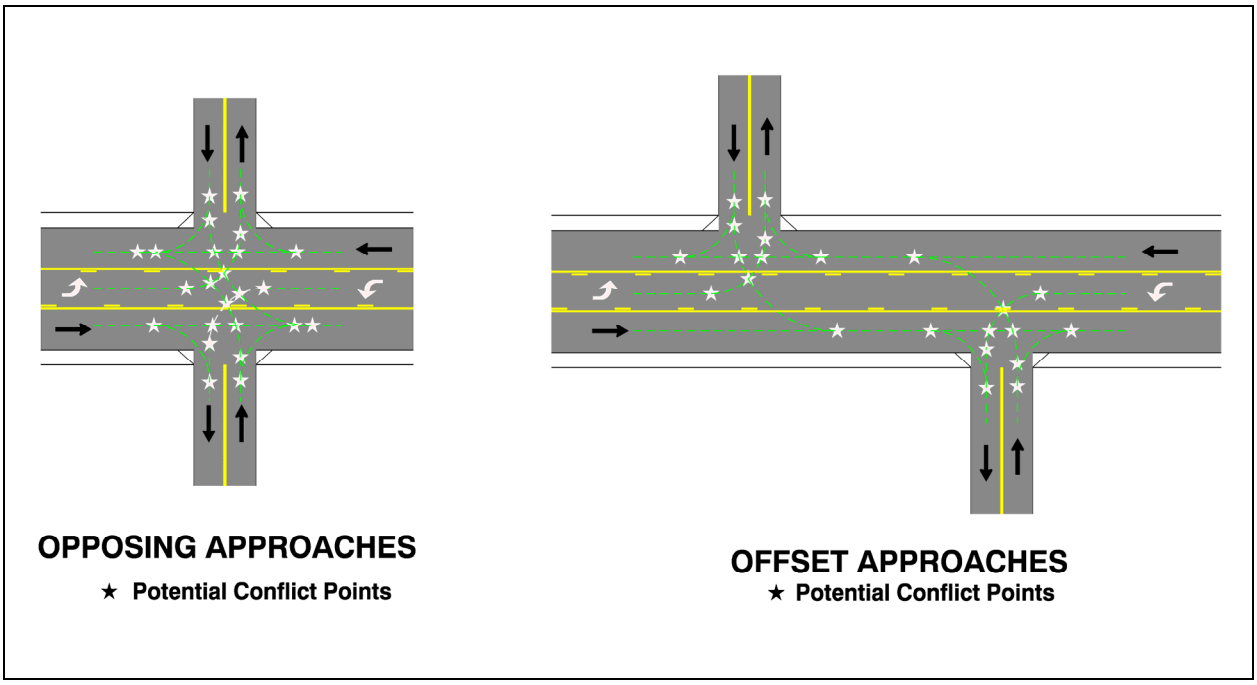


Figure 8-2: Offset Approaches

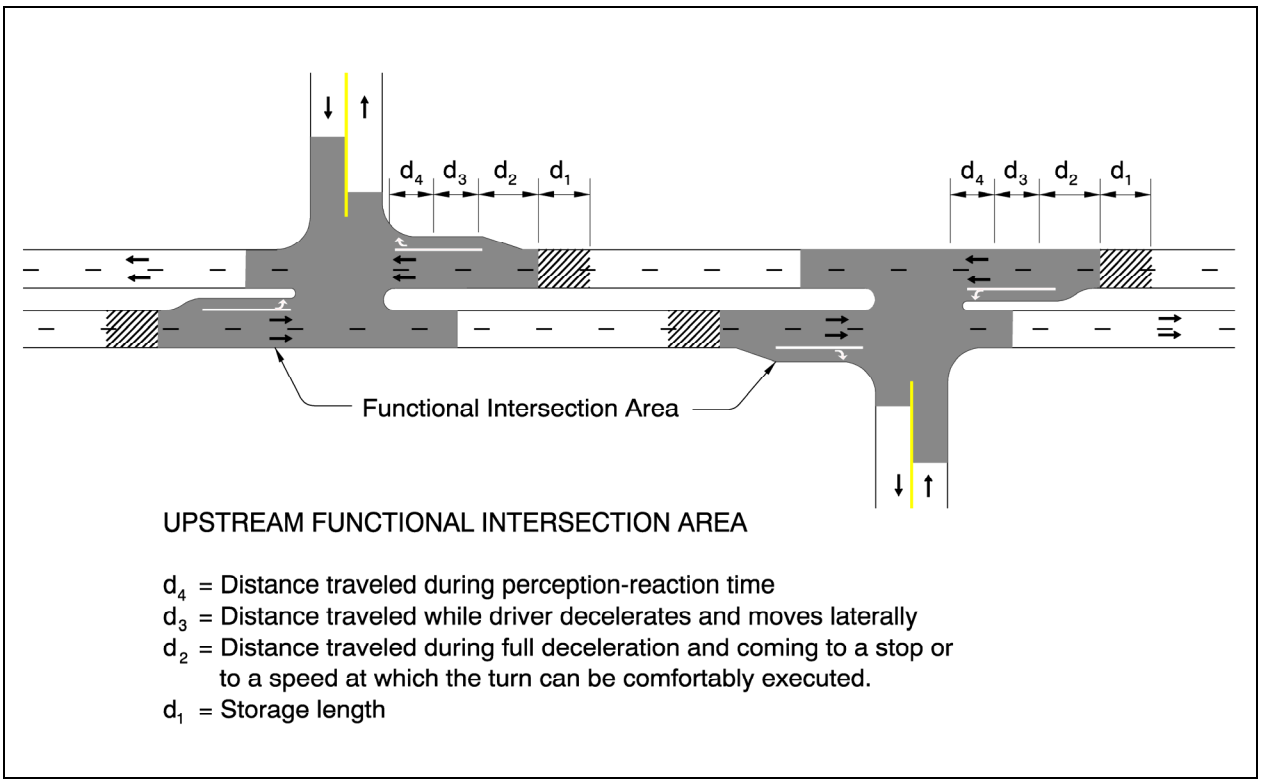


Figure 8-3: Functional Intersection Area

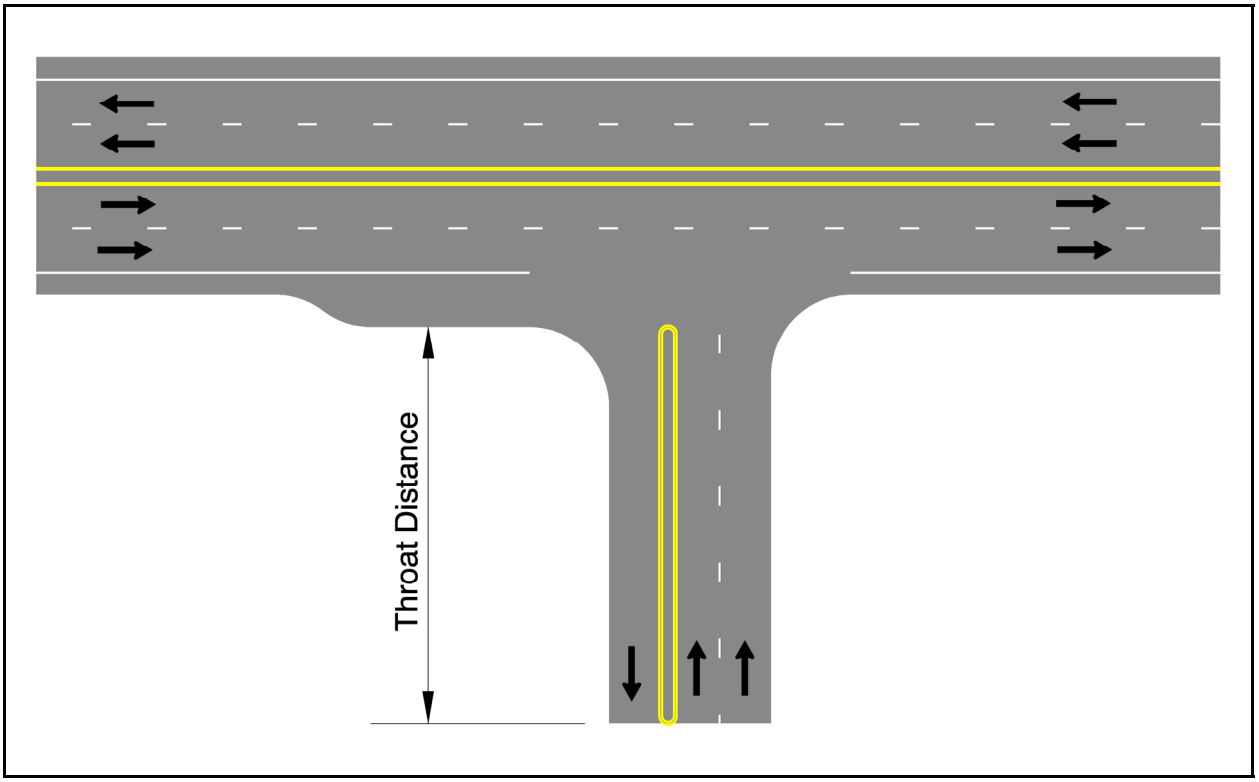


Figure 8-4: Throat Distance at Approaches

8.2.2.1 LEGAL CONSIDERATIONS FOR ROAD APPROACHES

The legal issues involved with approaches are specialized and complicated. Refer to the *"Access Management Manual"* for access rights and road approach issues. This manual includes information from *"Oregon Administrative Rules, Chapter 734, Division 51 – Access Management,"* that defines legal criteria relating to road approach permitting and design. Additional information on access management can be found in Section 2.6.

Table 8-1: Typical Private Approach Style and Width

Land Use Type	Approach Peak Hour Volume	Approach Style	Typical Throat Width[1]
SF Residential[2]	0 – 10	Dust Pan	16′
SF Residential[2]	11+	Dust Pan	24′
MF Residential	0 – 10	Dust Pan	16′
MF Residential	11 – 150	Dust Pan	24′ – 28′
MF Residential	151 – 300	Dust Pan[3]	36′ – 40'
MF Residential	301 – 399	Radius[4]	Variable[5]
MF Residential	400+	Radius	Variable[5]
Commercial	0 – 20	Dust Pan	24′
Commercial	21 – 150	Dust Pan	28′ – 32′
Commercial	151 – 300	Dust Pan[3]	36′ – 46′
Commercial	301 – 399	Radius[4]	Variable[5]
Commercial	400+	Radius	Variable[5]
Industrial		Dust Pan/Radius[6]	Variable[5]
Special Uses[7]		Radius	Variable[5]

Notes: SF = Single Family
MP = Multiple Family

[1] The typical throat widths are only to be used as guides to the designer or permit specialist. The throat width needs to be checked to ensure traffic movements are accommodated acceptably.

[2] Generally, multiple single-family residences don't share a single approach unless they are on a public road.

[3] The dust pan style designs are primarily to be used. However a radius style may be used if the traffic composition at the driveway contains a substantial number of recreational vehicles, buses, and single unit trucks, and the highway posted speed is greater than 35 mph, or access spacing each side is 660 feet or more.

[4] The radius style design should generally be used. However, a dust pan style may be considered where the highway posted speed is 30 mph or less and access spacing is 165 feet or less.

[5] The typical width is variable dependant upon approach style, design vehicle, and number of lanes.

[6] Special care should be used when determining the appropriate style. Some industrial uses operate similar to commercial uses and should use commercial style approaches and dimensions. Heavy industrial/warehouse uses that serve significant truck volumes should use a radius style.

[7] Special Uses include developments such as truck stops, amusement parks, stadiums, distribution centers, etc.

8.3 GENERAL INTERSECTION DESIGN

8.3.1 GENERAL DESIGN CONSDERATIONS

This section describes the standards and guidelines for the geometric design of traditional at-grade intersections including lane widths, shoulders, superelevation, skew angles, turning radii, left turn lanes, right turn lanes, channelization islands, curb extensions, and bicycle and pedestrian needs. Context of the roadway and roadside is important to the final intersection design. Contextual factors in the design of intersections include the adjacent land use, urban or rural condition, vehicle speeds, traffic volumes and highway operation. The ODOT Practical Design Policy of Safety, Corridor Context, Optimize the System, Public Support and Efficient Cost (SCOPE) can aid in applying context design to a project. (See Practical Design Policy)

Specific design issues and concerns related to signalized and unsignalized intersections are discussed in Sections 8.4 and 8.5, respectively. The design standards and considerations for modern roundabouts are contained in Section 8.6.

8.3.2 APPROACH GRADES

There are two types of approaches to state highways. Public road connections are one type of approach and private approaches such as driveway connections are the second category. For public roads, the approach grades of intersecting roadways with a state highway should be kept to a minimum. It is undesirable to have road connections along superelevated curved sections of state highway and these connections are discouraged. When this type of connection can not be avoided, special care must be taken by the designer to provide an adequate connection. It is preferable to have a relatively flat or slightly elevated roadway connecting with a state highway. This helps improve the visibility of the intersecting roadway and can also help control highway drainage.

In order to effectively match intersecting roadway grades with state highway grades, vertical curve alignments should be used on all approach connections. Generally the intersecting roadway's vertical alignment should match with the cross slope of the highway as long as the cross slope is less than 3 percent. Where the cross slope is equal to or greater than 3 percent a small break in the grade or vertical curve at the outer edge of shoulder not exceeding 2 percent may be acceptable. In addition, a 20 foot paved landing should be provided to aid an entering vehicle transition to the highway. The goal is to provide a connection that does not require vehicles to stop and enter the highway from a steep grade. The flatter the approach, the better, particularly for large vehicles. Due to acceleration and deceleration characteristics of various vehicle types using public roadways, grades of public road approaches at state highway connections greater than 3 percent should be avoided. However, in many locations due to

existing terrain or right-of-way constraints, constructing approach grades less than or equal to 3 percent may be costly or infeasible to accomplish. In these locations, a more practical threshold would be to provide a maximum grade on the connecting road of 6 percent. In locations where the connecting approach grade exceeds 6 percent, special care needs to be taken by the designer to provide adequate vertical transition from the steep road approach to the highway grade.

Due to typically expected operating conditions, driveway approaches to state highways can be constructed with greater differential changes in grade than public roadway connections. Figure 8-5 and Figure 8-6 provide design and layout information for an approach with sidewalk and without sidewalk. Additional information and options about the design and layout of sidewalks and driveway approaches is available from Oregon Standard Drawings. Pertinent standard drawings include RD715, RD725, RD730, RD735, RD740, RD745 and RD750.

Regardless of roadway connection type, where a marked or unmarked crosswalk exists, the cross slope should be held to 2 percent or less to meet ADA requirements. Figure 8-7 provides information about sidewalk ramps. In addition, adequate sight distance must be provided at all road connections.

NOTE: Crosswalks, whether marked or unmarked, exist across each approach to an intersection unless specifically closed by the road authority.

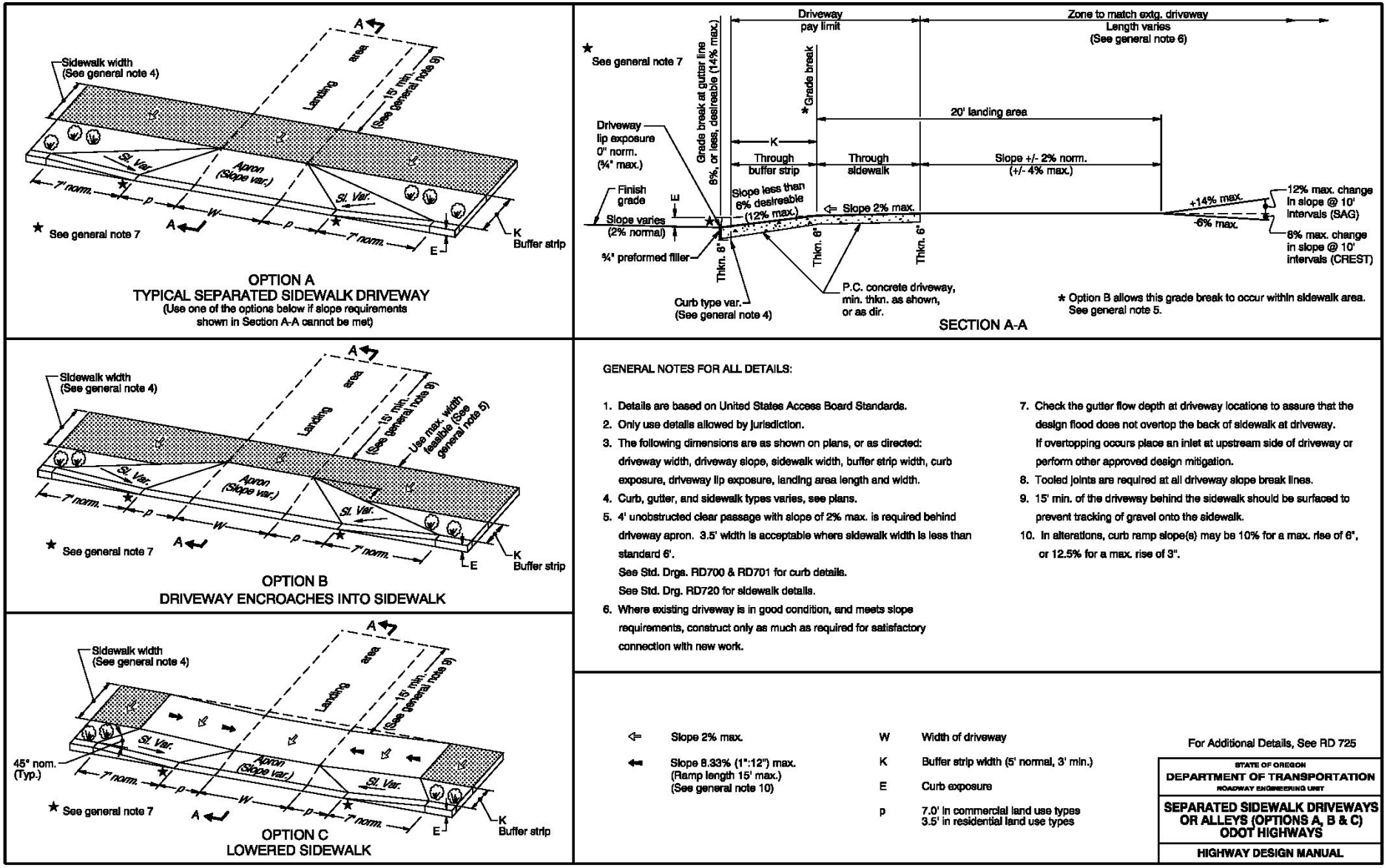


Figure 8-5: Driveway Approaches With Sidewalks

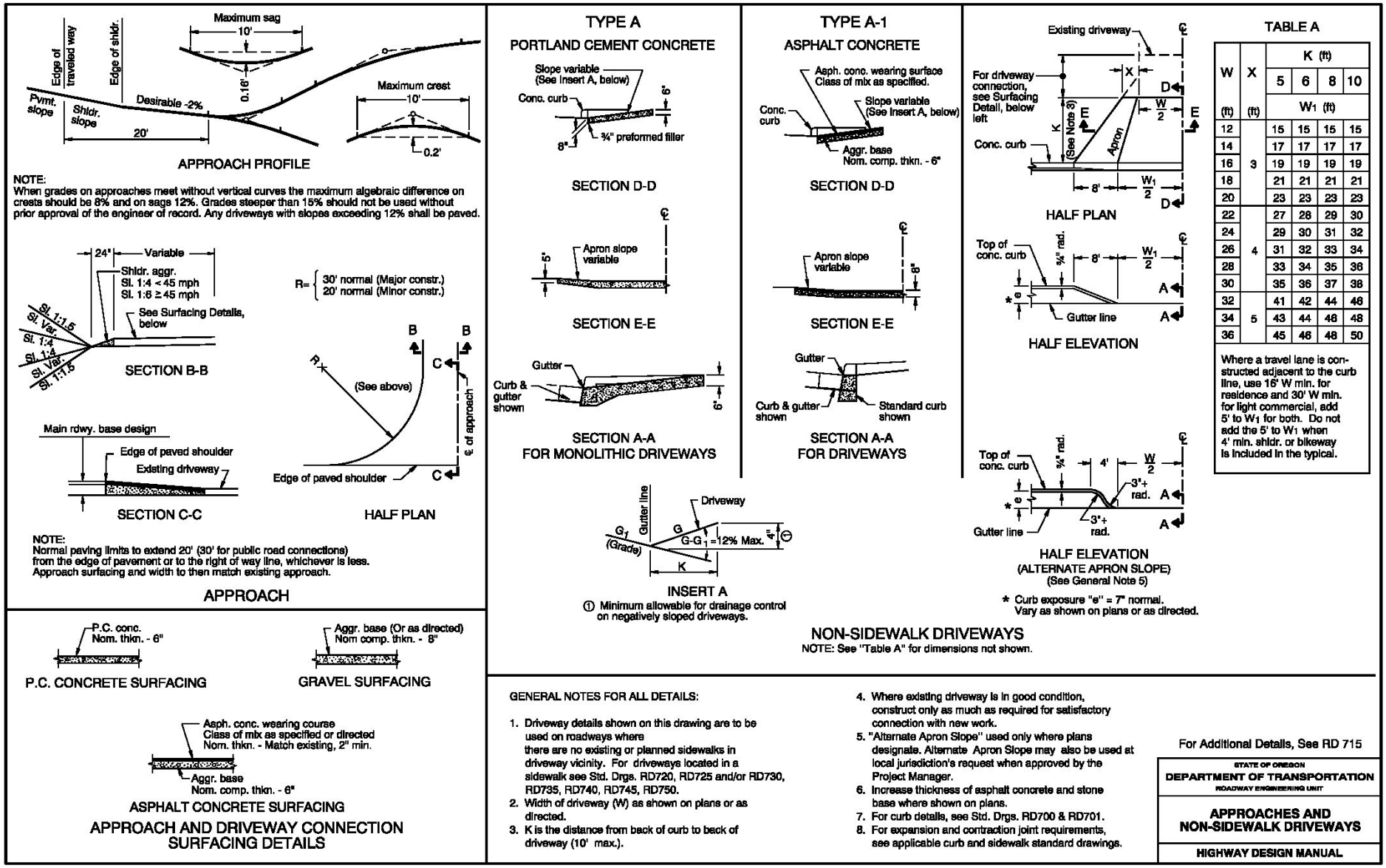
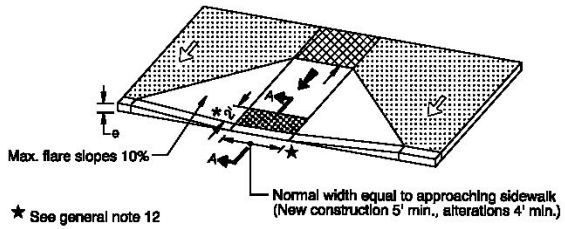
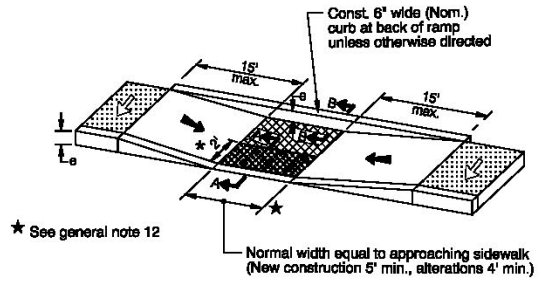


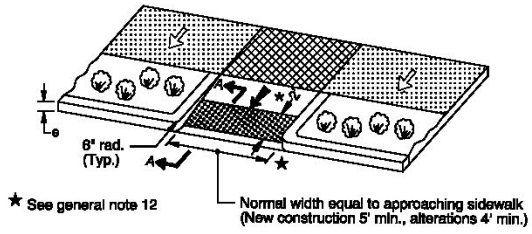
Figure 8-6: Driveway Approaches Without Sidewalks



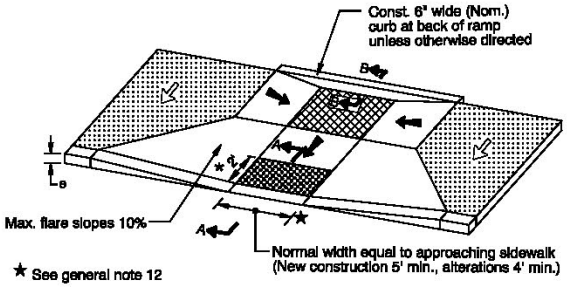
PERPENDICULAR SIDEWALK RAMP DETAIL
(Use "Parallel Sidewalk Ramp Detail" or "Combination Sidewalk Ramp Detail" when reqd. landing cannot be obtained)



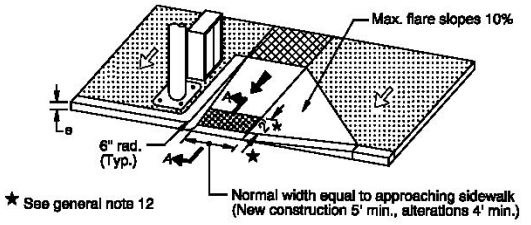
PARALLEL SIDEWALK RAMP DETAIL



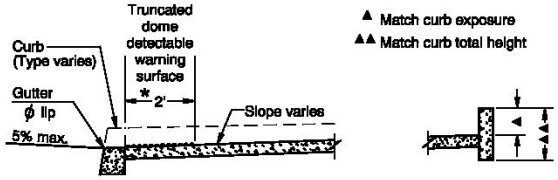
PERPENDICULAR SIDEWALK RAMP DETAIL
(THROUGH BUFFER STRIP)



COMBINATION SIDEWALK RAMP DETAIL



PERPENDICULAR SIDEWALK RAMP DETAIL (WITH SINGLE FLARE)
(Use "Parallel Sidewalk Ramp Detail" or "Combination Sidewalk Ramp Detail" when reqd. landing cannot be obtained)



SECTION A-A

SECTION B-B

Slope 2% max.

Slope 8.33% (1":12") max.
(See general note 10)

Truncated dome detectable warning surface

Landing Area (Min. level area 48" x 48")
For the purposes of this application,
a 2% maximum slope (For drainage) is considered level

* 2' See general note 5

GENERAL NOTES FOR ALL DETAILS:

1. Sidewalk ramp details are based on United States Access Board Standards.
2. See Std. Drgs. RD700 & RD701 for curbs. See Std. Drg. RD720 for sidewalks. See Std. Drgs. TM503 & TM530 for crosswalk markings, widths, etc.
3. Tooled joints are required at all sidewalk ramp slope break lines.
4. Sidewalk curb ramp slopes shown are relative to the true level horizon (Zero bubble).
5. Place truncated dome detectable warning surface in the lower 2' adjacent to traffic of throat of ramp only. For details not shown, see Std. Drg. RD759.
6. Side flares that are not part of the path of travel may be any slope.
7. Sidewalk flare is not necessary where the ramp is protected from pedestrian cross-travel.
8. For the purpose of this drawing, a curb ramp is considered "perpendicular" if the angle between the longitudinal axis of the ramp and a line tangent to the curb at the ramp center is 75° or greater.
9. Ramps for paths intersecting a roadway should be full width of path, excluding flares. When a ramp is used to provide bicycle access from a roadway to a sidewalk, the ramp should be 8' wide.
10. In alterations, curb ramp slope(s) may be 10% for a max. rise of 6", or 12.5% for a max. rise of 3".
11. For sidewalk ramp placement options, see Std. Drgs. RD756 & RD757.
12. Check the gutter flow depth at ramp locations to assure that the design flood does not overtop the back of sidewalk at ramp. If overtopping occurs place an inlet at upstream side of ramp or perform other approved design mitigation.
13. Only use details allowed by jurisdiction.

For Additional Details, See RD 755

STATE OF OREGON
DEPARTMENT OF TRANSPORTATION
ROADWAY ENGINEERING UNIT

SIDEWALK RAMP DETAILS

HIGHWAY DESIGN MANUAL

Figure 8-7: Sidewalk Ramp Details

8.3.3 TRAVEL LANE WIDTHS

Travel lane width through an intersection needs to remain constant. In general, the through travel lane width at channelized intersections is 12 feet as shown in Figure 8-9. For specific locations, the appropriate travel lane width is determined by the location (rural or urban), design speed, volume of trucks, highway designation and alignment. The rural or urban highway design chapters of this manual should be used to determine the appropriate through lane width. In Special Transportation Area (STA) designated roadway sections, 11 foot travel lane width is preferred, depending on functional classification, volume and nature of traffic, pedestrian mobility, freight mobility and accessibility goals. In other urban locations with significant constraints, 11 foot travel lane width may be allowable with approval. See Chapter 6 for guidance on the use of lane widths less than 12 feet. However, travel lane widths shall not be reduced through an intersection. Lane width approaching an intersection is to be maintained through the intersection.

When an intersection is a part of or connecting to a turning roadway, the lane widths may need to be increased to allow for large vehicle off tracking. Refer to chapters 3 and 9 of the AASHTO's *"A Policy on Geometric Design of Highways and Streets - 2011"* for more details of turning roadways.

Any reductions in existing lane widths will need to be investigated for freight mobility issues and comply with ORS 366.215, Creation of state highways; reduction of vehicle-carrying capacity. For guidance in complying with ORS 366.215, see ODOT guidance document "*Guidelines for Implementation of ORS 366.215, No Reduction of Vehicle-Carrying Capacity*" and the "*ODOT Highway Mobility Operations Manual*".

8.3.4 TRAVEL LANE ALIGNMENT

Similarly to through travel lane width, travel lane alignment should remain constant through an intersection. Shifting of lanes through an intersection is strongly discouraged and should only be done in extreme circumstances. The lane lines should line up throughout the entire intersection and not be offset. This helps to not only discourage actual lane changes through the intersection area, but also minimizes the possibility of a driver inadvertently encroaching on the adjacent lane. In cases where it is deemed necessary to shift a lane through an intersection, a maximum offset of 4 feet may be permissible. At signalized intersections, care must taken if lanes are shifted through the intersection. Excessive shifting of lanes may cause signal head mis-alignment with their respective lanes. Signal heads should be shifted to match the lane shift. If this can not be accomplished, then lane shift should be limited to 2 feet.

If shifting lanes through an intersection is necessary, it is advantageous to carry some form of lane marking, generally a skip stripe, through the intersection to inform drivers of the shift and help keep them aligned with the lanes. Review by Region Traffic staff and Technical Services Traffic-Roadway Engineering staff is required.

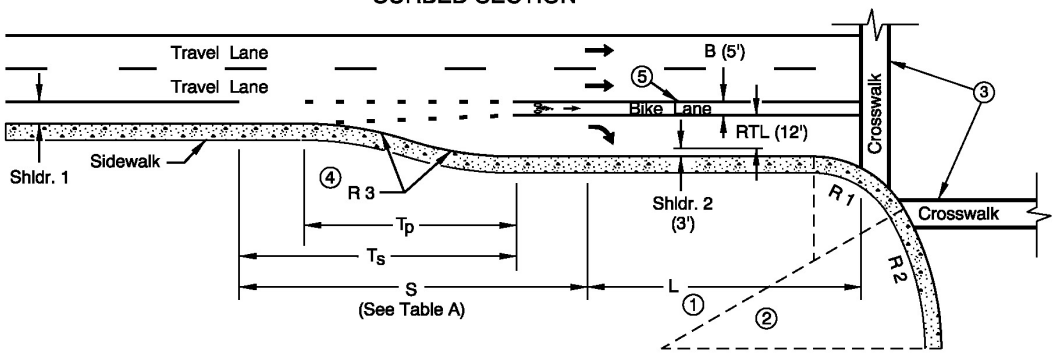
When a through lane drops downstream of an intersection, adequate length of the lane being eliminated needs to be established to allow the two traffic streams to merge safely and effectively. This distance may vary by location due to specific intersection operation, number of downstream access points, on-street parking or other constraints. Each location needs to be thoroughly investigated and an appropriate length for full lane width needs to be determined. Failure to provide adequate length for necessary maneuvers may impact intersection operation and expected capacity due to uneven lane balance. Anticipated lane utilization through the intersection may not occur if it is too difficult to merge downstream. Drivers who know the intersection may be reluctant to use the lane that is dropping if they have had difficulties merging downstream in the past and they may choose to merge into the downstream through lane prior to the intersection. This is particularly true for locations where a lane is added just prior to the intersection to increase intersection capacity and then immediately dropped downstream of the intersection too abruptly. Providing appropriate downstream lane length can be an effective tool to increase intersection capacity. Follow Manual on Uniform Traffic Control Devices (MUTCD) and ODOT Pavement Marking Design Guidelines for striping and signing requirements for lane reduction and merge layout.

8.3.5 SHOULDER WIDTHS

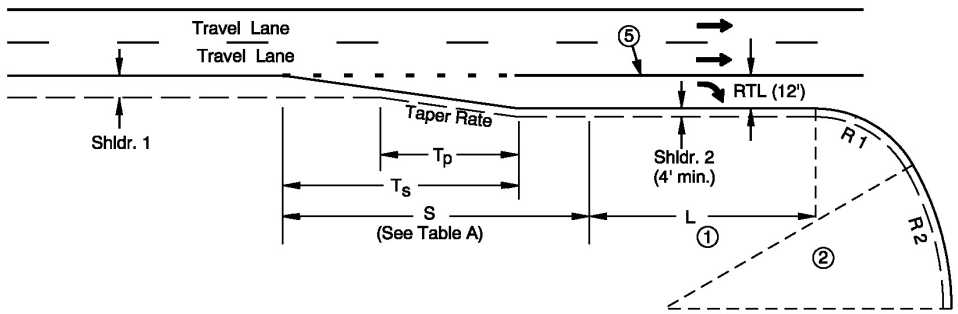
As with travel lanes, the width of shoulders should generally remain constant through an intersection. However, two-lane highways that are flared to provide left turn channelization may require shoulder width modifications Standard shoulder width should be utilized through intersections. In constrained locations where left turn channelization is being considered, the shoulder width may be reduced, but shall be no less than 4 feet. Reduction of shoulder width below the standard 6 foot width may require a design exception. When reducing shoulder width, bicycle accommodation needs to be addressed. The Oregon Bicycle and Pedestrian Design Guide provides information about shoulder widths and consultation with ODOT Bicycle and Pedestrian staff may provide additional appropriate design options. Shoulder widths will also require modifications where the intersection includes a right turn lane. In these situations, the shoulder should be designed to match the dimensions of Figure 8-8.

To be used in locations where a traffic investigation has determined a right turn lane to be warranted.

CURBED SECTION



NON-CURBED SECTION



1. Minimum Storage Length 'L'
   Unsignalized:
   Provide 50' minimum beyond end of 'T_S' distance.
   Signalized:
   Determined by traffic study.
2. Compound radii accomodate large design vehicles, yet minimize pedestrian crossing distance. See discussion on accommodating vs designing for the design vehicle under Turning Radii in this chapter.
3. See Traffic Section for placement of crosswalk.
4. R 3 for reversing curves used only in curbed sections.
5. Through bike lane is generally constructed and striped in sections with a posted speed of 45 mph or less, even where there is no approaching shoulder or bike lane. See Oregon Bicycle and Pedestrian Design guide.

TABLE A

Design Speed (mph)	Taper Rate	S (ft.)
25	8:1	155
30	8:1	155
35	9:1	175
40	10:1	190
45	11:1	215
50	12:1	265
55	13:1	320
60	14:1	385
65	15:1	450
70	15:1	520

DESIGN EQUATIONS AND VARIABLES

S = Deceleration distance for initial speed

T_P = Pavement or Curb Taper Length
= Taper Rate x [(B + RTL) + (Shldr. 2 - Shldr. 1)]

T_S = Striping Taper Length
= Taper Rate x (B + RTL)

R 3 = Reversing Curve Radius

$$= T_P \times \frac{\text{Taper Rate} + \frac{1}{\text{Taper Rate}}}{4}$$

B = Bicycle Lane Width (5' typ.)

RTL = Right Turn Lane Width (12' typ.)

DESIGN PROCEDURE

1. Decide on values for 'B', 'RTL', 'Shldr.1' and 'Shldr.2'
2. Determine 'S' from Table A
3. Calculate 'T_S'
4. Determine 'L' (see note 1). For unsignalized intersections, check to see that $S - T_S \geq 50'$. If so, L = 0. If not, $L = 50 - (S - T_S)$.
5. Calculate 'T_P' for layout of curb or edge of pavement

STATE OF OREGON
DEPARTMENT OF TRANSPORTATION
ROADWAY ENGINEERING UNIT

RIGHT TURN CHANNELIZATION

HIGHWAY DESIGN MANUAL

Figure 8-8: Right Turn Channelization

8.3.6 INTERSECTIONS ON CURVES AND SUPERELEVATION

It is undesirable to have an intersection located within a horizontal curve and the practice should be avoided. Intersections on curves present design challenges that affect superelevation, sight distance, driver comfort and vehicle stability. However, in many existing situations, intersections are present within highway curves and in many of these locations, these connections cannot be effectively relocated. Signalized intersections in curves compound operational problems, as well. Stopping traffic on steep cross slopes determined by main line design superelevation needs is undesirable due to the potential for slippage under ice conditions or potential load shifting on trucks.

When an intersection occurs within a highway curve, the highway superelevation should be kept to a minimum. However, the highway still needs to provide for safe movement of traffic through the intersection at highway speeds. As a result, the designer must balance the superelvation need of traffic on the main line in free flow conditions with operational issues of the intersection. In these types of locations, some designers prefer to merely limit maximum superelevation to 4%. However, in some cases, trying to hold the superelevation to 4% or less may result in design speeds less than desirable for a specific highway. A better solution is to determine an appropriate superelevation for a specific location based on needs at that location.

At a minimum, the superelevation at an intersection should provide speeds determined from the Comfort Speed matrix shown in Table 3-5 equal to the desirable design speed. This means that if the design speed for the highway segment is 45 mph, then the comfort speed for the curve at the desired superelevation must be at least 45 mph.

Example:

Using Table 3-4 Suburban Superelevation & Spiral Lengths and a design speed of 45 mph with an 8 degree curve, the design superelevation would be 6%. This may be an undesirable condition with a signalized intersection on a curve. An alternative is to use the Comfort Speed values from Table 3-5. Entering the table for an 8 degree curve and following across the row until the column for 45 mph is reached returns a 4% superelevation. This would reduce the design superelevation by 2% and may be an acceptable option.

When using an alternate superelevation design, care must be taken to determine that reducing superelvation does not compromise the overall geometry of the alignment and subsequently create a new problem while attempting to solve a current one. A design exception will be required to utilize an alternate superelevation design based on Comfort Speed in relation to Design Speed. It is critical to ensure that connections on the high side of a superelevated highway curve provide an approach with adequate sight distance. Ideally, intersection sight distance should be provided. Where this is not feasible or practical, as a minimum, stopping sight distance must be provided.

Another important consideration in designing a road connection on the high side of a horizontal main line curve is the comfort factor for side road traffic. Operation of the main line is the first

concern, but it is important to create a comfortable transition across the superlevation for the traffic entering onto the main line. Where possible, keeping supereleavation to a minimum on the main line while establishing grades on the connecting road to minimize vertical and lateral movement inside the vehicle entering onto the main line is desirable.

In addition to consideration of vehicles entering from the side road to the main line, main line traffic turning dynamics at intersections on curves must be evaluated as well. Main line turning vehicle dynamics and driver comfort also benefit from minimum superelavation when making turns onto side roads. Main line vertical grade can have great effect on turning dynamics. Negative (downhill) grades in conjunction with horizontal curvature and its respective superelevation can exacerbate turning forces acting on a vehicle. Not only can these forces be uncomfortable for drivers and passengers, in the case of trucks or other vehicles with higher centers of gravity like RVs and buses, these forces can cause loads to shift or, in extreme cases, cause roll over crashes.

When it is necessary to design or improve an intersection located on a horizontal curve, it is important to carefully analyze the interaction of the horizontal curvature and superelevation with all intersecting grades, grade breaks and vertical alignments on both the side road and the main line in relation to anticipated vehicle turning movements and dynamics. It is important to keep these forces and reactions to a minimum and within acceptable levels to ensure safe and effective operation of the intersection.

8.3.7 SKEW ANGLES

Roadway connections with a state highway should intersect at a 90 degree angle. 90 degree intersections maximize sight distance, improve safety, increase efficiency, and improve operations and safety of bike and pedestrian movements. In some situations however, obtaining a 90-degree intersection is impractical or excessive in cost. Where this is the case, skewed intersections may be unavoidable. Skew angles of up to 30 degrees from perpendicular may be justified. However, the amount of skew should be held to a minimum. Figure 8-17 shows an intersection with excessive skew and the intersection reconfigured to improve skew. Figure 8-18 shows skew configuration with right turn lanes and islands to accommodate pedestrian movements. The presence of large trucks needing to negotiate this type of intersection can have direct effect on the final design layout.

Several factors can help determine the amount of skew that is acceptable for any particular intersection. Intersections with all or most of the following characteristics might justify allowing a skew angle of up to 30 degrees.

1. Highway speeds are low, generally 35 mph or less;
2. Volumes on both the highway and intersecting roadway are low (at or below left or right turn channelization warrant limits);
3. Large vehicle turning movements are minimal;
4. Intersecting roadway has a functional classification of minor collector or below, and

5. Intersection sight distance is available.

For all other intersections not meeting criteria on this list, the maximum skew should be held to 15 degrees from perpendicular. Refer to AASHTO's *"A Policy on Geometric Design of Highways and Streets - 2011"*, pages 9-26 and 9-27, for possible alignment solutions to skewed intersections.

8.3.8 TURNING RADII

Turning radii are one of the most important design elements of intersections. The operations, safety, and efficiency of an intersection are controlled by the turning movements. If the turning vehicles are geometrically limited from completing the maneuver properly, the intersection will break down, capacity is limited, and accident potential will increase.

The appropriate design vehicle must be identified prior to designing the intersection turning movements. Selection of the appropriate design vehicle can sometimes be difficult. Issues to take into consideration in choosing a design vehicle include number and type of trucks, functional classification of the intersecting roadways, surrounding land use, consideration of future changes in land use and traffic, freight route designation, etc. See Chapter 2 for additional information on design vehicle selection. After determining the appropriate design vehicle, a decision needs to be made as to the level of design accommodation to be made. In other words, is the intersection radii to be designed for the design vehicle or merely to accommodate the design vehicle? The concept of designing for the design vehicle is to provide a path for the vehicle that is free of encroachments upon other lanes. Providing a design that only accommodates the design vehicle means that some level of encroachment upon other lanes is necessary for the vehicle to make a particular movement (see Figure 8-1). An example of an intersection that would need to be designed for trucks with no encroachment into adjacent lanes would be a stop controlled intersection with a state highway, the highway being two lane or multi-lane with higher speeds and/or high traffic volumes. If a traffic study concludes that finding a gap in multiple traffic flows is not possible, the intersection would need to be designed for the design vehicle so that the truck driver can turn from his lane into a single lane. Other factors to consider in turning radii are the affects on pedestrians and bicycles. Large radii create long crossing distances with increased exposure times. These conditions negatively impact pedestrian and bicyclist safety and may add time to signal timing cycles. Large radii also encourage motorists to take turns at higher speeds that can have an effect on intersection safety as a whole. In general, large vehicles are a small percentage of the vehicle types and users of an intersection. Designing intersections for large vehicle maneuverability may be of benefit for the large vehicle, but it tends to make the intersection less safe for the majority of the users of the intersection. Therefore, in consideration of the overall safety of the intersection, the design should only accommodate large vehicle operation in most cases. When it is necessary to design the intersection with large radii for larger vehicles, a balance needs to be obtained between the necessary radii and impacts to all intersection users.

Another item that must be decided is the turning radius of the design vehicle. The turning radius of the design vehicle determines the ease and comfort of making the turning maneuver. The smaller the turning radius, the larger the off-tracking of the vehicle and the slower the

speed. Forcing large vehicles to use very small turning radii forces the driver to perform a very slow maneuver that may not be in the best interests of the operation of the intersection. Generally the radius chosen is in line with the surrounding culture. Tighter radii are chosen for low and/or urban speeds, while larger radii are selected for higher speeds and rural intersections.

Once the design vehicle is selected and the level of design accommodation determined, then the intersection radii can be designed. Intersection radii should be kept as small as possible to minimize the size of the intersection and the pedestrian crossing distance. Any time the design vehicle is larger than a Single Unit (SU) truck or a bus, the designer may need to consider using a two-centered curve. Off-tracking templates or automated off-tracking programs should be used to determine the vehicle path. Once this path is identified, a two-centered curve can be developed which closely emulates this path. The designer may need to look at a range of vehicle turning radii and the subsequent intersection designs. This allows the designer to select the best design for the design vehicle while minimizing the size of the intersection.

Designers are encouraged to keep the size of intersections to a minimum. Often when accommodating large trucks, the intersection radii become very large. This can substantially increase the size of the intersection. Larger intersections generally have greater accident potential, are difficult to delineate, can be confusing, require more right-of-way, and significantly increase pedestrian and bicycle crossing times and distances.

8.3.9 LEFT TURN LANES

Providing a left turn lane at an intersection will significantly improve the safety of the intersection. Eliminating conflicts between left turning vehicles decelerating or stopping and through traffic is an important safety consideration. A left turn lane must be provided at all non-traversable median openings and they are strongly recommended to be installed at other intersections meeting the installation criteria. The left turn lane installation criteria are different for signalized and unsignalized intersections. Refer to Section 8.4, Signalized Intersections, and Section 8.5, Unsignalized Intersections, for the appropriate siting criteria. For additional information about siting criteria for left turn lanes, see the ODOT Analysis and Procedures Manual (APM). (http://www.oregon.gov/ODOT/TD/TP/APM.shtml)

Left turn lanes shall be 12 feet wide plus the appropriate traffic separator width and shy distance when required. The installation of a traffic separator at left turn lane locations is critical when there are access points to adjacent properties along the length of the left turn lane. The separator will protect the left turn lane operation and safety by eliminating the opportunity for vehicles to cross it when entering and exiting adjacent accesses. The width of the traffic separator is determined by several factors. If the median includes a raised curb design, the traffic separator width shall be a minimum of 4 feet. When pedestrians are to be accommodated on the raised portion of the median with separate phases for the crossing maneuver, the raised traffic separator width shall be 6 feet minimum. Medians that use raised curb also need to provide the appropriate shy distance from the curb and adjacent through travel lanes. The width of striped traffic separators is determined by the design speed of the highway and the

type of land use area. For design speeds of 55 mph or less, the striped separator shall be 2 feet and 4 feet for design speeds of 60 mph or greater. For more information on median design, refer to Section 4.3.

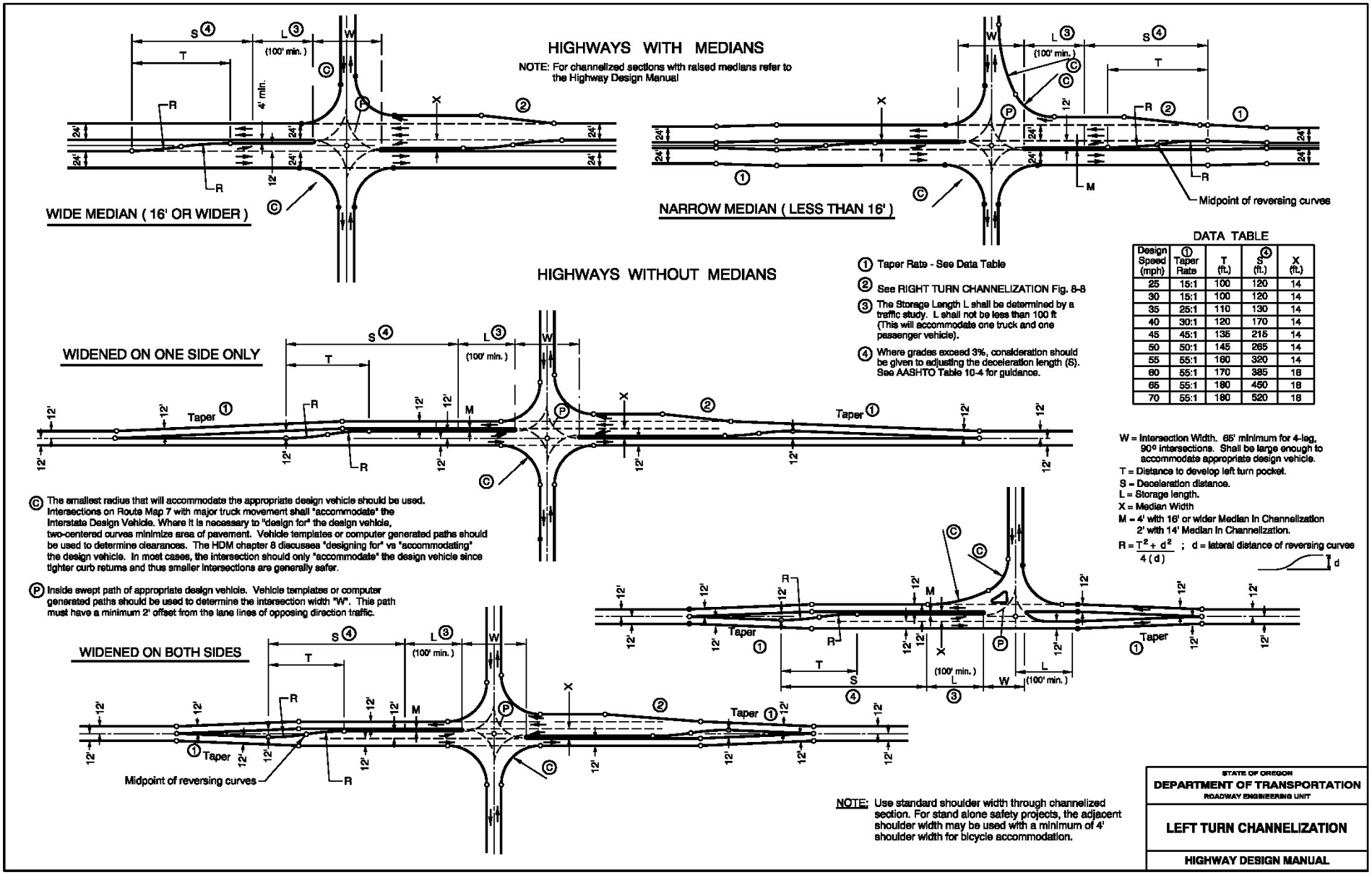
Development of left turn lanes should be in conformance with Figure 8-9. However, where the median width is developed non-symmetrically, a reversing curve may be used in lieu of the straight speed tapers. The reversing curve option can reduce the overall widening thereby saving construction costs and possibly saving right of way or significant features. Figure 8-9 depicts the standard left turn channelization design. Figure 8-10 depicts the reversing curve channelization option.

Left turn lanes should be striped in accordance with the ODOT Pavement Marking Design Guidelines. Essentially this means that the reversing curve entry taper shall be used for:

1. All dual left turn lanes;
2. All left turn lanes developed from sections without medians or with narrow medians, and
3. All left turn lanes located within wide median sections or CTWLTLs that have design speeds greater than 45 mph.

It is critical to the operation of intersections to provide adequate storage length for left turning vehicles out of the through traffic lanes. At a minimum, the turn lane should provide 100 feet of storage. The Region Traffic Engineering Unit and the Analysis Procedures Manual (APM) should be consulted to determine the appropriate storage length for specific intersections. For specific analysis procedure questions or interpretation of the APM or for complex projects requiring additional study, contact the ODOT Transportation and Analysis Unit (TPAU) for guidance or technical help on the particular project or methodology.

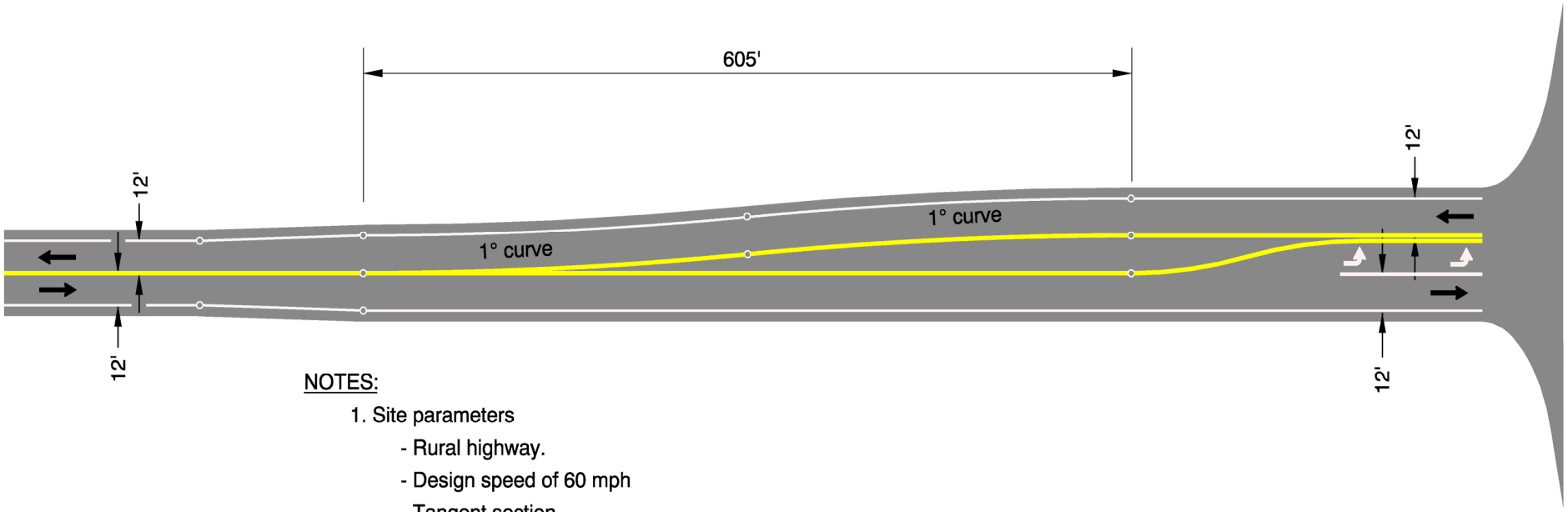
In some instances, dual left turn lanes may need to be considered. When designing dual left turn lanes, there must be dual receiving lanes on the connecting roadway with adequate length downstream prior to any merge points. The designer must determine the appropriate design vehicles to use for side-by-side operation through the turning movement. In rare locations, like at freeway ramp terminals leading to truck stops or warehousing districts, the design may need to be two WB-67 vehicles making the turn simultaneously. However, in most locations, a WB-67 and an SU vehicle side-by-side is adequate for design. In other locations where truck volumes are low, an SU vehicle and a passenger vehicle may be sufficient.



DATA TABLE

Design Speed (mph)	① Taper Rate	T (ft.)	④ S (ft.)	X (ft.)
25	15:1	100	120	14
30	15:1	100	120	14
35	25:1	110	130	14
40	30:1	120	170	14
45	45:1	135	215	14
50	50:1	145	265	14
55	55:1	160	320	14
60	55:1	170	385	18
65	55:1	180	450	18
70	55:1	180	520	18

Figure 8-9: Left-Turn Channelization



NOTES:

1. Site parameters
   - Rural highway.
   - Design speed of 60 mph
   - Tangent section
2. Used only when widening for channelization is done on one side of the highway.
3. Survey crew must lay out stripe locations.

Figure 8-10: Reversing Curve Option for Left-Turn Channelization

8.3.10 RIGHT TURN LANES

Speed differential between right turning traffic with through traffic can create significant safety problems at intersections. To reduce this conflict, installation of right turn lanes may be appropriate at some intersections. Right turn lanes also help improve traffic operations and mobility standards at some intersections. Installation of right turn lanes should be considered at intersections that meet the siting criteria. For information about siting criteria for right turn lanes, see the ODOT Analysis and Procedures Manual (APM).
(http://www.oregon.gov/ODOT/TD/TP/APM.shtml)

Not all intersections that meet the siting criteria should have right turn lanes installed. In urban situations, only significant public roads and large private approaches should be considered for installation of a right turn lane. A proliferation of right turn lanes along an urban arterial is undesirable for bicycles and pedestrians, creates an aesthetically unpleasing typical section, and may not improve safety throughout the section. Multiple right turn lanes could, in effect, create a continuous right turn lane, which is not desirable on state highways.

Right turn lanes should be designed in conformance with Figure 8-8. The right turn lane should be 12 feet wide with a shoulder of 3 feet or 4 feet for curbed or non-curbed sections respectively. In some instances right turn lanes could be considered a turning roadway. Turning roadways are usually thought of in relation to interchange ramps. However, according to AASHTO, turning roadways include interchange ramps and intersection curves for right-turning vehicles. The AASHTO publication, "*A Policy on Geometric Design of Highways and Streets - 2011*" has extensive information on turning roadway design including sections on minimum radii, control radii, corner islands, minimum edge of traveled way, lane configuration and swept paths.

When designing an urban right turn lane, through bicyclist movements need to be accommodated. By adding a bike lane to the left of the right turn lane, conflicts between right turning vehicles and through cyclists can be minimized. In addition, providing the bike lane between the through travel lane and the right turn lane better aligns the cyclist with the downstream shoulder or continuation of the established bike lane. However, creating a bike lane between the through lane and the right turn lane establishes a conflict point further back from the intersection where the paths of right turning vehicles and cyclists must cross. In this conflict area, the bike lane is generally marked with short skip striping. However, more recently, the MUTCD and FHWA have allowed this area to be colored green as an experimental condition to draw more attention to the conflict area. Region Traffic and Roadway sections, ODOT bicycle and pedestrian coordinators and the ODOT, Technical Services, Traffic-Roadway section should be consulted for current guidance if it is determined that using this experimental treatment in this location would be beneficial.

The standard width for a bike lane between a through travel lane and a right turn lane is 5 feet. This width is narrower than a standard bike lane against a curb. However, it is a minimum width and if the bike lane is too wide, it may appear to vehicle drivers as an added lane. Also, width added to a bike lane increases the overall width of the roadway section that must be crossed by pedestrians. Width of the right turn lane is critical as well. The standard width is 15

feet (12′ lane, 3′ shoulder) from the adjacent travel lane or bike lane to curb for an urban right turn lane. The additional 3 feet provides space for truck off-tracking and minimizes the need for a right turning truck to encroach on the adjacent lane when making the turn. In some instances, a 3 foot shoulder may not be adequate and additional width might be needed. However, that additional width has consequences. Right turn lane width in conjunction with bicycle lane width is a balance between providing enough space for the respective vehicle's lane use, but minimizing the crossing distance for pedestrians at an intersection.

In some instances, dual right turn lanes may need to be considered. When designing dual right turn lanes, there must be two lanes on the connecting roadway to turn into and there must be adequate length provided downstream before any lanes merge. The designer also must determine the appropriate design vehicles to use for side-by-side operation through the turning movement. In rare locations, like at freeway ramp terminals leading to truck stops or warehousing districts, that may need to be two WB-67 vehicles making the turn simultaneously. However, in most locations, a WB-67 and an SU vehicle side-by-side is adequate for design. In other locations where truck volumes are low, an SU vehicle and a passenger vehicle may be sufficient. When considering dual right turn lanes as an option, the Region Traffic Section should be consulted for input. Dual right turn lanes are also difficult for pedestrians and bicyclists to navigate. The Oregon Bicycle and Pedestrian Design Guide provides information in regards to dual right turn lanes. The ODOT Bicycle and Pedestrian coordinator should be consulted for guidance as well.

8.3.11 AT-GRADE RIGHT TURN ACCELERATION LANES

At-grade intersections generally should not have short tapers or acceleration lanes constructed for vehicles entering the state highway from a crossroad or another state highway. Acceleration lanes are generally only provided at grade separated facilities. However, in some situations acceleration lanes may be justified. The following criteria outlines where at-grade right turn acceleration lanes can be considered. All of the criteria must be satisfied and requires joint approval from the State Traffic-Roadway Engineer through the design exception process.

1. The posted speed on the main highway shall be 45 MPH or greater.

2. The V/C ratio of the right-turn movement without the acceleration lane shall exceed the maximum value listed in Tables 6 and 7 of the OHP for the corresponding highway category and location.

   (a) Exception 2a: If trucks represent at least 10% of all right-turning vehicles entering the highway, then the V/C criteria may be waived.

   (b) Exception 2b: If substandard sight distance exists at an intersection or right-turning vehicles must enter the highway on an ascending grade of greater than 3%, then the V/C criteria may be waived.

   (c) Exception 2c: If crash data in the vicinity of the intersection shows a history of crashes at or beyond the intersection attributed to right-turning vehicles entering the highway, then the V/C criteria may be waived.

3. The peak hour volume of right-turning vehicles from the side street onto the state highway shall be at least 10 vehicles/hour for Rural Expressways and 50 vehicles/hour for all other highways.

4. No other access points or reservations of access shall exist on both sides of the highway within the design length, taper, and downstream from the end of the taper within the decision sight distance, based on the design speed of the highway.

   (a) Exception 4a: If positive separation between opposing directions of traffic exist such as raised medians or concrete barriers, then access control is only needed in the direction of the proposed acceleration lane.

The State Traffic-Roadway Engineer shall determine if a right-turn acceleration lane proposal meets the above criteria. Proposals should be submitted to the State Traffic-Roadway Engineer and include an engineering investigation with data supporting the above criteria and a drawing encompassing the intersection and design length of the acceleration lane showing all access points and reservations of access to the highway. Only proposals for right-turn acceleration lanes from public streets will be considered. All right-turn acceleration lane proposals shall require the approval of the State Traffic-Roadway Engineer.

Special consideration should be given to cyclists and pedestrians. Acceleration lanes create an unexpected condition for both pedestrians and cyclists. Every reasonable effort should be made to create conditions that make the crossing safer and easier for pedestrians and cyclists. The acceleration lane shall be designed in accordance with Figure 8-11 "Right Turn Acceleration Lane from At-Grade Intersection".

Free-flow acceleration lanes may be considered in rural or suburban areas provided the turning radius is tightened and the angle of approach is kept as close to a right angle as possible. These combined elements will force right-turning drivers to slow down and look ahead, where pedestrians and bicyclists may be present, before turning and accelerating onto the roadway.

RIGHT TURN ACCELERATION LANE FROM AT GRADE INTERSECTION

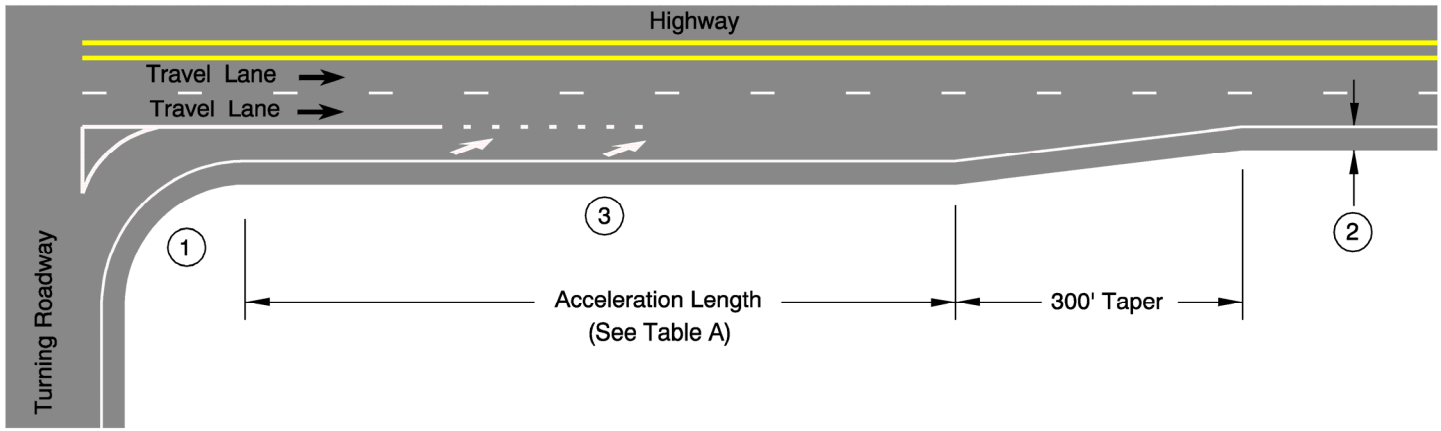


TABLE A
MINIMUM ACCELERATION LENGTH

Design Speed of Highway (mph)	Design Speed of Turning Roadway	
	Stop Condition	15 mph
50	720	660
55	960	900
60	1200	1140
65	1410	1350
70	1620	1560

TABLE B
MAINLINE GRADE AJUSTMENT FACTORS

Design Speed of Highway (mph)	% Grade	Upgrade	Downgrade
50	3% to less than 5%	1.3	0.65
55		1.35	0.625
60		1.4	0.6
65		1.45	0.6
70		1.5	0.6
50	5% and over	1.5	0.55
55		1.6	0.525
60		1.7	0.5
65		1.85	0.5
70		2.0	0.5

NOTES:

1. Radius shall accomodate design vehicles, yet minimize pedestrian crossing distance. Compound radii are recommended for larger design vehicles. Radii are measured to the edge of travel lane or face of curb.

2. Design shoulder width as required per Highway Design Manual

3. Use Table B to determine acceleration lenght adjustment due to grade.

STATE OF OREGON
DEPARTMENT OF TRANSPORTATION
ROADWAY ENGINEERING UNIT

RIGHT TURN ACCELERATION LANE FROM AT GRADE INTERSECTION

HIGHWAY DESIGN MANUAL

Figure 8-11: Right Turn Acceleration Lane from at Grade Intersection

8.3.12 MEDIAN ACCELERATION LANES

For ODOT purposes, a median acceleration lane is a lane added to the median of a roadway at an un-signalized intersection to allow left turning vehicles from a side road to gain speed and merge with main line traffic. Median acceleration lanes may seem like a reasonable solution to left turn problems onto busy, high speed roadways and, in some locations, they may be an acceptable feature. However, their use should be reserved for locations with specific needs. Improper installation of a median acceleration lane may create unanticipated problems greater than the problems the installation is attempting to solve. Any location where a median acceleration lane is proposed must be analyzed carefully before a median acceleration lane is considered to be appropriate. Overall, there is little definitive research or information available on the use or effectiveness of median acceleration lanes. What does seem to be known, however, is that location is of critical importance to the effective function of a median acceleration lane. Therefore, site specific analysis is paramount in determining the appropriateness of installing a median acceleration lane.

Median acceleration lanes function best on rural, multi-lane, free flowing roadways with ample median width and decision sight distance to accommodate not only the turning movements of all vehicle types, but to also provide the acceleration lane itself. Median width must be provided over a long enough distance to allow the accelerating driver to choose a gap in the traffic stream and merge smoothly prior to the end of the median acceleration lane. Median acceleration lane length will likely need to be longer than typical right side acceleration lane length in order to ensure adequate, comfortable and safe merge maneuvers into the traffic stream. Additional run-out length should be provided downstream of the median acceleration lane taper. This will provide a "bail out" area or escape route in the event that no adequate gap is available for the accelerating vehicle in the main line traffic stream. Median acceleration lanes are not appropriate for two lane roadways on the state highway system and shall not be installed on such facilities in either rural or urban locations. Figure 8-12 and Figure 8-13 provide information about Median Acceleration Lane layout.

Although not recommended, it may be possible to install a median acceleration lane on some limited access, divided, urban arterials or expressways with posted speeds of 45 mph or greater. However, this type of installation must be considered carefully. Median width and intersection spacing must be appropriate to allow the acceleration lane to function. In addition, there shall be no right side access points to the main line highway along the length of the median acceleration lane or within decision sight distance of the left side merge taper. Right side accesses along a section of roadway with a median acceleration lane on the left side create the scenario of the main line traffic being impacted from both sides of the roadway at the same time. Median acceleration lanes shall not be installed in locations with posted speeds below 45 mph. When speeds are below 45 mph, the differential of an accelerating vehicle and the traffic stream are not as great and a median acceleration lane does not provide added benefit.

As discussed in the preceding paragraphs, in limited situations, a median acceleration lane may provide an incremental improvement to a multi-lane expressway by providing left turning

vehicles an opportunity to accelerate and reduce speed differential before entering the traffic stream. This is particularly true where there are large numbers of left turning trucks. Where sufficient gaps exist in the main line traffic stream, a median acceleration lane is not needed and the cost of installation as well as potential environmental impacts of adding new impervious surface may not be justified. However, where there are few gaps in the main traffic stream and there is a high demand for left turning trucks or other large vehicles like RVs, motor homes or buses from the side road, a median acceleration lane may serve as an acceptable interim solution. A median acceleration lane is not a typical design. Contact Technical Services Roadway staff for information regarding the installation of median acceleration lanes. Before any median acceleration lane can be installed on the state highway system, approval from the State Traffic-Roadway Engineer must be obtained.

Consideration may be given to install a median acceleration lane when all of the following criteria are met:

1. A multi-lane, divided expressway or arterial highway with a posted speed of 45 mph or greater
2. Adequate Median width to allow for desirable dimensions as shown in Figure 8-12 and Figure 8-13
3. Large left turning volume from side road – particularly truck volumes and recreational vehicle
4. Insufficient gaps or inadequate intersection sight distance (Particularly AASHTO B1, Right Side)
5. No right side accesses onto main line along the length of the acceleration lane or within decision sight distance of the end of the taper
6. Significant crash history – particularly truck crashes

Table 8-2: Desirable Length of Full Width Medians: Acceleration Lane

Posted Speed (mph)	2/3 of Posted Speed (mph)	Desirable Length of Full Width Median Acceleration Lane, Rounded (ft.)
45	30	810
50	34	995
55	37	1203
60	40	1435
65	44	1680

Desirable Length Based on 200lb/hp Truck Accelerating to 2/3 posted speed
Minimum Median Acceleration Lane Length – 810′

The 200 pound per horsepower truck equates to the 85% truck in the national fleet based on studies reported in NCHRP Report 505, Review of Truck Characteristics as Factors in Roadway Design published in 2003. Table 29 in NCHRP Report 505 lists average acceleration capabilities for several different weight to power ratio classes of trucks. For the 200 pound per horsepower vehicles, the average acceleration listed is 1.22 ft/s2. The following formula for uniform acceleration was used to determine the desirable lengths for Median Acceleration Lanes listed in Table 8-2.

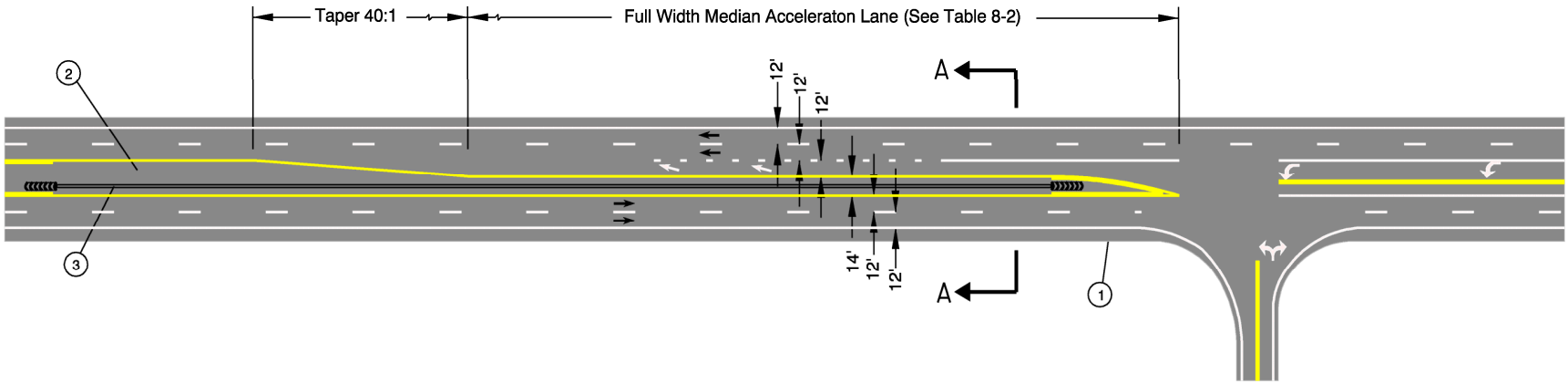
$$V_f^2 = V_i^2 + 2AS$$

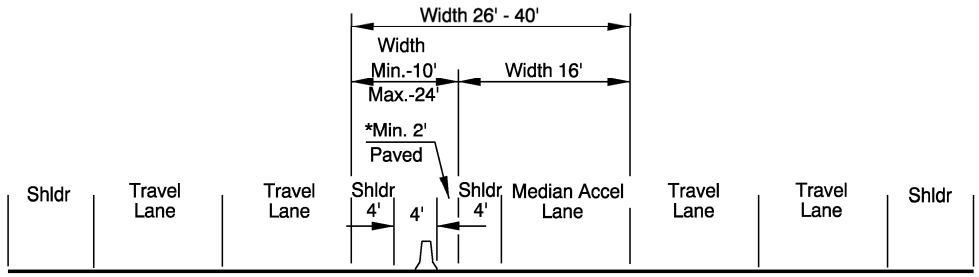
Where:

V_f = Final speed achieved at the end of distance S, ft/sec.
V_i = Initial speed, ft/sec. For Table 8-2, $V_i = 0$
A = Acceleration, ft/sec^2. A=1.22 ft/sec^2
S = Distance to accelerate to 2/3 of posted speed, Ft.

MEDIAN ACCELERATION LANE - NARROW MEDIAN

(26' - 40' Between Travel Lanes)





SECTION A-A

NOTES:

1. Flat Curve to Allow Design Vehicle to Enter Median Acceleration Lane Without Encroaching on Adjacent Lane
2. Inside Shoulder Extended for Runout Area
   Minimim Length - 300'
   Minimum Width - 10' (12' To Face of Barrier)
3. Median Barrier Treatment Concrete Barrier Shown Cable Barrier System May Be Optional (Final Placement and Layout With Project Design)

* Median Barrier Required For 26' to 40' Width Between Travel Lanes
Minimum 2' Paved Between Barrier and MAL Shoulder for 26' Width
For Widths Greater than 26' and Up To 40'- Paving Optional
For Widths Greater Than 40' - Median Barrier Not Required

STATE OF OREGON
DEPARTMENT OF TRANSPORTATION
ROADWAY ENGINEERING UNIT

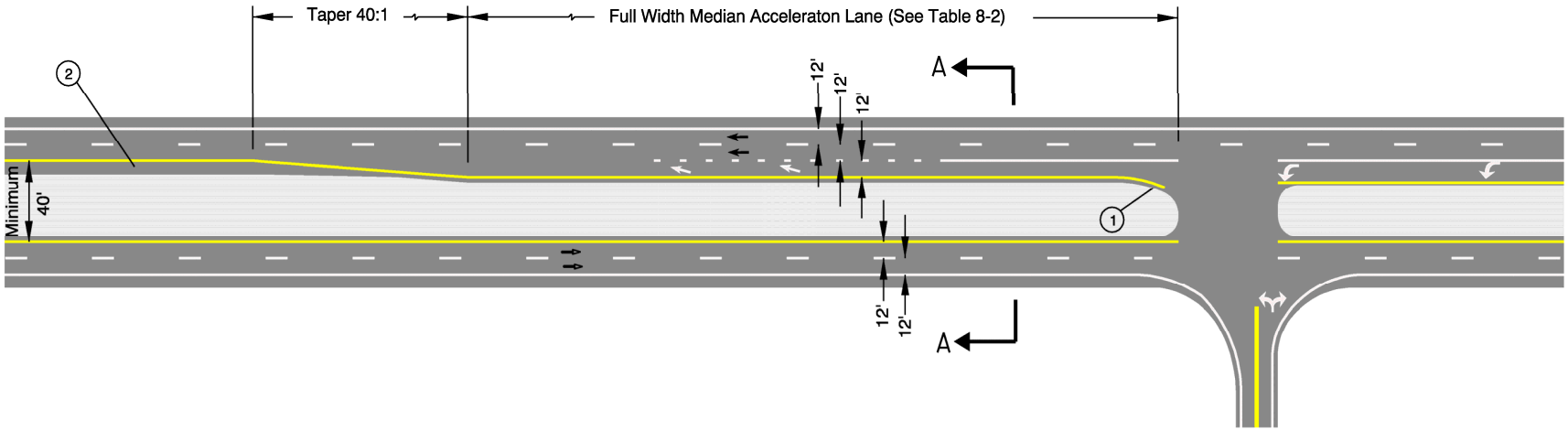
MEDIAN ACCELERATION LANE
NARROW MEDIAN

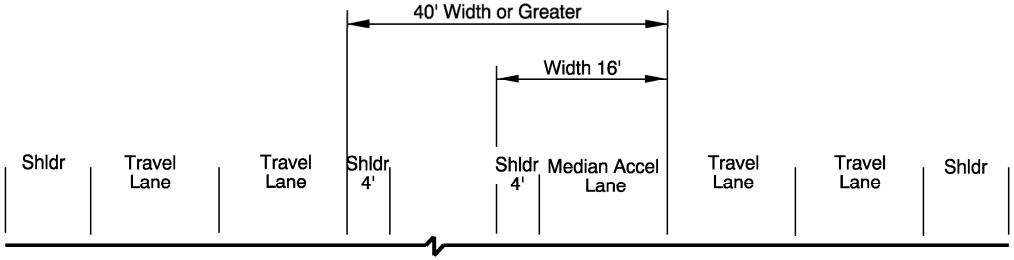
HIGHWAY DESIGN MANUAL

Figure 8-12: Median Acceleration Lane - Narrow Median

MEDIAN ACCELERATION LANE - WIDE MEDIAN

(40' or Greater Between Travel Lanes)





SECTION A-A

NOTES:

1. Flat Curve to Allow Design Vehicle to Enter Median Acceleration Lane Without Encroaching on Adjacent Lane
2. Inside Shoulder Extended for Runout Area
   Minimim Length - 300'
   Minimum Width - 10' (12' To Face of Barrier)

STATE OF OREGON
DEPARTMENT OF TRANSPORTATION
ROADWAY ENGINEERING UNIT

MEDIAN ACCELERATION LANE
WIDE MEDIAN

HIGHWAY DESIGN MANUAL

Figure 8-13: Median Acceleration Lane - Wide Median

8.3.13 LEFT TURN ADD LANES

A left turn add lane is a lane provided for vehicles turning left from a side road to accelerate and enter the main line traffic stream in a designated through lane. A left turn add lane should not be confused with a median acceleration lane. Although they may serve similar functions, there is a distinct difference. A median acceleration lane requires the left turning vehicle to merge into the through lane of the main line traffic stream. Where as, a left turn add lane creates a new and separate through lane for the left turning vehicle to enter that is independent of the existing through travel lane on the main line highway. This eliminates the need for the turning vehicle to merge into the existing through lane and creates a completely different operational characteristic from a median acceleration lane that reduces impacts on traffic in the existing through lane. Some form of physical separation between the add lane and the existing through travel lane should be provided for a length necessary to minimize speed differential between travel lanes. The first 600 feet should be a positive physical separation in the form of a raised separator or barrier, while the remaining length can be less physically separating in the form of rumble strips or a wide, solid paint stripe.

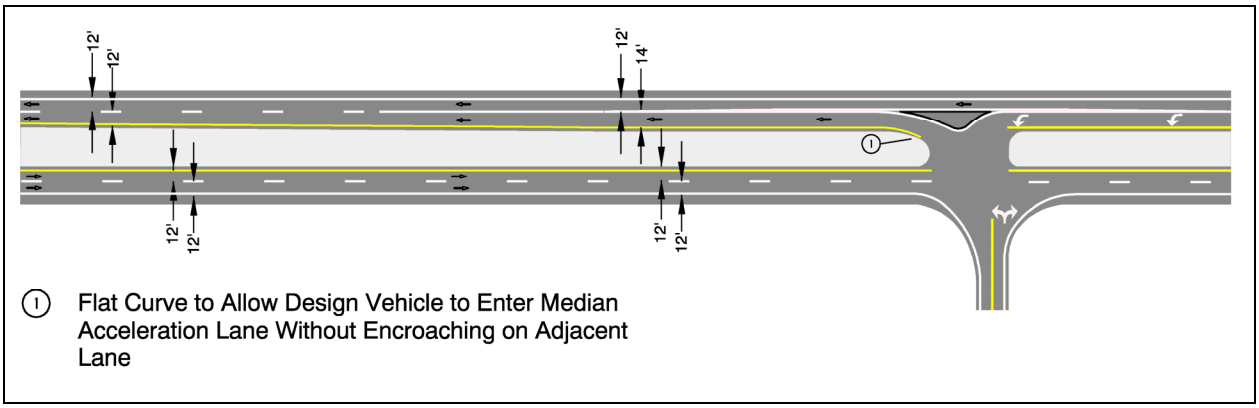


Figure 8-14: Left Turn Add Lane

8.3.14 CHANNELIZATION ISLANDS

Channelization islands help to direct turning traffic through an intersection. Channelization islands are a tool to help decrease the exposed crossing area of very large intersections. These islands can provide a refuge area for crossing pedestrians and offer a location for signal poles and sign posts. Where channelization islands are to accommodate poles or sign posts, the island should ideally have an area of at least 100 square feet. The minimum area shown on RD710 is 75 square feet.

Channelization islands are also useful for decreasing the crossing distance of pedestrians. When intersections are very wide, pedestrians must cross very long distances which increases their exposure time to traffic, reduces safety, and reduces efficiency of the signal due to the time necessary to cover the crossing maneuver. The designer should consider using channelization

islands where crossing distances are greater than 6 lanes wide. (Section 4.3.4.1 discusses raised medians and (Section 8.3.14 provides additional guidance on channelization islands). Channelization islands should be designed in conformance with Figure 8-22. Figure 8-15 provides additional information regarding pedestrian crossings and channelization islands.

In some rural locations, it may be advantageous to provide a moderate to higher speed right turn movement at major intersections. Channelization islands could also be used in these instances. When channelization islands are installed at high speed, rural locations, care must be taken to place these islands with adequate offset distance from the through travel lane. Figure 8-22 provides layout details for channelization islands. Adding raised channelization islands to intersections must be in compliance with ORS 366.215 and freight mobility needs. See ODOT guidance document "*Guidelines for Implementation of ORS 366.215, No Reduction of Vehicle-Carrying Capacity*" and the "*ODOT Highway Mobility Operations Manual*".

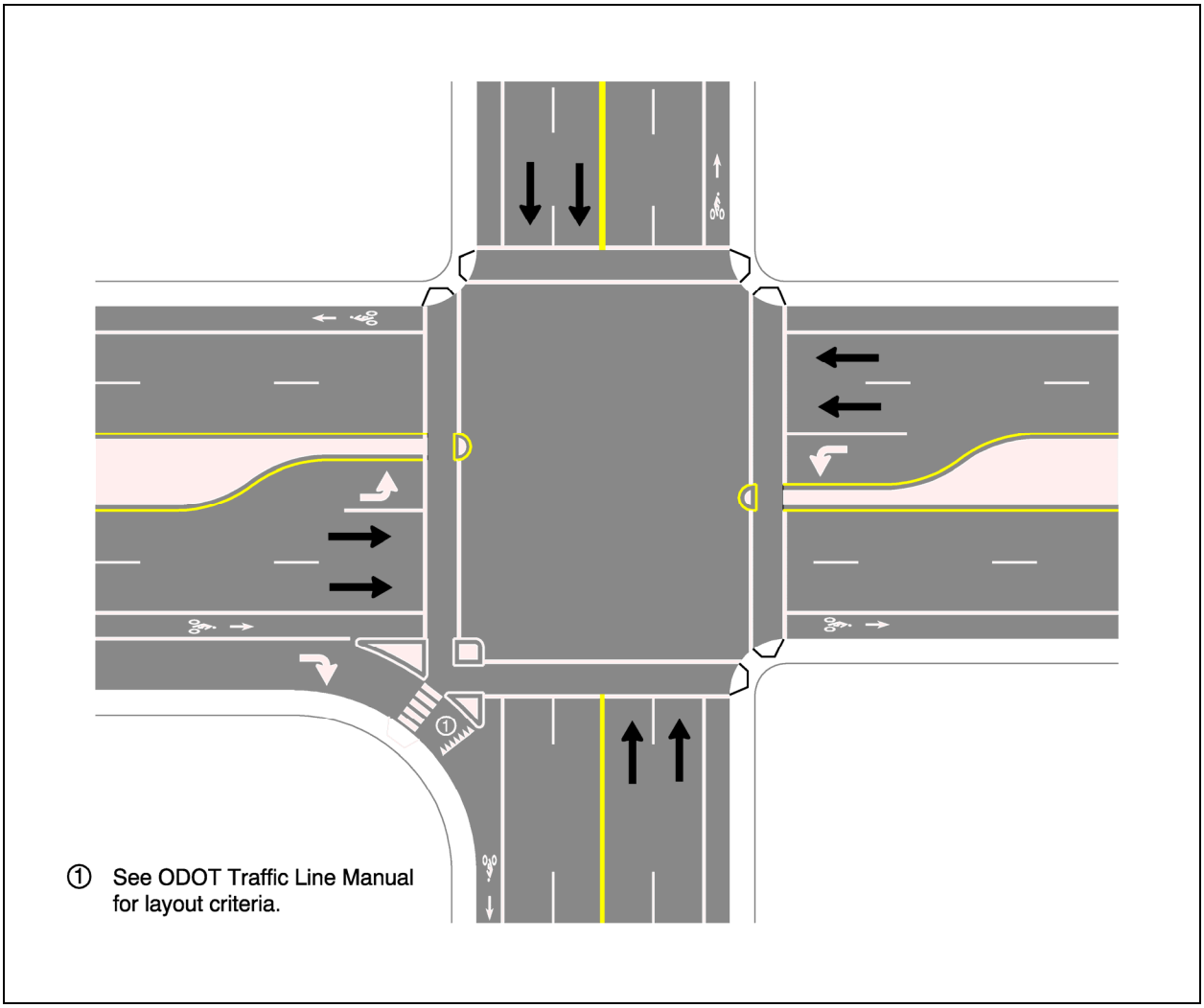


Figure 8-15: Typical Multi-Lane Channelized Intersection

8.3.15 CURB EXTENSIONS

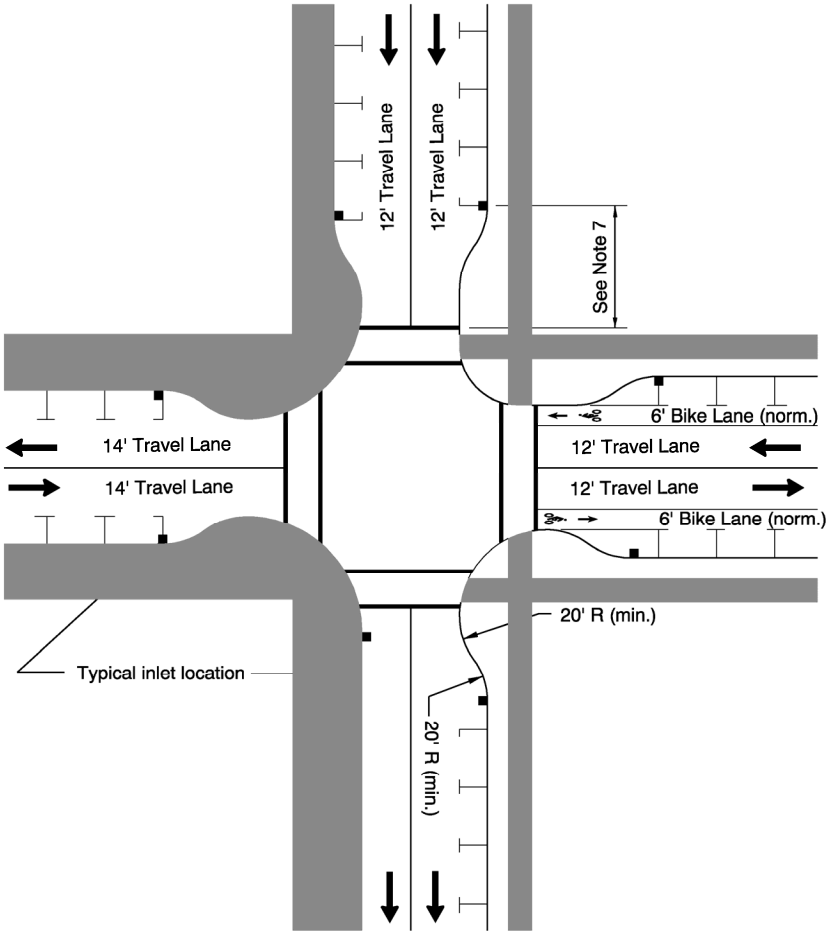
Curb extensions, also known as "bulb-outs," are good tools to help reduce the pedestrian crossing distances in areas with on-street parking. Curb extensions also increase pedestrian visibility, help control vehicular speeds, and give a "downtown look" to an urban area. Curb extensions are generally appropriate within slower speed compact areas, such as Special Transportation Areas (STAs) or Traditional Downtown/Commercial Business Districts. Curb extensions are generally considered at intersections, but they can also be utilized with great benefit at mid-block pedestrian crossings as well.

The curb extensions still must be designed to accommodate the appropriate design vehicle. However, due to the speed, traffic characteristics, and importance of alternative modes in these areas, the level of accommodation (see Section 8.3) of large vehicles is expected to be minimal. Curb extension design at proposed locations must meet the process and criteria outlined in ORS 366.215 and must meet freight mobility needs. See ODOT guidance document "*Guidelines for Implementation of ORS 366.215, No Reduction of Vehicle-Carrying Capacity*" and The "*ODOT Highway Mobility Operations Manual*".

Curb extensions should generally be constructed to the full width of the on-street parking. However, when no bike lane is present, the curbside travel lane should be at least 14 feet wide from the left side lane line to the face of the curb at the maximum extension point. Each curb extension design is different. Figure 8-16 contains several design concepts for consideration. Special consideration is required in many situations for addressing drainage in conjunction with curb extensions, especially in retrofit situations. Curb extensions should not block or narrow bicycle lanes and must provide adequate drainage along the curb line with no ponding of water at the sidewalk ramp entrance. For additional information on curb extensions, see Chapter 13, Section 13.5.2.4.

ORS 811.550(17) requires parking to be 20 feet from a marked or unmarked crosswalk and the MUTCD indicates parking should be 30 feet from the crosswalk at signalized intersections. Curb extensions can be used to provide the pedestrian benefits listed previously in this section as well as provide compliance for the required distance from crosswalks to on street parking.

CURB EXTENSIONS



General Notes

1. Intersections on state facilities should be designed to accommodate the appropriate design vehicle with the SU as the minimum. Corner curb radii are normally no less than 33' in cases where vehicles are turning right onto a roadway without a bike lane. The introductions of bike lanes, one-way streets and different combinations of turning movements may reduce the curb radius requirements to as low as 16'. These radii should be confirmed by the appropriate vehicle turning template or computer-generated turning paths.

2. Other curb radii should be no less than 20' to accommodate street maintenance equipment. This radius may be reduced if approved by local authorities.

3. Sidewalk ramps should be designed in accordance with Standard Drawing RD 755. Ramps should be located at the normal crosswalk location.

4. Storm water inlets should be placed at the point where the curb extension matches into the normal street section. Normally, 2 inlets are required, one at each end of the curb extension. Drainage may be required in the sidewalk area, depending on slopes and sidewalk design. Drainage may also be provided through the use of a slotted drain or a pipe through the curb extension.

5. Curb extension should be kept clear of obstructions that could obscure sight distance.

6. Curb extensions should not narrow bikelanes.

7. ORS 811.550(00) requires parking to be 20 feet from crosswalk for unmarked or marked crosswalks. MUTCD - 30 feet from crosswalk at Signalized intersection

STATE OF OREGON
DEPARTMENT OF TRANSPORTATION
ROADWAY ENGINEERING UNIT

CURB EXTENSIONS

HIGHWAY DESIGN MANUAL

Figure 8-16: Curb Extension

8.3.16 BICYCLE AND PEDESTRIAN NEEDS

The design of intersections takes into account the needs of bicyclists and pedestrians. The level and amount of design effort required to ensure adequate design for these modes will vary among different areas.

Intersection designs should try to keep the crossing distances and pedestrian exposure to a minimum. Pedestrians and motorists must be able to see each other clearly and understand how the other will proceed through the intersection. This can sometimes be difficult at major intersections that accommodate multiple turn lanes. When intersections become excessively large and complex, pedestrian safety is often at a higher risk. The designer should try to find mitigation measures to reduce the crossing distance.

The preferred method is to provide pedestrians with a crossing that can be completed in one movement. However, when pedestrians must cross an excessive number traffic lanes or a combination of excessive traffic lanes and a large skew angle, a pedestrian median refuge should be considered to enable the pedestrian to cross the street in two phases. A right turn channelization island should also be considered to reduce the pedestrians' exposure to both through and right turning vehicles. Curb extensions are a tool available to reduce the crossing distance for roadways with on-street parking. Median refuges and right turn channelization islands may be more appropriate in suburban locations, and curb extensions may be a more appropriate tool in more compact areas such as STAs or Commercial Business Districts. However, any of these tools could apply in a multitude of situations. A general rule of thumb is to consider pedestrian crossing remediation when the crossing distance exceeds 90 feet in typical urban environments such as Urban Business Areas (UBAs) and 72 feet in compact densely developed areas such as STAs.

ADA requirements shall be met in every intersection design. Issues such as proper ramps, location of pedestrian and signal poles, obstructions, fixed objects, drainage, etc., need to be reviewed and designed to accommodate all roadway and intersection users. Chapter 13, Pedestrian and Bicycle, has additional information on intersection accommodation.

8.3.17 INTERSECTION DESIGN AFFECTING PEDESTRIANS

There are several aspects of intersection design that impact the safety, comfort or access needs of pedestrians. For each identified issue, measures that can be used to mitigate these effects will be proposed.

8.3.17.1 EXCESSIVE SKEWS

Skewed approaches have several negative effects for pedestrians:

1. They make the crossing longer;

2. They enable motorists to make a turn at high speeds;
3. They force entering motorists to look backwards for conflicts, so that a pedestrian approaching from the other direction is out of sight, and
4. They place crossing pedestrians with their backs to approaching traffic.

The best way to mitigate for a skew is to reconfigure the intersection at or close to a right angle. If sufficient right of way is not available for total reconfiguration, the negative effects can be mitigated with a curb extension in the flat-angle corner(s). Figure 8-17 shows an example of an intersection with excessive skew and the intersection reconfigured with improve skew angle. If a curb extension isn't feasible, then use the tightest possible radius in the flat-angle corner(s).

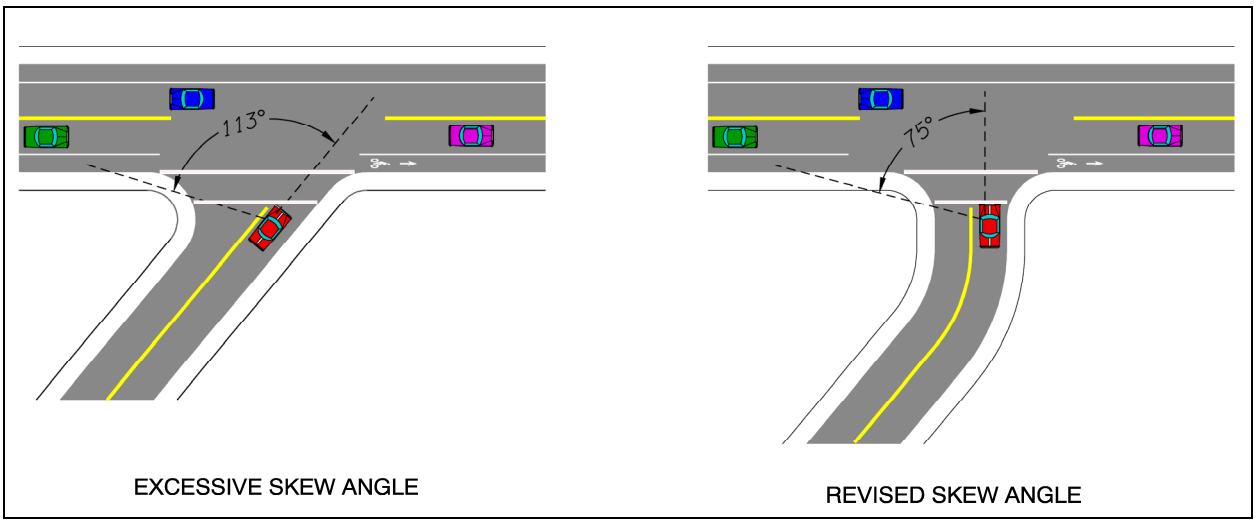


Figure 8-17: Skew Angle and Field of View

8.3.17.2 LONG CROSSWALKS

Long crosswalks are a problem for all road users for several reasons:

1. The pedestrian is exposed to conflicts longer;
2. It is difficult for some people to see pedestrian signals if they are too far away, and
3. The capacity of the intersection is reduced if the signal cycle is governed by the pedestrian crossing time.

Several methods may be considered, individually or jointly, to reduce crosswalk lengths:

1. Narrow the cross-section;
2. Provide curb-extensions on streets with parking;
3. Reduce the skew of the intersecting street, and

4. Minimize curb radius.

If the overall crosswalk length cannot be reduced, or the above techniques still do not provide sufficient reductions, then consider placing a refuge island(s) to enable the pedestrian to cross in two or more phases. Pedestrians should not be forced into a two-phase crossing; rather, the option should be available should they be stranded on a refuge island. Always provide a pedestrian push-button on islands. Pedestrian median refuges are strongly recommended when crossing more than 6 lanes. The Region Traffic Section and the Technical Services Traffic Unit should be consulted when considering the installation pedestrian refuge islands.

8.3.17.3 ISLANDS GEOMETRY

An island placed between a slip lane and through traffic can offer pedestrians a refuge, but if it is poorly designed, the geometry can encourage drivers to make turns at high speeds without looking for pedestrians. This can be mitigated by a design that brings the motorist to the intersecting street at close to a right angle, rather than a skew. This forces the driver to slow down, and enables the driver to see the crossing pedestrian. Figure 8-18 shows an example of a reconfigured right angle design skewed flat angle design. The type of design chosen varies depending upon the right turn vehicle accommodation. In many cases the presence of large trucks prohibits the use of this treatment. See ODOT guidance document "*Guidelines for Implementation of ORS 366.215, No Reduction of Vehicle-Carrying Capacity*" and the ODOT "*Highway Mobility Operations Manual*".

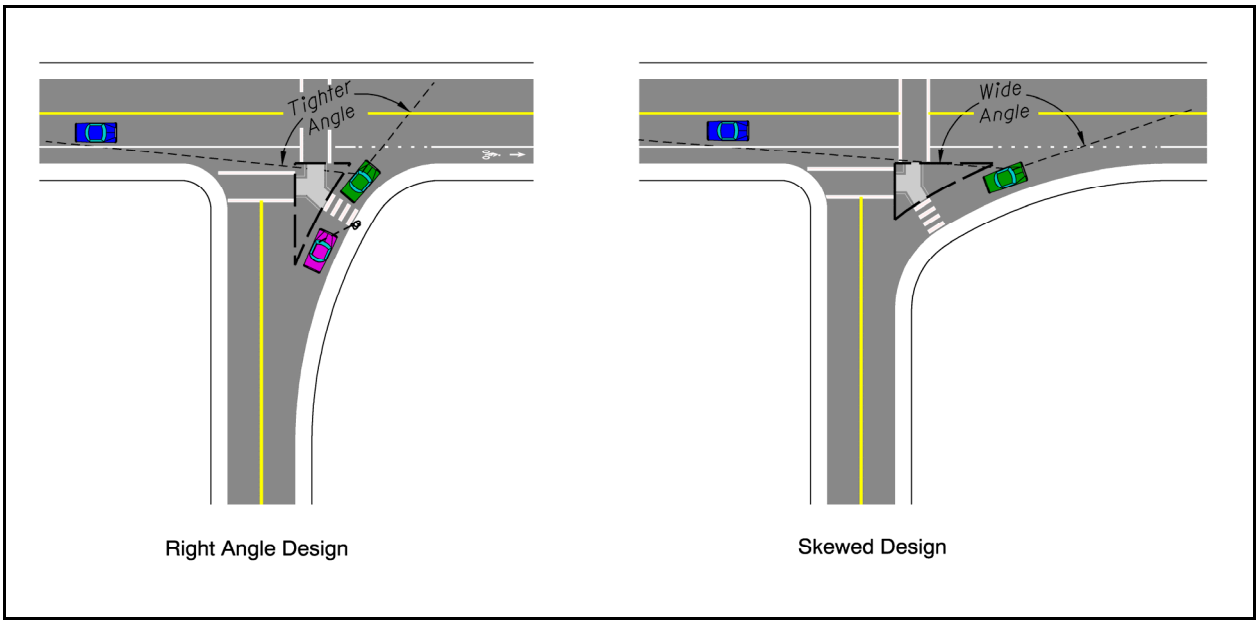


Figure 8-18: Island Geometry

8.3.17.4 CORNER RADII

Large corner radii present several problems for pedestrians:

1. They make the crossing longer;
2. They enable motorists to make a turn at high speeds, and
3. They make it very difficult to line up the sidewalks, crosswalks and curb cuts.

Designers should try every possible technique to minimize the corner radii at intersections in urban areas. Refer to the techniques described in Section 8.3.8, Design Considerations, Turning Radii.

Choosing the appropriate radius is often dependent on factors other than strict interpretation of design parameters. For example, it may be acceptable to design to a tight radius on approach streets with very little truck traffic, even if that means that the occasional truck may have to encroach into traffic to make a turn. Where there is a higher volume of truck traffic turning, a balance needs to be maintained between a large enough radius to accommodate truck turning, but a small enough radius to keep speeds of smaller turning vehicles low; thereby, minimize impacts to pedestrians and bicyclists.

8.3.17.5 CROSSWALK AND RAMP PLACEMENT

Crosswalk and ramp placement becomes a concern when an intersection is skewed, or if the corner radii are too large, especially with curb-tight sidewalks. The pedestrian expects the sidewalk, the curb ramp and the crosswalks to be in a reasonably straight line. The natural crossing point will be a continuation of the sidewalk.

Again, large corner radii create very long crosswalks. The designer may then be tempted to move the crosswalk away from the intersection, where the crossing is shorter, and crosswalks and curb rapms are perpendicular to the curb. This creates a new problem, as the crosswalk is offset from the intersection. The crossing pedestrians may not be visible to turning motorists, or pedestrians may ignore the crosswalk markings and walk where they are less inconvenienced. In other circumstances, squaring up the crossing may be the appropriate treatment. The best solution is to tighten up the intersection as much as possible.

In most instances, the best design will be arrived at through an iterative process., Imagining the natural path a pedestrian will take, while anticipating the various vehicle turning movements that may conflict with a pedestrian will help a designer reach optimal visibility of pedestrians and reasonable crossing distances., Examining driver and pedestrian expectations where pedestrian/vehicle conflicts may occur will help a designer better accommodate pedestrian crossings.

Another consideration is trying to ensure that sidewalks are separated with a buffer strip. This has two advantages: the extra separation will place the sidewalks between the offset crosswalk

and the curb-tight crosswalk described above, and a curb ramp traced through the buffer strip will more effectively channel pedestrians to the right crossing point.

8.3.17.6 CURB RAMPS - PLACEMENT AND NUMBER

U.S. Access Board guidance on compliance with the Americans With Disabilities Act (ADA) recommends two curb ramps at each corner of an intersection on new construction, and reasonable efforts should be made to install two on retrofit projects. Two curb ramps enable people in wheelchairs and other mobility aids to enter a crosswalk directly, without having to turn 45° in the roadway. Two curb ramps also make it easier to construct them perpendicular to the curb, as required. An additional advantage to utilizing two curb ramps is they better line up between the crosswalk and the adjacent sidewalk than a single curb ramp does. This allows vision impaired pedestrians a straight path to follow to reach the sidewalk, rather than having to deviate from the crosswalk alignment to find the single ramp located away from the crosswalk to sidewalk path. However, on corners with larger radii, generally radii greater than 30 feet, placing two curb ramps may make it difficult to align everything correctly. In these situations, after other mitigation has been tried, placing one diagonal ramp may work better. Figure 8-19 is an example of number of curb ramps based upon radius size, crossing distance and location. However, regardless of radius, the designer should strive to place two ramps for each corner when it is feasible. Whatever the final design, the designer needs to provide the most effective method available to ensure continuity for people with disabilities to traverse the distance between the crosswalk and the sidewalk. See applicable ODOT Standard Drawings for accessible island, accessible sidewalk and accessible ramp options and design. Additional information about providing acceptable access to public rights-of-way can be found in the publication, Special Report: Accessible Public Rights-of-Way, Planning and Designing for Alterations that was produced by the Public Rights-of-Way Access Advisory Committee (PROWAAC).

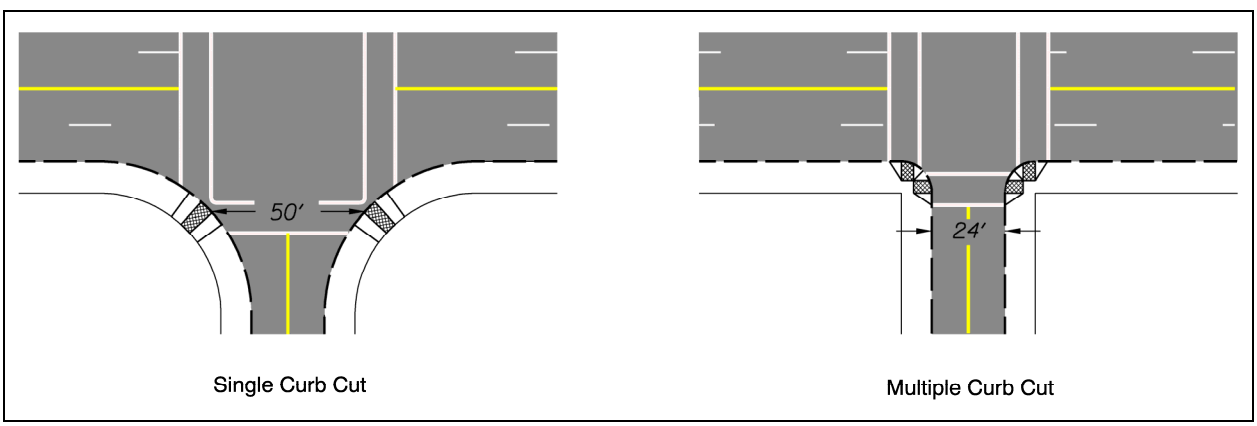


Figure 8-19: Crosswalk Ramp Placement

8.3.17.7 SIGNAL POLE PLACEMENT

Signal poles must be placed in a location where they do not interfere with pedestrians' path of travel. But, they must be placed in a manner that all pedestrians are able to conveniently reach the signal control push-buttons. There are special placement criteria for accessibility that must be followed to be in compliance with the Americans with Disabilities Act. The designer should work with the Region Traffic Unit and the Technical Services, Traffic-Roadway Section concerning placement of signal poles.

Placing the poles correctly is made easier with tight corner radii, sidewalks separated with a buffer strip, and two curb ramps per corner. As the radius increases, it becomes more difficult to place the pole out of the ramps and out of the walking area, but still within reach. The best location for a signal pole is between the two ramps. If that is not feasible, the pole can be placed in the back of walk. This may make it difficult for pedestrians to reach the push-buttons. In this situation, consider placing a pedestrian pole at a more convenient location, preferably between the two curb cuts. In all locations, signal poles and pedestrian buttons must be installed to meet accessibility requirements.

On corners with one curb ramp, it may be best to place the pole at the back of curb, while ensuring that there is a minimum 4 foot level area between the pole and the top of the ramp. Under no circumstances should poles be placed in a curb cut, or in the level landing at the top of a ramp. Figure 8-20 provides a general example of signal pole placement with parallel style sidewalk ramps. See the "*ODOT Traffic Signal Policy and Guidelines*" as well as the MUTCD, 4E.08 for additional detailed information on signal pole placement.

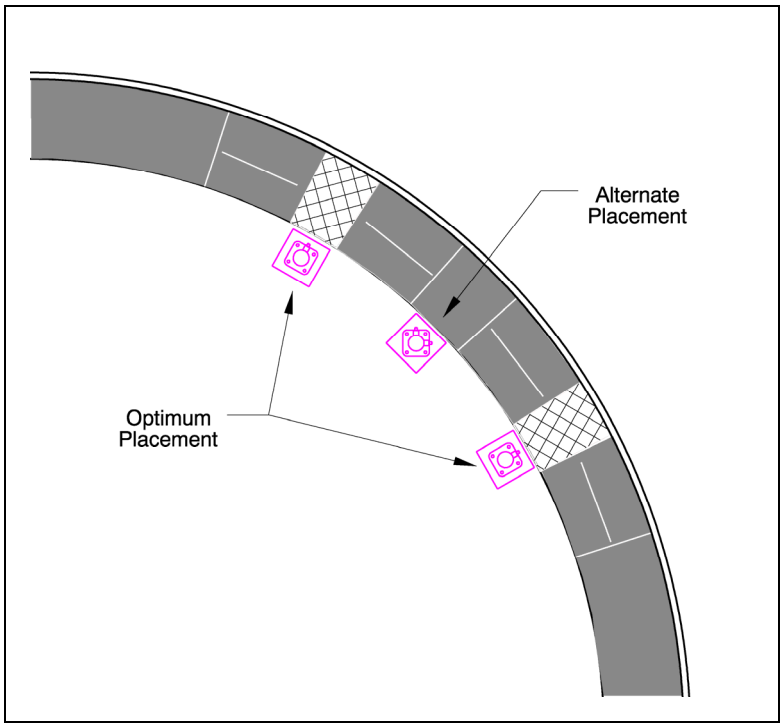


Figure 8-20: Signal Pole Placement

8.3.17.8 FREE-FLOW ACCELERATION (ADD) LANES

This type of intersection treatment should be avoided in urban areas. Free-flow acceleration lanes are generally not allowed for at-grade intersections in accordance with Section 8.3 General Intersection Design. They create an unexpected condition for both pedestrians and cyclists. Free-flow acceleration lanes are different than at-grade right-turn acceleration lanes described in Section 8.3.11. A free-flow acceleration lane provides a lane for traffic to make the turn and enter the acceleration lane without stopping. This implies priority for the turning vehicle over other roadway facility users and is generally not appropriate in urban locations. Use of free-flow lanes is strongly discouraged where pedestrians and bicyclists are expected to cross the lane.

If a free-flow acceleration or add lane is provided for capacity reasons, then every reasonable effort should be made to create conditions that make any adjacent crossings safer and easier for pedestrians and cyclists. Crossings should occur prior to vehicle acceleration locations where vehicle speed is low and adequate sight distance must be provided for a driver to see pedestrians and bicyclists crossing the lane.

Most of the design principles offered in previous sections on right turn lanes would apply to free-flow lanes also: tighten the turning radius, narrow the lane, and keep the angle of approach as close to a right angle as possible. These three elements combined will force drivers turning right to slow down and look ahead, where pedestrians and bicyclists may be present, before turning and accelerating onto the roadway.

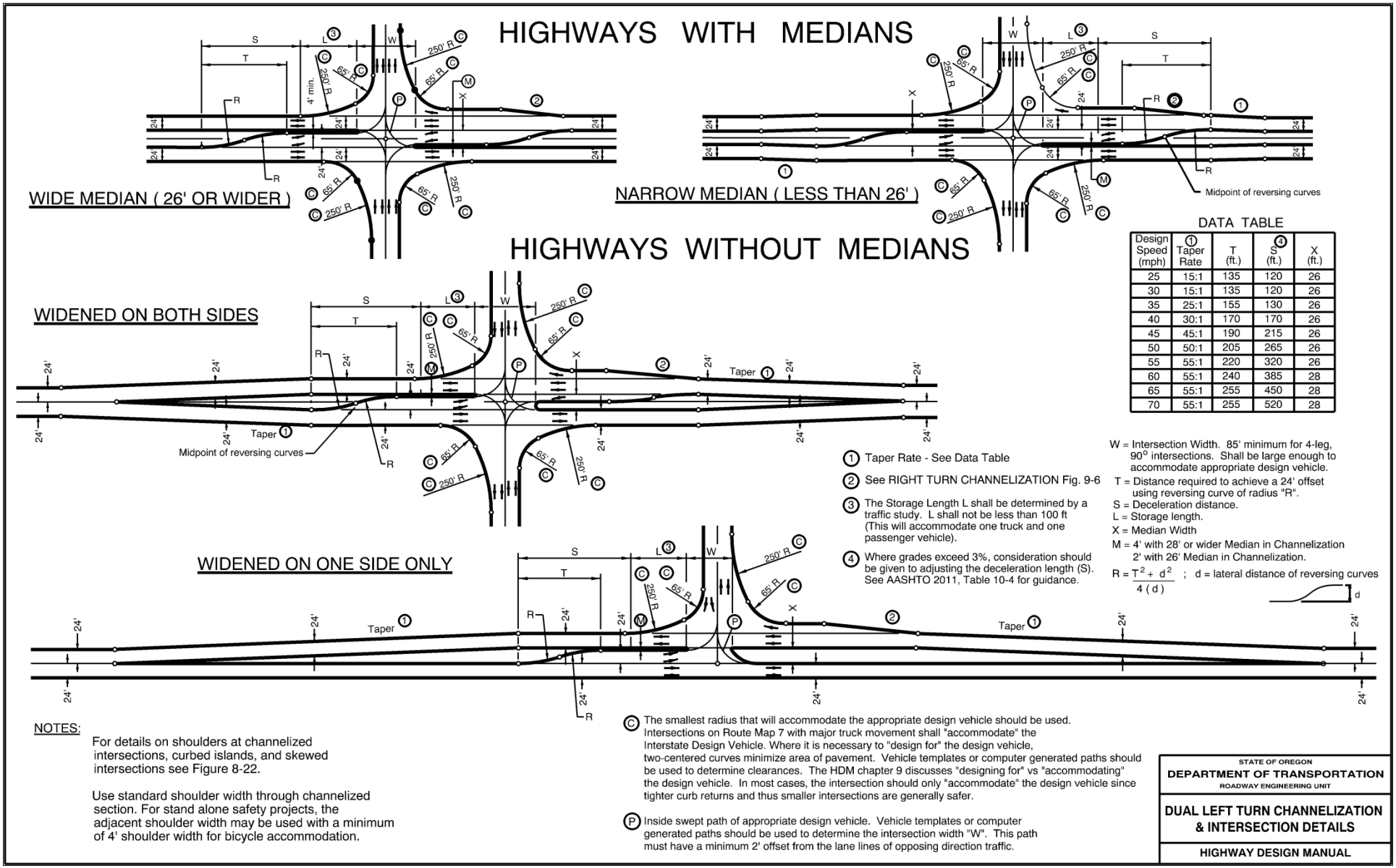
8.4 SIGNALIZED INTERSECTIONS

Signalized intersection design will need to consider the following issues in addition to the design standards for general intersection design that were discussed in Section 8.3. Specific roadway design items of interest at signalized intersections include left turn lanes, right turn lanes, bicycle accommodation and pedestrian needs. It will be necessary for the designer to coordinate with the Region Traffic Unit and the Traffic-Roadway Section of Technical Services to meet these specific design needs.

8.4.1 LEFT TURN LANES

Most signalized intersections will have left turn lanes. When left turning traffic is allowed from a two way highway at a signalized intersection, a left turn lane must be provided. Providing a traffic signal phase for left turning traffic is determined by Traffic Engineering Section (see "*ODOT Traffic Signal Policy and Guidelines*").

When the left turning volume is very large, a single left turn lane may not be able to handle the volume and still provide an acceptable mobility standard or safety. In these instances, a dual left turn lane may be needed. Requests for dual left turn lanes must be approved by the State Traffic-Roadway Engineer (see OARs 734-020-0135 and 0140 for criteria). When designing dual left turns lane, there must be dual receiving lanes on the connecting roadway with adequate length downstream prior to any merge points. The designer must determine the appropriate design vehicles to use for side-by-side operation through the turning movement. In rare locations, like at freeway ramp terminals leading to truck stops or warehousing districts, that may need to be two WB-67 vehicles making the turn simultaneously. However, in most locations, a WB-67 and an SU vehicle side-by-side is adequate for design. In other locations where truck volumes are low, an SU vehicle and a passenger vehicle may be sufficient. Dual left turn lanes should be designed in conformance with Figure 8-21. The Region Traffic Section should be consulted when considering the design of a dual left turn lane as well.



DATA TABLE

Design Speed (mph)	① Taper Rate	T (ft.)	④ S (ft.)	X (ft.)
25	15:1	135	120	26
30	15:1	135	120	26
35	25:1	155	130	26
40	30:1	170	170	26
45	45:1	190	215	26
50	50:1	205	265	26
55	55:1	220	320	26
60	55:1	240	385	28
65	55:1	255	450	28
70	55:1	255	520	28

① Taper Rate - See Data Table

② See RIGHT TURN CHANNELIZATION Fig. 9-6

③ The Storage Length L shall be determined by a traffic study. L shall not be less than 100 ft (This will accommodate one truck and one passenger vehicle).

④ Where grades exceed 3%, consideration should be given to adjusting the deceleration length (S). See AASHTO 2011, Table 10-4 for guidance.

W = Intersection Width. 85' minimum for 4-leg, 90° intersections. Shall be large enough to accommodate appropriate design vehicle.
T = Distance required to achieve a 24' offset using reversing curve of radius "R".
S = Deceleration distance.
L = Storage length.
X = Median Width
M = 4' with 28' or wider Median in Channelization
2' with 26' Median in Channelization.

$$R = \frac{T^2 + d^2}{4(d)}$$; d = lateral distance of reversing curves

NOTES:

For details on shoulders at channelized intersections, curbed islands, and skewed intersections see Figure 8-22.

Use standard shoulder width through channelized section. For stand alone safety projects, the adjacent shoulder width may be used with a minimum of 4' shoulder width for bicycle accommodation.

Ⓒ The smallest radius that will accommodate the appropriate design vehicle should be used. Intersections on Route Map 7 with major truck movement shall "accommodate" the Interstate Design Vehicle. Where it is necessary to "design for" the design vehicle, two-centered curves minimize area of pavement. Vehicle templates or computer generated paths should be used to determine clearances. The HDM chapter 9 discusses "designing for" vs "accommodating" the design vehicle. In most cases, the intersection should only "accommodate" the design vehicle since tighter curb returns and thus smaller intersections are generally safer.

Ⓟ Inside swept path of appropriate design vehicle. Vehicle templates or computer generated paths should be used to determine the intersection width "W". This path must have a minimum 2' offset from the lane lines of opposing direction traffic.

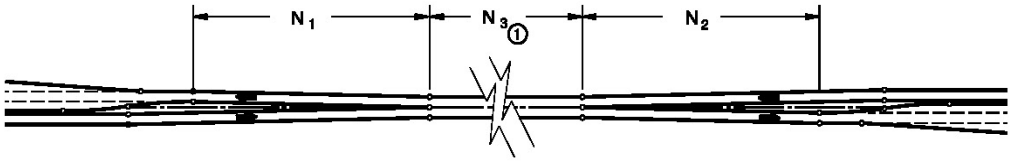
STATE OF OREGON
DEPARTMENT OF TRANSPORTATION
ROADWAY ENGINEERING UNIT

DUAL LEFT TURN CHANNELIZATION & INTERSECTION DETAILS

HIGHWAY DESIGN MANUAL

Figure 8-21: Dual Left Turn Channelization

CHANNELIZATION & INTERSECTION DETAILS



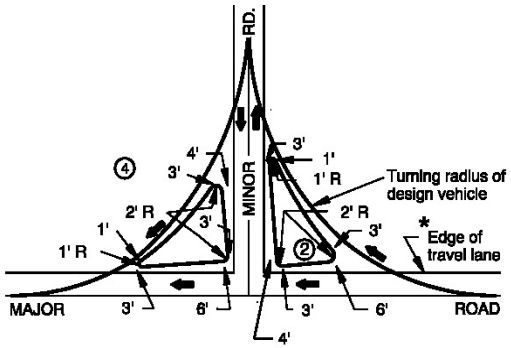
DETERMINATION OF REFUGE CONTINUATION FOR HIGHWAYS WITHOUT MEDIANS

① $N_3 \geqq N_1 + N_2$
N_3 represents the length of section between tapers for two channelized intersection. When N_3 is less than $N_1 + N_2$ a continous, full width median should be carried between the two intersections.

② Taper from 6' offset at approach end to 3' min. at exit end. Taper not to exceed 15:1 or be less than 50:1. On small island exit offset will be greater than 3'. Triangular islands should be at least 75 sq. ft. and preferably 100 sq. ft.

③ See Figure 8-8 RIGHT TURN CHANNELIZATION

④ See ODOT Traffic Line Manual for additional layout criteria.



Islands should be checked against all turning movements of design vehicle.

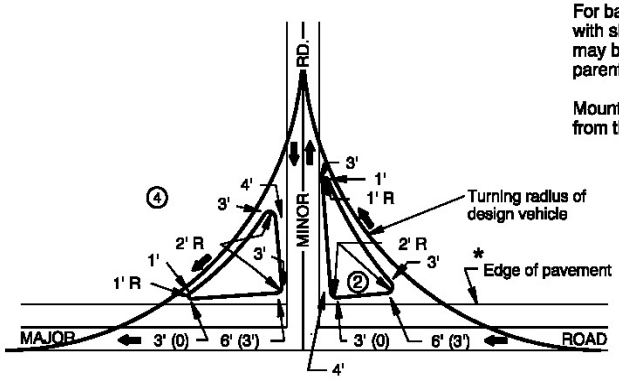
MOUNTABLE CURB ONLY

CURBED ISLANDS

(When approved by Roadway Engineer.)

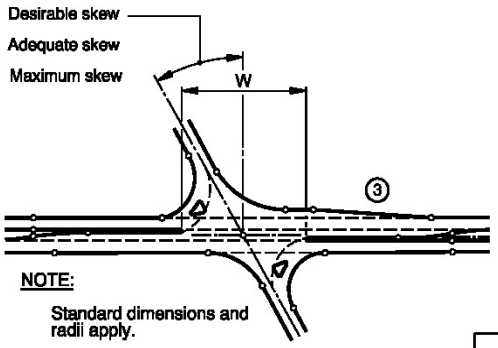
* Barrier curbed islands shall be offset from the edge of pavement extended. For barrier curbed islands, on roadways with shoulders wider than 6', offsets may be reduced to the values shown in the parentheses.

Mountable curbed islands may be offset from the edge of travel lane.



BARRIER or MOUNTABLE CURB

0°- 15° Desirable skew
15°- 21° Adequate skew
21°- 30° Maximum skew



NOTE:
Standard dimensions and radii apply.

SKEWED INTERSECTION

STATE OF OREGON
DEPARTMENT OF TRANSPORTATION
ROADWAY ENGINEERING UNIT

CHANNELIZATION & INTERSECTION DETAILS

HIGHWAY DESIGN MANUAL

Figure 8-22: Channelization & Intersection Islands Details

8.4.2 RIGHT TURN LANES

There are no specific warrants for installation of a right turn lane at a signalized intersection. A rule of thumb is to install a right turn lane when peak hour right turn volume is 200 or more. Installation of a right turn lane at signalized intersections should be justified by engineering analysis. The Region Traffic Section and The Transportation Planning Analysis Unit (TPAU) should be consulted where right turn lanes might be necessary.

It is critical to the operation of signalized intersections that adequate storage length for right turning vehicles (out of the through traffic lanes) be provided. The storage length needs to accommodate the 95% queue distance through the design life of the project. The 95% queue length means that there is only a 5% probability that the actual volume of vehicles will exceed the storage available. In areas where obtaining the 95% queue distance is impractical, the designer should provide as much storage as possible. Consideration should be given to shortening the entrance taper to lengthen the available storage. Any exceptions, however, will require an approval from the State Traffic-Roadway Engineer. For individual intersection or operational projects, the Region Traffic Engineering Unit should be contacted to determine the appropriate storage lengths needed. For complex or environmental study projects, the Transportation Planning Analysis Unit (TPAU) can be contacted to help determine the appropriate storage lengths or give guidance or technical help on the particular project or methodology. At some intersections, right turn demands might be so large that dual right turn lanes may be necessary. The Analysis Procedures Manual, Region Traffic, and the Technical Services Traffic Engineering Section must be consulted and the approval of the State Traffic-Roadway Engineer obtained prior to installation of dual right turn lanes (see OARs 734-020-0135 and 0140). Where dual right turn lanes are required, follow the guidelines shown in Figure 8-23. Dual right turn lanes can create additional crossing issues for bicycle and pedestrian movements. When dual right turn lanes are proposed, bicycle and pedestrian movements must be considered and adequately addressed. Contact the ODOT Bicycle and Pedestrian Facility Specialist for information about providing appropriate facilities.

In addition to bicycle and pedestrian considerations at dual right turn lane locations, the designer also must determine the appropriate design vehicles to use for side-by-side operation through the turning movement. In rare locations, like at freeway ramp terminals leading to truck stops or warehousing districts, that may need to be two WB-67 vehicles making the turn simultaneously. However, in most locations, a WB-67 and an SU vehicle side-by-side is adequate for design. In other locations where truck volumes are low, an SU vehicle and a passenger vehicle may be sufficient. When considering dual right turn lanes as an option, the Region Traffic Section should be consulted for input. When designing dual right turn lanes, there must be two lanes on the connecting roadway to turn into and there must be adequate length provided downstream before any lanes merge.

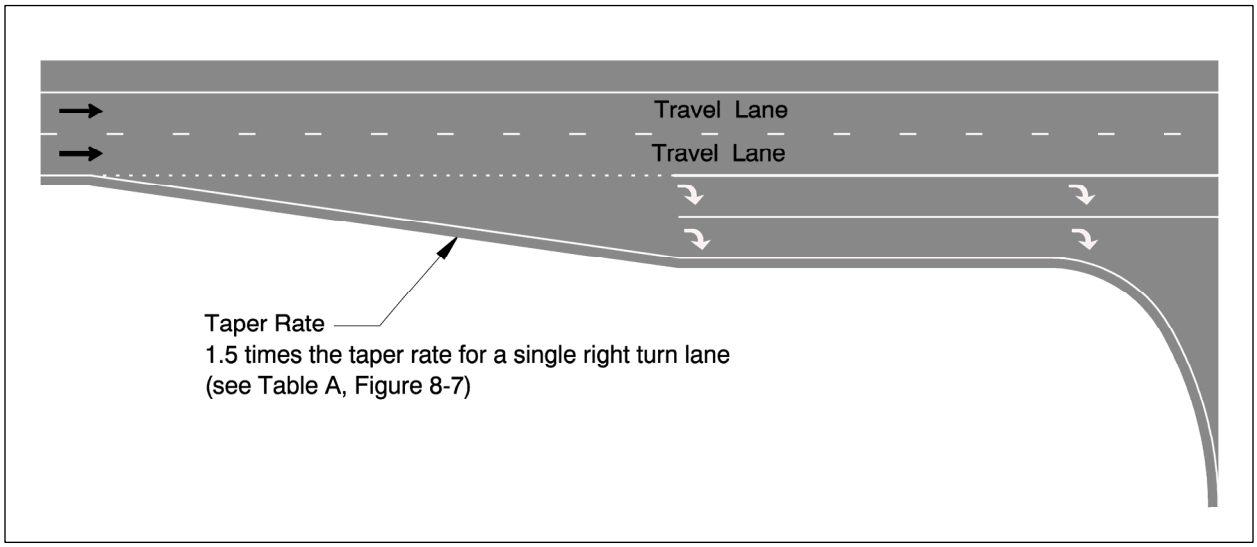


Figure 8-23: Dual Right Turn Channelization

8.4.3 BICYCLE AND PEDESTRIAN NEEDS

Signalized intersections need to provide marked pedestrian crossings at all approaches and provide bicycle connectivity and continuity. There may be some locations where full access may not be appropriate. Locations where exceptions to full access may be considered are:

1. Intersections that include multiple left or right turn lanes,
2. Intersections with one or more legs being one way roadways, and
3. Intersections that are a 'T' configuration.

However, even at these locations, bicycle and pedestrian needs and movements must be addressed and some level of accommodation is expected. The idea is to only close a crossing where a turn movement has a direct protected green arrow conflict with a crossing pedestrian. Only the State Traffic-Roadway Engineer can close a legal pedestrian crossing. The Region Traffic Section and the Traffic Engineering Section of Technical Services should be contacted early in the project to determine the appropriate pedestrian crossing locations.

8.5 UNSIGNALIZED INTERSECTIONS

This section covering unsignalized intersection design is intended to enhance the discussion about general intersection design criteria covered in Section 8.3. Left turn lanes, right turn lanes, bicycle access and pedestrian movements will need to be specifically considered and accounted for when designing unsignalized intersections. The level and amount of design effort required to ensure adequate design for these modes will vary among different areas. Because of the complexity of urban areas, a higher level of effort is needed to ensure that these design needs are adequately addressed.

8.5.1 LEFT TURN LANES

Left turn lanes at unsignalized intersections must meet the siting criteria to justify installation. Regardless of the funding source, the Region Traffic Engineer must approve all unsignalized channelized left turn lanes. The designer should work with the Region Traffic Unit in locations where left turn lanes are being considered. For information about siting criteria for left turn lanes, see the ODOT Analysis and Procedures Manual (APM).
(http://www.oregon.gov/ODOT/TD/TP/APM.shtml)

8.5.2 RIGHT TURN LANES

Unsignalized intersections and private approach roads must meet the installation criteria prior to constructing a right turn lane. Regardless of the funding source, the Region Traffic Engineer must approve all unsignalized right turn lanes.

Since the right turning vehicles only have to yield to pedestrians at unsignalized intersections, there is no need to provide vehicle storage at an unsignalized right turn lane. The one exception is where vehicular storage may be required where the right turn lane is next to an at grade railroad crossing. For information about siting criteria for right turn lanes, see the ODOT Analysis and Procedures Manual (APM).
(http://www.oregon.gov/ODOT/TD/TP/APM.shtml)

8.5.3 BICYCLE AND PEDESTRIAN NEEDS

Bicycle movements must be considerd at all unsignalized intersections. There are a variety of methods available to provide adequate bicycle connectivity and continuity at these types of locations. For information, see the "*Oregon Bicycle and Pedestrian Design Guide*".

By law, every intersection is a legal crossing location for pedestrians. This is true whether the crossing is marked or unmarked. Therefore, it is important to ensure that pedestrian needs are included in the intersection design, particularly in urban areas. The marking of crosswalks shall meet the guidelines and recommendations of the ODOT Traffic Manual and the ODOT Traffic Line Manual.

8.6 MODERN ROUNDABOUTS

8.6.1 GENERAL

This section provides basic information and site criteria on both single lane and multi-lane roundabouts. Please contact the Technical Services, Traffic-Roadway Section for additional design criteria and recommendations.

Traffic signals, stop signs and modern roundabouts are all forms of intersection control. Signal control and stop control are more established forms of intersection control and are well known to motorists, pedestrians and bicyclists. Signal control and stop control function by separating out individual traffic movements at an intersection. Each road user takes a turn or is delegated time and reasonable opportunity to move through the intersection. However, intersections controlled by signals and signs do not always afford the most efficient or most safe operation. When traffic volumes are low, signals can cause unnecessary delay by stopping traffic flow when conflicts do not exist. When traffic volumes are high, stop signs can cause long queues and extended delay. In addition, when motorists, pedestrians or bicyclists make mistakes or push the limits at signalized or stop controlled intersections, the results often cause severe injury crashes or fatal crashes. Modern roundabout controlled intersections have the potential to function much more efficiently and safely than signal controlled or stop controlled intersections. They do not stop traffic flow unnecessarily. By design, roundabouts allow for more consistent flow by slowing all vehicles through the intersection. By reducing delay, they improve vehicle fuel efficiency and reduce vehicle emissions at the intersection as well. Modern roundabouts can also be safer than signalized or stop controlled intersections. By reducing speeds and keeping traffic flowing in the same direction, both crash frequency and severity have been shown to be reduced when compared to other intersection control types. Roundabouts have been shown to be safer for pedestrians and bicyclists as well.

However, roundabouts are not as prevalent as signals or stop signs and some people are unsure how to use them. As a result, they approach roundabouts with concern, both when discussing proposed installations and when encountering one on the highway. In some cases, drivers remember circular intersections of the past that were called “traffic circles” or “rotaries”. Many of these older circular intersections did not function well. As a result, many drivers have negative impressions of circular intersections that carry over to the present. By their design, however, modern roundabouts eliminate the undesirable design features of older traffic circles or rotaries and create an efficient and effective intersection control option with specific characteristics. The distinctive characteristics of a modern roundabout that separate it from a traffic circle or rotary include a raised central island with a circulatory roadway, raised splitter islands at the entry to introduce deflection to the vehicle path, and yield control for approaching vehicles, rather than having the circulating traffic yield to the entering traffic as was the case with older style traffic circles or rotaries. In various locations around the United States, operations at many of the original traffic circles and rotaries have been improved by

incorporating some of the modern roundabout concepts into them where feasible. In some locations, the older style traffic circles have been removed entirely. Figure 8-24 details several major roundabout elements.

Studies have shown, even in communities where the initial majority viewpoint concerning the installation of roundabouts was negative, once roundabouts were installed and the community became used to driving them, the roundabouts have become a popular form of safe and effective intersection control and the community viewpoint changed to positive for the installation of roundabouts.

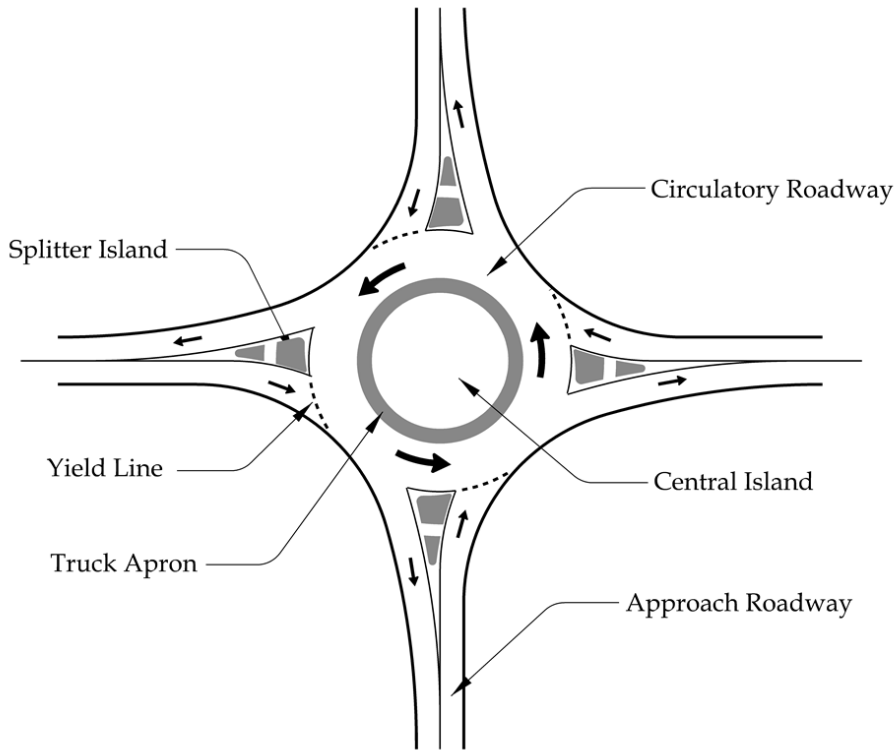


Figure 8-24: Elements of a Roundabout

8.6.2 OVERVIEW

Roundabouts have been proven as a viable alternative to traffic signals at many intersections. Several studies comparing roundabouts to traffic signals or two-way stop controlled intersections have demonstrated consistent results in determining that roundabouts can provide significant safety improvements. Their combined findings indicate:

1. Reduction of fatalities by more than 90%;
2. Reduction of injuries by up to 75%;
3. Reduction of all crashes by a third or more; and

4. Increases in pedestrian and bicyclist safety due to slower vehicle speeds.

Additional information concerning roundabouts and their safety performance can be found through information provided by the Federal Highway Administration website *"FHWA Safety – Roundabouts"* and through research results from the Insurance Institute for Highway Safety (IIHS).

All roundabouts greatly reduce conflicts at intersections and increase safety when compared to signal controlled or stop controlled intersections. However, due to differences in inherent characteristics of single lane and multi-lane roundabouts, there are differences in the potential safety improvements between them. Both single lane and multi-lane roundabouts reduce fatal and serious injury crashes. Single lane roundabouts have greater reduction in intersection conflict points than multi-lane roundabouts and, therefore, tend to have greater reduction in overall crash rates than multi-lane roundabouts. Since there is more than one travel lane in a multi-lane roundabout, multi-lane roundabouts have the potential for sideswipe crashes that single lane roundabouts do not have. However, since speeds are slow, these crashes are generally less severe than the higher speed "T-bone" and head-on crash types that occur at signalized or stop controlled intersections. Therefore, even though multi-lane roundabouts may have a greater preponderance of side-swipe crashes than a single lane roundabout, they are still a safer alternative than a multi-lane signalized intersection because severity of crashes is greatly reduced, while providing the necessary intersection capacity.

There are three conflict types that can occur at multi-lane roundabouts that do not occur at single lane roundabouts and they can lead to sideswipe crashes. They are categorized as:

1. Driver fails to maintain lane position through the roundabout *(Note: ORS 811.292 and ORS 811.370 have provision for "commercial motor vehicles" to operate outside a single lane in a multi-lane roundabout when necessary.)*
2. Entering driver fails to yield properly and enters next to a vehicle exiting the roundabout
3. Driver turns or exits from the incorrect lane and crosses the path of a vehicle in the outside lane

These types of driver error are not unique to roundabouts and similar errors can also occur at conventional intersections. However, with good roundabout geometric design consistent with appropriate entry and exit angles, vehicle deflection and sight distance as well as effective striping and signing, the first two can be minimized thereby further improving safety over conventional, multi-lane intersections.

Along with the potential safety benefits they provide, roundabouts can also reduce congestion and delay. They have been shown to be efficient during both peak and non-peak hours. Other distinct advantages of roundabouts include the following:

1. Reduced pollution and fuel use through smoother flow and fewer stops;
2. Significant life-cycle cost savings when compared to traffic signals due to no signal equipment installation and reduced maintenance costs; and

3. Can provide traffic calming and general speed reduction, while supporting urban and rural community values through quieter operation and by providing a traffic control solution that is both functional and aesthetically pleasing.

As stated earlier in this section, some features of multi-lane roundabout design are significantly different from single lane roundabout design and some techniques used in single lane roundabout design may not directly transfer to multi-lane roundabout design. However, several principal objectives should be achieved when designing any roundabout. The following principles should be the goal of roundabout designs:

1. Provide slow entry speeds and consistent speeds through the roundabout utilizing vehicle path deflection.
2. Provide the appropriate number of lanes and lane assignments to achieve adequate capacity, lane volume balance and lane continuity for necessary vehicle movements.
3. Provide smooth channelization that is intuitive to drivers that results in vehicles naturally using the intended lanes.
4. Provide adequate design and accommodation for all vehicle types expected to use the roundabout, including freight and transit vehicles.
5. Design to include the needs of pedestrians and bicyclists.
6. Provide appropriate sight distance and visibility for driver recognition of the intersection and potential conflicts with other roadway users both motorized and non-motorized.

The Transportation Research Board (TRB) and the FHWA have published a useful guidance document entitled Roundabouts: An Informational Guide, Second Edition that is also NCHRP Report 672. It can be found on the TRB/NCHRP website.

For proposed roundabouts on state highways in Oregon, staff should familiarize themselves with FHWA guidance documents, the Oregon Highway Design Manual, including Section 8.6 Modern Roundabouts, the Roundabout Selection Criteria And Approval Process (Section 8.6.3 of the HDM and Section 6.26.2 of the ODOT Traffic Manual) as well as pertinent sections of the Analysis and Procedures Manual (APM) published by TPAU.

Before proceeding to the Roundabout Selection Criteria And Approval Process, a thorough alternatives analysis should have been completed in the form of an Intersection Traffic Control Study showing that a roundabout is a viable alternative when compared to other types of intersection traffic control. Refer to the Intersection section (Section 6.13) of the ODOT Traffic Manual for more detail on how to conduct this type of analysis. Capacity for the proposed roundabout should be analyzed for the appropriate peak hour flow(s).

8.6.3 ROUNDABOUT SELECTION CRITERIA AND APPROVAL PROCESS

Roundabouts can be proposed for a variety of reasons including, safety improvements, operation improvements, community livability, traffic calming, aesthetic gateway treatments,

etc. The State Traffic-Roadway Engineer has been delegated the authority to approve the installation of roundabouts on State Highways. Requests for roundabout evaluations are a collaborative process between the Region Traffic Unit and Region Roadway Unit. All roundabout requests sent to the State Traffic-Roadway Engineer for consideration shall be jointly sent by the Region Traffic Manager and Region Roadway Manager, accompanied by an Engineering Investigation that includes purpose, need and intent of installation of the proposed roundabout. In addition, the Engineering Investigation shall address the considerations as described in the following discussion.

Once the State Traffic-Roadway Engineer receives a request, the Traffic-Roadway Section will coordinate a review with other technical staff from Technical Services and the Transportation Planning Analysis Unit (TPAU) to make a recommendation to the State Traffic-Roadway Engineer. If the information provided is insufficient or not appropriate in methodology (as determined by the Department) the State Traffic-Roadway Engineer may request further analysis.

The approval process for Roundabouts is divided into two phases: Conceptual Approval and Design Approval. The State Traffic-Roadway Engineer will make the decision whether Roundabouts will receive Conceptual Approval and move to the Design Approval phase. The State Traffic-Roadway Engineer will make the final decision on the approval of the geometric design in the next phase. Conceptual Approval must follow ODOT procedures that assure the roundabout can accommodate freight movement on the highway and this requires the Region to have conversations with the freight industry through the freight mobility committee review process (*ORS 366.215; OAR 731-012*). The State Traffic-Roadway Engineer will make the final decision on the approval of the geometric design in the Design Approval phase.

Conceptual Approval will constitute official approval under the Delegated Authorities of the State Traffic-Roadway Engineer for a roundabout to be used as traffic control at a particular intersection. For Conceptual Approval, an Intersection Traffic Control Study addressing all pertinent considerations described in this section will be required. In addition, a Conceptual Design of the intersection shall be submitted to the State Traffic-Roadway Engineer for review by Traffic-Roadway Section staff. Conceptual Approval will not be granted until Traffic-Roadway Section staff verifies that Region has followed the ODOT procedures related to vehicle carrying capacity (*ORS 366.215; OAR 731-012*).

Design Approval will constitute the final approval phase of the roundabout at a particular intersection. The geometrics of roundabout designs (including channelization plans) must be submitted to the State Traffic-Roadway Engineer for review and approval.

The Department has developed a list of considerations that should be addressed in the Engineering Investigation that is submitted for proposed roundabout locations. These considerations should not be interpreted as roundabout warrants nor should they be considered pass/fail criteria for installation of a roundabout. Rather, they have been identified as important considerations to take into account when proposing roundabout intersections on state highways.

1. Freight Mobility needs should be sufficiently defined and addressed prior to Conceptual Approval.
2. Motorized user mobility needs must be balanced with the mobility needs of non-motorized road users. The ability for bicyclists and pedestrians to safely move through the roundabout intersection is equally important as the mobility needs of motorized vehicles. Bicyclists should be given the option to use either the circulating roadway with other vehicles or the pedestrian crossings outside the circulatory roadway. Special design considerations should be given for the pedestrian crossings at the entrances and exits on all legs of the roundabout where vehicles are either decelerating to enter the roundabout or accelerating to exit the roundabout. Multi-lane roundabouts, like other multi-lane intersections, have potential for "multiple threat" conflicts between vehicles and pedestrians, particularly vision impaired pedestrians. The Public Rights-Of-Way Accessibility Guide (PROWAG) has identified the need for pedestrian-activated crossing capability at multi-lane roundabouts. Although not explicitly required at this time, rulemaking is proposed and it is prudent to design a multi-lane roundabout for easy installation of the necessary equipment in the future. Crosswalk placement, striping, installing conduit as well as identifying and reserving necessary equipment locations even though final installation of all the equipment is not necessary at this time, is good design practice and can save money in the future. Additional information can be obtained by reviewing the PROWAG document available from the FHWA Civil Rights website under Programs/ADA/Section 504.
3. Roundabout design should consider the needs and desires of the local community including speed management and aesthetics.
4. Intersection safety performance should be a primary consideration when pursuing a roundabout for intersection control. Predicted reductions in fatal and serious injury crashes should be compared with other types of intersection control such as traffic signals or other alternatives supported by crash modification factors (CMF) from the AASHTO Highway Safety Manual.
5. Roundabout entrance geometry, circulating geometry and exit geometry should be designed to allow the design vehicle to traverse the roundabout in a reasonable and expected manner commensurate with best design practices as shown in NCHRP Report 672, Roundabouts: An Informational Guide, Second Edition and the ODOT Highway Design Manual. This design should utilize a representative template of the design vehicle and the vehicle path should be demonstrated through the use of computer generated path simulation software,
6. Roundabouts should meet acceptable v/c ratios for the appropriate Design Life. (See the Design Life subsection for possible exceptions to this consideration.)
7. Roundabouts proposed for the state highways with posted speeds higher than 35 mph will require special design considerations (e.g. longer splitter islands, landscaping, possibly reversing curve alignments approaching the roundabout, etc.) to transition the roadside environment from higher to lower speeds approaching the roundabout intersection.
8. For Roundabouts with more than 4 approach legs, special design considerations should be made for the layout of the approach legs.

9. Roundabout proposals should address how roundabout operations would impact the corridor immediately upstream and downstream from the roundabout intersection. (If the proposed roundabout is in a location where exiting vehicles would be interrupted by queues from signals, railroads, draw bridges, ramp meters, or by operational problems created by left turns or accesses, these problems should be addressed by the Engineering Investigation.

For brevity, the following is summarized from the ODOT Traffic Manual, Section 6.26 Roundabouts and included in a bulleted, step-wise listing. For the full text, reference the ODOT Traffic Manual.

Steps in the Roundabout Selection Criteria and Approval Process include:

1. Perform an engineering Investigation including a comprehensive Intersection Traffic Control Study. In addition to site specific intersection data, the investigation should include comparisons of intersection control types (i.e. stop controlled, signal controlled, roundabout, etc.)
2. Determine design Life – generally 20 years for STIP projects and 10 years for development review.
3. Submit a scaled Conceptual Design of the proposed roundabout to the State Traffic-Roadway Engineer for approval including roundabout type, geometry, topography, influence area, approximate right-of-way required as well as other pertinent design information and impacts. Figure 8-24 illustrates major design elements of a roundabout.
4. After Concept Design Approval has been obtained, submit a refined Design Package to obtain Design Approval from the State Traffic-Roadway Engineer. This Design Package should include:
    a. Channelization plans, completed per the Department's guidance for roundabout striping found in the Traffic Line Manual and for splitter islands found in the Highway Design Manual.
    b. A summary of the documented design decisions including how the requirements of *ORS 366.215* and *OAR 731-012* (Reduction of Vehicle Carrying Capacity) are being met.
    c. Identified deviations from design standards where design exceptions might be needed.
    d. Roundabout geometric data, including:
        - Approach, entry, exit, and circulating design speeds for all approach legs including any bypass legs for right-turning vehicles. Bypass legs should be designed for speeds no more than 5 mph greater than the design speed of the circulatory roadway in order to accommodate bicycles and pedestrians crossing the bypass leg;
        - The design vehicle for each movement and accommodations for other special vehicles (e.g. permitted loads, farm equipment, etc.);

- A table or drawing summarizing the roundabout design details, including inscribed diameter, central island diameter, truck apron designed to accommodate the appropriate design vehicle for the roundabout, and cross slope of the circulating roadway;
- Detailed drawings showing the fastest path for each movement, with speed and radius for each curve;
- A table summarizing stopping and intersection sight distance on each leg; and
- Computer generated paths showing design vehicle and largest oversize vehicle movements (freight routes will help identify the oversized loads that could be expected).

5. Detailed drawings of the splitter islands on each leg. These should include pedestrian and bicycle accommodation, ramps, etc.
6. Preliminary signing and illumination plans.

8.6.4 DESIGN CONSIDERATIONS

It is the intent of the Department to ensure that the geometric design of roundabouts adheres to principals that encourage lower speeds, where appropriate, and improves safety for all users. These principals will also have traffic-calming benefits on the road system. It must be recognized that the design of a roundabout is an iterative process. Geometric layout may need to be refined several times before capacity and safety requirements can be achieved. Engineering judgment will be required to refine the layout.

The following discussion points present some basic design considerations for modern roundabouts. Additional design details and layout considerations can be obtained through consultation with the Traffic-Roadway Section of Technical Services. Roundabout designs on the state highway system shall use NCHRP Report 672, Roundabouts: An Informational Guide, Second Edition and the ODOT Highway Design Manual to determine design criteria and compliance with design standards. Where design considerations may conflict, the ODOT Highway Design Manual criteria will be used to resolve the conflict.

8.6.4.1 DESIGN VEHICLE

When designing intersections on the state highway system, ODOT makes a distinction between "designing for" and "accommodating for" large vehicles. The design vehicle for intersections on state highways is the WB-67 class Interstate Truck also known as the Interstate Design Vehicle. Vehicles larger than the WB-67 class are accommodated as necessary. In the design of roundabouts, as with other highway facilities, layouts should provide accommodation for the largest vehicles likely to use the facility. The primary consideration for designing a roundabout to allow large vehicles to satisfactorily traverse it is to select both the appropriate design vehicle and, if necessary, the appropriate accommodation vehicle. Once the vehicles have been

selected, the necessary design for entrance geometry, circulating geometry and exit geometry can be provided.

When designing a roundabout on the state highway system, the designer:

1. Shall coordinate with ODOT Motor Carrier Transportation Division and appropriate highway user groups to determine type and frequency of large vehicle traffic expected to use the roundabout.
2. Shall use a WB-67 Interstate Truck as the design vehicle, unless it has been determined through coordination with ODOT Motor Carrier Transportation Division and appropriate highway user groups that a smaller vehicle is acceptable.
3. Shall consider and accommodate as necessary, based on conversations with ODOT Motor Carrier Transportation Division and appropriate highway user groups, the need of over-dimensional vehicle passage through the roundabout.
4. Shall design entrance geometry, circulating geometry and exit geometry for all roundabouts, single lane and multi-lane, to allow the design vehicle to traverse the roundabout in a reasonable and expected manner commensurate with best practices as shown in NCHRP Report 672, Roundabouts: An Informational Guide, Second Edition and the ODOT Highway Design Manual. It is also important to remember that *ORS 811.292* and *ORS 811.370* have provision for "commercial motor vehicles" to operate outside a single lane in a multi-lane roundabout when necessary.
5. Shall design the roundabout using representative templates for the design vehicle and for any vehicles being accommodated with the design. This design will utilize the representative templates to demonstrate vehicle accommodation and vehicle pathway through the roundabout by using computer generated path simulation software.
6. Shall coordinate with ODOT Motor Carrier Transportation Division and other highway user groups throughout the design process to ensure all roundabout user expectations are being considered, including bicycle and pedestrian needs.

8.6.4.2 DESIGN SPEED AND TARGET SPEED

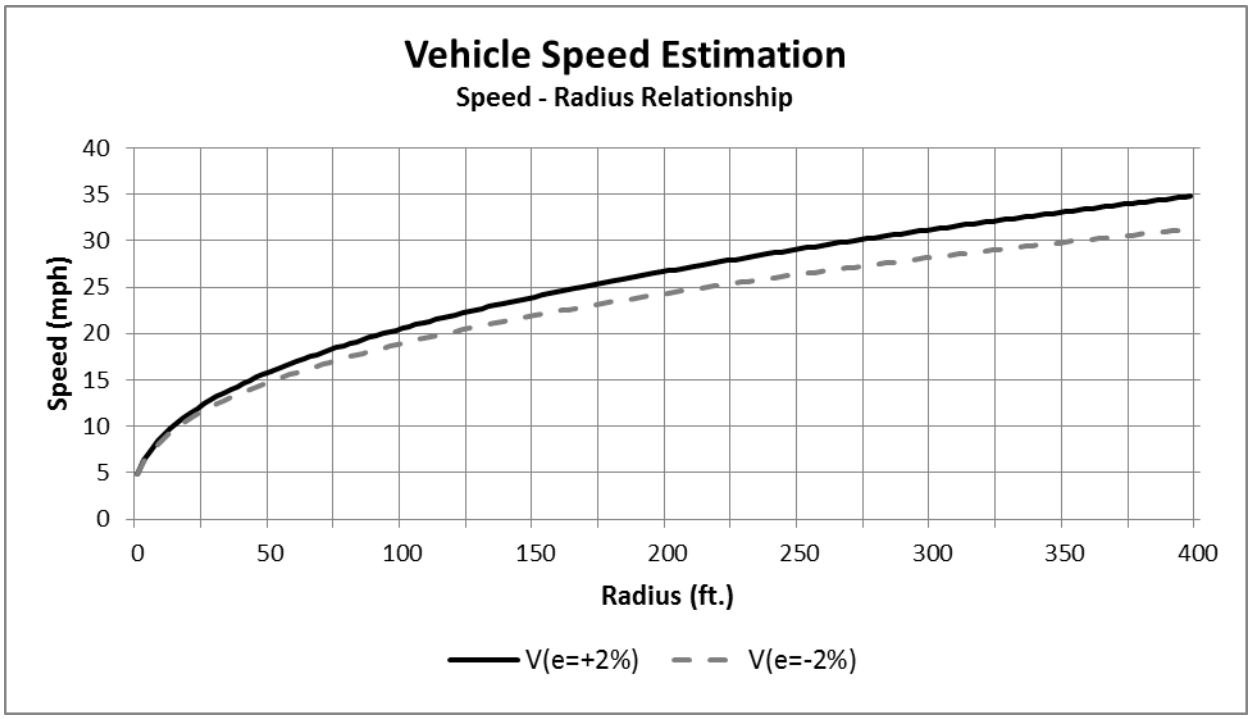


Figure 8-25: Estimated Vehicle Speed and Radius Relationship – Fastest Path

Highway designers generally use a selected design speed when designing roadway elements for a project. However, in the traditional sense of highway design, the term design speed doesn't necessarily relate well to roundabouts. Controlling speed plays an important part for safety at roundabouts. Roundabouts are purposely designed so that traveling speeds are restricted to a low and consistent speed through the roundabout. Figure 8-25 demonstrates estimated vehicle speeds based on the relationship of path geometry in the terms of radius and superelevation to corresponding theoretical velocity when calculating fastest paths through a roundabout. Superelevation for the path through a roundabout is considered to be a typical positive two percent at entrance and exit and a typical negative two percent along the circulating roadway. Table 8-3, is a tabular form of the path speed/radius relationship based on 25 foot increments in radius and the typical positive and negative two percent superelevation. The vehicle speed values shown on the graph in Figure 8-25 and in Table 8-3 are determined by utilizing the simplified equations shown in TRB Report 672, Roundabouts: An Informational Guide, Second Edition where $V=3.4415R^{0.3861}$ for e= +2% and $V=3.4614R^{0.3673}$ for e= -2%. These simplified forms are derived from the basic equation for velocity and minimum radius from the AASHTO document A Policy on Geometric Design of Highways and Streets;

$$\boldsymbol{V = \sqrt{15R(e+f)}}$$

They are only valid for superelevation values (e) of +2% and -2%. Side Friction Factor (f) varies with speed as shown in Figure 3-6 (Side Friction Factors Assumed for Design) in the AASHTO

2011, A Policy on Geometric Design of Highways and Streets and is accounted for in the equations. In an actual design, if superelevation is greater or less than the assumed positive and negative two percent shown in Figure 8-25 or Table 8-3, then theoretical fastest path speeds for the specific design will need to be calculated using the AASHTO minimum radius equation, design superelevation (e) and friction factor (f) values from the 2011 AASHTO Figure 3-6, Side Friction Factors Assumed for Design.

Table 8-3: Speed, Radius Relationship

Radius (ft.)	V(+2%) (mph)	V(-2%) (mph)
25	12	11
50	16	15
75	18	17
100	20	19
125	22	20
150	24	22
175	25	23
200	27	24
225	28	25
250	29	26
275	30	27
300	31	28
325	32	29
350	33	30
375	34	31
400	35	31

Speed (V), Radius (R) Relationship Equations:

Equation 8-1 $\mathbf{V = 3.4415R^{0.3861}}$ For e= 2% (NCHRP Report 672)

Equation 8-2 $\mathbf{V = 3.4614R^{0.3673}}$ For e= -2% (NCHRP Report 672)

Equation 8-3 $\mathbf{V = \sqrt{15R(e+f)}}$ (AASHTO Minimum Radius)

The design speed of the roundabout intersection should not be confused with the design speed of the highway. In many cases, the design speed of the approaching roadway may be greater than the speed for which the roundabout will be designed. Therefore, it is advantageous to use the term target speed when designing the roundabout layout. This will eliminate confusion with the approach road design speed. Target speed can be considered the speed of the "fastest path" of a vehicle through the roundabout. There are five critical path radii used to determine fastest path movements through a roundabout. The fastest path of a vehicle is a theoretical

analysis of entrance radius (R_1), the circulating radius (R_2), exit radius (R_3), left turn radius (R_4) and right turn radius (R_5). Figure 8-26, denotes the five critical radii that determine fastest path calculations for a roundabout. Figure 8-27(a) and Figure 8-27(b) demonstrate the method and assumptions used to calculate a fastest path through a roundabout. On the state highway system, maximum theoretical entry approach speeds for single lane roundabouts should be 25 mph. For multi-lane roundabouts maximum theoretical entry approach speeds should be limited to 30 mph. Target speeds for single lane roundabouts should be between 15 and 20 mph and between 20 and 25 mph for multi-lane roundabouts. Theoretical speeds through the roundabout (entry, circulation, exit) should be kept consistent with no greater differential than 10 mph to 15 mph maximum between entry and exit. For smaller diameter roundabouts found on local jurisdiction highways, these theoretical speeds may need to be reduced to fit the smaller design.

A safely designed roundabout should have geometry that accommodates all traffic movements at the chosen approach and target speeds, thereby maximizing safety benefits and minimizing the area needed for installation.

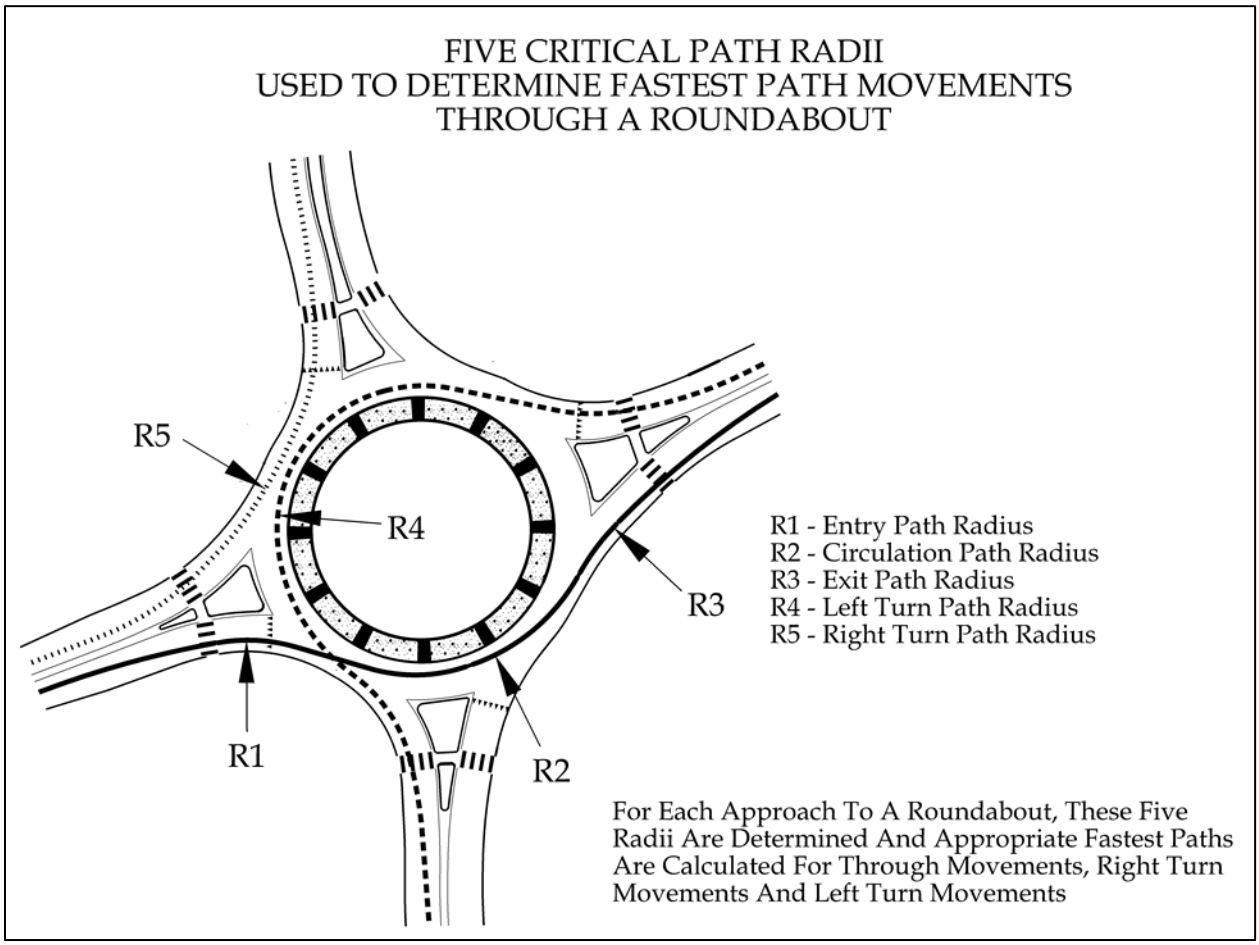


Figure 8-26: Five Critical Path Radii for Fastest Path Analysis

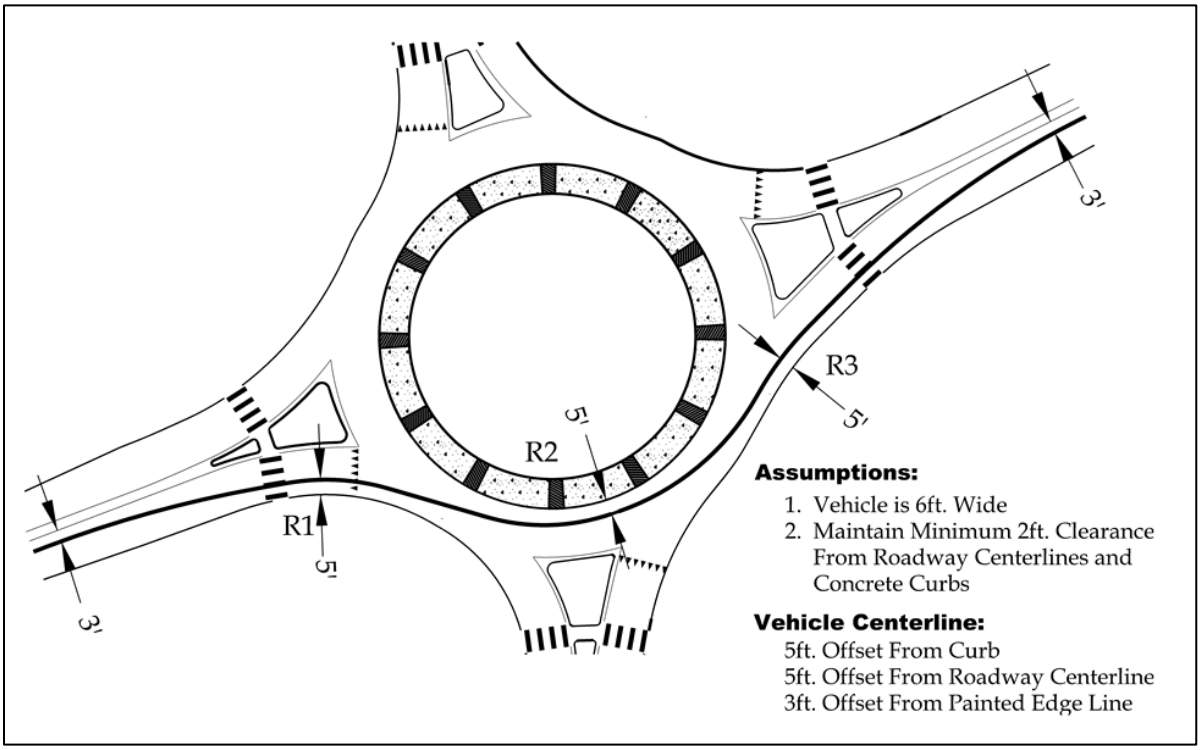


Figure 8-27(a): Fastest Vehicle Path Through a Single Lane Roundabout

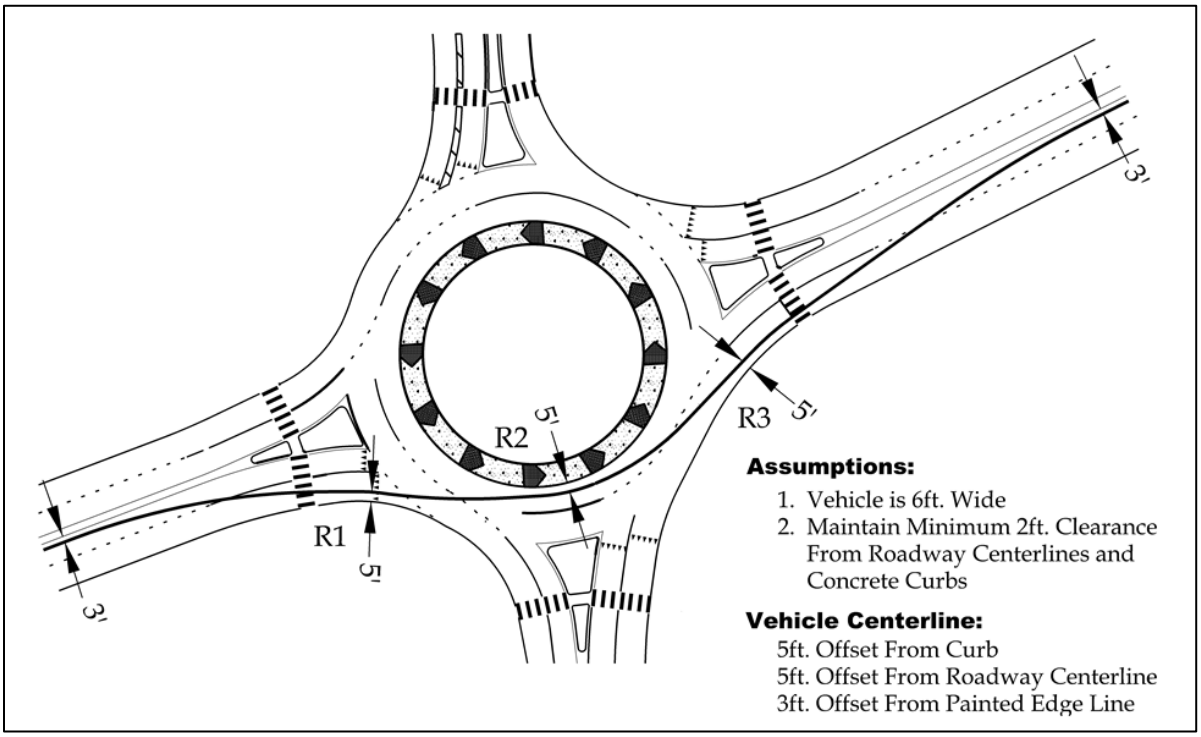


Figure 8-27(b): Fastest Vehicle Path Through a Multi-Lane Roundabout

8.6.4.3 INSCRIBED CIRCLE AND CENTRAL ISLAND

The inscribed circle is the outside edge of travel of the circulatory roadway. The central island is the raised area surrounded by the circulatory roadway. There are two areas of a central island, the mountable truck apron and the non-traversable, center, raised area. Figure 8-28 shows a typical cross-section of a roundabout including the truck apron, circulating roadway and central island.

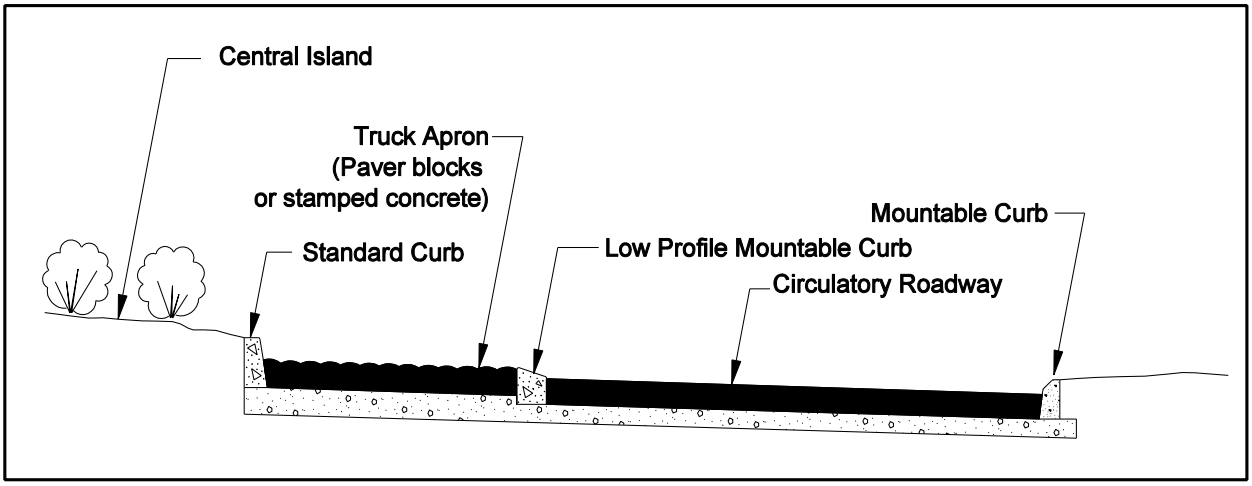


Figure 8-28: Roundabout Cross-Section

The Interstate Design Vehicle (WB-67 class truck) is the standard design vehicle for roundabouts on the state highway system. Vehicles larger than a WB-67 vehicle will be accommodated at roundabouts where necessary as determined through conversation with ODOT Motor Carrier Transportation Division and appropriate highway user groups. The truck apron is a key roundabout design element to provide passage and accommodation of the design vehicle and larger vehicles through the roundabout. Encroachment onto the truck apron is permitted and encouraged in order for large vehicles to effectively traverse a roundabout; however, vehicles smaller than the Interstate Design Vehicle may be accommodated without encroachment. To minimize circulatory roadway width for single lane roundabouts, some states use the design philosophy that the circulatory roadway should be only wide enough to allow passage of a standard bus without using the truck apron and therefore, all larger vehicles would use the truck apron for off-tracking. This is a good "rule of thumb" for initial design to minimize the circulatory roadway width if deemed necessary. However, each roundabout should be designed to provide the most appropriate design elements for the traffic stream expected to use it. In some locations where high proportions of heavy vehicles are expected, the design of adequate circulatory roadway width with minimal use of the truck apron may be appropriate. It is anticipated that these locations would be the exception and few in number, since increasing circulatory roadway width or inscribed diameter to accommodate large vehicles within the circulatory roadway will generally increase the fastest path speeds through the roundabout for smaller vehicles, thereby potentially negating some of the safety benefits afforded by roundabouts. A balance must be maintained between accommodating large vehicles and the safe, effective passage of general traffic for which the roundabout is intended.

NCHRP Report 672, Roundabouts: An Informational Guide, Second Edition lists ranges of acceptable inscribed diameters for both single lane and multi-lane roundabouts. For a WB-67 vehicle and a single lane roundabout, suggested inscribed diameters are from 130 feet to 180 feet and for multi-lane roundabouts the suggested range is from 165 feet to 220 feet for 2-lane roundabouts and up to 300 feet for 3-lane roundabouts. However, NCHRP Report 672 was written to cover roundabouts in all applications including national highways, state highways and local jurisdictions. Therefore, the entire range of diameters may not be appropriate for state highways. For general design parameters on the state highway system, the minimum inscribed circle diameter for a single lane roundabout accommodating the Interstate Design Vehicle is 165 feet and the minimum inscribed circle diameter for a multi-lane roundabout accommodating the Interstate Design Vehicle is 200 feet. If a smaller vehicle than a WB-67 class vehicle has been deemed the appropriate design vehicle, a smaller inscribed diameter may be acceptable. Use of inscribed diameters smaller than the minimums described above require design concurrence and/or design exceptions. Contact the Technical Services, Traffic-Roadway Section for guidance.

Table 8-4: Inscribed Diameters

	INSCRIBED DIAMETER				
	NCHRP Report 672			ODOT Minimum	
Design Vehicle	Single Lane	Multi-Lane		Single lane	Multi-Lane
		2-Lane	3-Lane		
WB-67	130 ft - 180 ft	165 ft - 220 ft	up to 300 ft	*165 ft	*200 ft

* Design Exception Required For Smaller Inscribed Diameters

In addition to design vehicle considerations, there are many other factors to consider when determining the inscribed diameter. There may be locations where a smaller inscribed diameter is appropriate to accomplish overall intersection control goals. These locations should be considered on a case-by-case basis and designed accordingly to achieve the necessary intersection control. These designs may be based on a smaller design vehicle if deemed appropriate through conversation with ODOT Motor Carrier Transportation Division and the requisite highway user groups. If a WB-67 class vehicle is the design vehicle and a smaller diameter than the standard diameter is proposed, then the truck apron may need to be widened for accommodation. However, widening the truck apron will reduce the central Island diameter and may create undesirable visibility and sight lines across the roundabout. If a non-standard inscribed diameter is proposed for a design, contact the Technical Services, Traffic-Roadway Section for guidance.

Once the inscribed diameter has been established, circulatory roadway width and truck apron width can be determined. The circulatory roadway is the area between the outside curb and the truck apron. This is the area where the majority of traffic will traverse the roundabout. For single lane roundabouts, circulatory roadway widths should provide adequate width for most vehicles to comfortably maneuver through the roundabout, provide for some off-tracking of

larger vehicles up to the design vehicle, but not be so wide that drivers may feel there is more than one lane in the roundabout.

For all roundabouts, circulatory roadway width is based on the number of entering lanes and the turning requirements of the design vehicle. Generally, the circulating width should be at least as wide as the maximum entry width and in some cases it may be appropriate to increase the width up to 120 percent of entry width. The recommended circulatory roadway width for a single lane roundabout on the state highway system is 21 feet, excluding the truck apron width. For multi-lane roundabouts, the suggested circulating width is 14 feet to 16 feet per lane or 28 feet to 30 feet for a two-lane roundabout on the state highway system. The suggested circulatory roadway widths are based on general design characteristics. Circulating widths for specific designs should be checked using design vehicle turning characteristics and overall intersection control parameters governing the intended need for the roundabout installation. Circulatory roadway width should not jeopardize intended speed control of a roundabout.

Central island truck aprons are an integral design element of a roundabout that provides accommodation for large vehicles while maintaining deflection and design controls for general traffic to achieve effective roundabout design at an intersection. A truck apron should be designed in such a way that mounting over by a passenger car would feel uncomfortable but not unsafe. Truck aprons shall be designed to allow for efficient transition to and from the circulatory roadway. Modified, low profile curbs no higher than 3 inches shall be used for delineation and transition between the circulatory roadway and the truck apron. Curbs for the truck apron shall be installed flush with the circulatory roadway. See Figure 8-29.

Truck apron width is determined by turning requirements of the design vehicle and other large vehicles being accommodated through the roundabout. Vehicle paths can be simulated using computer software to determine off-tracking needs.

In general, past design practice set cross-slope of the truck apron at 2% from the roundabout center to the apron curb (-2%). However, more recent design philosophy is leaning to utilizing a 1% cross-slope to better accommodate specific large vehicle

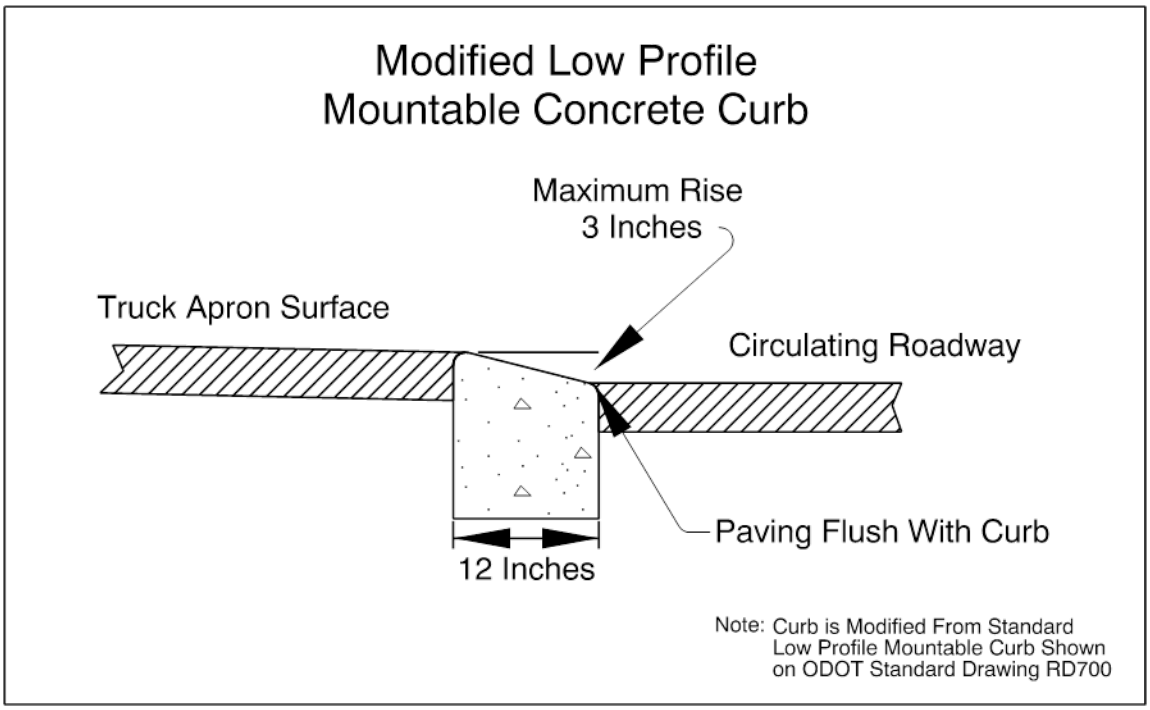


Figure 8-29: Truck Apron Mountable Concrete Curb

combinations. Truck apron cross-slope needs to be carefully determined in order to not introduce undesirable dynamics to large vehicles as they traverse the apron. This is particularly true when accommodating low–boy trailers, oversize loads, loads with high centers-of-gravity or loads that can shift, like bulk liquid loads. Low-boy trailers can pose particular problems with the vertical profile between the apron and the circulating roadway. Some low-boy trailers have only six inches of clearance from the ground to the bottom of the trailer frame. Truck apron cross-slope should be only as steep as necessary to provide adequate drainage. Smooth transitions between the circulating roadway and the apron are crucial to effective design and in most all cases should not be greater than 2% in differential slope.

Cross-slope of the circulating roadway is also usually at 2% outward (-2%) keeping the truck apron and circulating roadway relatively parallel with each other. Figure 8-30(a) Illustrates typical truck apron and circulating roadway cross-slope. Advantages to this cross-slope design include:

1. Raising the central island and improving its visibility,
2. Lowering circulating speeds by introducing adverse superelevation,
3. Minimizing breaks in the cross-slope of the entrance and exit lanes. And
4. Helping drain surface water to the outside of the roundabout minimizing the drainage system.

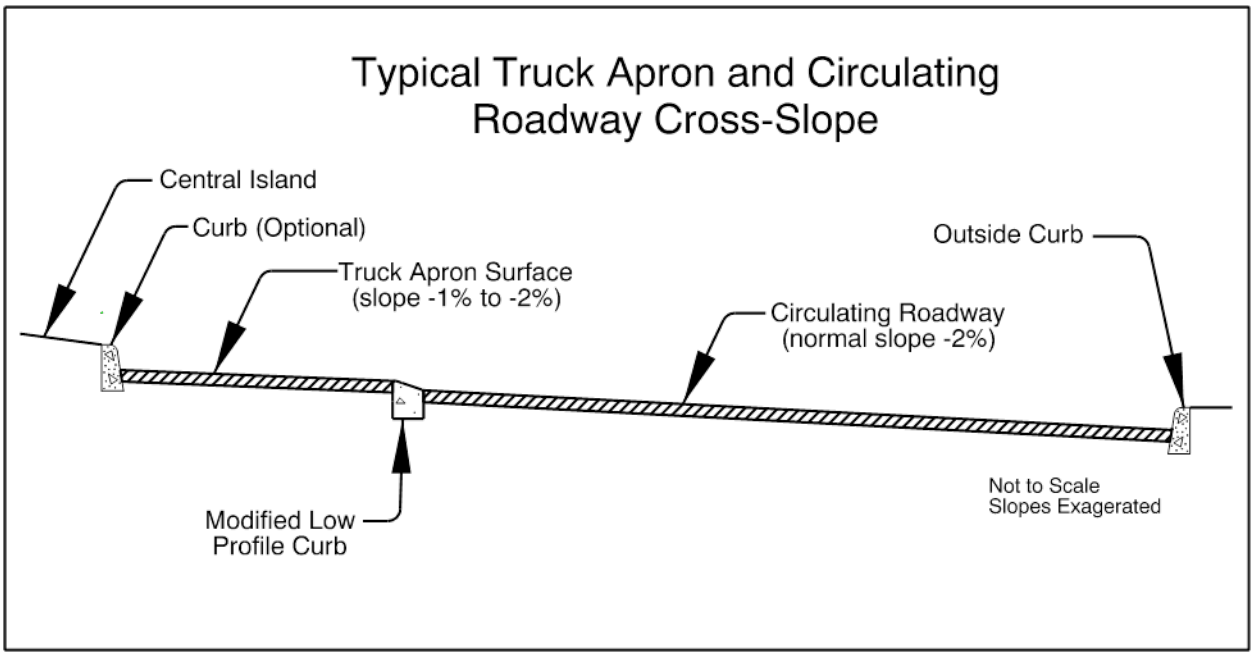


Figure 8-30(a): Typical Truck Apron and Circulating Roadway Cross-Slope

In the past, significantly altering the cross-slope relationship between the truck apron and the circulating roadway was generally not an accepted practice. However, more recent research and analysis investigating varying this relationship from the typical -2% across the truck apron and circulatory roadway has shown there may be some benefit to certain vehicle movements through roundabouts, as well as potential drainage benefits. Some agencies have opted to slope the truck apron inward toward the central island. In locations subjected to high incidence of precipitation, this option can reduce runoff across the circulating roadway. This can also have a beneficial effect of less ice buildup on the circulating roadway in colder climates. Depending on adjacent geometry of a particular roundabout, sloping the truck apron inward can also have a positive effect in minimizing the potential for load shifting.

Some agencies are developing roundabout geometries that include a crown section on the circulating roadway. In this option, the inner portion of the circulating roadway is sloped inward towards the truck apron and the outer portion is sloped outward away from the truck apron. The crown section is usually divided into two-thirds of the circulating roadway width sloping inward and one-third sloping outward. The roadway width could also be divided in a half inward and a half outward scenario. Figure 8-30(b) illustrates the crowned circulating roadway concept.

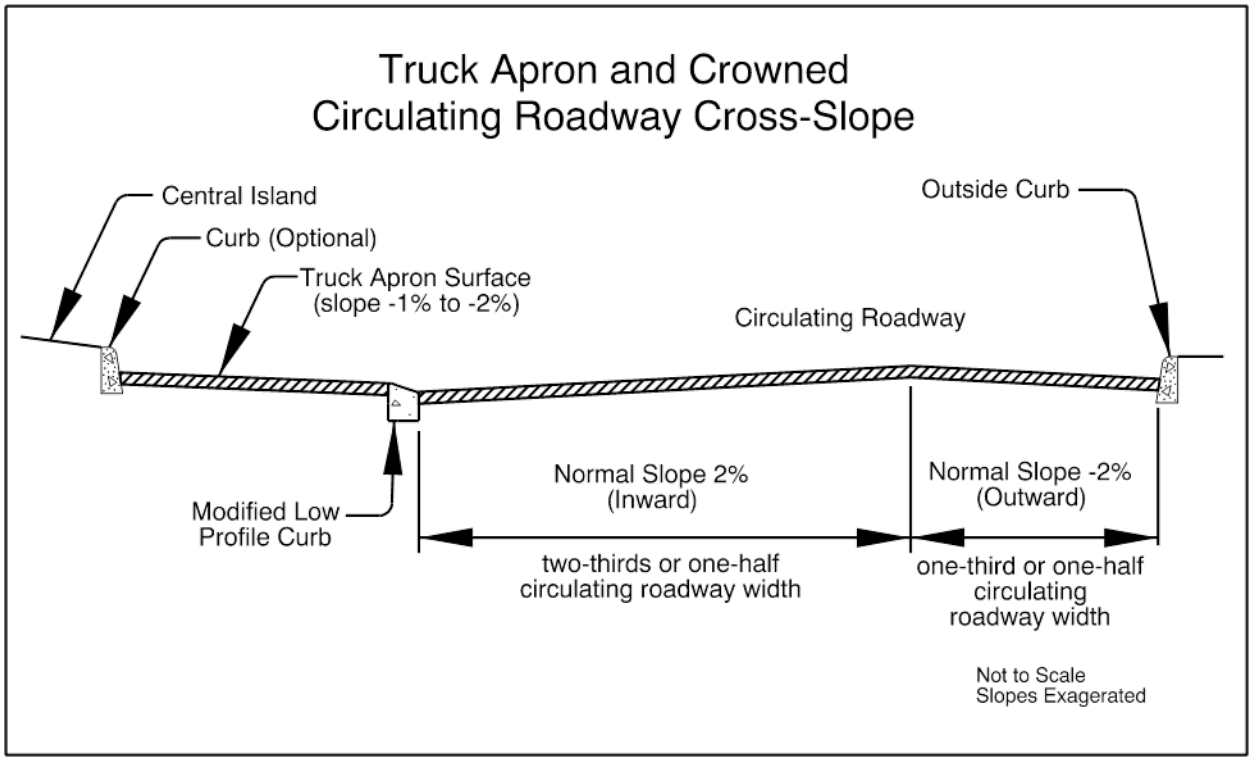


Figure 8-30(b): Truck Apron and Crowned Circulating Roadway Cross-Slope

Agencies that are developing these alternative cross-sections feel they may be of benefit in accommodating oversize and overweight vehicles at roundabouts. The theory is to minimize vertical movement as a large vehicle transitions on and off the truck apron. Disadvantages to using a crowned circulating roadway section are;

1. More inlets are required to handle the drainage and the drainage system is more complex with the potential for increased maintenance.
2. The crown section introduces a break point in the vehicle path at entrances and exits that must be adequately blended for both comfort and vertical clearance problems
3. Sloping the circulating roadway inward reduces or eliminates the adverse superelevation of the fastest path through the roundabout. This can increase vehicle speeds on the circulating roadway.

The alternative roundabout cross-sections discussed in this section are not the preferred cross-section for roundabouts on the state highway system in Oregon. They are discussed here because some agencies are using them and they seem to have benefits in certain locations. However, their use is not wide spread and more information is needed to understand if there are unforeseen negative impacts.

However the cross-section of a roundabout is designed, the vertical profile that a vehicle traversing a roundabout follows is a critical piece of the overall roundabout design. Designers must analyze the design profile for the paths of all vehicles that will be using the roundabout.

This is particularly important for large vehicles that will need to utilize the truck apron and for low-boy trailers with limited ground clearance. The vertical clearance can be checked by drawing a chord across the truck apron in the position of the trailer's swept path. It is also important to analyze vertical clearance along the circulatory roadway itself. In some cases, the warping of the profile to blend transitions at exits and entrances can create high spots that a turning trailer may contact under dynamic loading or twisting of the trailer frame.

There is no set truck apron width. It needs to be wide enough to accommodate appropriate vehicle movements. A 10 foot width is a good starting point. Large vehicles making left turns will generally have the greatest off-track. Apron width may need to be increased to accommodate this move for some vehicles. Truck aprons and the corresponding central island do not necessarily need to be round. There are examples of oval shaped central islands and odd shaped aprons that have been used to accommodate specific vehicles. Truck aprons utilizing "cut-out" central island sections have also been employed in order to optimize truck movements at some locations. Figure 8-31 illustrates modifying the truck apron and central island to accommodate truck movements.

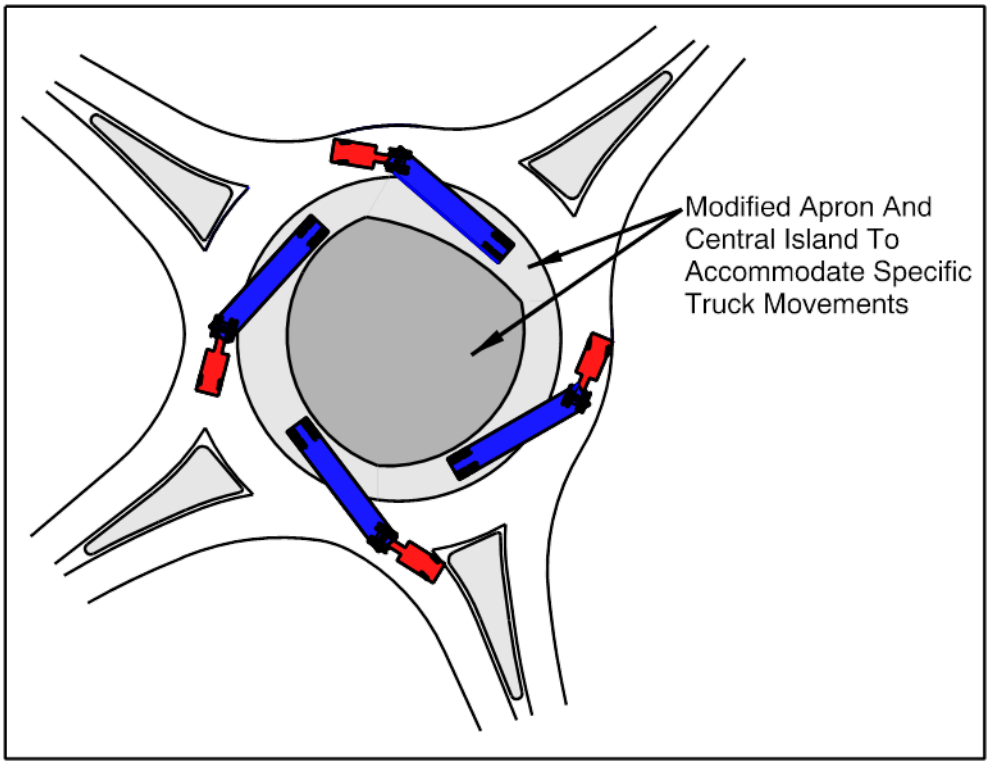


Figure 8-31: Modified Truck Apron

Modifying the central island and truck apron can be beneficial in small diameter roundabouts by keeping the footprint small and still provide accommodation for large vehicles. This can also work well at normal sized roundabouts that accommodate oversize vehicles. However, care must be taken in not creating an apron wider than necessary. Widening the truck apron will decrease the remaining raised center area. One important reason for the raised center area is to provide a visual screen using vegetation to restrict visibility from one side of the roundabout to

the other. The center area needs to be visible to approaching drivers to indicate to them the existence of the roundabout. If an approaching driver can see across the roundabout, there may be a tendency to think the road continues straight through the intersection and the driver may be unaware of the necessity to deviate and maneuver around the circulatory roadway. Long range approach visibility of the central island is important at all roundabouts, but it is paramount at rural locations where approaching vehicles are traveling at a greater speed differential between normal roadway speed and roundabout entrance speed. A driver needs time to understand and slow down on approach to the entrance.

In a positive sense, wider aprons can increase sight distance to the left for a driver judging a gap when entering a roundabout. Balance needs to be maintained between a truck apron wide enough to accommodate vehicles and aid in entering sight distance, but not create visibility or recognition problems for approaching traffic. If a roundabout's inscribed diameter needs to be in the smaller end of the suggested NCHRP 672 range for design, a wider apron may be necessary to accommodate large vehicles. Designing for these situations needs careful consideration to ensure compromises made do not negatively affect overall roundabout performance.

8.6.4.4 ENTRY/EXIT GEOMETRY AND LAYOUT

Entrance and exit geometry and layout are critical to effective roundabout design. There are four key considerations when designing roundabout entrances and exits. They include;

1. Approach alignment;
2. Angle between approaches;
3. Entry/exit width, and
4. Entry/exit curve radii.

A. Approach Alignment

There are three general types of approach alignment. They include;

1. Alignment offset left of center;
2. Alignment with center, and;
3. Alignment offset right of center.

Figure 8-32 illustrates the three alignment types.

1. Alignment Offset Left of Roundabout Center
    a. Advantages
        - Increased deflection for better entry speed control

- Potential for larger entry radii to better accommodate large vehicles with smaller inscribed diameters
- May reduce impacts to right side of approach roadway

b. Disadvantages

- Potential for tangential exit or increased exit radii creating less speed control on exit
- May create greater impacts to left side of approach roadway

2. Alignment With Center of Roundabout

a. Advantages

- Reduces alignment changes along approach roadway to keep impacts centered
- May provide for more consistent entry and exit radii and more consistent speed
- Centers approach on roundabout center and may make roundabout more visible to approaching drivers.

b. Disadvantages

- May require a slightly larger inscribed diameter to maintain speed control compared to left offset style
- May be more difficult to control approach speeds

3. Alignment Offset Right of Center

a. Advantages

- May Improve view angles in some locations
- May help in large inscribed diameters, if speed can be controlled

b. Disadvantages

- Less potential for appropriate deflection to control entry speed
- Decreases exit radii creating greater speed differential through roundabout
- Creates potential for uncomfortable forces acting on vehicle occupants

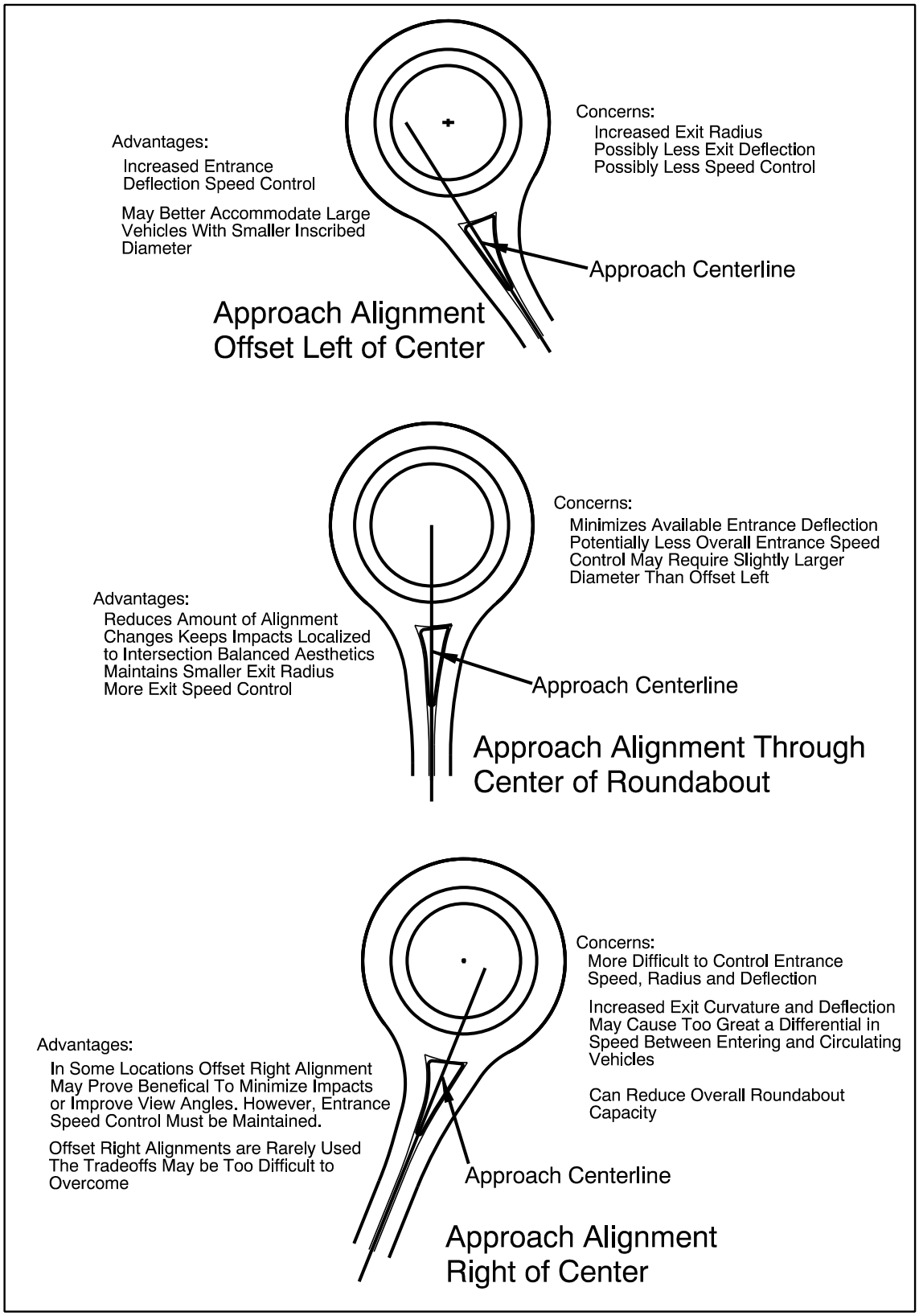


Figure 8-32: Approach Alignment

Of the three types of approach alignments discussed, alignments offset left or alignments with the center are preferred for roundabout design on state highways. Approach alignments offset right are discouraged and should not be used. Offset right alignments will require design concurrence through the ODOT Technical Services, Traffic-Roadway Section and the state Traffic-Roadway Engineer.

B. Angle Between Approaches

As with stop controlled or signalized intersections, the angle between approaches is important to the overall design of a roundabout. All approaches should be designed as perpendicular to each other as possible. This approach design will help ensure sufficient separation between two adjacent legs. Approaches built too close together, can lead to potential traffic conflicts due to the entering driver being unaware of an entering vehicle on the upstream approach leg. In addition, if two successive approaches meet at an angle significantly greater than 90 degrees, it will often result in excessive speed of right turning vehicles. Alternatively, if two successive approaches form an angle significantly less than 90 degrees, then the difficulty for larger vehicles to successfully move through the turn is increased. Figure 8-33 demonstrates difficulties with approach angles too great or too small.

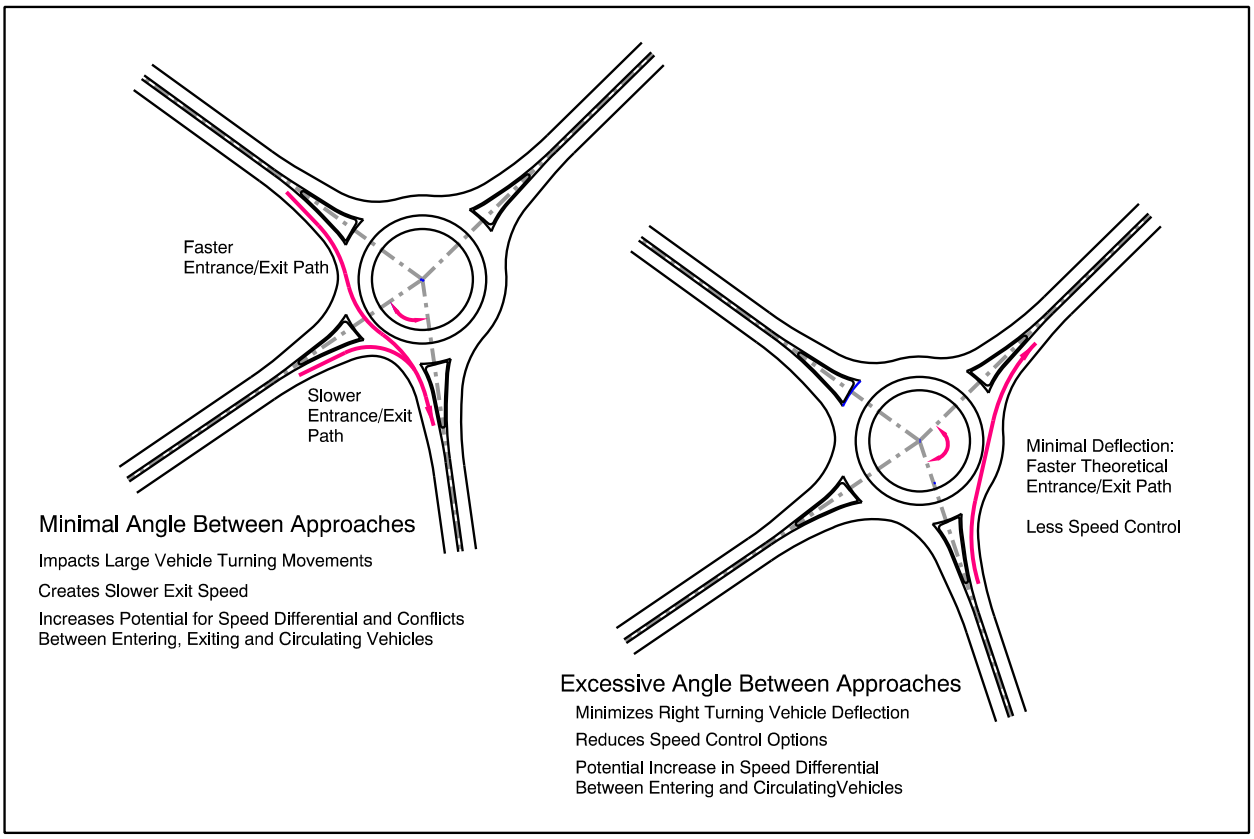


Figure 8-33: Angle Between Approaches

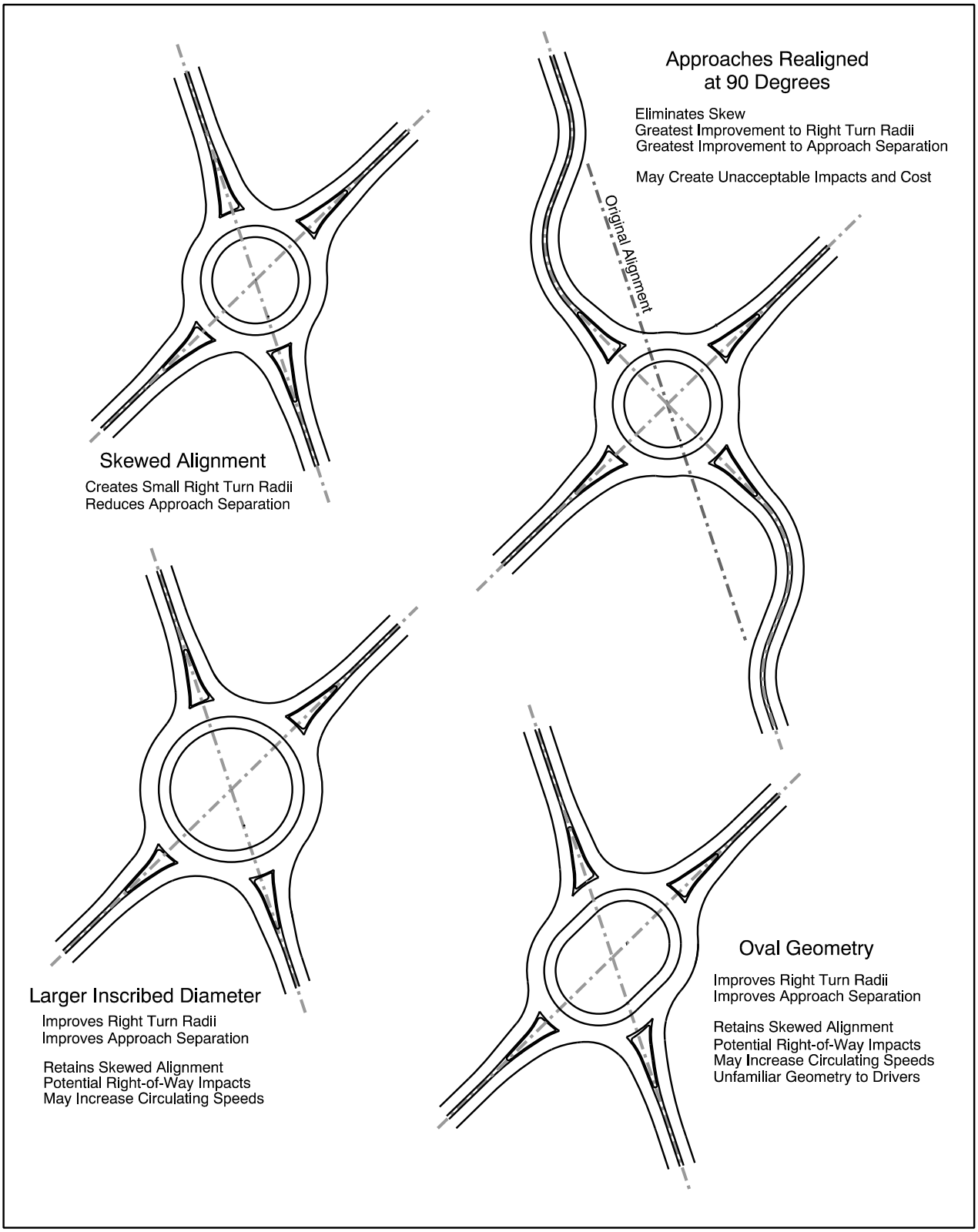


Figure 8-34: Skewed Alignments

As with designing any intersection improvement, conventional or roundabout, it may be difficult if not impossible to provide perpendicular approach connections. Right-of-way, topography and existing structures are only a few of the potential restrictions and conflicts designers face when trying to improve skewed intersection alignments. When it is not possible to re-align approaches to 90 degrees, it may be possible to increase the inscribed diameter or to change the overall geometry from a circle to an oval to achieve a balance between entry design, exit design and speed control. However, care must be taken to not compromise the overall roundabout design or project parameters. Increasing the inscribed diameter or developing an oval roundabout can improve adjacent approach geometry, but these designs can also increase roundabout speeds to the point of negatively impacting the overall design. Also, an oval geometry may have greater right-of-way impacts as well as being too unfamiliar to drivers, thereby creating the potential for confusion. Figure 8-34 illustrates a skewed alignment and the three options to make approach alignment improvements and the potential trade-offs when using them. A fourth option could be a combination of these design adjustments. Improving the skew with a minor alignment change and a small increase in inscribed diameter may be sufficient to provide acceptable approach geometry, while minimizing impacts to adjacent properties. For simplicity in presenting the concepts, illustrations in the previous figures all have the individual approach alignments meeting at the center of the roundabout. Using approach alignments other than center alignments as shown in Figure 8-32 could also help to create acceptable overall approach spacing at skewed locations. Even though a roundabout contains skewed approaches, it may still provide improved safety and operations over the existing skewed intersection it is replacing.

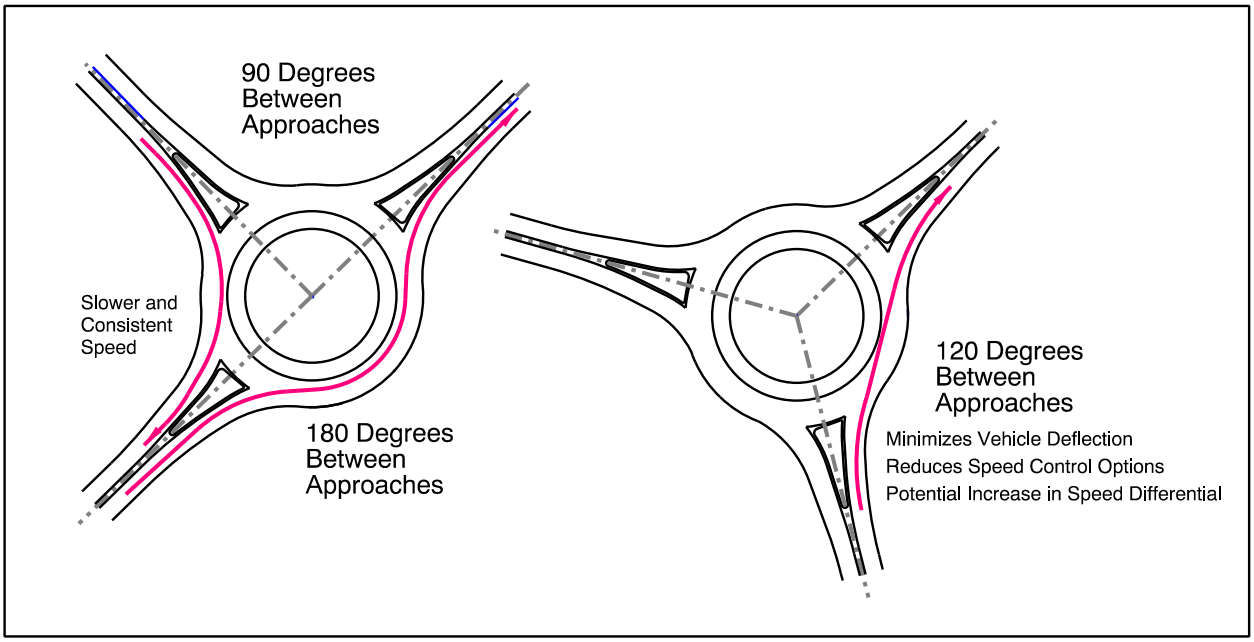


Figure 8-35: Three Legged Approaches

By their nature, roundabouts with 3 or 5 (or more) approaches can be difficult to provide appropriate deflection, speed control and right turning radii. Roundabouts with only three

approaches may have large angles between approaches allowing for less deflection and higher entrance and exit speeds. Roundabouts with five or more approaches present challenges not so much in achieving deflection, but in providing sufficient turning radii at some or all right turn movements, as well as challenges providing preferred entry design. For roundabouts with three approaches, in order to achieve appropriate deflection and speed control, it is preferred, as much as possible, to align two of the approaches at 180 degrees with each other and the third approach at 90 degrees with the other two rather than aligning all three at 120 degrees with each other. Figure 8-35 depicts three legged roundabout approach alignment.

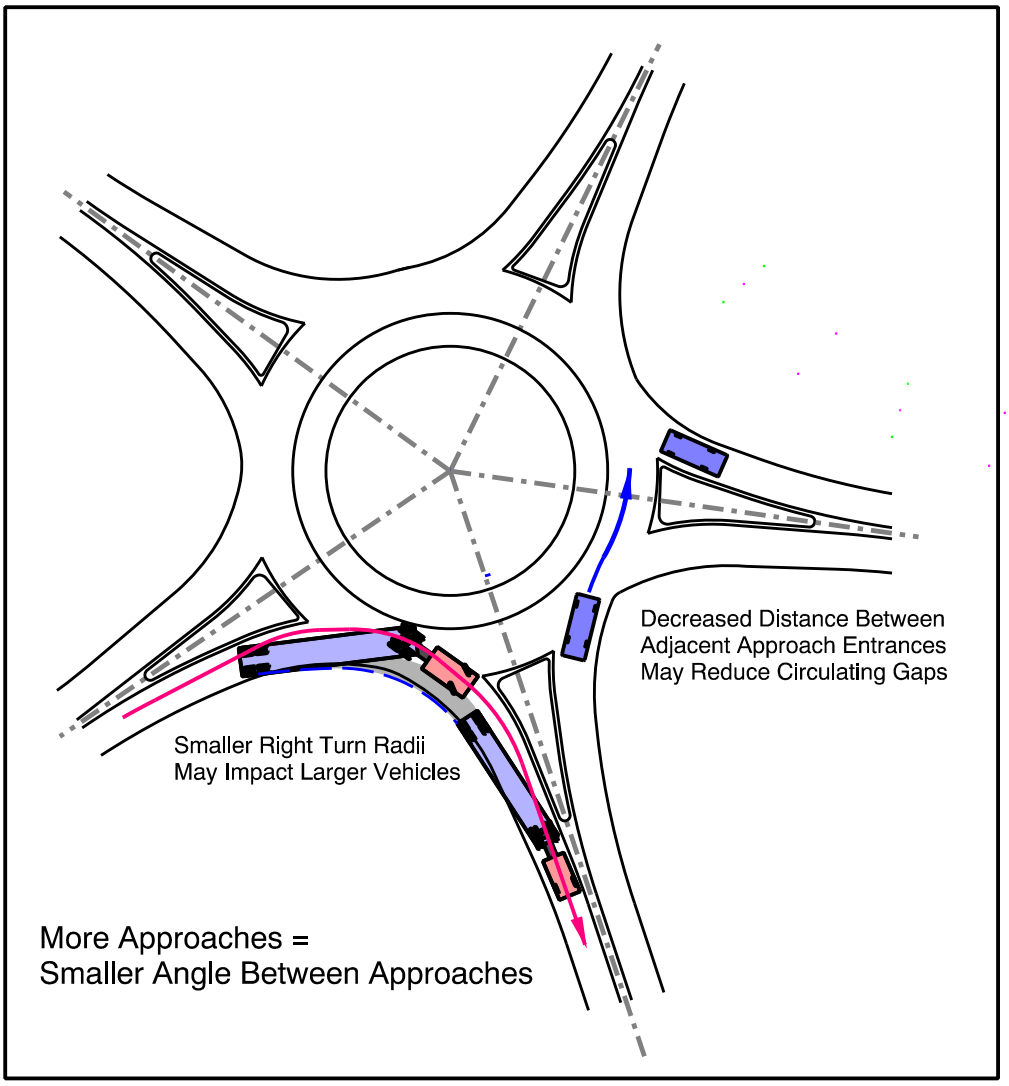


Figure 8-36: Five Approach Roundabout

Figure 8-36 portrays a roundabout with five approaches and some of the inherent problems with roundabouts comprised of more than four legs. Roundabouts with more than four approaches present challenges with approach angles and with entry and exit parameters. In general, the more approaches there are, the smaller the angle between the approaches. These

roundabouts will need special design considerations to achieve an effective design. Contact the Technical Services Roadway Unit to discuss options when laying out a roundabout with more than four approach legs.

C. Entry and Exit Width

Entry width and exit width are also important factors in creating effective roundabout design. These widths are dictated by the needs of the traffic stream based principally on the design vehicle. However, vehicle needs must be balanced against necessary speed management and pedestrian crossing needs. Single lane roundabouts generally employ widths between 14 ft. and 18 ft. Although, in some locations, these widths may be increased if deemed appropriate. For multi-lane roundabouts, required entry and exit widths depend on the number of lanes entering or exiting. Typical widths for a two-lane roundabout range from 24 ft. to 30 ft. However, as with single lane roundabouts, these widths may be increased for specific vehicle accommodation when necessary, keeping in mind the balance with other roundabout design needs and parameters.

D. Entry and Exit Geometry

Along with entry width and exit width, entrance and exit geometry helps control speed in roundabout design. Entrance and exit geometry can have an effect on capacity and safety. Entrance radii designed too small may potentially create single vehicle crashes due to abrupt changes in vehicle path alignment. Entrance curve radii set too large may increase entry speeds and a fastest path greater than desired. Entrance radii are generally in a range from 50 ft. to 100 ft. However, there is no single appropriate radius for all designs. Entrance radius should be appropriate to control entrance speed, but still provide the necessary room for large vehicles to enter the circle without hitting the curb. For some locations, compound radii may be the best solution.

Exit radii are generally larger than entrance radii to allow for consistent or slightly increased flow at the exit. Exit radii should not be designed smaller than entrance radii. When exit radii are smaller than entrance radii, the potential exists for congestion and crashes at the exit. However, if exit radii are too large, speeds may be too great at the downstream pedestrian crossing. Exit pathways must balance exit speed in relation to predicted fastest path speeds from entrance and circulating geometries along with pedestrian crossing needs. Research has demonstrated correlation between observed exit speed and a vehicle's ability to accelerate on the circulating roadway as it approaches the exit to the roundabout. Approach alignments left of center are beneficial for entrance geometry deflection and entrance speed control, but they can also have a tendency to create flatter horizontal exit geometry that may have potential for greater acceleration and higher than acceptable speed upon exiting the roundabout. Roundabout designers must provide a consistent and controlled path for vehicles to enter, traverse and exit a roundabout at an appropriate speed. It may take several design iterations to achieve acceptable entrance and exit geometry for a roundabout location.

The generally accepted method to predict entrance and exit speed for design is to use the speed, radius relationship as previously discussed in section 8.6.4.2. However, research projects from 2004 and 2007 have developed an alternate method of predicting vehicle speeds for entrances and exits of roundabouts. These research projects observed vehicle operation at roundabouts throughout the country and determined that in some locations, the actual vehicle speeds observed did not match predicted speeds. The intent of the two research projects was different, but they both developed an alternate method to match observed speeds with predicted design speeds at roundabout exits. The method is based on the standard Newtonian equation for uniform acceleration. Although equations were developed for both entrance speed and exit speed, it is recommended by NCHRP Report 672, Roundabouts: An Information Guide, Second Edition that the standard method using the speed, radius relationship should be used for prediction of entrance speed, while the alternate method may be used for exit speed.

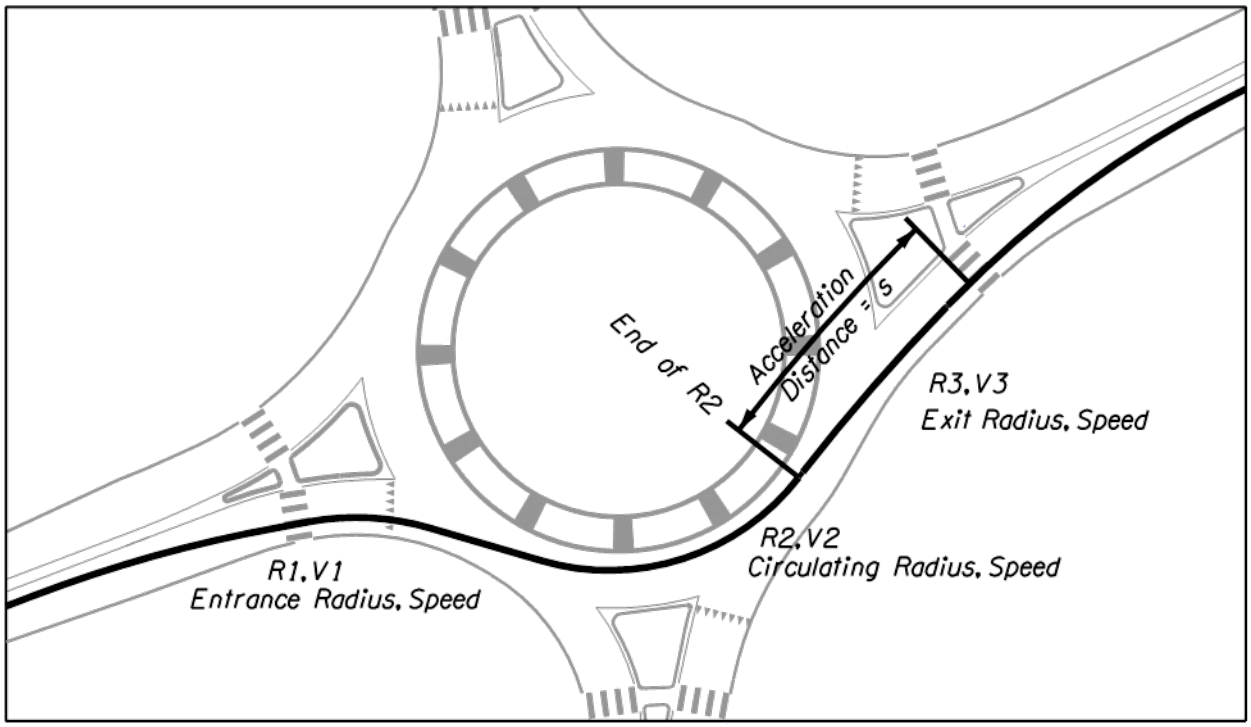


Figure 8-37: Exit Geometry – Alternate Speed Prediction Method

Newtonian Equation for Uniform Acceleration to Predict Roundabout Exit Speed

Equation 8-4 $$V_f^2 = V_i^2 + 2aS$$ (Figure 8-37)

Where: V_f = Final R3 Speed, ft/s (V3 – exit speed)
V_i = Initial R2 Speed, ft/s (V2 – circulating speed)
a = Acceleration , ft/s^2
S = Distance, ft (End of R2 to Crosswalk)

Since, as a general rule, larger exit radii will increase the overall roundabout capacity by allowing exiting vehicles to exit faster than entering vehicles, some roundabout designs incorporate a large exit radius that creates an almost tangential alignment for exiting vehicles. The concept is to maximize flow at the exit and, thereby, create greater gaps for entering vehicles. These designs are based on the alternate method of exit speed prediction using uniform acceleration calculations. This may work well to increase capacity and designers who prefer this type of design feel that opening up the exit geometry may provide drivers with a better line of sight to pedestrians and the crosswalk area as well. However, the potential for loss of consistent speed control at the downstream crosswalk is a major disadvantage. Limiting the acceleration distance and determining appropriate acceleration rates are critical to predicting potential exit speed with these types of designs. See Appendix P, Analysis for Roundabout Entrance and Exit Geometry, for additional information and discussion about larger radius or tangential roundabout exits and the proposed alternate calculation method.

There is significant discussion between roundabout designers about the best method to determine exit geometry and to control exit speed within design parameters. As a result, currently there is no definitive answer to what is the best method to predict entrance and exit speed when designing a roundabout. Research has shown that in some cases where exit radii are not excessively large and/or acceleration distances are short limiting a vehicle's ability to accelerate prior to the exit crosswalk, opening up exit geometry may not have a great effect on exit speed. However, relaxed exit geometry that increases acceleration distances and acceleration rates can potentially have significant effects on a vehicle's speed at the exit crosswalk thereby impacting pedestrian movements and, potentially, pedestrian safety. This is particularly true for multi-lane roundabouts in off-peak times when a vehicle's fastest path may cross adjacent lanes. In any roundabout layout, it is the designer's responsibility to provide vehicle alignments that consistently control vehicle speeds from entrance to exit in an effective manner for all modes of transportation utilizing the roundabout. For this reason, ODOT's preferred method of design is to use smaller, radial alignments for entrance and exit layout when predicting vehicle speed into, through and out of a roundabout. There may be some rural locations where pedestrian activity is expected to be low or locations where pedestrian activity is restricted or prohibited that a large radius or tangential exit design might be acceptable. However, for roundabouts designed on the state highway system, appropriate radius values that effectively provide design entrance, circulating and exiting speeds shall be determined using the speed, radius relationship discussed in section 8.6.4.2 of the ODOT Highway Design Manual using Equation 8-1, Equation 8-2 or Equation 8-3, Figure 8-25 or Table 8-3 to determine appropriate fastest paths for roundabout design. For additional guidance on roundabout entrance and exit geometry design, contact the ODOT Technical Services, Traffic-Roadway Section.

E. Entrance and Exit Aprons

Depending on overall geometry, large vehicles can have difficulties negotiating entrances and exits to roundabouts. Like aprons added to central islands to aid vehicle off-tracking, truck aprons positioned on the entrance and/or exit curves have been utilized at some roundabout locations to accommodate potential off-tracking needs. While these aprons are advantageous

for the movement of large vehicles through the roundabout, they can be counter-productive for the roundabout as a whole by providing an alternate fastest path that allows too great a speed for smaller vehicles, thereby, diminishing the overall effectiveness of the roundabout. These types of entrance and exit aprons should not be a general design element included in all roundabout designs. Rather, their design should be approached with caution and should be reserved for when they are needed as a necessity to accommodate specific vehicles. Effective entrance and exit geometry to control speeds of smaller vehicles must be maintained along with the design of truck entrance aprons. Figure 8-38(a) demonstrates an oversize vehicle off-tracking onto an entrance apron.

When entrance or exit aprons are used, they need to be designed to allow access by large vehicles, but designed to discourage their use by smaller vehicles in order to maintain the overall roundabout design parameters. Entrance and exit apron design is similar to central island truck apron design (See Figure 8-29). Using entrance and/or exit aprons may create potential design compromises that need to be understood and analyzed as appropriate for the overall roundabout design at any specific location. Entrance and exit aprons should only be used when all other design options have been evaluated and they are the only reasonable alternative to provide accommodation for large vehicles through the roundabout. Figure 8-38(b) demonstrates an oversize vehicle swept path through a single lane roundabout utilizing an entrance apron.

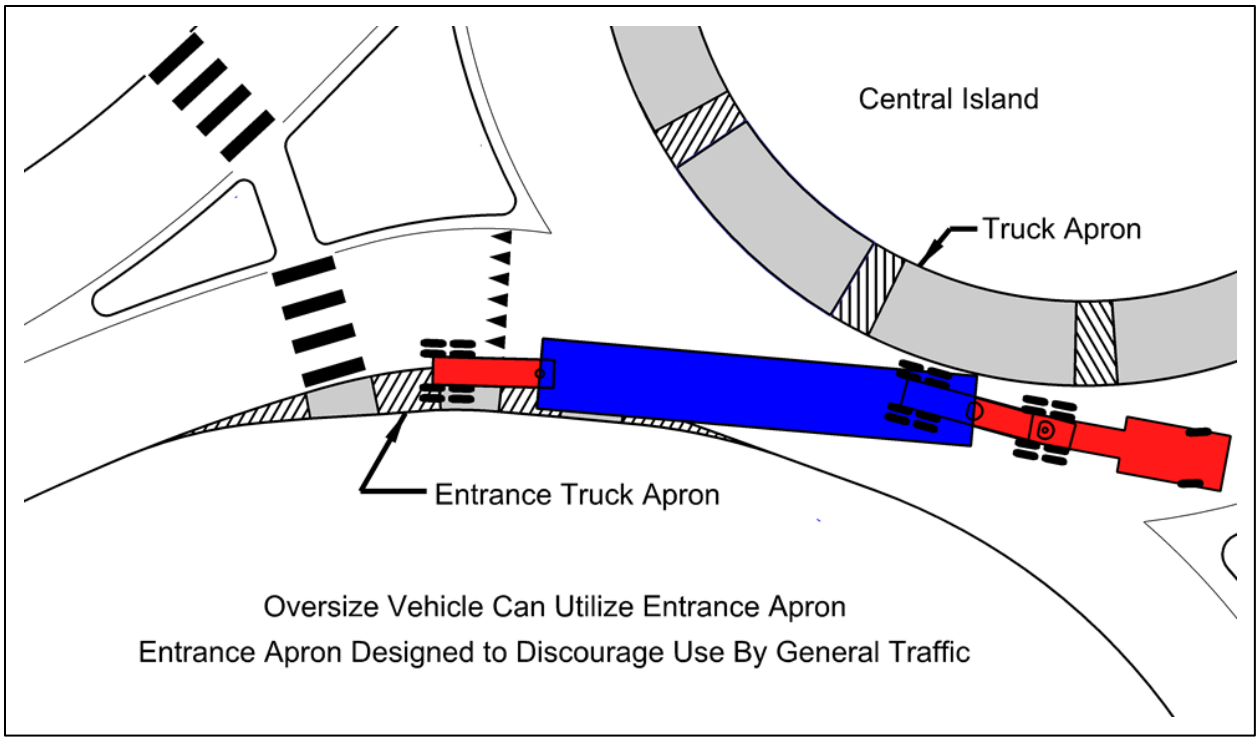


Figure 8-38(a): Oversize Vehicle Entrance Apron

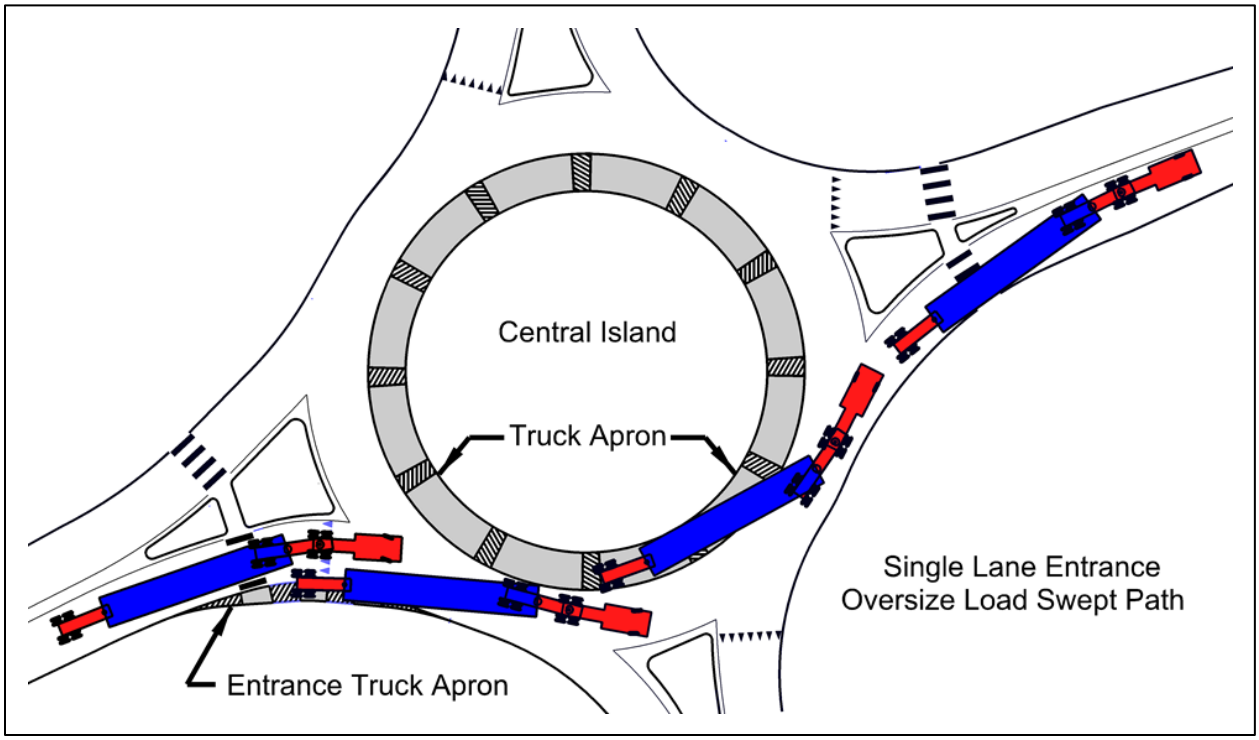


Figure 8-38(b): Swept Path of Oversize Vehicle Using an Entrance Apron

F. Splitter Island

The purposes of splitter islands are to:

1. Help alert drivers of the upcoming roundabout, regulate entry and exit speed;
2. Physically separate entering and exiting traffic, minimize potential for wrong-way movement;
3. Introduce deflection into vehicle paths; and
4. Provide a refuge for pedestrians, and a place to mount traffic signs.

Although a length of 100 ft. is desirable, the minimum length of the island in an urban location measured along the approach should be 50 feet long to provide sufficient protection for pedestrians. Longer islands or extended raised medians should be used in areas with high approach speeds. For these locations, median and splitter island combined length should be based on the distance needed to comfortably decelerate from roadway speed to the desired entrance speed to the roundabout. A separation between the yield line on the circulatory roadway and the pedestrian crossing is crucial to safety and operation. This separation distance helps split up the decision points of yielding to a pedestrian and picking a gap in the vehicular flow of the roundabout. It is recommended that the pedestrian crossing be located at least 35 – 40 feet from the yield line to the center of the crosswalk. The recommended crosswalk width is 10 feet. The opening through the splitter island should be 6 feet in length at the center of the

crosswalk. Typically, the splitter island will have a cut through design to accommodate pedestrians. Figure 8-39 shows an example of a splitter island at a single lane roundabout.

For multi-lane roundabouts, entry geometry is typically established first to identify a design that adequately controls fastest-path speeds, avoids path overlap and accommodates large vehicles. The splitter islands are then developed in conjunction with the entrance and exit designs to provide adequate median width for pedestrian refuge and sign placement requirements. For more information specific to overall design of multi-lane roundabouts, refer to the following section specific to multi-lane roundabout design.

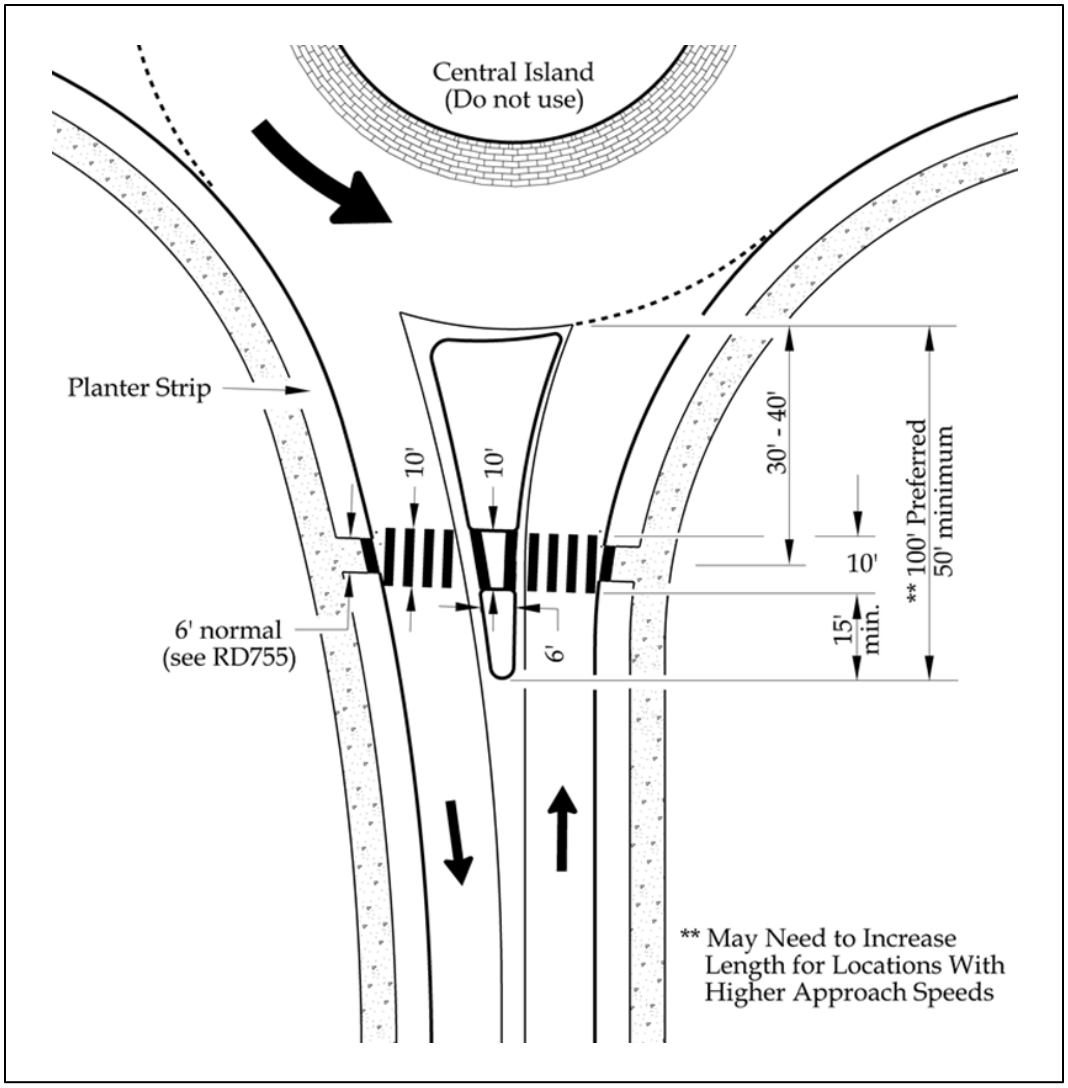


Figure 8-39: Minimum Splitter Island Dimensions, Single Lane Roundabout

8.6.5 MULTI-LANE ROUNDABOUTS

8.6.5.1 MULTI-LANE ROUNDABOUT CONFIGURATION

Since many design features of roundabouts are integral to both single lane and multi-lane roundabouts, the previous discussion about roundabout design elements did not specify explicit information about single lane roundabouts or multi-lane roundabouts, but rather discussed the design elements themselves in more general terms for both applications. However, there are a few unique design needs at a multi-lane roundabout that are not shared with single lane roundabouts. As a result, multi-lane roundabout design presents a greater challenge to the designer.

In the past, roundabouts were classified as single lane, double lane and, in extreme cases, triple lane roundabouts. The intent was to have equal lanes entering and exiting assuming balanced flow between intersecting roadways. However, as roundabout design has evolved, general intersection control principles are being applied to roundabout design. In conventional intersection design, it is not required to have an equal number of lanes at each leg. Intersection lane configuration is based on the needs of the traffic movements through the intersection. If one leg has a high volume of left turn traffic, a dedicated left turn lane may be designed for that leg as well as a through lane. This, in effect, creates a two lane entrance, while the through lane may align with only one lane on the opposite leg exiting the intersection. Likewise, if one leg has a high volume of right turn traffic, a dedicated right turn lane or even a "free right" slip lane might be designed to improve operation. The same concepts are now being applied to roundabout design and the term "multi-lane roundabout" has replaced the previous "double lane" or "triple lane" nomenclature. The term multi-lane covers a wider range of various lane configuration options that a design might employ to better tailor the design to the specific intersection control required for a specific location. However, as a result, because lane configuration on entrance and exit may be specific to a particular move at a particular exit, signing and striping of multi-lane roundabouts must convey to drivers which lane they need to be in to negotiate the roundabout successfully. The information contained in the signing and striping must be understood by the approaching driver far enough in advance of the roundabout to safely make the appropriate lane choice. If drivers are positioned in the correct lane for their destination when entering the roundabout, the lane striping and guidance will get them to the appropriate exit.

Some multi-lane roundabout configurations may appear complex to an approaching driver. When examining a design in plan view it may be easy to see how the lanes flow. However, at driver eye level that may not be the case. The designer must keep in mind what drivers see, or don't see, as they approach the roundabout and what must they see to understand how to get to the appropriate exit for their journey. Efficient, effective and well placed signing, striping and lane markings are critical to convey that information to motorists in modern multi-lane roundabout design. Figure 8-40(a) and Figure 8-40(b) portray examples of multi-lane roundabout design with various entrance and exit lane configurations. These layouts are hypothetical and are intended to provide guidance and illustration for potential options to meet traffic control needs at a given location.

These multi-lane roundabout layouts are not all inclusive and other configurations may fit a particular location better. Individual designers will need to design for the needs of the site for which the roundabout is being designed. Some of the entrance and exit options shown in the figures would only be employed at unique or high volume locations. As with any intersection design, it is important to only provide what is necessary to meet the control needs of the traffic movements. It is good design practice to keep the layout and operation of a multi-lane roundabout as simple as possible, while still providing the necessary control functions to allow smooth, efficient traffic flow. Additional information about roundabout lane configuration and striping can be found in the 2009 Edition of the MUTCD, Chapter 3C Roundabout Markings.

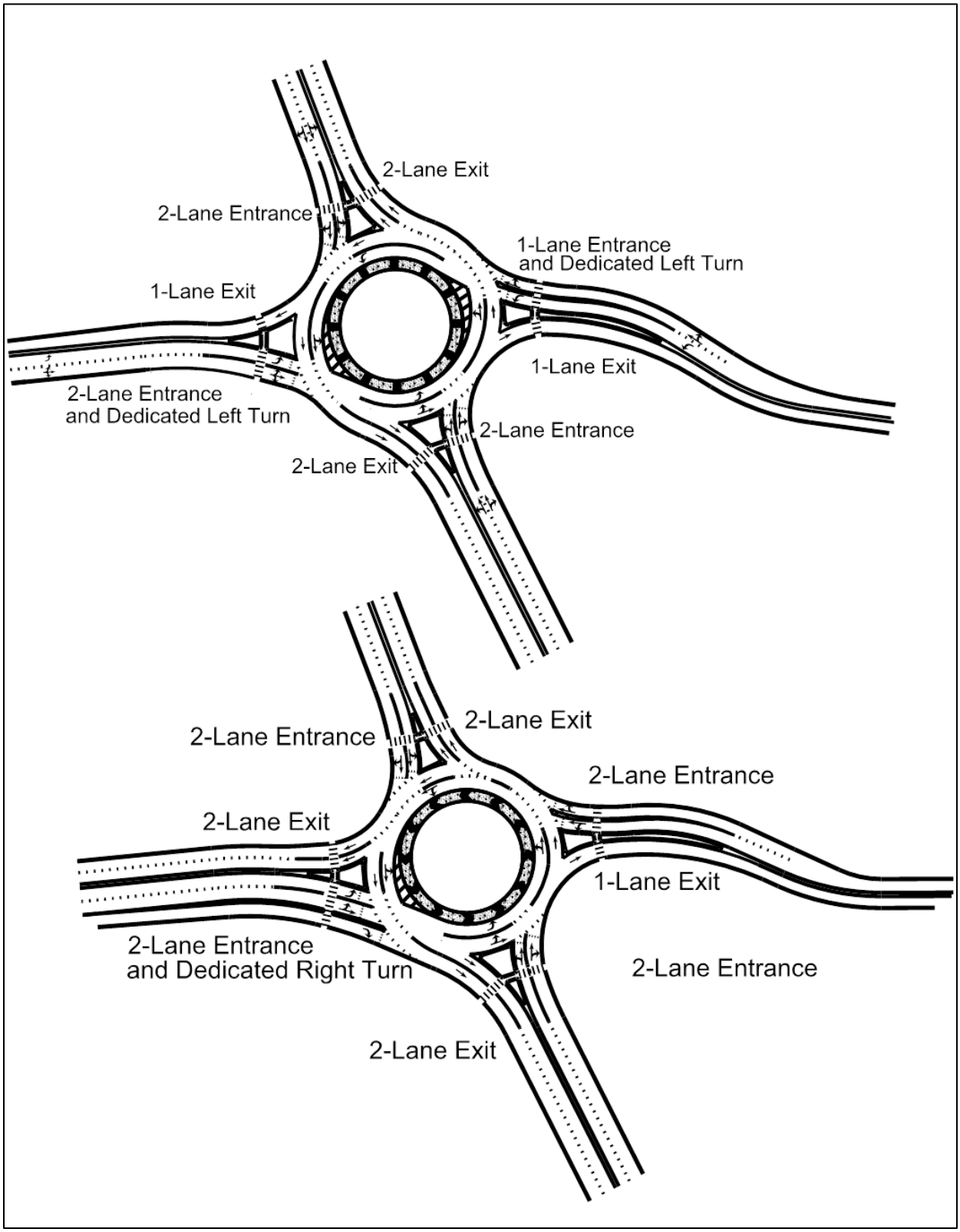


Figure 8-40(a): Various Multi-Lane Roundabout Entrance and Exit Options

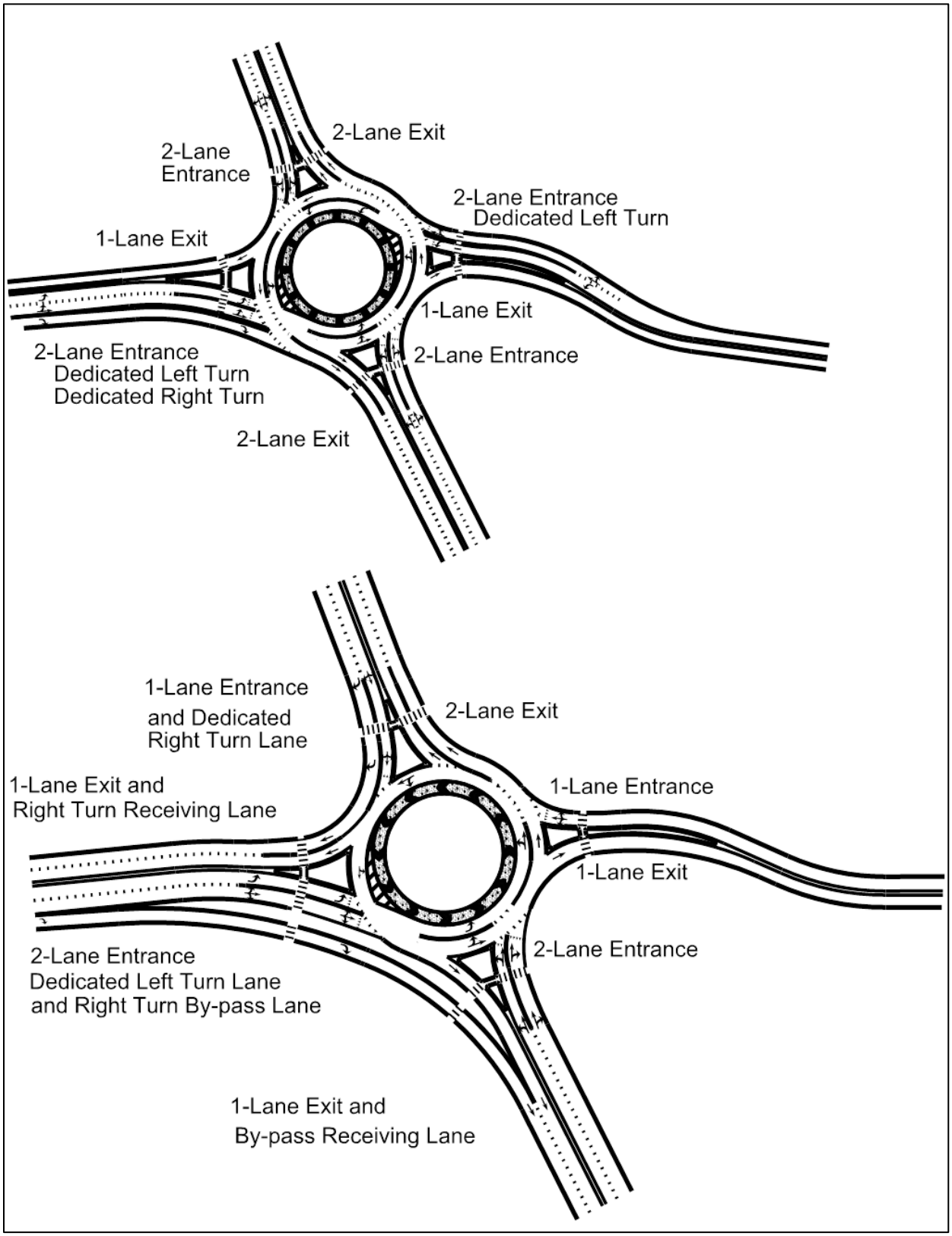


Figure 8-40(b): Additional Multi-Lane Roundabout Entrance and Exit Options

8.6.5.2 PATH OVERLAP

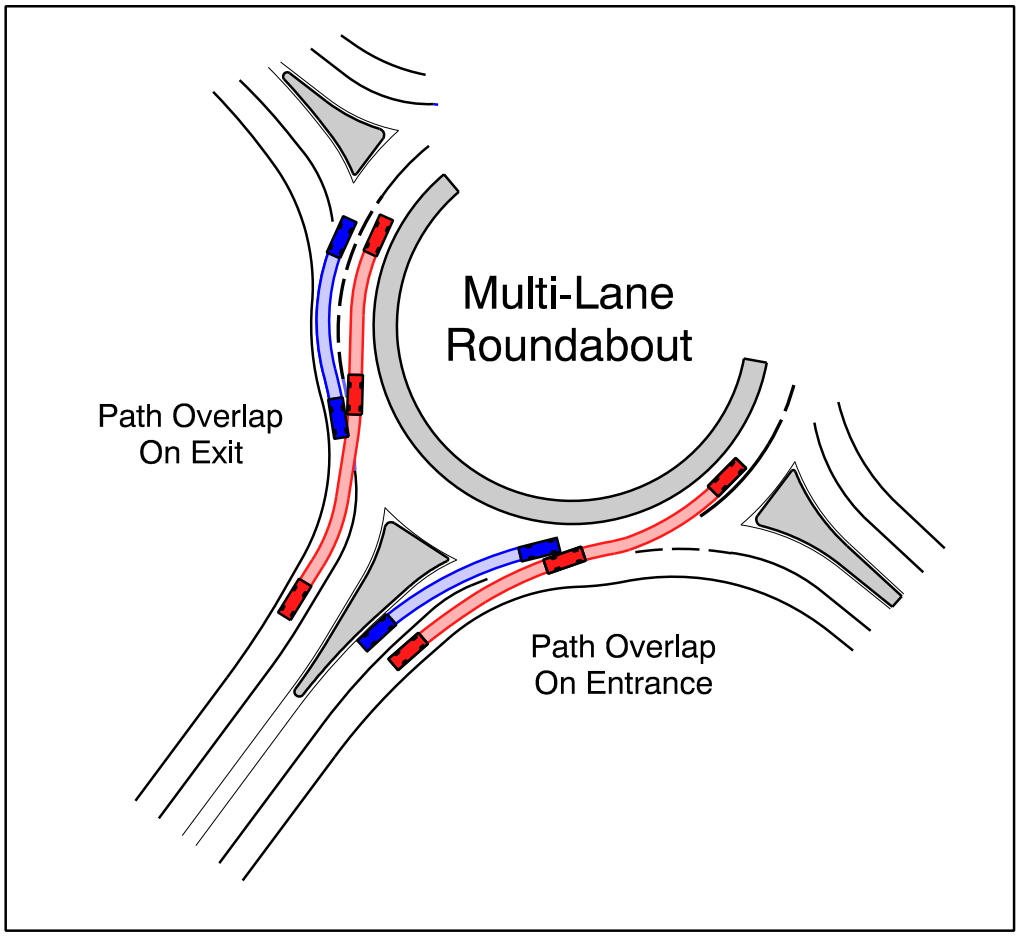


Figure 8-41: Path Overlap

Path overlap is another unique design concern present with multi-lane roundabouts. Figure 8-41 demonstrates the effect of path overlap at a multi-lane roundabout. Entrance design, central island design and exit design must be balanced to provide a consistent, comfortable flow when designing both single lane and multi-lane roundabouts. Multi-lane roundabouts, however, pose a greater problem with entry and exit design. Because more than one lane enters and exits the circulating roadway at multi-lane locations, a phenomenon known as path overlap can occur. Vehicle path overlap occurs when the natural path of a vehicle crosses into the adjacent lane. It generally happens at entrances to roundabouts, but can also occur at exits or even along the circulating roadway itself. The natural path of a vehicle is the path a driver seeks based on comfort due to the applied forces to the vehicle from the roadway geometry. The natural path is determined by approach geometry, entrance radii and entrance width. To avoid path overlap and potential side-swipe crashes at a multi-lane roundabout, the entry design for the approach lanes must provide a comfortable path for drivers to keep their vehicles in one lane and not encroach on the adjacent lane. While proper entry curvature is a key factor in avoiding path overlap, there is no single method for creating a desirable vehicle path

alignment. It may take several iterations of design elements to finalize an appropriate vehicle path to provide a smooth transition from entrance to circulating roadway to exit that eliminates path overlap.

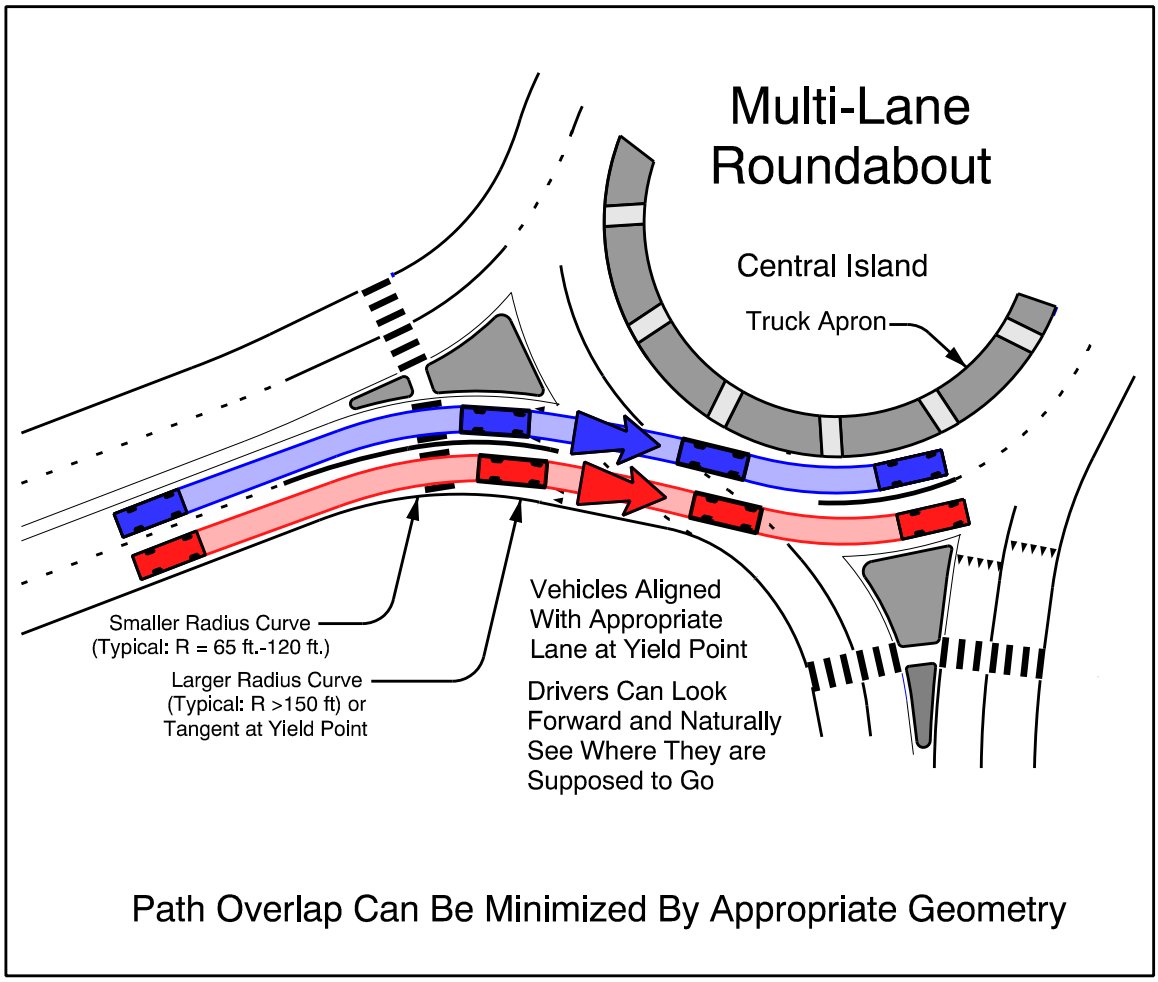


Figure 8-42: Minimizing Path Overlap

As a general starting point, entrance radii should be greater than 65 ft. and less than 120 ft. Compound curve sets or a single curve in series ahead of a tangent may prove beneficial in creating a successful design that balances desired speed constraint, provides large vehicle accommodation and addresses bicycle and pedestrian needs while directing the entering driver to the appropriate lane through the multi-lane roundabout. Figure 8-42 illustrates geometry that can minimize path overlap. The general idea is to create entrance geometry that slows the entering vehicle to the desired entry speed and then comfortably leads it to the appropriate circulating lane with a smooth transition to the circulating roadway and another smooth transition from the circulating roadway to the exit radius out of the roundabout.

8.6.5.3 LARGE VEHICLE ACCOMMODATION

Large vehicles must be able to negotiate a multi-lane roundabout. As with single lane roundabouts, truck aprons around the central island are used to aid large vehicle movements through multi-lane roundabouts. While ORS 811.292 and ORS 811.370 provide for "commercial motor vehicles" to operate outside a single lane in a multi-lane roundabout when necessary, it is beneficial to design multi-lane roundabouts to allow larger vehicles to remain in one lane as much as possible. However, this need must be balanced with the overall effectiveness of the roundabout. Providing too much room may encourage faster path speeds for passenger vehicles when truck volumes are not present. One way to help keep large vehicles from encroaching on the adjacent lane at the entrance to a multi-lane roundabout, while keeping entrance width to a minimum is to provide a section of "Gore Striping" between the entrance lanes. Figure 8-43(a) and Figure Figure 8-43(b) depicts a WB-67 swept path at a roundabout entrance that utilizes gore striping. The drawings show a truck entering from either lane utilizing the striping to minimize encroachment of the adjacent lane.

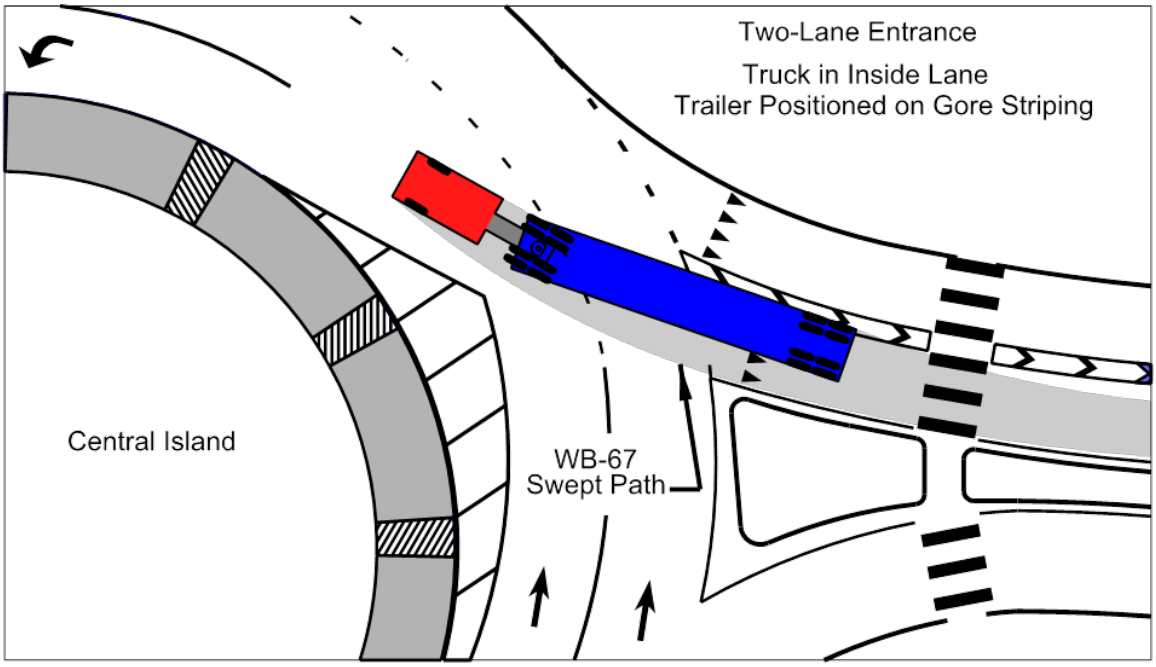


Figure 8-43(a): WB-67 Entering in the Inside Lane, Using Gore Striping

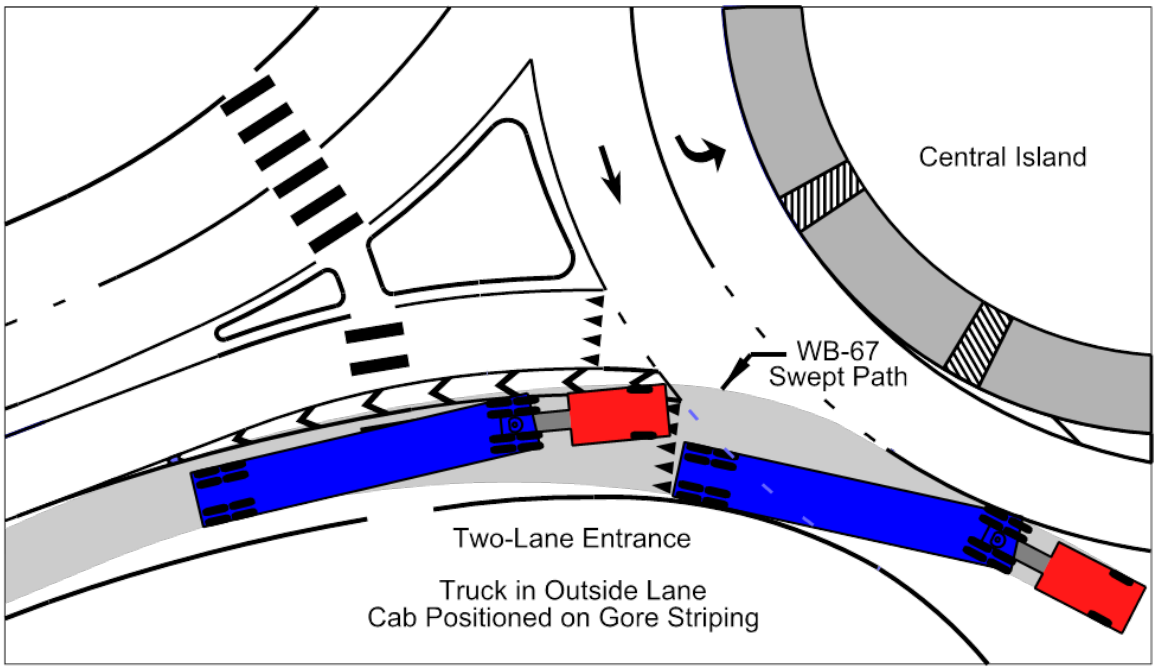


Figure 8-43(b): WB-67 Entering in the Outside Lane, Using Gore Striping

8.6.6 MULTI-MODAL ROAD USERS

8.6.6.1 PEDESTRIANS

The accommodation and safety of pedestrians at roundabouts is dependent on the following design features:

1. Slow speeds, achieved through sufficient deflection.
2. Separation of conflicts, achieved by placing the crosswalk away from the yield line of the circulatory roadway by 26–40 feet (approx. one car length); and
3. Breaking up the pedestrian crossing movements, achieved by placing a splitter island at each leg.

Sidewalks provide pedestrian accessibility at roundabouts. Standard sidewalk width of 6 feet should be used with greater widths as necessary. Where ramps will provide bicyclists access to use the sidewalks and crosswalks with pedestrians, 10 feet or more is appropriate for sidewalk width. When pedestrians and bicyclists share a sidewalk, appropriate multi-use or shared path guidelines are employed for the design. See the ODOT Highway Design Manual (HDM) section 13.7 and Chapter 7 of the Bicycle and Pedestrian Design Guide in Appendix L of the Highway Design Manual for shared use pathway design guidance.

Sidewalks should be set back from the edge of the circulatory roadway whenever possible using landscaped buffer zones. Landscape strips provide more benefits than just aesthetic value. They provide increased comfort for pedestrians, an area for snow storage and a buffer to allow for the overhang of large vehicles, if necessary, as they traverse the roundabout. Set backs also help direct pedestrians to appropriate crosswalks, rather than crossing to the center island or cutting across the circulatory roadway. In addition, vision impaired persons can use the landscape strip to guide them to the crosswalk. Recommended set back widths should be 5 feet. The minimum recommended set back is 2 feet. Grass or low shrub type vegetation should be the choice for plantings. They provide the visual and tactile delineation, but also allow drivers to see pedestrians on the sidewalk and at crosswalks. Taller plantings may block driver sight distance and mask the presence of pedestrians. Roundabout Signing and vegetation placement must be coordinated in order to ensure signs are not obscured as vegetation grows over time. Legible signs, easily understood by drivers are an important feature of modern roundabouts.

When a buffer zone is not incorporated in the design and a curbside sidewalk must be used, a continuous detectable edge treatment should be included along the street side of the sidewalk to guide pedestrians to the ramps and crossing areas. Examples of edge treatments include chains, fencing or railings. For additional information, see the document "Public Rights-of-Way Accessibility Guidelines"(PROWAG), Section R306.3.1.

Research has shown multi-lane roundabouts to be safer for pedestrians than signal controlled, multi-lane intersections. Vision impaired pedestrians may find crossing multi-lane roundabout connections to be difficult, due to limited or masked audible cues to traffic movements. However, this would not be dissimilar to multi-lane, mid-block crossings or multi-lane, uncontrolled intersection crossings as well. When appropriate, multi-Lane Roundabouts benefit from the installation of special traffic control devices (Signals, Pedestrian Hybrid Beacons or Rectangular Rapid Flash Beacons) at crosswalk locations to accommodate pedestrians with vision impairment.

The Public Rights-of-Way Accessibility Guidelines (PROWAG), Section R306.4 published by the United States Access Board indicates that roundabouts with multi-lane street crossings shall have accessible pedestrian signals. Section R209 of PROWAG defines "accessible pedestrian signal". As such, not all traffic control devices meet the criteria shown in section R209 to be compliant with the PROWAG. Currently, the PROWAG has not been officially adopted by the United States Department of Justice. Therefore, at this time, there is some flexibility in terms of absolute requirements for accessibility and types of equipment to provide accessibility when designing a multi-lane roundabout. However, while not actually installing signalization equipment at this time, it would be both beneficial and prudent for potential future signalization requirements to incorporate signalization design criteria to the greatest extent possible with all designs. The designer should consider what would be required to retrofit a signal into the proposed multi-lane roundabout layout. Consideration should be given to signal pole placement, signal head visibility, and controller cabinet location as well as conduit, wiring and operational needs. At the very least, the roundabout design should be as easily adaptable as possible in the future to include the requirements for accessibility as defined in the PROWAG should they become mandatory. Check with the Region Traffic Unit and the Traffic–Roadway Section of Technical Services for applications and acceptable devices.

8.6.6.2 BICYCLISTS

In general, bicyclists will be given a choice to enter a roundabout as a vehicle and occupy a lane until exiting the roundabout, or to use the sidewalks and crosswalks as pedestrians. Occupying a lane through the roundabout will, in most cases, be the most expedient method of traversing a roundabout. However, riding with traffic in a roundabout may not be comfortable for many bicyclists. For these bicyclists, a ramp is provided for them to exit the bike lane on approach to the roundabout and use the sidewalk and crosswalks in the manner of a pedestrian. It is generally recommended that only experienced bicyclists, comfortable riding with traffic, use the travel lane through a roundabout.

In single lane roundabouts, occupying a lane through the roundabout is less complicated than occupying a lane in a multi-lane roundabout. With a single lane roundabout, bicyclists will generally be traveling at relative speed to other vehicles on the roadway. Since it is easier to command the lane in a single lane roundabout, there is less chance of a bicyclist being cut off at an exit by a motorist. Also, bicyclists are more visible to motorists in a single lane roundabout, as there is less room and less distraction for vehicle drivers.

Multi-lane roundabouts pose greater challenges to bicyclists when occupying a lane to navigate through them. The greater complexity of multi-lane roundabouts may cause bicyclists to be less visible to motorists. Bicyclists will have a greater challenge in controlling the lanes in a multi-lane roundabout and there is greater potential to be cut off at an exit. Depending on roundabout configuration and bicyclist destination, a bicyclist may need to enter the roundabout in the left lane of a multi-lane roundabout. This may not be familiar or expected by other roundabout users. When considering bicycle access and movement through a multi-lane roundabout, it is important to remember that *ORS 811.292* and *ORS 811.370* have provision for "commercial motor vehicles" to operate outside a single lane in a multi-lane roundabout when necessary. Like other vehicle drivers traversing a roundabout, bicyclists must not pass or ride beside a commercial vehicle.

If bicyclists choose to ride with traffic through any roundabout, single lane or multi-lane, they should be afforded the same roundabout design concepts as motor vehicle drivers. They are expected to be a vehicle and should not be given individual direction to maneuver in a manner unexpected or different than a motor vehicle. They should be provided with efficient, safe and effective means of traversing the roundabout, as are other roundabout users. Bicyclists choosing to use the travel lane through a roundabout should be given ample space and distance to merge into the travel lane prior to the roundabout entry to allow motorists time to recognize them. Under no circumstances should a bike lane be carried into or through a roundabout. Providing a bike lane up to the actual circulatory roadway entrance will compound the merge maneuver for the bicyclist and create a conflict point between the bicyclist and motorist who are both concentrating on entering a gap in roundabout traffic. Providing a bike lane within a roundabout will only increase potential conflicts between vehicles and bikes at roundabout exits creating a potentially less safe condition than if bicyclists use the

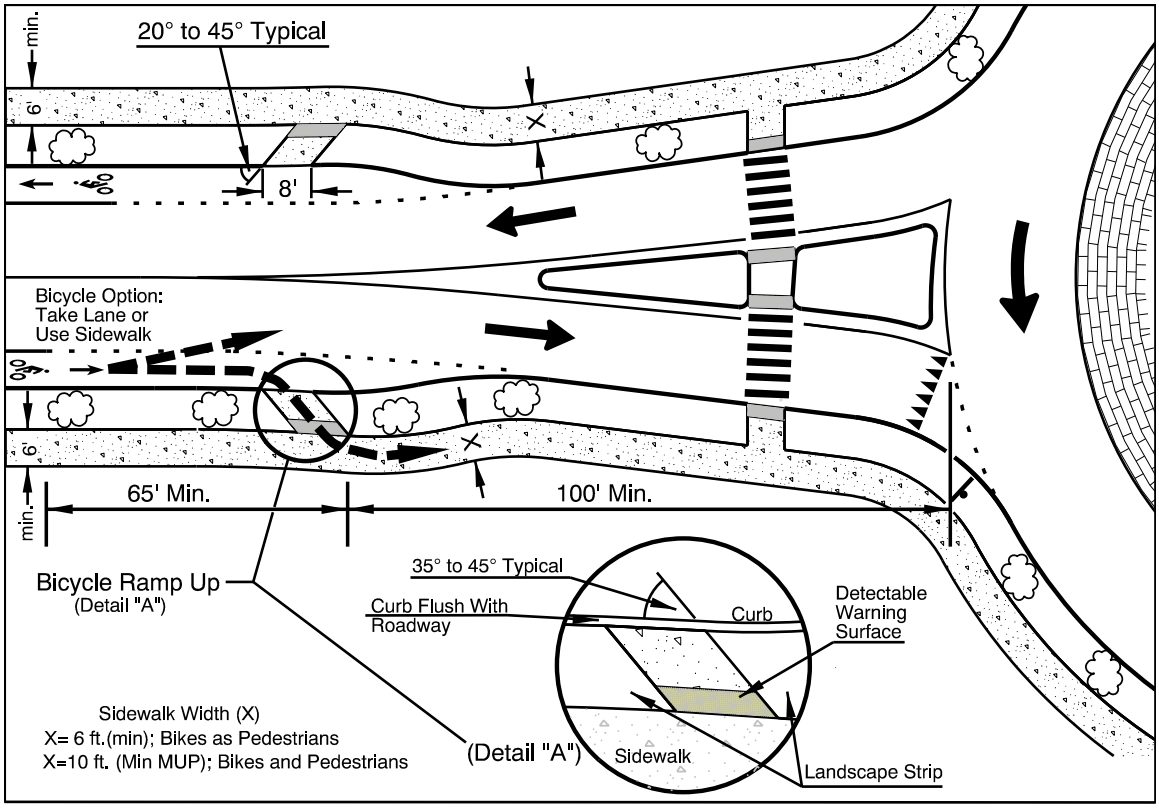


Figure 8-44: Bike Curb Cut

travel lane. Figure 8-44 provides direction for roundabout approach legs that have a shoulder or bike lane. The shoulder/bike lane should terminate at a distance sufficient to allow bicyclists to merge into traffic before drivers' attention is on roundabout traffic coming from the left. Curb ramps should be placed where the shoulder/bike lane terminates, allowing bicyclists to access the sidewalk should they choose to utilize it and the crosswalks to traverse the roundabout. The bike lane should end 165 feet in advance of the yield line and curb ramp width should be a minimum of 8 feet. General design practice attempts to keep roundabout entrances relatively flat with a suggested maximum grade of 4 percent. However, this is not always possible due to existing topographic conditions. Even a maximum grade of 4 percent sustained over a long enough distance can slow a cyclist. Approach grade and expected cyclist speed in relation to vehicle speed at the lane merge point is an important design consideration when designing for bicyclists to use the travel lane through a roundabout.

Bicycle ramps can be confused with pedestrian ramps by vision impaired pedestrians. Detectable warning surfaces should be included on bicycle ramps. It is preferred to locate bicycle ramps in a landscape strip or buffer area and a detectable warning surface should be placed at the top of the ramp, adjacent to the sidewalk. In these locations, the ramp is considered as part of the traveled way that needs to be detectable.

The least desirable location for the bicycle ramp is within the sidewalk itself. When placement of the ramp within the sidewalk is unavoidable, the detectable warning surface is placed at the

bottom of the ramp, adjacent to the curb and care must be taken to ensure the ramp is not a tripping hazard in the pedestrian pathway along the sidewalk.

Minimum sidewalk width is 6 feet. However, sidewalks that include bicycle traffic mixed with pedestrian traffic should be increased to at least 10 feet in width to allow for a minimum width multi-use pathway condition. If sidewalks are limited to a 6 foot width, then bicyclists should walk their bikes as a pedestrian. In locations where bicycle riding on the sidewalk is prohibited by statute, appropriate signage is necessary to inform bicyclists.

Bicycle ramps up from the roadway to the sidewalk should be placed at a 35 degree to 45 degree angle with the roadway allowing bicyclists to use the ramp, while discouraging them from entering the sidewalk area at too great a speed. Since the bicycle ramp is not a pedestrian ramp, its slope is not limited to a maximum of 1 in 12 (8.33%). If necessary, the slope may be greater than 1 in 12. Ramps steeper than 1 in 12 can be a clue for vision impaired pedestrians to differentiate between the bicycle ramp and the pedestrian ramp. Steeper ramps can also help slow bicycle traffic as it enters the sidewalk zone. In general, ramps should only be as steep as necessary to fit the location with a potential maximum of 1 in 5 (20%) in extreme circumstances. Bicycle ramps from the sidewalk down to the roadway at roundabout exits can be placed with an angle as small as 20 degrees with the roadway since it is not necessary for a bicyclist to slow upon entry to the roadway. A flatter angle can be beneficial in allowing a bicyclist to enter the bike lane or travel lane at a relative speed to traffic. However, some discernible angle is necessary to provide information to vision impaired pedestrians that the bicycle ramp is not the pedestrian ramp.

Some roadways leading up to a roundabout location may have been designed utilizing a separated or protected bicycle facility like a cycle track, side path or multi-use path. Depending on the actual cycle track or path design, there may be several options for providing accommodation for bicyclists to navigate the roundabout. For guidance in melding the bicycle facility design with the roundabout design, contact the ODOT bicycle and pedestrian facility specialist in the Technical Services, Traffic-Roadway Section.

8.6.6.3 TRANSIT CONSIDERATIONS

While it is possible to effectively locate roundabouts on transit corridors, placement of actual transit stops in proximity to roundabouts is problematic for smooth operation of both the transit system and the roundabout. The placement of bus or other transit stops near roundabouts should be consistent with the needs of the users and the desired operations of the roundabout. Stops should be close to passenger generators or destinations, and pedestrian crossings of the roundabout legs should be minimized. A bus or transit stop is best situated:

1. On an exit lane, in a pullout just past the crosswalk; or
2. On an approach leg 60 feet upstream from the crosswalk, in a pullout; or
3. On a single lane entrance leg, just upstream from the crosswalk, if the traffic volume is low and the stopping time is short. This location should not be used on two-lane

entrances (In the interest of pedestrian crossing safety, a vehicle should not be allowed to pass a stopped bus).

Bus pullouts or transit stops shall not be located in the circulatory roadway on the state highway system.

Although rare, there are locations in other jurisdictions where fixed transit lines (light rail, Bus Rapid Transit) have been provided with independent alignment through roundabouts. The best practice for the state highway system is to avoid placing a fixed transit line through a roundabout, However, when it cannot be avoided, care must be taken when establishing the transit alignment so as to not diminish the performance of the roundabout. The design can be successful. However, care must be taken to determine the transit schedule and its impact on the traffic flows at the roundabout. The interaction between the transit vehicles and normal traffic must be considered for present volumes and patterns as well as anticipated future transit and traffic needs.

8.6.6.4 TRUCKS IN ROUNDABOUTS

Freight transport is a vital function of the state highway system. Improperly designed roundabouts can impede freight traffic. Roundabouts on the state highway system must be designed to accommodate the necessary movement of freight. The WB-67 class "interstate" truck will be the basic design vehicle for roundabouts on the state highway system. A smaller design truck might be appropriate on some sections of highway. If a vehicle smaller than a WB-67 is anticipated to be used as the roundabout design vehicle, discussions with ODOT Motor Carrier Division and representatives of the trucking industry will be necessary in order to reach a final determination of feasibility.

From time to time, oversize/overweight (OSOW) loads may need to move through a roundabout location and these loads will need to be accommodated in an acceptable manner. In order to create an overall roundabout design that will accommodate the anticipated OSOW vehicles at a particular roundabout, discussion between the designer, Technical Services staff, ODOT Motor Carrier and trucking industry representatives will be necessary in order to determine appropriate loads to consider and how best to accommodate their movement through the roundabout. Section 8.6.4.1, page 8-59 provides general information about roundabout design vehicles and accommodation vehicles.

There may be locations where a smaller diameter roundabout is required that may also need to allow for OSOW vehicle traffic or a location may need to allow for unique or specialized loads to pass through the proposed roundabout. For these situations, there are several alternative design concepts that provide special access and movement through the roundabout. Contact the Technical Services, Traffic-Roadway Section for assistance in designing these unique and special access locations. In most cases they will require design concurrence and may need additional design approval from the state Traffic-Roadway Engineer.

APPENDIX

P

ANALYSIS FOR ROUNDABOUT ENTRANCE AND EXIT GEOMETRY

P.1 WHITE PAPER; ROUNDABOUT ENTRANCE AND EXIT GEOMETRY

Entrance and exit geometries play an important role in controlling speed and movement of a vehicle through a roundabout. In general, providing roundabout alignments that increase flow at the exit may provide increased gaps in the circulating traffic stream and may provide greater opportunities for entering vehicles. Currently, there is significant discussion between roundabout designers about the best method to determine exit geometry and to control exit speed within design parameters. The discussion centers around the prediction of vehicle speed and how to calculate appropriate values for design. The standard method has been to utilize the speed, radius relationship as shown in Figure P–1. The graph was derived using the basic equation for velocity and minimum radius from the AASHTO document A Policy on Geometric Design of Highways and Streets; V= $\sqrt{15R(e+f)}$, where superelevation, e, is held to +2% and -2% with side friction factor, f, values assumed for general design.

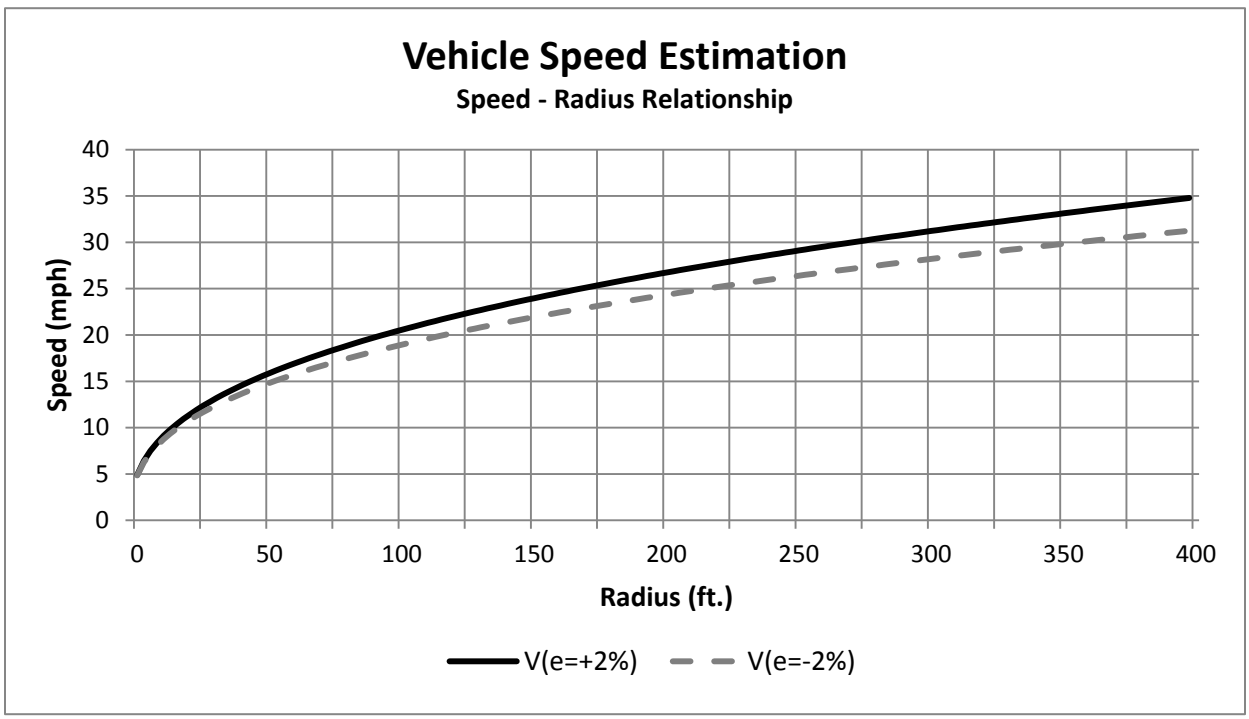


Figure P–1: Estimated Vehicle Speed and Radius Relationship

Table P–1 is a tabular form of the values in Figure P–1 reported at 25 ft. radius intervals. In addition, NCHRP Report 672 Roundabouts: An Informational Guide, provides simplified

equations to calculate speeds for given radii as well. Equation 1 is for +2% superelevation and Equation 2 is for -2% superelevation.

Table P–1: Speed, Radius Relationship

Speed (V), Radius (R) Relationship Equations

Equation 1
NCHRP Report 672
$V=3.4415R^{0.3861}$; e= 2%

Equation 2
NCHRP Report 672
$V=3.4614R^{0.3673}$; e= -2%

Equation 3
AASHTO Minimum Radius
$V=\sqrt{15R(e+f)}$

Radius (ft.)	V(+2%) (mph)	V(-2%) (mph)
25	12	11
50	16	15
75	18	17
100	20	19
125	22	20
150	24	22
175	25	23
200	27	24
225	28	25
250	29	26
275	30	27
300	31	28
325	32	29
350	33	30
375	34	31
400	35	31

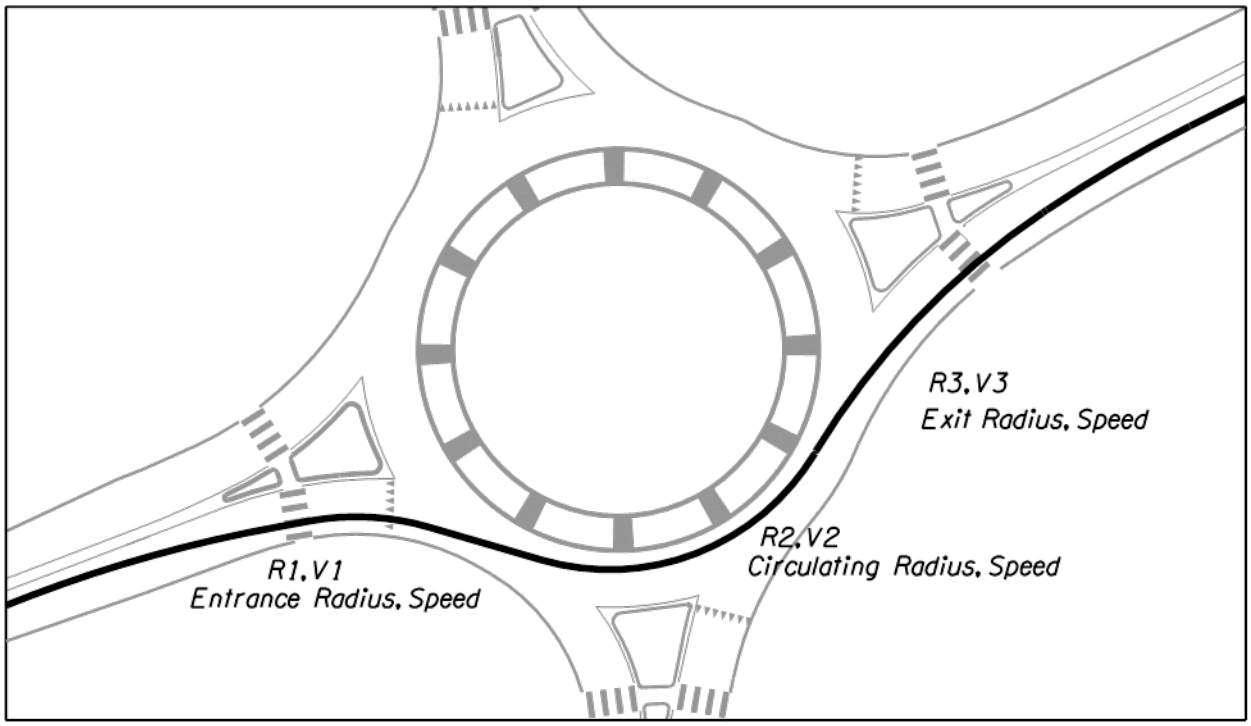


Figure P–2: Vehicle Path Through a Roundabout - Speed, Radius

For superelevation other than +/- 2%, Equation 3, AASHTO Minimum Radius needs to be used with an appropriate side friction factor, f.

However, there is thought that exit radii designed too small to reduce predicted exit speed in an attempt to focus on pedestrian safety may unnecessarily limit overall roundabout capacity. This leads to the question, then, how to calculate appropriate exit radii to maximize capacity and still protect pedestrian movements at the downstream crosswalk?

P.1.1 RESEARCH FOR ALTERNATE CALCULATION METHOD

Alternate Design Methods for Pedestrian Safety at Roundabout Entries and Exits: Crash Studies and Design Practices in Australia, France, Great Britain and the USA Bill Baranowski, Edmund Waddell (2004)

Research done in 2004 by Bill Baranowski of Roundabouts USA and Edmund Waddell of Michigan DOT investigated entrance and exit geometry in order to determine appropriate roundabout alignments to increase capacity without negatively effecting pedestrian safety. The investigation determined that R_1 and R_2 values along with vehicle acceleration from R_2 through R_3 may play more of a role in exit speed than exit radius, R_3, alone. The researchers looked at the circulation radius, speed; R_2,V_2 relationship, the distance from the end of the R_2 radius to the exit crosswalk and the potential acceleration of a vehicle over that distance.

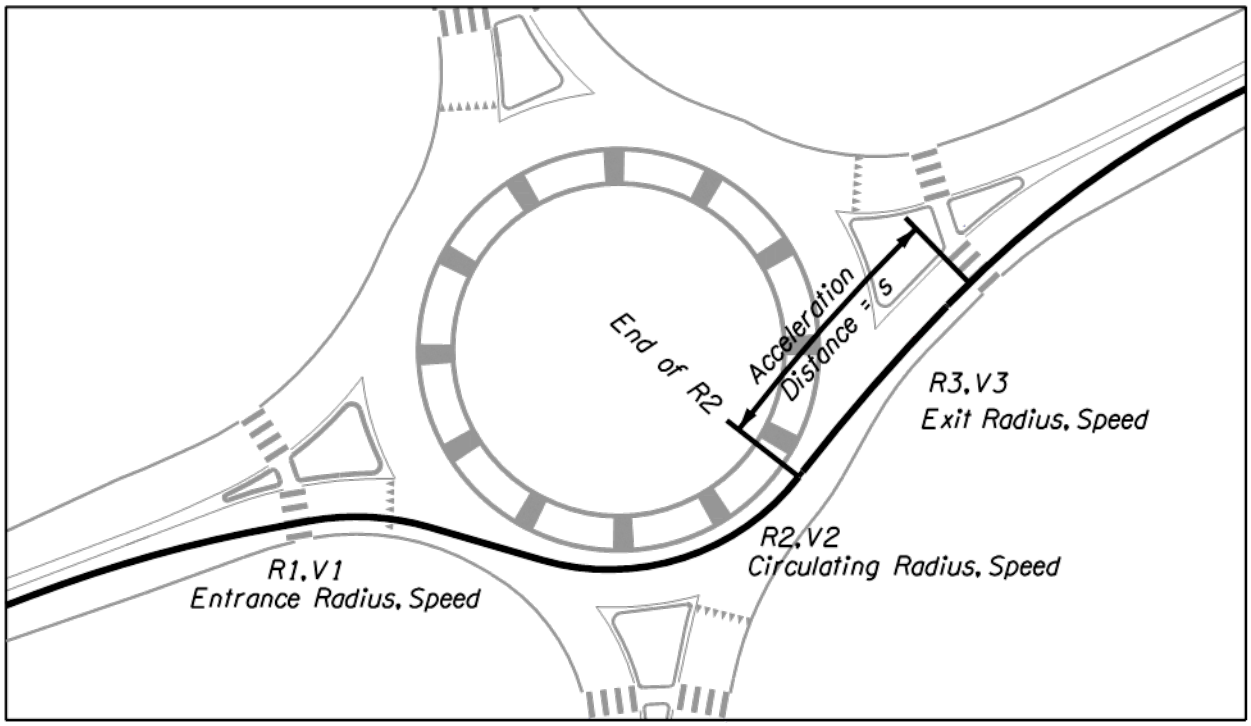


Figure P–3: Vehicle Path Through a Roundabout Speed,Radius, Acceleration Distance

The research assumed an exiting vehicle is capable of accelerating along a given R_3 radial path with an acceleration rate of 3.5 ft/s^2 and also assumed acceleration starts at the end point of R_2. The standard Newtonian equation for uniform acceleration was used to compute potential vehicle speeds at the exit crosswalk.

Newtonian Equation for Speed and Acceleration

$$\mathbf{V_f^2 = V_i^2 + 2aS}$$

Where: V_f = Final R_3 Speed, ft/s (V_3, Exit Speed)
V_i = Initial R_2 Speed (V_2, Circulating Speed)
a = Acceleration, (3.5 ft/s^2)
S = Distance, ft (End of R_2 to Crosswalk)

After analyzing theoretical roundabout layouts and investigating several existing roundabouts, the researchers concluded that the R_2,V_2 radius, speed relationship and vehicle acceleration from R_2 to the crosswalk as a vehicle exits a roundabout has more effect on the vehicle speed at the exit crosswalk than a tighter exit radius using only the radius, speed relationship for R_3 alone. The theory then is that exit geometry (radius) can be relaxed to increase overall capacity and not appreciably affect pedestrian activity or safety at the exit crosswalk by increased vehicle speed. This may prove to be true for small acceleration distance values coupled with relative radius values in order to predict and control maximum potential exit speed. However, effectively controlling this relationship may not always be easily accomplished

While the theory may have validity, it is only one analysis and appropriate application is critical to its effectiveness for speed prediction and control. Two key variables in the calculation are the distance available to accelerate prior to the exit crosswalk and the acceleration rate itself. If available acceleration distance is kept short, the exit speed may not be greatly affected. However, in larger diameter roundabouts, the available distance to accelerate may have an appreciable effect on exit speed. This may be particularly true for multi-lane roundabouts. The acceleration rate chosen for design will also have an effect on the predicted speed. The research used a rate of 3.5 ft/sec^2 for exit speed calculations. This is not a particularly fast rate of acceleration and may be acceptable for a curvilinear acceleration rate for small to moderate radii transitioning to the exit. However, some roundabout designs are utilizing large exit radii that become almost tangential. In these designs, it would be expected that vehicles would be accelerating from R_2 to the exit at a rate greater than 3.5 ft/sec^2. NCHRP Report 672, Roundabouts: An Informational Guide uses 6.9 ft/sec^2 for an acceleration rate in similar equations. This is nearly twice the rate used in the Baranouski/Waddell research and may be a better estimation when considering that the current vehicle fleet is capable of maximum performance, straight line acceleration rates of 9 ft/sec^2 for a four cylinder compact car to over 20 ft/sec^2 for a high performance eight cylinder vehicle with the average for all vehicles about 13 ft/sec^2. (See Table P–2 attached, Maximum Performance – Straight Line Acceleration by Vehicle)

The Baranowski/Waddell research is significant in that it shows the role R_2 can play in controlling exit speed when alignments incorporate smaller curvilinear radii and short acceleration distances between R_2 and the exit crosswalk. However, for larger radius or tangential exits, the acceleration rate for predicted speed calculations may need to be increased to better represent conditions as available acceleration distances increase.

P.1.2 NCHRP REPORT 572, ROUNDABOUTS IN THE UNITED STATES

Rodegerdts, Blogg, Wemple, Myers, et al (2007)

NCHRP Report 572 was a research project that investigated roundabouts in the United States and analyzed their operation. Authors of NCHRP Report 572 collected data from 103 roundabouts from around the United States. One of their findings indicated that observed entry and exit speeds did not always correlate well to the predicted entry and exit speeds determined for a given roundabout using the speed, radius relationship. The predicted speeds tended to be greater than the observed speeds. This was particularly evident for roundabouts with tangential or large entrance or exit radii. However, the speed, radius relationship did well in predicting observed circulating speeds through the R_2 and the R_4 pathways around the central island. It is unclear as to why the speed, radius relationship is effective to predict speeds for pathways around the central island radius, but is not as effective when predicting speeds in relation to entry and exit radii when correlated to observed speeds at specific roundabouts. From their observations and analysis, the authors developed equations that, in some locations, may better predict entry and exit speeds based on vehicle deceleration and acceleration ability.

Like the previous research work done in 2004, these equations include vehicle deceleration and acceleration parameters based on observations and analysis and use the standard equation for uniform acceleration as a basis. These equations are also presented in NCHRP 672, Roundabouts: An Informational Guide, second edition (2010) to calculate predicted values for V_1 and V_3 along a vehicle's fastest path as it enters and exits a roundabout. The guide suggests these equations can be used as an alternative to using values derived from the simplified speed, radius relationships. However, as a cautionary statement, since predicted V_2 values derived from the speed, radius relationship seem to correlate to observed V_2 values, there may be other factors involved like driver behavior, driver expectation, driver familiarity, etc. affecting the correlation of predicted exit speeds and observed exit speeds rather than straight forward correlations to radial path, speed or acceleration.

Equation 4 – Alternative Entrance Speed Calculation, V_1

$$\boldsymbol{V_1 = \frac{1}{1.47}\sqrt{(1.47V_2)^2 + 2a_{1,2}d_{1,2}}}$$

V_1 = entry speed, mph
V_2 = circulating speed based on path radius, mph
$a_{1,2}$ = deceleration between point of interest along v_1 path and mid-point of V_2 path, = -4.2 ft/s^2
$d_{1,2}$ = distance between point of interest along V_1 path and mid-point of V_2 path, ft.

The deceleration rate of -4.2 ft/s^2 for entry speed was developed from the observed driver/vehicle behavior at the researched sites. While this equation had better correlation predicting entry speed with observed speed, the authors also included the following statement in NCHRP 572:

> *"However, given the hesitancy currently exhibited by drivers under capacity conditions, the observed entry speeds may increase over time after drivers acclimate further. Therefore, the research team believes that an analyst should be cautious when using deceleration as a limiting factor when establishing entry speeds for design. Furthermore, the research team believes that a good design should rely more heavily on controlling the entry path radius as the primary method for controlling entry speed, particularly for the fastest combination of entry and circulating path (typically the through movement)."*

NCHRP Report 672, Roundabouts: An Informational Guide, second edition also addresses this concern and states:

> *"Analysts should use caution in using deceleration as a limiting factor to establish entry speed for design. To promote safe design, deflection of the R1 path radius should be the primary method for controlling entry speed. Therefore, while Equation 6-3 may provide an improved estimate of actual speed achieved at entry, for design purposes it is recommended that predicted speeds from Equation 6-1 be used."*
>
> *(Note: In this White Paper, NCHRP Report 672 Equation 6-3 and Equation 6-1 are reported as Equation 4 and Equation 1 respectively)*

Similar to entry speed, NCHRP Report 572 developed an equation that utilizes vehicle acceleration ability for predicting exit speed based on the standard uniform acceleration equation to better correlate predicted exit speed with observed exit speed for investigative purposes. As with the deceleration rate for entry speed, the report developed a vehicle exit acceleration value of 6.9 ft/s^2 from observed information.

Equation 5 – Alternative Exit Speed Calculation, V_3

$$\boldsymbol{V_3 = \frac{1}{1.47}\sqrt{(1.47V_2)^2 + 2a_{2,3}d_{2,3}}}$$

V_3 = Exit Speed, mph
V_2 = circulating speed based on path radius, mph
$a_{2,3}$ = average acceleration between midpoint of V_2 path and the point of interest along V_3 path = 6.9 ft/s^2
$d_{2,3}$ = distance along vehicle path between midpoint of V_2 path and the point of interest along the V_3 path, ft..

The authors of NCHRP 572 did not provide a caveat for not using the alternate V_3 calculation method for design as was provided for the alternate V_1 calculation method. There is no explanation provided in the report to indicate why one calculation may be considered more valid than the other. One must remember the reason for the derivation of these equations. The intent was to provide a prediction of exit speed that better correlated to observed exit speed at roundabout locations. The use of these equations lies in the assumption that since the predicted exit speed using the speed, radius relationship is greater than the observed speed, there must be something affecting the speed, radius relationship at exits. Acceleration rates were determined to make a better correlation. However, it works fine for R_2,V_2 and R_4,V_4 predicted and observed values. There may be other driver behavior factors that also affect observed R_1,V_1 and R_3,V_3 relationships. The authors are concerned this is the case with entrance speed and the same may be true for exit speed. The derived equations use a single deceleration or acceleration rate determined from observed data. Applying these acceleration rates to large radius or tangential exits and small radius, tight curvilinear exits equally may not produce effective design results in both cases. Using the same rates for both exit types assumes acceleration in a straight line or in a large radius is the same as acceleration in a tighter curvilinear path. This may not be the case. Therefore, lowering the acceleration rate for smaller radius paths seems reasonable. The research done in 2004 used 3.5 ft/s^2 as an acceleration rate for their investigation into exit geometry. This seems a more reasonable acceleration rate for smaller radial paths. NCHRP 572 uses 6.9 ft/s^2 as an acceleration rate. This seems reasonable for larger radius or tangential exits and seems to represent where, by observation, American drivers currently feel comfortable when exiting a roundabout. However, will this rate increase as drivers become more familiar with roundabouts? This is a concern of the authors of NCHRP Report 572 for V_1 values.

In addition to determining an acceptable acceleration rate, the other two critical variables in these equations are the V_2 speed and the distance, d, over which the deceleration or acceleration can take place. Therefore, if a large radius or tangential exit is designed for a roundabout, the R_2 value must provide the appropriate design V_2 and the acceleration distance must be effective in limiting a vehicle's potential downstream speed to design values.

Figure P–4 is a hypothetical roundabout layout based on real roundabout dimensions that portrays potential differences in speed between a smaller curvilinear exit and a more tangential exit. The vehicle path alignment shown from lower left to upper right (green) assumes radii for R_1 and R_2 that provide a 20 mph V_1 and V_2. The curvilinear R_3 exit radius is shown as both 175 ft. and 125 ft. for illustrative purposes and correlates to a V_3 speed of 25 mph and 22 mph respectively. These V_3 values are based on the speed, radius equations discussed previously in this report and is shown in Table P–1, Figure P–1. For comparison, the speed, acceleration equation was used to calculate a predicted V_3 exit speed along the radial R_3 path. Since the exit radius is small, using the 3.5 ft/s^2 acceleration rate discussed previously and coupled with the relatively short acceleration distance shown, a predicted V_3 of 25 mph was determined. This is equal to the value predicted for V_3 using the speed, radius relationship for a 175 ft. exit radius. This is in line with the conclusions of the 2004 research report. However, keep in mind, this geometry has a smaller curvilinear alignment with a short acceleration distance that helps limit a vehicle's ability to accelerate. For comparison, increasing the acceleration rate for the calculation to the NCHRP Report 572 value of 6.9 ft/s^2 yields a predicted speed of 29 mph at the crosswalk. This is beginning to reach the unacceptable level for speed at the crosswalk when considering pedestrian safety.

Large radius or tangential exit geometry set for increased capacity or exit geometry opened up due to skewed approach alignments or other site specific parameters that might dictate positioning of roundabout elements may have equal or greater impact to potential vehicle speeds at the crosswalk.

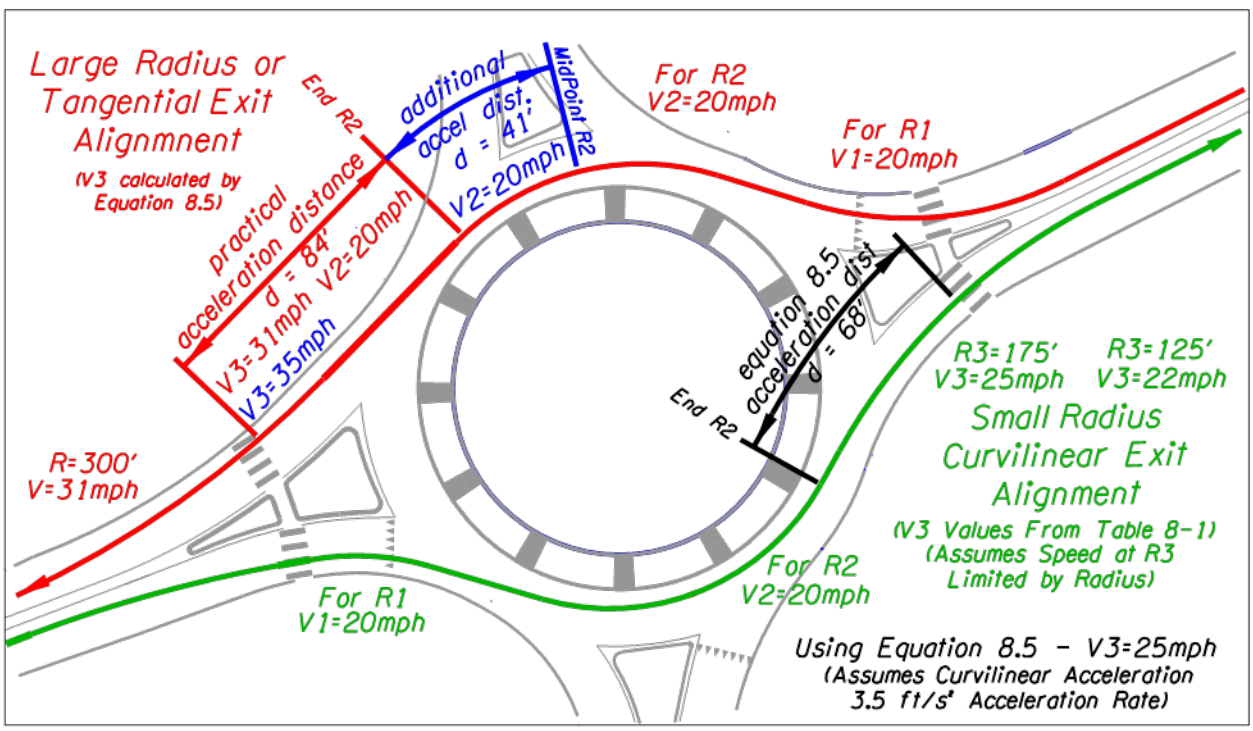


Figure P–4: Exit Geometry – Comparison Tangential and Small Radius

The vehicle path shown on the opposite side of the roundabout from upper right to lower left (red) in Figure P–4 also assumes radii for R_1 and R_2 that provide a 20 mph V_1 and V_2. However, the V_3 value of 31 mph is based on the potential for vehicle acceleration from the end of R_2 to the crosswalk. This distance is shown as a "practical acceleration distance", d, and for this layout is equal to 84 ft. This distance assumes a driver does not accelerate until reaching the end of the circulating path radius R_2. This is the approach the researchers in 2004 preferred. However, the equation parameters listed in NCHRP 672, Roundabouts: An Informational Guide, second edition define the acceleration distance as the distance from the midpoint of the V_2 path and a point of interest along the V_3 path. The point of interest is the downstream crosswalk in this analysis. Adding the additional acceleration distance back along the path to the midpoint of R_2 and assuming a vehicle is capable of accelerating at 6.9 ft/s^2 along this reversing radial to tangential path, yields a total distance of 124 ft. that a vehicle can accelerate prior to the downstream crosswalk increasing the calculated V_3 speed to 35 mph. These calculated speeds are 6 mph and 10 mph faster than the predicted V_3 speed of 25 mph at the tighter curvilinear exit on the opposite path of the roundabout. Either of these speeds would be considered excessive for design at the downstream crosswalk. This exemplifies the need to limit the acceleration distance, d, to provide acceptable exit speed if a tangential or large radius design is used.

P.1.3 CONCLUSION

The two research projects discussed both used uniform acceleration in their calculations. However, they each used different rates of acceleration. Baranowski and Waddell used 3.5 ft/s^2 for acceleration. NCHRP Report 573 used 6.9 ft/s^2, which is almost double the rate used by Baranowski and Waddell. Both these rates appear to be rates that were field observed by the authors of the reports. The difference may be attributed to the focus of the individual research. Baranowski and Waddell were studying roundabout locations where they considered exit radii to be excessively tight to restrict speeds. Therefore, the observed rates of acceleration were compatible with the geometry. In the case of NCHRP Report 572, the authors were trying to correlate observed exit speed with predicted speed and they noted there was a greater discrepancy when the exit radius was large – predicted speed greater than actual observed speed. In these cases, it appears the acceleration rate was determined to match the observed speed and the 6.9 ft/s^2 value they determined in 2007 may in fact be a comfortable rate for American drivers at larger radius exits. This is further borne out when looking at potential 0 – 60 mph maximum performance characteristics of the current vehicle fleet. Table P–2 is a listing of maximum performance and straight line acceleration of various late model production vehicles ranging from 4 cylinder compact cars to high performance 10 cylinder "muscle cars". The data was collected from the on-line automotive sight AutoRooster at http://www.autorooster.com. The site reports 0-60 times for a variety of current vehicles. The corresponding accelerations were calculated and added to the table as 60 mph acceleration values in ft/s^2. The acceleration values ranged from 9.09 ft/s^2 for a 2008 Honda Civic, 4-cylinder vehicle to 24.50 ft/s^2 for a 2010 Dodge Viper, 10-cylinder vehicle. The mathematical average for all the vehicles in the table is 12.89 ft/s^2. This indicates that the 6.9 ft/s^2 value determined from observed speeds in NCHRP Report 572 may be an acceptable overall value as

a "comfortable" acceleration rate to most drivers, since the average in Table 2 of 12.89 ft/s^2 was determined from maximum, straight line performance.

Currently, there is no definitive answer to what is the best method to predict entrance and exit speed when designing a roundabout. Research has shown that in some cases where exit radii are smaller and/or acceleration distances are short limiting a vehicle's ability to accelerate prior to the exit crosswalk, opening up exit geometry may not have a great effect on exit speed. However, relaxed exit geometry that increases acceleration distances and acceleration rates can potentially have significant effects on the exit crosswalk impacting pedestrian movements. This is particularly true for multi-lane roundabouts in off-peak times when a vehicle's fastest path may cross adjacent lanes. In any roundabout layout, it is the designer's responsibility to provide vehicle alignments that consistently control vehicle speeds from entrance to exit in an effective manner for all modes of transportation utilizing the roundabout. For this reason, after the above discussion, it seems reasonable to use roundabout entrance and exit alignments that limit a driver's ability to accelerate prior to the exit crosswalk and it appears that a good method to do that is the standard radius, speed relationship.

Table P–2: Maximum Straight Line Acceleration Performance by Vehicle

Maximum Performance - Straight Line Speed, Acceleration
Data From AutoRooster (autorooster.com/0-60-times)

Vehicle Data	0-60 (sec)	1/4 mile (sec)	60 mph dist (ft)	60 mph acel (ft/sec^2)
2008 Honda Civic, 4cyl	9.7	17.1	427.8	9.09
2010-12 Nissan Versa, 4 cyl	9.4	18.3	414.5	9.38
2013 Ford Escape, 4 cyl	9.3	17.4	410.1	9.48
2011-14 Chevy Cruze, 4 cyl	9.0	16.5	396.9	9.80
2009-12 Toyota Corolla. 4 cyl	8.9	16.7	392.5	9.91
2010-13 Chevy Tahoe, 8 cyl	8.5	16.9	374.9	10.38
2013 Ford Fusion, 4 cyl	8.5	16.9	374.9	10.38
2014 Ford Focus, 4 cyl	8.5	16.7	374.9	10.38
2012 Toyota Camry, 4 cyl	8.3	15.6	366.0	10.63
2011-12 Dodge Caravan, 6 cyl	8.1	16.7	357.2	10.89
2014 Chevy Impala, 6 cyl	8.1	16.3	357.2	10.89
2012-14 Ford Explorer, 4 cyl	7.8	15.9	344.0	11.31
2013 Honda Accord, 4cyl	7.7	15.8	339.6	11.45
2013 Nissan Altima, 4 cyl	7.1	15.5	313.1	12.42
2012 Mercedes S Class, 6 cyl(D)	7.0	15.3	308.7	12.60
2013 Toyota Avalon, 6 cyl	6.8	15.3	299.9	12.97
2012 Mercedes C Class, 4 cyl	6.8	15.3	299.9	12.97
2011-13 Ford F-150, 6cyl	6.5	15.3	286.7	13.57
2012-13 BMW 5 Series, 4 cyl	6.1	14.5	269.0	14.46
2012-13 Chevy Camero, 6 cyl	6.0	14.4	264.6	14.70
2009-12 Nissan Maxima, 6 cyl	5.8	14.4	255.8	15.21
2012-12 BMW 3 Series, 4 cyl	5.6	14.4	247.0	15.75
2011-13 Ford Mustang, 6 cyl	5.3	14.0	233.7	16.64
2014 Chevy Corvette, 8 cyl	3.9	12.1	172.0	22.62
2008-10 Dodge Viper, 10 cyl	3.6	11.9	158.8	24.50

Avg, 12.89 ft/s^2