5.0 GENERAL DESIGN STANDARDS AND DESIGN ELEMENTS

- **General**

The first section of this chapter provides the designer with background information on design standards. Information is presented concerning project type and the appropriate design standards to use. Project types are defined to assist the designer in applying the proper standards to the project. The rest of the chapter provides general design element information that applies to almost every project. Information on issues such as design speed, sight distance, superelevation, and clear zone are discussed. Later chapters on design standards are broken down into specific design areas such as rural design, urban design, freeway design, and intersection design.

5.1 DESIGN ELEMENTS

5.1.1 DESIGN SPEED

Design speed is a selected speed used to determine the various geometric design features of the roadway. The selected design speed should be consistent with the speeds that drivers are likely to expect on a given highway. The design speed of a project has a direct impact on the cost, safety, and quality of the finished project. With the exception of local streets, the chosen design speed in rural areas should be as high as practicable to attain a specified degree of safety, mobility, and efficiency while taking into consideration constraints of environmental quality, social and political impacts, economics, and aesthetics. In urban situations, the design speed should generally be higher than the posted speed of the particular section of roadway and consider land use, pedestrian needs, safety, and community livability. Care must be taken to not confuse design speed with operating speed, posted speed, 85th percentile speed, or running speed.

The selection of a design speed for any given project is dependent on several factors. These factors include traffic volume, geographic characteristics of an area, functional classification of the roadway, number of travel lanes, 85th percentile speed, roadway environment, adjacent land use, and type of project being designed. Design speeds are generally selected in increments of 10 km/h.

When selecting an appropriate design speed the designer should not only look at the roadway section in question but also adjacent sections to the proposed project. Within the project, the chosen design speed should be applied consistently throughout the section keeping in mind the speed a driver is likely to expect. This is very important when dealing with horizontal and vertical alignments, superelevation rates, and spiral lengths. For example a project with a selected design speed of 90 km/h consists of multiple horizontal curves. All horizontal curves should be designed for 90 km/h along with the appropriate superelevation and spiral length for the design speed. The proper use of design speed creates consistent roadways and expectations for the driver. Due to economical or environmental reasons all curves may not be able to achieve a 90 km/h design speed. In those cases it is important that the driver be advised of the lower speed condition ahead with the use of curve warning signs.

Finally, selecting the appropriate design speed for a particular section must consider transition areas from rural to urban environments. Providing a smooth and clear transition from high rural speed conditions to urban environments is critical in controlling drivers' perceptions of the areas they are entering. These transitions alert users of the changing environment, and control vehicular speeds as they enter various urban environments. The most common and effective transitions are those that establish a different roadway culture such as sidewalks, buffer strips, and raised medians. Another common technique for transition areas is visual narrowing of the roadway. This can be accomplished with raised islands, buffer strips, and landscaping.

5.1.2 85th PERCENTILE SPEED

The 85th percentile speed is that speed at or below which 85 percent of the drivers operate their vehicles and is considered the maximum safe speed for a given location. The 85th percentile speed assists in determining the posted speed. The designer should be aware that the posted speed and corresponding 85th percentile speed may not be the same. The posted speed may be set below the 85th percentile speed. All non-statutory posted speeds are determined by a speed study. The designer should check with the Traffic Management Section for speed study information when using 85th percentile and posted speeds in design. Measuring the 85th percentile speed in the field can provide additional information to the designer in selecting the appropriate design speed.

5.1.3 SELECTING PROJECT DESIGN SPEED

Section 5.1.1 discusses design speed and the factors involved in the selection of appropriate design speed. Design standards for design speeds of the different highway sections are located in the following chapters:

- Freeway (Urban and Rural) Chapter 6
- Rural Non-Freeway Chapter 7
- Urban Non-Freeway Chapter 8

- **85th Percentile/Posted Speed and Design Speed (In Urban Environments)**

Table 5-1 provides correlation between 85th percentile/posted speed and design speed. This table is intended to be used to assist the designers in selecting urban design speeds for preservation and enhancement type projects. This table is not to be used to select design speeds for rural projects or 4-R projects. This table can be used by the designer as a guide to assist in determining the proper design speed for roadway projects that have existing posted speeds. As previously discussed, the posted speed may not be representative of the 85th percentile speed. The designer should contact Traffic Management Section or the Region Traffic Unit to obtain the 85th percentile speed determined in the speed zone investigation. If the posted speed is lower than the 85th percentile speed, the designer should select the 85th percentile speed in determining the design speed. The designer should keep in mind the age of the speed zone study to determine if roadway characteristics have changed and there is a need to update the speed study. The design speeds below are minimums for the given posted speed condition. On some higher speed facilities (i.e. expressways of 80 km/h and above), the designer may consider other factors such as highway function and safety to determine the design speed.

Table 5-1
Urban Non-Freeway/Non-Expressway Design Speeds

85th Percentile/Posted Speed	Design Speed
20 mph	40[1] km/h
25 mph	50 km/h
30 mph	60 km/h
35 mph	60 km/h
40 mph	70 km/h
45 mph	90 km/h
50 mph	100 km/h

[1] 40 km/h design speed is only appropriate for local road classification.

5.2 SIGHT DISTANCE

- **General**

Sight distance is unobstructed distance of roadway ahead visible to the driver. There are multiple types of sight distance that include stopping sight distance, passing sight distance, decision sight distance and intersection sight distance. It is critical that sight distance issues be properly developed and applied to projects. Sight distance shall be provided as shown in Tables 6-1, 7-1, 7-2, and 8-1 through 8-5 as applicable.

Horizontal sight distance shall be checked when designing slopes and retaining walls or where median barriers, center piers, structure screening or screen plantings are used. Combinations of slight horizontal curvature with crest vertical curves may seriously diminish sight distance where high curb or planting is used. Slopes, walls and other side obstructions shall be set back from the pavement edge to provide at least minimum stopping sight distance for a driver in the traffic lane nearest the obstruction. The possibility of future conversion of shoulders or parking areas to driving lanes should be considered.

Intersections at grade shall be provided with at least minimum stopping sight distance and preferably intersection sight distance for the design speed. Sufficient sight distance should be provided so that the entering vehicle may cross or make a turn without significant slowing of the through traffic.
Horizontal sight distance, as measured 600 mm above the centerline of the inside lane at the point of obstruction, must at least equal the safe stopping sight distance. This assumes there is little or no vertical curvature. When the normal cut bank reduces the horizontal sight distance below the safe stopping sight distance for the design speed, the cut bank should be flattened or benched. For horizontal alignment on freeway detours, see Section 5.6.

5.2.1 STOPPING SIGHT DISTANCE

Stopping sight distance is the minimum distance required for a vehicle traveling at a particular design speed to come to a complete stop after an obstacle on the road becomes visible. Stopping sight distance is normally sufficient to allow an alert and prudent driver to come to a hurried stop under normal circumstances. Stopping sight distance is measured from the driver's eye (assumed to be 1080 mm above the roadway surface) to an object 150 mm above the roadway surface (**ODOT has maintained the 150 mm height of object**). Stopping sight distance is the summation of two distances: the distance traveled by a vehicle from the time the driver sees an object that requires a stop to the instant the brakes are applied, and the distance required to stop the vehicle from the time the brakes are applied. These two distances are called brake reaction distance and braking distance. Table 5-2 contains the safe stopping sight distance standards.

Stopping sight distance must, at a minimum, be obtained on all vertical and horizontal alignments. Figures 5-1 and 5-2 show the minimum stopping sight distance requirements for crest and sag vertical curves (See Table 9-5 for sight distance on ramps). Figure 5-3 indicates the minimum stopping sight distance for horizontal curves. Care must be taken to ensure that these minimum distances are obtained in project design. Roadside elements such as cut slopes, guardrail, tunnels, retaining walls, bridgerail, and barriers can obstruct the view of the driver and must be properly located to ensure that proper stopping sight distance is achieved.

Highway grades can have a significant affect on safe stopping sight distances. Refer to Exhibit 3-2 on page 115 of the 2001 AASHTO's *"A Policy on Geometric Design of Highways and Streets-2001"*, for more information about the effects of grades on stopping sight distances.

Table 5-2
Safe Stopping Sight Distance

Design Speed	Safe Stopping Sight Distance
40 km/h	50 m
50 km/h	65 m
60 km/h	85 m
70 km/h	105 m
80 km/h	130 m
90 km/h	160 m
100 km/h	185 m
110 km/h	220 m
120 km/h	250 m

Source: 2001 AASHTO

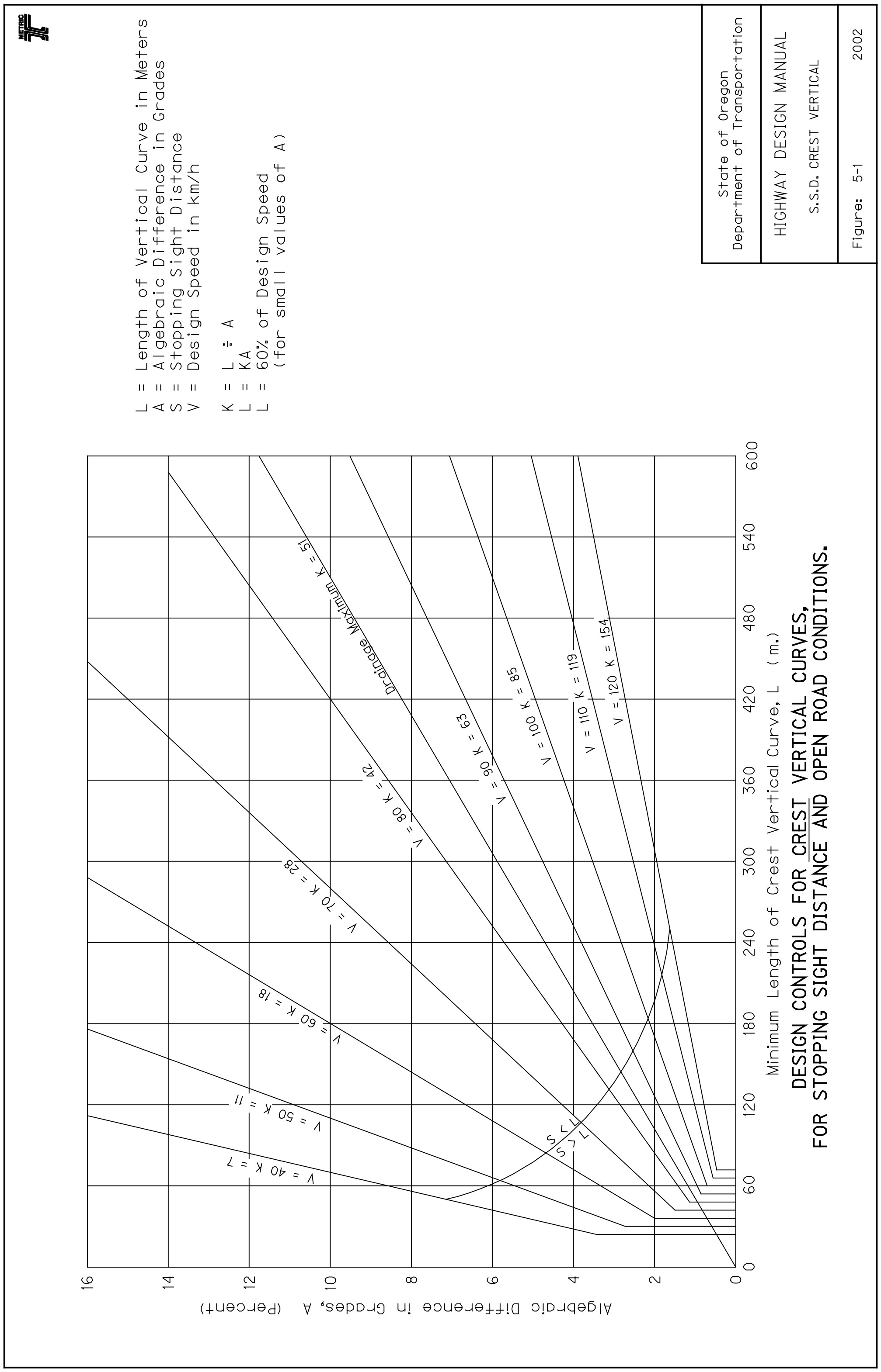
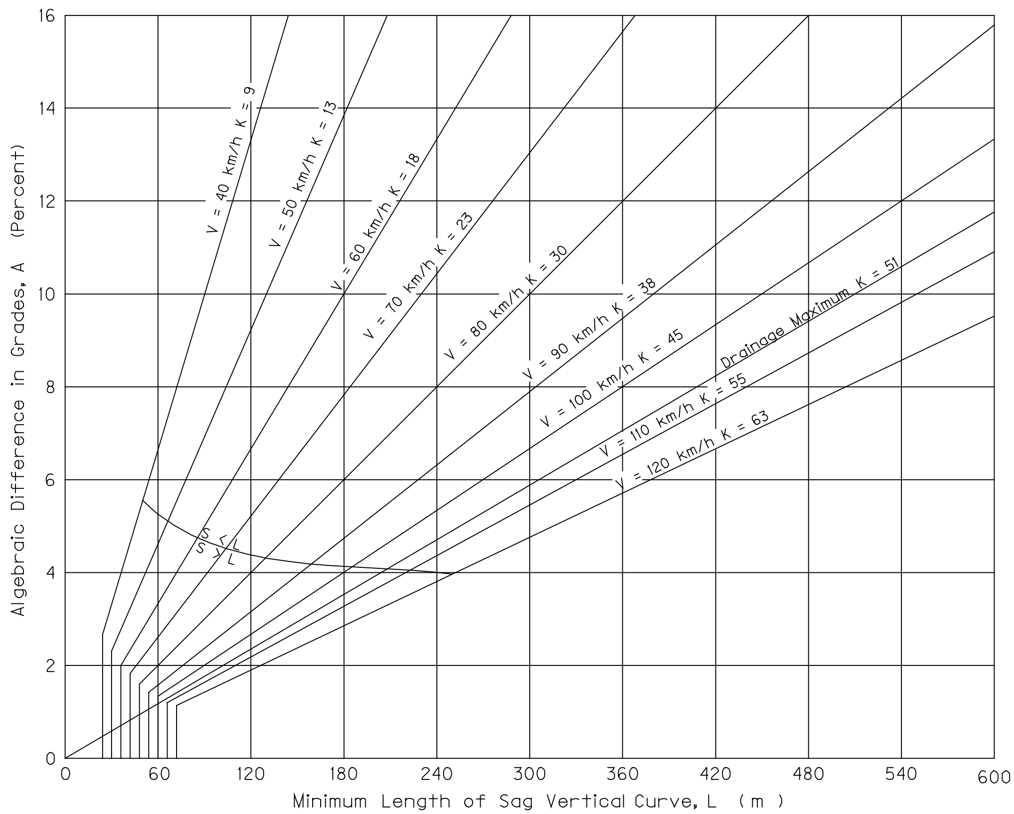


Figure 5-1
SSD Crest Vertical Curve



DESIGN CONTROLS FOR SAG VERTICAL CURVES.
OPEN ROAD CONDITIONS.

L = Length of Vertical Curve in Meters
A = Algebraic Difference in Grades
S = Stopping Sight Distance
V = Design Speed in km/h

K = L ÷ A
L = KA
L = 60% of Design Speed
(for small values of A)

State of Oregon
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HIGHWAY DESIGN MANUAL

S.S.D. SAG VERTICAL

Figure: 5-2 2002

Figure 5-2
SSD Sag Vertical Curve

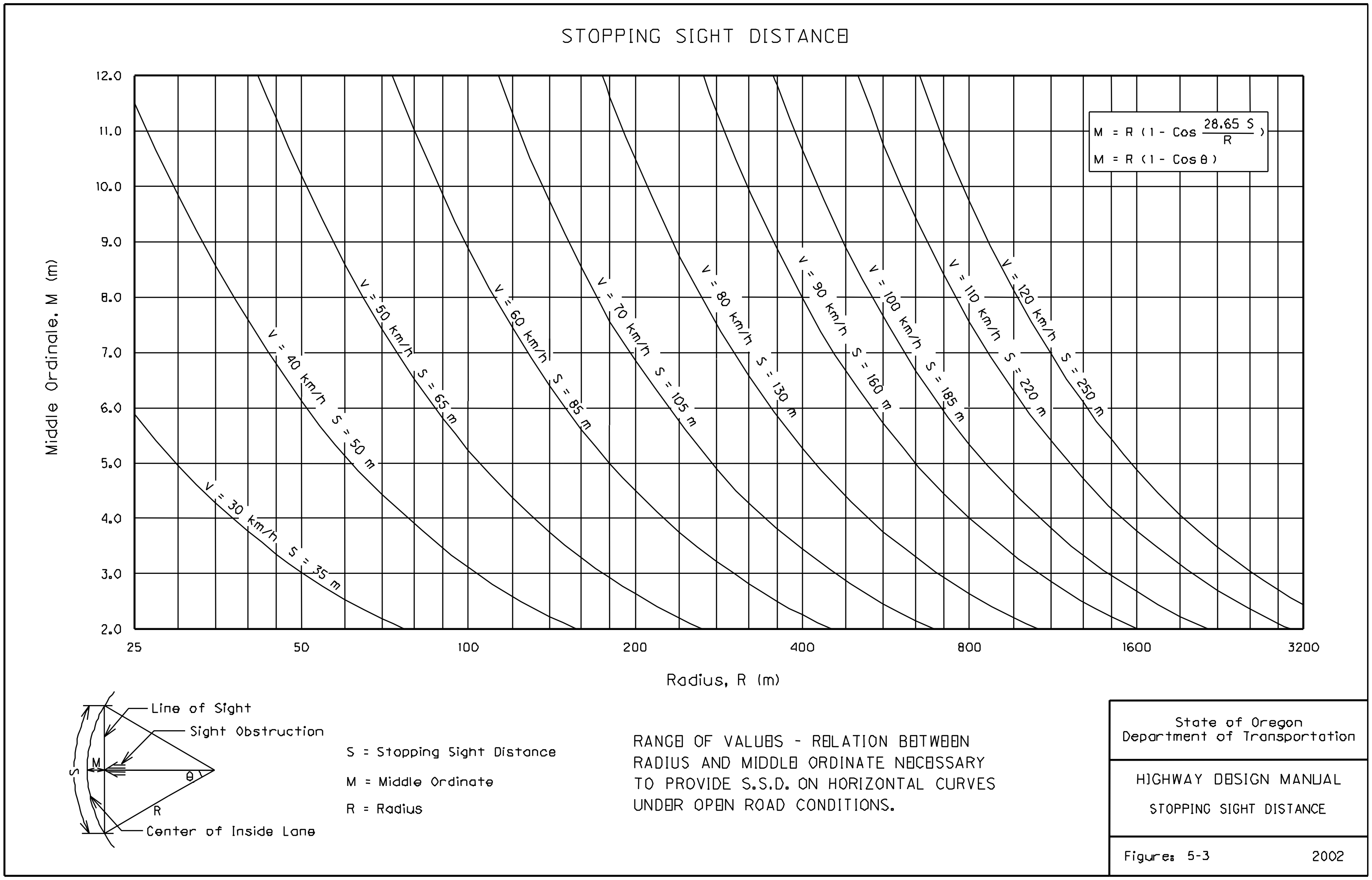


Figure 5-3
SSD On Horizontal Curves

5.2.2 DECISION SIGHT DISTANCE

Many times the elements of the roadway become complex and require additional distances for drivers to make the proper maneuver. Stopping sight distance may not be adequate when drivers must process complex roadway information in an instance or when the roadway information is difficult to decipher or unexpected. Decision sight distance should be provided at locations where multiple information processing, decision making, and corrective actions are needed. Sample locations where decision sight distance is needed include unusual intersection or interchange configuration and lane drops. Decision sight distance is calculated using the 1080 mm eye height and the 150 mm object height that is also used for stopping sight distance. Pages 115-117 of the 2001 AASHTO's *"A Policy on Geometric Design of Highways and Streets-2001"* provide more information on decision sight distance.

5.2.3 INTERSECTION SIGHT DISTANCE

Obtaining intersection sight distance is important in the design of intersections. Intersection sight distance is considered adequate when drivers at or approaching an intersection have an unobstructed view of the entire intersection and of sufficient lengths of the intersecting highways to permit the drivers to anticipate and avoid potential collisions. Sight distance must be unobstructed along both approaches at an intersection and across the corners to allow the vehicles simultaneously approaching, to see each other and react in time to prevent a collision. Intersection sight distance is determined by using a 1080 mm eye height and a 1080 mm height of object.

Intersection sight distance should be obtained at every road approach, whether it be a signalized intersection or private driveway. In no case should the sight distance be lower than safe stopping sight distance.

When reviewing intersection sight distance, items such as building clearances, street appurtenances, potential sound walls, landscaping, and other roadway elements must be taken into consideration in determining and obtaining the appropriate sight distance at intersections. Railroad and rail crossings should be treated in the same manner as roadway intersections in determining intersection sight distance.

Pages 665 – 669 of the 2001 AASHTO's *"A Policy on Geometric Design of Highways and Streets-2001"* indicate intersection sight distance requirements for traffic turning left, crossing, or turning right onto a major highway.

5.2.4 PASSING SIGHT DISTANCE

Passing sight distance is the minimum distance required for a vehicle to safely and comfortably pass another vehicle. An assumption made for passing sight distance includes the passing vehicle accelerating to a speed of 15 km/h above the vehicle being passed and the oncoming vehicle not reducing speed. A 1080 mm height of eye of the passing vehicle and 1080 mm height of object are used for measuring passing sight distance. If adequate passing sight distance opportunities cannot be accommodated in the project design, passing lanes or climbing lanes should be considered. Pages 118-126 of the 2001 AASHTO's *"A Policy on Geometric Design of Highways and Streets-2001"* provide more information on passing sight distance.

5.3 HORIZONTAL & VERTICAL ALIGNMENT

5.3.1 HORIZONTAL ALIGNMENT

- **General**

Horizontal alignment should provide for the safe and continuous flow of traffic at a uniform speed over substantial lengths of highway. The design of tangent highway sections and the curves that connect the tangents affect safe vehicle operating speeds, sight distances, passing opportunities and highway capacities. Decisions on alignment also have a major impact on the cost of a project. Consistent alignment practices should be maintained, i.e., sudden changes from tangents and gentle curves to sharp curves should be avoided. Beginning or ending curves on bridges should be avoided.

The combination of horizontal alignment and sight obstructions should be checked by designers. Horizontal curves through cut areas, through tunnels, and at intersections with minimum building set-backs should be analyzed carefully to determine that stopping and intersection sight distances are being met. Figure 5-3 provides design speed, stopping sight distance, and line of sight requirements for horizontal curves.

- **Curves**

Curve calculations shall be based on the radius definition for a circular curve. The project location should be assessed to determine the appropriate radius of curvature. Minimum radius of curvature can be found in Tables 6-1, 7-1, 7-2, and 8-1 through 8-5. Sufficient curve length must be used in open country to prevent the appearance of a "kink" in the line.

The use of compound curves will be permitted only upon approval of the Roadway Engineering Manager. Compound curves are multiple radius curves directly connected one to another without any transition. When approved, intermediate spiral segments shall be used which shall have an "A" value equal to or less than the standard for the sharper curve.

Broken back curves are hazardous and should be avoided. "Broken back curves" are curves connected with a short segment of tangent. When the use of "broken back" alignment with a short tangent cannot be avoided, consideration must be given to designing the tangent section so that all travel lanes slope in the same direction as the superelevation of the curves. This avoids the introduction of two flat spots on the travel lane toward the outside of the curves and may prevent the development of an unsightly dip on the edge of the pavement that can affect driver comfort and drainage (see Figure 5-4). Generally this treatment should be used when the length of the "normal cross slope" on the intervening tangent would be 150 m or less.

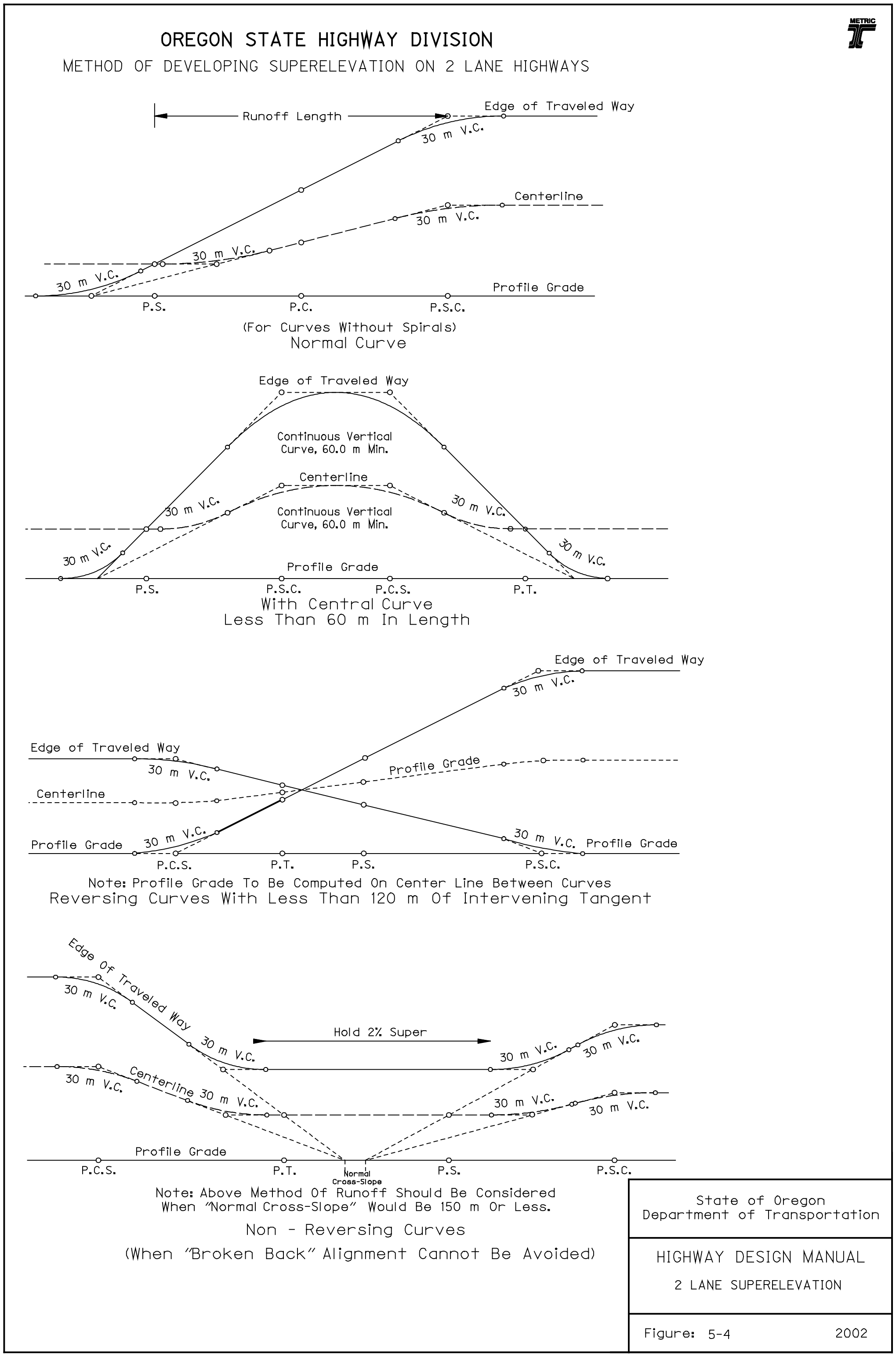


Figure 5-4
Developing Superelevation On 2-Lane Highways

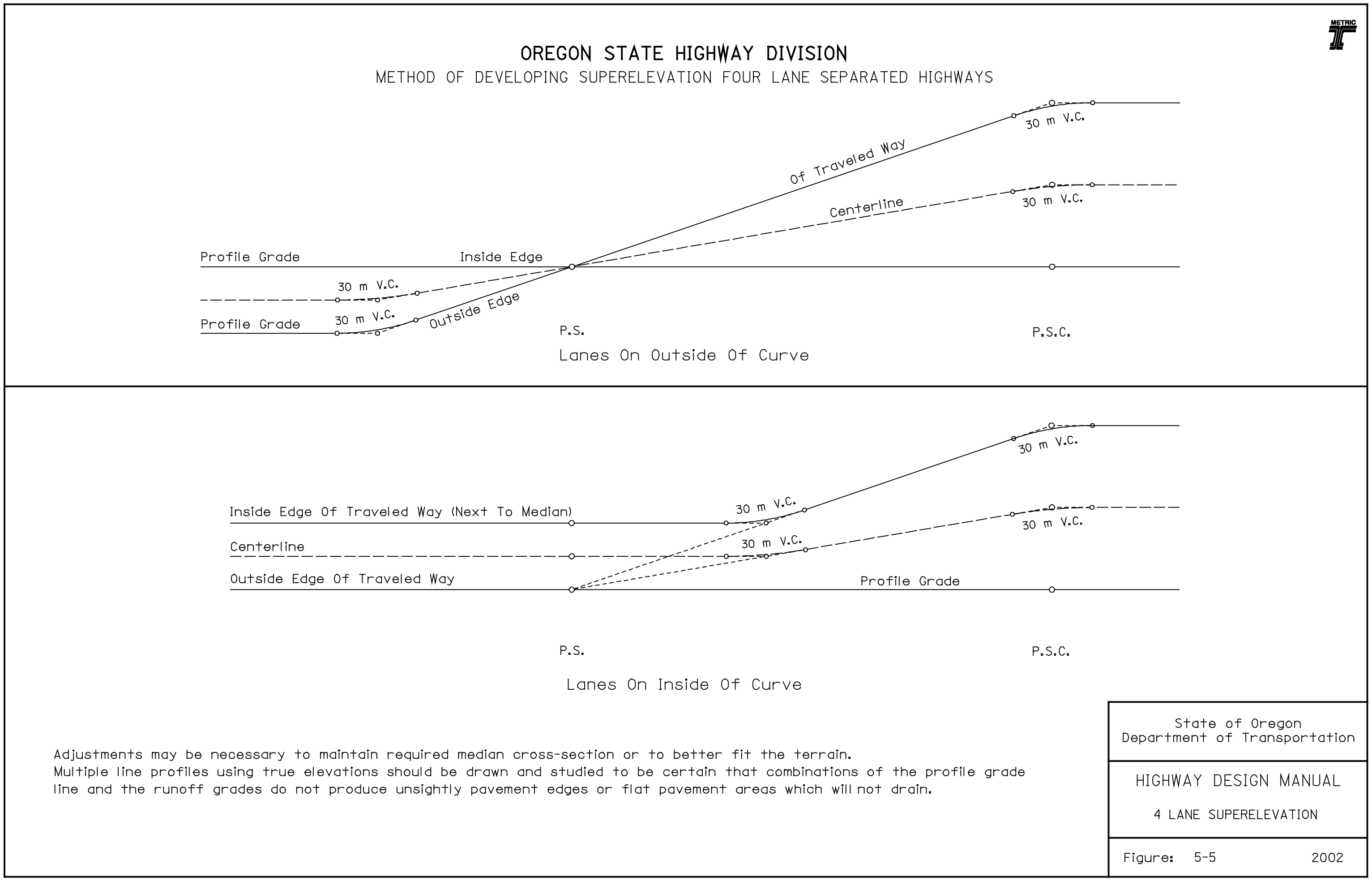


Figure 5-5
Developing Superelevation On 4-Lane Highways

- **Spirals**

Spirals provide a transition between tangents and curves and between circular curves of substantially different radii (spiral segment). The natural path of a vehicle entering a curve is to drive a spiral. Spirals also provide a location for developing surperelevation. All curves with a radius less than 2000 m shall be spiraled and so shown on the plans submitted. This applies to secondary as well as to primary highways. Curves with a 2000 m or flatter radius are not required to be spiraled. It is recommended that spirals be used for curves with a radius of 2000 m or flatter to assist in developing the superelevation runoff. When unspiraled curves with a radius less than 2000 m are approved by the Roadway Engineering Manager, the length of the runoff should be equal to the normal spiral length for the curve.

<u>Longer spirals</u> than the standard may be used wherever advantage in their use is apparent. Consideration should be given to using longer spirals appropriate for a section with additional lanes when future widening is anticipated. <u>Shorter spirals</u> may be used in certain instances when conditions do not permit the standard spiral length. The three formulas given in Tables 5-3 through 5-5 can be used to determine the minimum spiral length. Use of a spiral length from one of the three minimum spiral length formulas will require a design exception.

Double spirals may not be used. The minimum length of the simple curve between spirals shall be 15.0 m. At times it may be appropriate to install a spiral segment to transition from one central curve to another central curve. These are called compound curves. The spiral segment assists in providing a smooth transition between very close curves.

The proper combination of horizontal curve, spiral transition length, and superelevation rate provides for good design practice. The type and locations of the facility (urban or rural in nature) will dictate the proper combination of curve, spiral, and superelevation rate. The designer should be aware that some computer-aided design tools only provide a graphical representation of spiraled alignments and do not draw accurate spirals.

On some low speed non-superelevated roadways, the use of spirals may not be warranted. Low speed (30 km/h or less) local streets and special purpose roads without superelevation can warrant a design exception for exclusion of spirals. Chapter 5 of the 2001 AASHTO's *"A Policy on Geometric Design of Highways and Streets-2001"* provides additional background surrounding local streets and locations where spirals may not be needed.

Table 5-3 shall be used to determine proper spiral lengths and superelevation rates for freeways and rural highways (2 lane section). This table should also be used for constrained rural mountainous locations. The spiral details are available in the *ODOT Metric Alignment Guide*, 1995.

Table 5-4 shall be used for urban locations where design speeds range from 40–60 km/h and the maximum superelevation rate is 4%. Table 5-5 shall be used for transition areas between urban/suburban and rural areas and design speeds range from 70-80 km/h, with a maximum superelevation rate of 6%.

- **Superelevations**

In the design of highway curves, it is necessary to obtain a balance between the various factors involved, such as speed, curvature and centrifugal force. The use of superelevation provides the major method for counteracting side-friction, which is essential to the development of an acceptable design.

The rate of superelevation will vary depending upon the type of facility, urban or rural nature of the roadway, and design speeds. In combination with spiral lengths, superelevation rate charts have been developed. The Superelvation rates and spiral lengths shown in Table 5-4 (e max of 4%) should be used for urban environments where the design speeds are 60 km/h or less. In rural to urban/suburban transition areas and urban expressways environments where design speeds are 70 km/h to 80 km/h, Table 5-5 (e max of 6%) should be used. For freeways and open road rural highways, Figure 5-6 and Table 5-3 shall be used. This table also includes superelevation rates for turning roadways and constrained rural arterial sections. The designer needs to be aware of the type of roadway in question and apply the proper superelvation rate.

Table 5-3 should also be used for rural areas as where snow and ice conditions prevail. Elevations over 900 m can be considered where snow and ice prevail. Other locations, such as the Columbia River Gorge could be considered for discussion as a snow and ice area. Effort should be made to limit the radius of curvature to that of the corresponding 8% superelevation. For example, if the design speed is 110 km/h, the maximum radius where snow and ice prevail would be 600 m. If a sharper curvature must be used, the superelvation would be held at 8% with the understanding that the curve would need to be posted at lower than the design speed. In this situation, the safe speed figure (Figure 5-7) can be used to determine the safe speed of any curve at 8% superelevation. Use of the 8% superelevation where snow and ice prevail should be approved by the Roadway Manager.

The superelevation runoff shall conform to the standard method of development shown in Figures 5-4 and 5-5. The standard method of superelevation development for ODOT is rotation around the profile grade. The profile grade is carried along the low side edge of travel. The exception to this standard method is curves on grades of 4% or greater. In these situations, the superelevation should be developed according to Section 5.3.4. There may also be occasions on 3R projects where it may be appropriate to rotate profile grade around centerline. If rotation about centerline is considered, a three-line profile should be run in order to prohibit the forming of drainage ponds on the low side edge of pavement. On curves with radius greater than 2000 m (w/o spirals), the runoff length should be equal to the normal spiral length for the curve. It is not normal practice to calculate super runoff for curves flatter than 5000 m radius.

In the design of runoff, the use of multiple line profiles is suggested. Multiple line profiles should be used for design purposes and submitted with the job whenever special problems such as grade controls at road approaches, building elevations or interchange designs are encountered.

In cases where less than 60 m of main curve is left after spiraling, the superelevation along the main curve shall be determined by joining the runoffs in the center of the curve and using a continuous vertical curve of a length equal to twice the length of the main curve, with a minimum vertical curve length of 60 m.

On multi-lane divided highways, each direction may have an independent alignment. In these situations, the superelevation for one direction should be independent of the other to minimize run-out lengths. Each direction should follow the superelevation rules contained in this section for the number of lanes on each alignment.

When the tangent distance between reversing curves is less than 120 m, the runoff of the superelevation shall be adjusted so that the edges of pavement and the centerline fall on a uniform grade between the Point of Curve to Spiral (PCS) of the first curve and the Point of Spiral to Curve(PSC) of the second curve. This shall be accomplished by transferring the profile grade from the lower edge of the traveled way to the centerline for the transition and back to the lower edge of the traveled way for the second curve (see Figure 5-4). The resulting profile grade shall be shown on all profiles.

The standard superelevation shall be used on climbing lanes, except that if extreme climatic conditions warrant it, reduced superelevation may be used on a climbing lane on the high side of a curve. However, such an exception must be approved by the Roadway Engineering Manager.

Minimum radius is a limiting value of curvature for a given design speed and is determined from maximum rate of superelevation and the maximum allowable side friction factor. It may become advantageous during the design of an urban project to select a curve/superelevation relationship which is not shown on Figure 5-6 or Table 5-3. This may occur when reducing superelevations in an urban situation to accommodate cross streets, sidewalks and R/W constraints. The allowable minimum radius of curvature calculated for a combination of factors not in these tables can be found by referencing *A Policy on Geometric Design of Highways and Streets - 2001*

- **Safe Speed Chart**

The Safe Speed Chart shown in Figure 5-7 represents the vehicle speed, radius and superelevation at the point where the driver begins to experience an unacceptable level of discomfort. The data in this chart does not represent a design standard. Design standards for superelevation are provided in Tables 5-3 through 5-5. This chart is provided as a tool to evaluate existing or proposed sections for safety and operation. It can also be used for supporting data as part of a design exception.

Table 5-3
Standard Spiral Lengths and Superelevations

2 Lane Rural Roads And Freeways				2 Lane Constrained Rural Mountainous Arterial Sections			
km/h	**Radius (meters)**	**Length (meters)**	**"e" % slope**	**km/h**	**Radius (meters)**	**Length (meters)**	**"e" % slope**
120	8000	n/a	Crown	70	210	120	11
120	5000	n/a	Crown	70	200	120	11
120	3000	120	2	70	190	120	11
120	2000	120	4	60	180	120	11.5
120	1500	120	4	60	170	120	11.5
120	1200	120	5	60	160	120	11.5
110	1000	120	6	60	150	120	11.5
110	900	120	6	60	145	120	11.5
110	800	150	6	60	140	100	11.5
110	700	150	7	60	135	100	11.5
110	650	150	7	60	130	100	11.5
110	600	150	8	60	125	100	11.5
100	550	150	8.5	60	120	100	11.5
100	500	150	8.5	60	115	100	11.5
100	475	150	9	50	110	100	12
100	450	150	9	50	105	100	12
100	425	150	9.5	50	100	85	12
100	400	150	9.5	50	95	85	12
90	380	150	10	50	90	75	12
90	360	150	10	50	85	75	12
90	340	150	10	50	80	75	12
90	320	150	10	50	75	75	12
90	300	150	10	40	70	75	12
90	290	150	10	40	65	75	12
80	280	120	10.5	40	60	75	12
80	270	120	10.5	40	55	75	12
80	260	120	10.5	40	50	75	12
80	250	120	10.5				
80	240	120	10.5				
80	230	120	10.5				
80	220	120	10.5				

The above spiral lengths are for a two lane traveled way. (Two 3.6 m lanes) For multi-lane highways that are superelevated in the same plane between the edges of the traveled way, the following factors should be used to obtain the required spiral length:

3 lanes - 1.2 times the above value
4 lanes - 1.5 times the above value
5 lanes - 1.8 times the above value
6 lanes - 2.0 times the above value

When standard length spirals cannot be obtained, use the formulas below to determine minimum spiral lengths by runoff, comfort, & aesthetics. Use the longest spiral solution of the three formulas and round up to the next 5 m value.

Superelevation Runoff: $Ls = we/2s$

Where: e = Superelevation rate in %
Where: w = width, (edge of travel to edge of travel)
Where: s = relative slope in percent (as follows)

s = 0.75 @ 30 km/h
s = 0.70 @ 40 km/h
s = 0.65 @ 50 km/h
s = 0.60 @ 60 km/h
s = 0.55 @ 70 km/h
s = 0.50 @ 80 km/h
s = 0.48 @ 90 km/h
s = 0.45 @ 100 km/h
s = 0.42 @ 110 km/h
s = 0.40 @ 120 km/h

Comfort Control: $Ls = v^3/28R$

Where: v = Velocity in km/h
Where: R = Radius of curve

Aesthetic Control: $Ls = v/1.8$

Where: v = Velocity in km/h

	40 km/h			50 km/h			60 km/h			
R	e%	L2	L4	e%	L2	L4	e%	L2	L4	R
7000	NC	-	-	NC	-	-	NC	-	-	8000
5000	NC	-	-	NC	-	-	NC	-	-	5000
3000	NC	-	-	NC	-	-	NC	-	-	3000
2000	NC	-	-	NC	-	-	NC	-	-	2000
1500	NC	-	-	NC	-	-	NC	-	-	1500
1200	NC	-	-	NC	-	-	RC	40	60	1200
1000	NC	-	-	NC	-	-	RC	40	60	1000
900	NC	-	-	NC	-	-	RC	40	60	900
800	NC	-	-	RC	40	60	2.1	40	60	800
700	NC	-	-	RC	40	60	2.3	50	75	700
650	NC	-	-	RC	40	60	2.4	50	75	650
600	RC	40	60	2.1	40	60	2.5	50	75	600
550	RC	40	60	2.2	40	60	2.6	50	75	550
500	RC	40	60	2.3	50	75	2.7	50	75	500
475	RC	40	60	2.4	50	75	2.7	50	75	475
450	RC	40	60	2.4	50	75	2.8	60	90	450
425	2.1	40	60	2.5	50	75	2.9	60	90	425
400	2.1	40	60	2.5	50	75	2.9	60	90	400
380	2.2	40	60	2.6	50	75	3.0	60	90	380
360	2.3	50	75	2.6	50	75	3.1	60	90	360
340	2.3	50	75	2.7	50	75	3.2	60	90	340
320	2.4	50	75	2.8	60	90	3.2	60	90	320
300	2.4	50	75	2.8	60	90	3.3	60	90	300
290	2.4	50	75	2.9	60	90	3.4	70	105	290
280	2.5	50	75	2.9	60	90	3.4	70	105	280
270	2.5	50	75	3.0	60	90	3.5	70	105	270
260	2.5	50	75	3.0	60	90	3.5	70	105	260
250	2.6	50	75	3.0	60	90	3.6	70	105	250
240	2.6	50	75	3.1	60	90	3.6	70	105	240
230	2.6	50	75	3.2	60	90	3.7	70	105	230
220	2.7	50	75	3.2	60	90	3.7	70	105	220
210	2.7	50	75	3.3	60	90	3.8	70	105	210
200	2.8	60	90	3.3	60	90	3.8	70	105	200
190	2.8	60	90	3.4	70	105	3.9	70	105	190
180	2.9	60	90	3.5	70	105	3.9	70	105	180
170	3.0	60	90	3.5	70	105	4.0	70	105	170
160	3.0	60	90	3.6	70	105	4.0	70	105	160
150	3.1	60	90	3.7	70	105	4.0	70	105	150
145	3.1	60	90	3.7	70	105	R min. 150 m			145
140	3.2	60	90	3.8	70	105				140
135	3.2	60	90	3.8	70	105				135
130	3.3	60	90	3.8	70	105				130
125	3.3	60	90	3.9	70	105				125
120	3.4	70	105	3.9	70	105				120
115	3.4	70	105	3.9	70	105				115
110	3.5	70	105	4.0	70	105				110
105	3.5	70	105	4.0	70	105				105
100	3.6	70	105	4.0	70	105				100
95	3.7	70	105	R min. 100 m						95
90	3.7	70	105							90
85	3.8	70	105							85
75	3.9	70	105							75
70	3.9	70	105							70
65	4.0	70	105							65
60	4.0	70	105							60
55	R min. 60 m									55
50										50

In recognition of safety considerations, the use of e Max=4% should be limited to urban conditions.

e Max 4%

e - Superelevation in percent
Ln - Standard spiral length for n lanes
L2 - Standard spiral length for two lanes

L4 - Standard spiral length for four lanes with up to 2.4 m median

L3=1.25(L2) , L5=1.8(L2) , L6=2(L2)
(Check for minimum length)

When standard length spirals cannot be attained,use the formulas below for minimum spiral lengths by runoff, comfort, & aesthetics: Use the longest spiral solution of the three formulas and round to the nearest higher even 5 meters.

Superelevation Runoff: Ls=we/2s
Where: e=superelevation rate in percent
Where: w=width, edge of travel to edge of travel
Where: s=relative slope in percent
s=0.70 @ 40 km/h
s=0.65 @ 50 km/h
s=0.60 @ 60 km/h

Centrifugal Control: $Ls=V^3/28R$
Where: V=velocity in km/h and R= radius of curve

Aesthetic Control: Ls=V/1.8
Where: V=velocity in km/h

METRIC

Table 5-4
e Max 4%

	70 km/h			80 km/h			
R	e	L2	L4	e	L2	L4	R
8000	NC	-	-	NC	-	-	8000
5000	NC	-	-	NC	-	-	5000
3000	NC	-	-	NC	-	-	3000
2000	NC	-	-	NC	-	-	2000
1500	NC	-	-	2.2	50	-	1500
1200	2.2	50	75	2.7	60	90	1200
1000	2.6	50	75	3.1	60	90	1000
900	2.8	60	90	3.4	70	105	900
800	3.1	60	90	3.6	70	105	800
700	3.4	70	105	4.0	80	120	700
650	3.6	70	105	4.1	80	120	650
600	3.8	70	105	4.3	80	120	600
550	4.0	80	120	4.5	90	135	550
500	4.2	80	120	4.8	90	135	500
475	4.3	80	120	4.9	90	135	475
450	4.4	80	120	5.0	90	135	450
425	4.6	90	135	5.2	100	150	425
400	4.7	90	135	5.3	100	150	400
380	4.8	90	135	5.4	100	150	380
360	5.0	90	135	5.6	110	165	360
340	5.1	100	150	5.7	110	165	340
320	5.2	100	150	5.8	110	165	320
300	5.4	100	150	5.9	110	165	300
290	5.5	100	150	5.9	110	165	290
280	5.5	100	150	5.9	110	165	280
270	5.6	110	165	6.0	110	165	270
260	5.7	110	165	6.0	110	165	260
250	5.7	110	165	R min. 255 m			250
240	5.8	110	165				240
230	5.9	110	165				230
220	5.9	110	165				220
210	6.0	110	165				210
200	6.0	110	165				200
190	R min. 195 m						190
180							180
170							170
160							160
150							150
145							145
140							140
135							135
130							130
125							125
120							120
115							115
110							110
105							105
100							100
95							95
90							90
85							85
75							75
70							70
65							65
60							60
55							55
50							50

e Max 6%

e - Superelevation in percent
Ln - Standard spiral length for n lanes
L2 - Standard spiral length for two lanes

L4 - Standard spiral length for four lanes
with up to 2.4 m median

L3=1.25(L2), L5=1.8(L2), L6=2(L2)
(Check for minimum length)

When standard length spirals cannot be attained, use the formulas below for minimum spiral lengths by runoff, comfort, & aesthetics: Use the longest spiral solution of the three formulas and round to the nearest higher even 5 meters.

Superelevation Runoff: Ls=we/2s
Where: e=superelevation rate in percent
Where: w=width, edge of travel to edge of travel
Where: s=relative slope in percent
s=0.55 @ 70 km/h
s=0.51 @ 80 km/h

Centrifugal Control: $Ls=V^3/28R$
Where: V=velocity in km/h and R= radius of curve

Aesthetic Control: Ls=V/1.8
Where: V=velocity in km/h

Use of e Max=6% is appropriate in transition areas and on urban expressways with mixed controls.

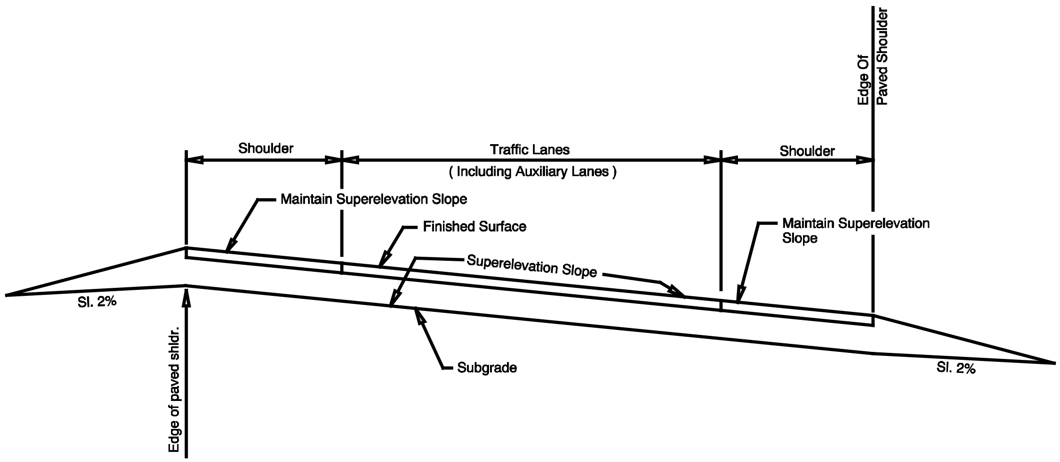
METRIC

Table 5-5
e Max 6%

5.3.2 VERTICAL ALIGNMENT

Vertical curves shall provide sight distance at least equal to the safe stopping sight distance for the indicated design speed. The vertical sight distance is the distance from the operator's eye, assumed to be 1080 mm above the pavement to the point ***150 mm*** above the pavement. The minimum lengths of vertical curves which may be used for the various design speeds are shown in Figures 5-1 and 5-2. It is desirable to increase the length of vertical curves over that shown whenever it is economically possible.

Note that the object height for safe stopping sight distance is 150 mm. This value is different than AASHTO's *"A Policy on Geometric Design of Highways and Streets-2001"*. ODOT uses a lower object height for stopping sight distance calculations to afford drivers the opportunity to stop for objects often found on Oregon State Highways. Using a lower object height also provides for smoother and safer vertical curves.



STANDARD SUPERELEVATION

Radius (m)		Superelevation of Traffic Lanes (%)	Design Speed (km/h)
Max.	Min.		
∞	3615	Normal Crown	120
3610	2835	2	120
2830	2015	3	120
2010	1415	4	120
1410	1180	5	120
1175	785	6	110
780	640	7	110
635	580	8	110
575	490	8.5	100
485	430	9	100
425	385	9.5	100
380	290	10	90
285	215	10.5	80

NOTE:

In URBAN AREAS super rates should be based on the roadway design speed and Section 6.0 of Highway Design Manual. Additional data on superelevations can be found in "A Policy on Geometric Design of Highways and Streets, AASHTO - 2001".

Superelevations for Design Speeds less than 80 km/h may be found in Table 4-7 of the ODOT Highway Design Manual.

For 4 and 6 lane highways with paved medians that are superelevated on the same plane as the travel lanes, the length of spirals and runoff rates are modified as shown in Table 4-7 of the ODOT Highway Design Manual.

NOTE: All material and workmanship shall be in accordance with the current Oregon Standard Specifications

OREGON STANDARD DRAWINGS

ROADWAY CROSS SLOPES
SUPERELEVATED SECTIONS

2002

REVISIONS

DATE	DESCRIPTION

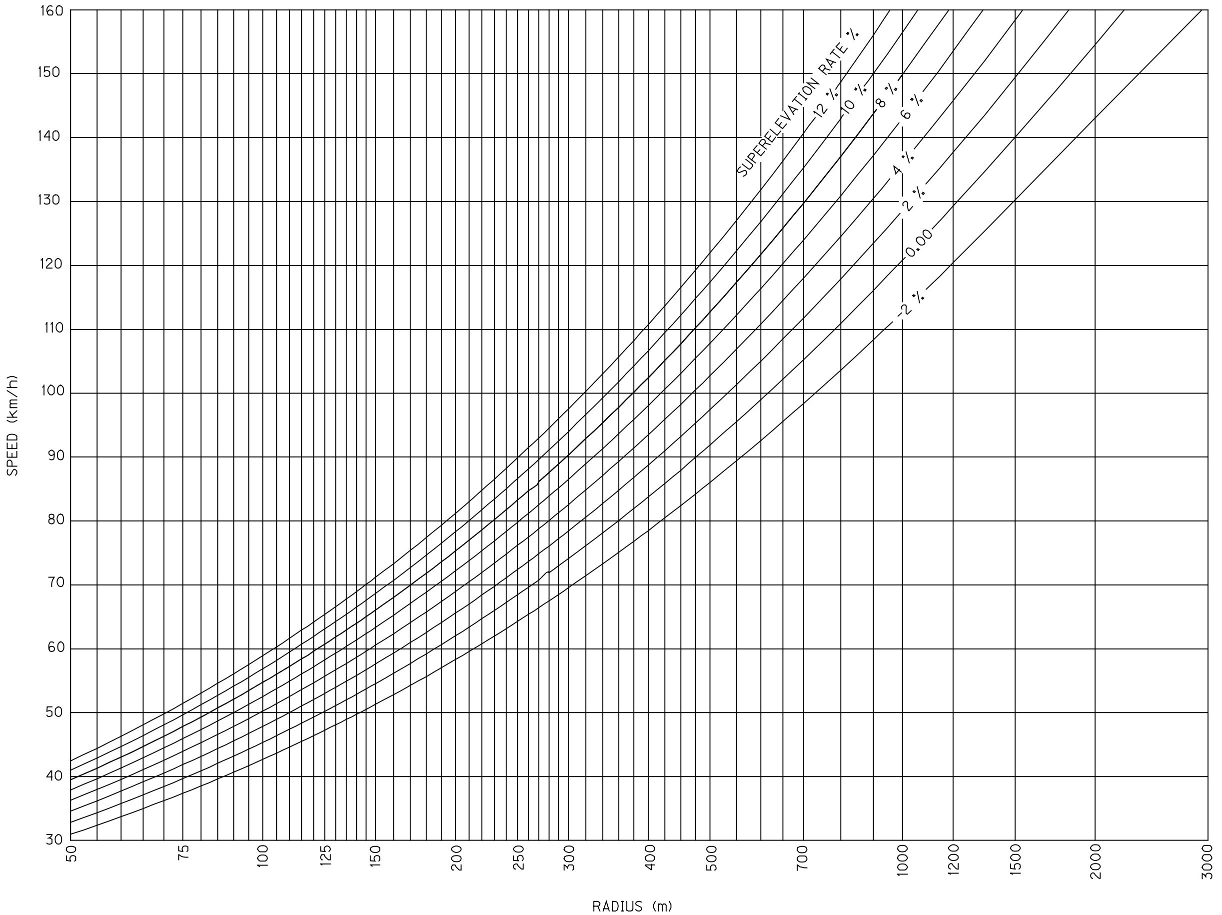
RD200

Figure 5-6
Standard Superelevation

SAFE SPEED CHART

$$R = \frac{V^2}{1.27(e+f)}$$

e = Rate Of Roadway Superelevation
f = Side Friction Factor
V = Vehicle Speed, km/h
R = Radius, m



SPEED	FRICTION FACTOR
30	17.1
40	16.5
50	15.9
60	15.3
70	14.7
80	14.0
90	13.5
100	12.8
110	12.2
120	11.5

State of Oregon
Department of Transportation

HIGHWAY DESIGN MANUAL

SAFE SPEED CHART

Figure: 5-7 2002

Figure 5-7
Safe Speed

5.3.3 COMBINED HORIZONTAL AND VERTICAL ALIGNMENT

The combined effect of the horizontal and vertical alignment must be considered during design of a highway (see AASHTO's *"A Policy on Geometric Design of Highways and Streets – 2001").*

When designing for the coordination of horizontal and vertical alignment, the following issues need to be considered.

- Curvature and grades should be balanced. Tangent alignment mixed with steep grades or flat grades with excessive curvature is poor design. A balance of both elements leads to uniform operation, aesthetically pleasing, and safe designs.
- Vertical and horizontal alignments should complement each other.
- Sharp horizontal curves should not be located at or near the top of a crest vertical curve.
- Sharp horizontal curves should not be located at the low point of a sharp vertical curve.
- Horizontal and vertical curvature should be as flat as possible in the area of intersections to allow for proper sight distance.

On summits with both horizontal and vertical curves, the horizontal curve should be longer than the vertical curve. There should not be more than one vertical curve within a horizontal curve. It is desirable to provide a tangent grade on tangent alignment. Once the sight distance is broken by a curve in either the vertical or the horizontal alignment, there is little value in maintaining a tangent. The ideal alignment extends from control point to control point without unnecessary curvature between. However, extremely long tangents are believed to cause problems, due to driver boredom. It is almost impossible to provide safe passing sight distance on vertical curves, therefore safe stopping sight distance should be provided. (See AASHTO's, *"A Policy on Geometric Design of Highways and Streets-2001"*)

In the design of two-lane arterials, the alignment and profile should provide sections for safe passing at frequent intervals.

5.3.4 GRADES

On grades of 4% or over, the profile grade shall be carried at the edge of the traveled way to the right of the centerline ascending the grade. The superelevation is obtained by raising the center and left side of the roadway on curves turning to the right going up hill, and by lowering the center and left side of the roadway on curves turning to the left. Where this rule applies and the horizontal curve passes over a summit, the profile grade shall be carried on the outside of the curve developing superelevation by lowering the center and inside edge of the roadway. For grades less than 4%, the standard method of superelevation shall be in conformance with Section 5.3.1.

Maximum grade for principal arterials and expressways shall be as shown in Tables 7-1, 7-2, and 8-1 through 8-5. Maximum grade for freeways shall be as shown in the ODOT 4R/New Freeway Standards Table 6-1.

It is important to take into account the impact from grades in the different design elements such as acceleration and deceleration lanes, safe stopping sight distance, passing sight distance, and intersection sight distance. Exhibit 3-2 in AASHTO's *"A Policy on Geometric Design of Highways and Streets-2001"*, page 115 shows the effects of grade on stopping sight distance. The designer should refer to this table and take into account the impact of project grades on stopping sight distance.

5.4 CROSS SECTION

- **Roadway**

The Standard Roadbed Sections (Figures 6-1, 6-4, 7-1) and the ODOT 4R/New Standards (Tables 6-1, 7-1, 7-2, 8-1 through 8-5) give the dimensions to be used for the design of the roadway. These include shoulders, travel lanes, and medians. Frontage roads shall be designed in accordance with the anticipated traffic and their location.

When the distance computed for the lateral support of the surfacing material is a fractional distance, the lateral support slope distance is rounded up to the next even 0.1 m increment.

In cases of very rugged terrain and where grading costs are high, consideration should be given to using steeper slopes or curb sections for lateral support. The use of either must be approved by the Roadway Engineering Manager. Curbs should be avoided on rural highways.

When the slope at the edge of the surfacing material is 1:6 and continuous sections of guard rail are required, consideration may be given to reducing the surfacing material slope to a minimum of 1:3 behind the guard rail when the width is critical. This may apply in the case of railway encroachments, high fill, or very high cost right of way.

- **Curbs and their Location**

When curbs are used on any freeway, or rural highways they should be mountable. Only the low profile mountable curb has been approved for freeway application. The low profile mountable curb, mountable curb, and mountable curb and gutter curb are approved for other locations. Full shoulder width shall be provided and paved to the same depth as the main roadway.

Where a standard curb is introduced, it should be curved away from the edge of the travel lane on the end of the curbed section approached by traffic. It need not be curved away where traffic leaves the curbed section. When curbs are used on highways with narrow shoulders, the beginning of a curb on the right shall be offset a minimum of 1.8 m. On the left, the offset shall not be less than 1.0 m greater than the normal curb offset. Refer to ODOT Standard Drawing RD220.

Where roadway grades are 0.2 percent or less, monolithic curb and gutter design (curb and gutter curb, and mountable curb and gutter curb) shall be used. On grades greater than 0.2 percent, low profile mountable curb, standard curb, or mountable curb should be used. Refer to ODOT Standard Drawings RD700 and RD705.

Although curbs are typically installed in urban areas, there may be instances where curbs are not installed due to water quality reasons. The Hydraulics Unit should be contacted for discussion on curbs and water quality issues.

- **Roadside Barriers**

Where right side roadside barriers are used, the standard right shoulder width will be increased to provide the 0.6 m "E" offset. This applies to all divided arterial locations, freeway (including ramps), or non-freeway. When the right hand shoulder is 3.6 m or greater, the 0.6 m “E” offset is not required, as a 3.6 m right side shoulder is adequate to park a disabled vehicle. The 0.6 m offset applies to both concrete barrier and guardrail.

The 0.6 m “E” distance is not added to the left side shoulder except under the following conditions:

- On freeways when the standard shoulder is 3.0 m.
- Four lane mainline section of roadways using concrete median barrier when the left side shoulders (1.8 m or less) of the opposing lanes are separated by only barrier.

This standard does not require the additional 0.6 m “E” for the left shoulder at spot roadside barrier locations such as bridges and interchange areas unless the above criteria is met. Interchange ramps with left side roadside barriers do not require the 0.6 m “E” offset.

Exceptions to the 0.6 m "E" widening may be approved by the Roadway Engineering Manager when the additional shoulder widening is not practical.

For more information on roadside barrier design and location refer to Section 5.8.

- **Ditches**

Figures 6-1, 6-4, and 7-1, outline the typical ditch section for rural highways, and urban and rural freeways. These typical sections create a standard roadside ditch that is 0.15 m deep. The peak discharge, longitudinal slope, and ground cover for each ditch affects the ditch capacity. On steep slopes shear stresses on the ditch bottom should be evaluated to assure the ditch does not erode. The discharge contributing to ditches runs off from areas from within the right of way, but this area is often small compared to runoff from outside the right of way. Evaluate each ditch for significant flows from off-site. The standard 0.15 m deep ditch should be used on all projects unless the calculated peak flows indicate insufficient capacity or instability. The use of a flat bottom ditch may be appropriate in locations to satisfy water quality treatment requirements. Flat bottom ditches are typically 1.0 m wide at the ditch bottom with standard surfacing slopes. Flat bottom ditches may also be appropriate in open freeway medians. Additional information on ditches is provided in Section 10.1.

- **Earthwork**

When the standard sections do not provide for stable slopes and roadbed, a special design is necessary. The design shall be based on soil tests and other factors and must have the approval of the Geotechnical Engineer.

Care in the design of individual cuts and fills must be used when varying the rate of the slope due to height variations in order to avoid unsightly, irregular faces.

Table 5-8 below provides guidance for widening of fill sections where there is a concern for the stability of slopes.

Table 5-8
Embankment Widening on High Fills

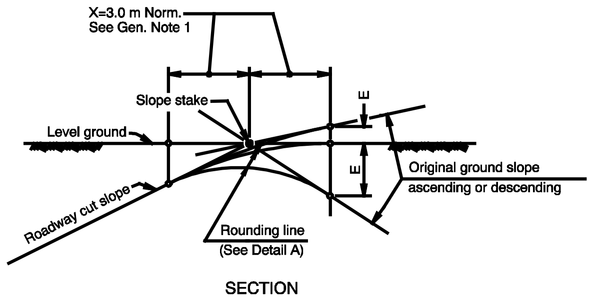
Fill Height (Meters)	Widening of Subgrade as Appropriate, Each Side of Centerline (Meters)
0-6	No Widening
6-9	0.3
9-12	0.6
12-15	0.9
Over 15	1.2

Fill height is to be considered as the difference in elevation between the subgrade shoulder and the adjacent toe of slope.

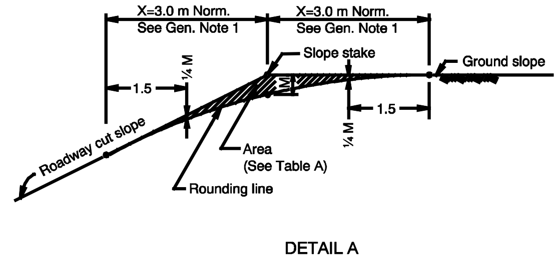
- **Rounding Cutbanks**

Cut slopes shall be designed to blend in with the surrounding terrain. This is accomplished by rounding the top of the cutbanks as shown on Figure 5-8, also as specified in the *Standard Specifications for Highway Construction - 1996* (Section 00330). The rounding limits also have an impact on right of way requirements.

ROUNDING OF CUTBANKS



SECTION



DETAIL A

GENERAL NOTES:

1. Extend slope rounding 3.0 m or to right of way line, whichever is less. Use the same X dimension on both sides of slope break point.

TABLE A

	E	CUT SLOPE									
		1 : 1		1 : 1.5		1 : 2		1 : 3		1 : 4	
	(m)	M (m)	AREA (m^2)	M (m)	AREA (m^2)	M (m)	AREA (m^2)	M (m)	AREA (m^2)	M (m)	AREA (m^2)
ASCENDING GROUND	1.8-2.1	0.69	2.14								
	1.5-1.8	0.76	2.32								
	1.2-1.5	0.84	2.60								
	0.9-1.2	0.91	2.79	0.53	1.67						
	0.6-0.9	0.99	3.07	0.61	1.86						
	0.3-0.6	1.07	3.25	0.69	2.14	0.50	1.49				
	0-0.3	1.14	3.53	0.76	2.32	0.57	1.77				
DESCENDING GROUND	0.0-0.3	1.22	3.72	0.84	2.60	0.65	1.95	0.46	1.39		
	0.3-0.6	1.30	3.99	0.91	2.79	0.73	2.23	0.53	1.67	0.44	1.30
	0.6-0.9	1.37	4.18	0.99	3.07	0.80	2.42	0.61	1.86	0.52	1.58
	0.9-1.2	1.45	4.46	1.07	3.25	0.88	2.69	0.69	2.14	0.59	1.77
	1.2-1.5	1.52	4.65	1.14	3.53	0.95	2.88	0.76	2.32	0.67	2.04
	1.5-1.8	1.60	4.92	1.22	3.72	1.03	3.16	0.84	2.60	0.74	2.23
	1.8-2.1	1.68	5.11	1.30	3.99	1.11	3.34	0.91	2.79	0.82	2.51
	2.1-2.4	1.75	5.39	1.37	4.18	1.18	3.6	0.99	3.07	0.90	2.69
	2.4-2.7	1.83	5.57	1.45	4.46	1.26	3.81	1.07	3.25	0.97	3.07
	2.7-3.0	1.91	5.85	1.52	4.65	1.34	4.09	1.14	3.53	1.05	3.16

Rounding not required within these limits.

o All dimensions are in meters unless otherwise noted.

NOTE: All material and workmanship shall be in accordance with the current Oregon Standard Specifications

OREGON STANDARD DRAWINGS

SLOPE ROUNDING

2002

REVISIONS	
DATE	DESCRIPTION

RD230

Figure 5-8
Rounding Of Cutbanks

5.5 MEDIAN DESIGN (Non-Freeway)

- **General**

Highway medians are important design elements that can significantly impact the safety, function, and/or efficiency of a highway. Highway medians provide separation of opposing traffic streams, separation of turning and through traffic, safety buffer and recovery area, positive longitudinal guidance, and positive control of turning movements. Some median designs improve pedestrian crossings by providing a refuge for pedestrians crossing, minimizing the exposure time to traffic and reducing the crossing distance. Other benefits may include enhanced aesthetics and reduced headlight glare. This section will discuss the design elements and standards for various median treatments on roadways **other than freeways**. Freeway median design is covered in Chapter 6.

Medians can be either traversable or non-traversable designs. Traversable medians are those which do not physically prevent vehicles from crossing or entering the median. These include Continuous Two Way Left Turn Lanes (CTWLTLs) and painted medians. A non-traversable median is designed to discourage or prevent vehicles from crossing the median except at designated locations. Examples of non-traversable medians include raised curb, concrete barrier, or depressed medians.

- **Continuous Two Way Left Turn Lanes**

Continuous Two Way Left Turn Lanes (CTWLTLs) are often used in urban areas to provide full movement access to adjacent properties and roadways while minimizing impacts of left turning vehicles on through traffic. CTWLTLs are a reasonable tool to improve system safety and efficiency for roadways with low to moderate traffic volumes and speeds. CTWLTLs should generally not be used on roadways with traffic volumes over 28,000 vehicles a day or speeds of 70 km/h or more. Under these types of conditions, the preferred median treatment is a non-traversable median that controls left turn movements.

Continuous left turn lanes should be considered only on roadways where:

1. Access to adjacent properties is desired and not otherwise precluded.
2. Left turning vehicles stopped in travel lanes may present an unexpected obstacle.
3. Left turning vehicles significantly reduce roadway capacity.
4. Property access points are clearly defined and the safety of pedestrian traffic is given the highest priority.
5. Passing opportunities on two-lane roadways are not appreciably reduced.

When the use of a continuous left turn lane is deemed appropriate, the following design features should be considered.

1. The volume of left turning vehicles should not exceed the available storage nor create a high conflict potential in the turn lane.

2. The continuous left turn lane should not extend through a railroad crossing or signalized intersection.
3. Horizontal and vertical alignment should be considered in the design of the continuous left turn lane to maximize sight distance.
4. The design of the continuous left turn lane and other median treatments should be consistent within a given highway section.
5. Care should be given to avoid overlapping left turns. This may require relocating or offsetting approach points.

CTWLTL's Design Standards

1. The width of a CTWLTL shall be 4.2 m where the design speed is 70 km/h or less. For design speeds greater than 70 km/h, the width shall be 4.8 m. In an urbanizing area where the posted speed is 45 mph, the use of a 4.2 m CTWLTL may be an appropriate treatment due to the urbanizing nature of the roadway and may provide a transition from a higher speed section of roadway to a slower speed section.
2. The striping of CTWLTLs shall be in conformance with ODOT's Striping Manual.
3. Where CTWLTLs are widened at intersections to provide for double left turn lanes, the width should be 7.8 m when the design speed is 70 km/h or less and 8.4 m when the design speed is greater than 70 km/h. Standard drawings RD 215 and Figure 9-15 provide more detail on CTWLTLs.

- **Painted Medians**

Painted medians are generally narrower than CTWLTLs. This type of median is typically 1.2 m to 3.0 m in width and utilizes double solid yellow lines to define the median area. Painted medians are intended to prohibit vehicles crossing the median or using it as a CTWLTL. This type of median control may be used on moderate volume and speed highways in rural areas. In these situations, the painted median is often used as a precursor to installing a non-traversable median such as a concrete barrier. In urban areas however, this median treatment should be used carefully. This treatment should be limited to urban areas where no adjacent property approach exists and intersection spacing is very long, 800 m or longer. Generally these conditions will only be present on limited access highways. The major concern is that the painted median will be used as a CTWLTL and may increase accident experience due to the narrow width.

- **Non-Traversable Medians**

Raised Medians

Raised medians are the preferred type of median treatment for most Statewide NHS and some Regional highways (See *Oregon Highway Plan,* Appendix D for Highway Classification information). Raised medians should also be considered on other highway classifications where the safety and operational benefits are significant and where improved pedestrian crossing opportunities are desired. Refer to the Median Policy from the Oregon Highway Plan for more information on raised median locations. Raised medians can be designed with either curbs or concrete barriers. Curbed raised median designs are the preferred treatment in urban areas as they are often more aesthetic than the concrete barrier and provide pedestrian crossing opportunities. However, the concrete barrier may be a more appropriate treatment in rural areas with high speeds or where right of way is constrained. Most of the design elements of this chapter apply to either type of median design. The remainder of this section will describe design standards and guidelines for both types of raised medians. In addition, raised curbed medians are described as two sub-sets. Full width medians refer to the curb to curb dimensions of the median between intersections or over long distances. A median traffic separator is that portion of the median that defines left turn channelization areas.

Raised Median Design Standards

1. Median Width **(note: median widths include the raised portions only and do not include shy distance or left side shoulder)**

   The width of raised medians is variable between intersections. Factors such as pedestrian accommodation, landscaping, and right of way control median widths.

   (a) The minimum median traffic separator width at intersections is 1.2 m when pedestrians are not to be accommodated in the median and the design speed is greater than 70 km/h. For design speeds 70 km/h and below , the median traffic separator can be reduced to 0.6 m in constrained locations. However, because of the improved visibility, a median traffic separator width of 1.2 m is preferred even when the design speed is less than 70 km/h.

   (b) Medians and median traffic separators must be designed to accommodate pedestrians mid-way across an intersection when crossing more than 6 lanes or 6 lanes and a 20-degree skew angle or more. The number of lanes includes turn and through lanes. Changes in the median traffic separator will impact the overall median width.

   (c) When pedestrians are to be accommodated mid-way, the median or median traffic separator width shall be as follows:

Design Hour Ped. Volume	Width
= 100	1.8 m
= 101	2.4 m

   (d) Where left turns are not accommodated over a significant length, 800 m or longer, the minimum raised curb median width should be no narrower than 1.8 m. Where left turn accommodation is provided at intersections the minimum median width shall be that necessary to provide a 1.2 m median traffic separator, a 3.6 m left turn lane and the appropriate shy distance for opposing traffic. (See Table 5-9 for shy distance requirements.) The intent is to minimize the hour glass effect of widening the median at intersections and narrowing between.

(e) Where intersection spacing is relatively short, left turn bays often become back to back in nature. It is desirable to have some full width median between the left turn bays. The full width median allows for better visibility of the driver and also allows a place to install signing. Figure 5-9 shows an example of a full width median. The minimum full width median section should be as follows:

Design Speed	Length of Full Width
= 50 km/h	20 m
60 km/h	30 m
70 km/h	40 m
= 80 km/h	50 m

(f) The minimum median width to accommodate landscaping is 1.8 m. Care should be taken to not use landscaping that impairs sight distance. There should also be a planting setback. The use of trees in a raised median are typically not recommended and should only be considered in urban situations where the design speeds are low and where the vegetation can carefully be chosen. If used, the trees should be of a type that remains small in trunk diameter, 100 mm or smaller

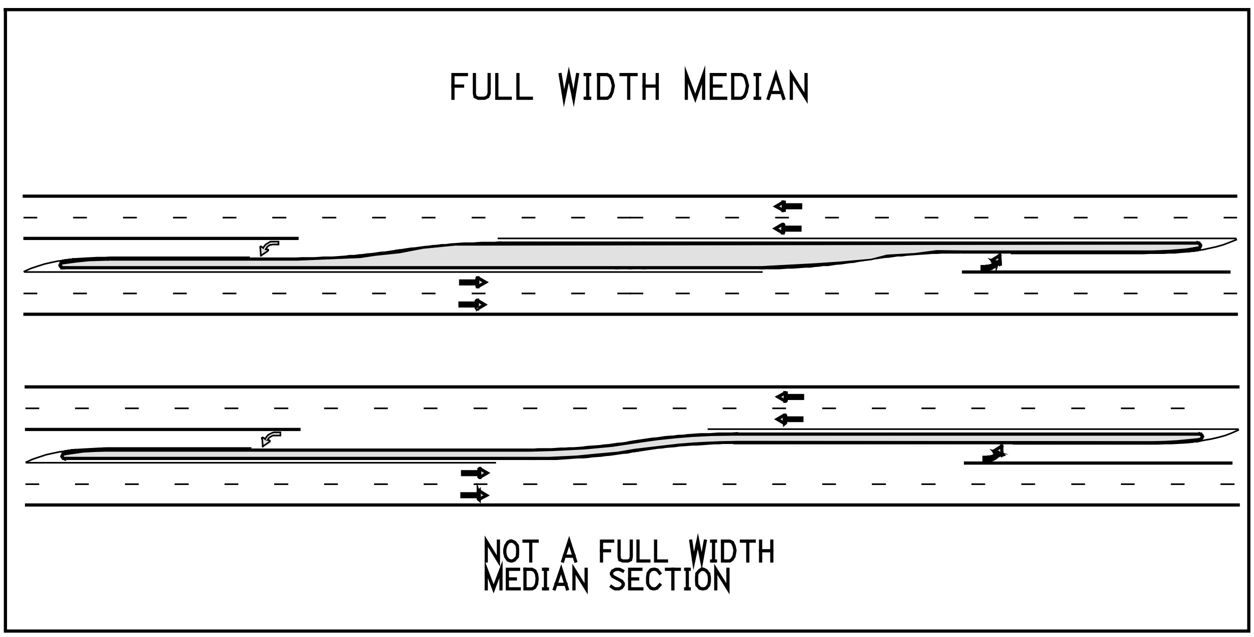


Figure 5-9
Full Width Median

2. Shy Distance From Raised Medians

Whenever barriers, such as curbs, are introduced into the roadscape it is desirable to provide a buffer space. This buffer helps improve safety of the users, traffic flow, and operational efficiency. This buffer is often referred to as Shy Distance. Table 5-9 establishes the shy distance requirements from raised medians. This table is not to be used for determining the shy distance for expressways (see Tables 7-1 and Table 8-1).The table also applies to left side shy distance for other conditions such as curbed sections on one-way roadways.

Table 5-9
Left Side Shy Distance

Design Speed (km/h)	Shy Distance (m)
40	0.3
50	0.3
60	0.6
70	0.6
80	0.9
90+	1.2

When raised curb or concrete barrier medians are not continuous, an additional 0.3 m of shy distance should be added to the values above.

3. Sight Distance

Sight distance at both unsignalized and signalized intersections is critical to provide a safe and efficient median opening. It is desirable to provide intersection sight distance at all median openings. However, in many situations, this is not practical. The designer is encouraged to provide the highest level of sight distance practical. Sight distance is covered in more detail in Section 5.2.

4. Landscaping Accommodation

Landscaping is an important feature to raised curb medians. Landscaping enhances the visibility of the median as well as the aesthetics. Two major concerns with landscaping are sight distance and maintenance. Sight distance concerns are crucial at both signalized and unsignalized intersections. The maintenance concerns include the amount of maintenance and cost. However, not all landscape techniques are labor intensive. Many types of vegetation are considered native and require almost no special care. In addition, landscaping features such as paving blocks, bricks, rocks, or other materials are relatively maintenance free. The following are important design elements to consider when landscaping medians:

(a) It is desirable to provide a vertical element within the median to increase visibility. Vegetation or mounding of earth, blocks, or bricks should extend 200 mm above the top of curb height. However, to ensure sight distance lines are preserved, vegetation or mounding should not extend higher than 600 mm above the pavement surface within the functional area of intersections. Sight distance must also be preserved where pedestrian crossings are provided mid-block.

(b) Sideslopes within the median for mounding shall be no steeper than 1:3 and preferably flatter.

(c) Trees shall be trimmed to provide a clear height of 3.0 m above the pavement surface.

(d) A planting set back of 0.3 m to 0.6 m should be considered where median width allows. The planter strip should be structural to support maintenance equipment. This could minimize the maintenance requirements or ease maintenance operations, such as mowing.

(e) Consider using planter boxes rather than continuous vegetation to reduce maintenance. Planter boxes are also effective treatments for improving median visibility. Planter boxes may either be flush or raised. Raised planter boxes should be 150 mm or less above the curb height.

(f) ODOT's Landscape Architecture Group should approve all vegetation plans.(See Figure 5-10 for Landscaping Accommodation)

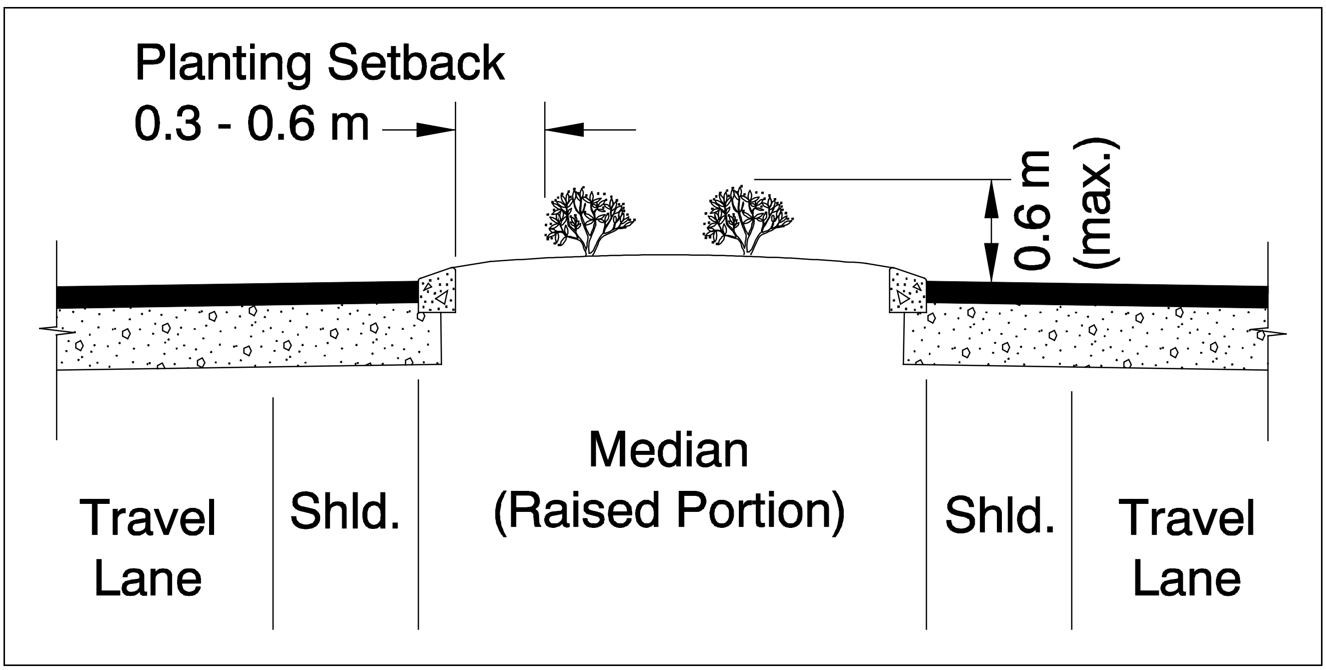


Figure 5-10
Landscaping Accommodation

5. End Treatments

Starting and ending raised median treatments can create conflict areas to roadway users and must be designed carefully. Raised median sections should be designed with logical starting and ending points within a given section of highway. Haphazardly placing small sections of raised median throughout a highway segment may offset any safety benefits and may actually increase the accident frequency over that anticipated without any median treatment. In urban situations it is preferred to have the median begin and end at an intersection. Rural areas may not allow this intersection approach. In these cases, the designer is to determine logical termini based upon the intended function of the median and roadside character of the highway. It is important to remember that raised medians are a barrier and can be a roadway hazard. End treatments are critical to ensuring the appropriate and safe function of the raised median.

Concrete barriers generally require an impact attenuator to protect the ends. The type of attenuator used must conform to the ODOT approved materials list. The most current version of AASHTO's *"Roadside Design Guide"* can provide additional information regarding end treatment design for concrete barriers.

Raised curbed medians generally do not require any special end treatments. However, in high speed situations, design speeds over 70 km/h, and where pedestrian accommodation in the median is not required, the curb line should be tapered to 50 mm. This tapered section should be accomplished over 4 m. Standard Drawing RD705 provides additional detail for this tapered treatment.

Two other concerns about end treatments are pedestrian refuges and truck off-tracking. The preferred design, when providing a pedestrian refuge for crossings at intersections, is to utilize the cut-through option. This treatment requires a protective nose area that should be at least 1.2 square meters or more. The nose can be designed with either a semi-circle or half bullet type design. The semi-circle design type is only recommended for median traffic separator widths of 1.2 m or less. Wider medians should utilize the half bullet type design to better facilitate truck turning movements. All end treatment designs need to consider the off-tracking characteristics of the appropriate design vehicle. The designer must use caution when providing a pedestrian refuge and using the half bullet type nose design. The half bullet design may reduce the available refuge for pedestrians. In some situations, the crossing may need to be moved back slightly to provide a full width refuge. This is especially prevalent where the nose must be moved back to provide for adequate truck turning movements. Figure 5-11 provides additional detail regarding end treatments for raised curb medians.

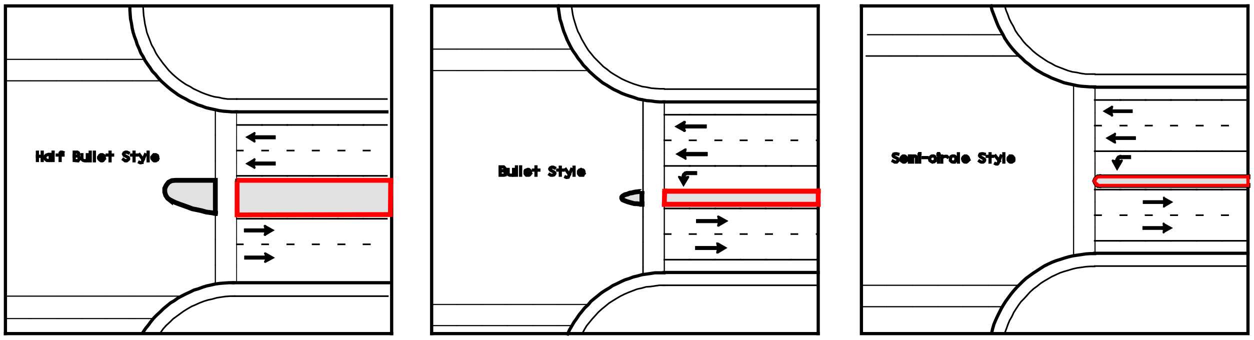


Figure 5-11
End Treatments

6. Accommodating U-Turns

The use of a raised median significantly reduces the opportunities for vehicles to make left turns. To facilitate traffic's ability to reach destinations on the left side of the highway, U-turn opportunities need to be included with the design. The preferred approach is to provide U-turn capabilities at signalized median openings. This approach offers greater protection for the U-turning vehicles. The second option is to utilize an unsignalized median opening. This approach should be used in conjunction with a jug handle design. Executing a U-turn through the oncoming traffic lanes creates a greater exposure to the U-turning vehicle and through traffic and should be avoided in high volume or high-speed conditions. When accommodating U-turning vehicles, the designer needs to consider the following:

(a) Speed of the highway.
(b) Volume of traffic opposing and executing the U-turn.
(c) The design vehicle to be accommodated.
(d) The adjacent roadside culture, and
(e) The opportunity to use existing roadways to accommodate U-turn movements.

A left turn lane shall always be included when accommodating U-turning vehicles. U-turn movements are never to be allowed out of a through travel lane. Section 5.11 provides additional information and illustrations for accommodating U-turns.

The Traffic Management Section should be consulted when considering accommodating U-turns on state highways. U-turns must be located with respect to legal requirements [ORS 810.130(3), ORS 811.365, OAR 734-02-0025]. In addition, the State Traffic Engineer must approve all U-turns at signalized intersections.

7. Type of Curb

When using raised curb medians, the designer needs to determine the appropriate curb type. The preferred curb type is the mountable curb. Mountable curb is a design that provides some protection for pedestrians, landscaping, or other objects in the median, while also enhancing the aesthetics of the median. Low profile mountable curb shall only be used where the median is 4.8 m or larger measured curb to curb. The use of low profile mountable curb also requires substantial mounding for visibility and safety. Standard curb can be substituted for mountable curb when desired by the project team when design speeds are less than or equal to 70 km/h. The use of standard curb may also be appropriate for urban or urbanizing areas where the posted speed is 45 mph.

5.6 TRAFFIC CONTROL

- **Traffic Operation During Construction**

Maintenance of traffic during construction must be included in the planning of all highway improvement projects. A traffic control plan (TCP) shall be developed and submitted with all projects to assure the safety of the traveling public, cyclists, pedestrians, and protection of workers during all stages of construction. The plan should provide a minimum width of 8.4 m for two-way traffic and 5.0 m for one-way traffic, exclusive of traffic control devices. Acceptable widths less than these must be approved by the Traffic Control Plan Engineer. Special consideration for wider clearances should be given when alternate routes for overwidth vehicles are not available or convenient.

The TCP should include the appropriate standard traffic control drawings and for more complex projects, stage construction plans. The standard traffic control drawings may be modified to meet specific project requirements. Special provisions and an itemized cost estimate shall be provided.

The TCP must consider existing traffic speed, access to connecting roads and streets, roadway width, roadway alignment (both horizontal and vertical), traffic characteristics (i.e., percent trucks, percent RV's, farm use, wide loads), annual peak traffic volume vs. roadway capacity during construction, overwidth and overheight vehicles, temporary striping and stripe removal, scope of the work, and duration of each construction operation. Temporary concrete barrier or temporary guard rail shall be used when conditions meet barrier warrants in the *"Roadside Design Guide"*. Appropriate end treatments and anchors shall be specified. Necessary temporary signing and other traffic control devices must be included.

- TCP Location Narrative Report information

1) Existing traffic signal locations.
2) Locations of proposed or mandatory material sources or disposal sites.
3) Problems related to proximity of business.

4) Need for parking prohibitions (also obtain and submit written approval of prohibitions from local jurisdictions).
5) List local special public events, with dates and projected traffic volumes. Note required traffic control measures (i.e., No Lane Closures, No Work, etc.).
6) Any other special problems or considerations that affect traffic flow and Temporary Traffic Control.

• Location Notes and Plan Sheets

1) Copy of the project sign log including locations and legends of all existing traffic control signs.
2) Existing typical sections showing locations of lane striping, guard rail and concrete barrier, especially in critical or problem areas.
3) Electrical power sources where required by TCP.
4) Locations of existing no-passing zones and length of existing solid-line no-passing zone striping.
5) Photographs and photo log of project. Show mainline roadway and major road approaches. Also include high traffic volume business approaches.
6) Plan and profile showing existing horizontal and vertical alignments.
7) All intersecting roads and streets. Show intersections with mainline for 0.8 km preceding and 0.8 km following the work area. Identify and label collector and arterial street connections.

• Detour plan (when appropriate)

1) Proposed detour route (also obtain and submit written approval of detour route from local jurisdiction).
2) Detour route signing.
3) Oversize load signing if appropriate.

- **Median Crossovers for Temporary Detours**

A minimum radius of 870 m should be used for the reversing curves whenever possible. In addition, the width of shoulders on the detour cross-overs shall equal those on the approaching roadway (usually 1.8 m left and 3.0 m right). Past experience using sharper curves, has shown that traffic delay and interruptions occur in sharper transition sections.

Traffic Section must be consulted to determine the number of travel lanes that must be maintained during construction. This may generate a need for additional temporary surfacing wherein the travel lane and shoulder width requirements need to be analyzed on a project by project basis.

- **Stream Crossing Detours**

The crossing of streams as part of a detour has an impact to water shed and salmon issues. If a steam crossing detour is required, it should be done via clear spanning or using a pile support structure first. The use of granular fill material placed in the stream should only be used if location or extreme cost prohibit the clear spanning of the stream. If the use of fill material around culverts is allowed and approved by the appropriate agencies, a clean granular material shall be used. If stream crossing detours are part of a project the appropriate state and regulatory agencies and the Environmental Section project representative should be contacted for proper course of action.

- **Detour Design Speeds**

Design of detours must take into account the expectations of the driver in addition to the many other physical and geometric limitations or design considerations. Care should be taken so that a motorist is not surprised by a detour that requires unreasonable reductions in speed from the normal operating speed of the roadway. This may not be an issue where the adjacent roadway sections control traffic speeds entering the detour. Where traffic speeds are particularly high, especially on straight sections of highway, detour designs with 30 or 35 mph posted curves should be avoided. Minimum curve design should be for 70 km/h design speeds to permit a reasonable reduction of speed from 90 to 100 km/h to the detour speed of 70 km/h.

For detour design speed, the pre-construction posted speed should not be reduced by more than the following:

Freeways:	15 km/h maximum reduction
4-Lane Highway:	Rural- 15 km/h maximum reduction
	Urban- 20 km/h maximum reduction
2-Lane Highway:	Rural- 20 km/h maximum reduction
	Urban- 30 km/h maximum reduction

There may be situations where studies indicating prevailing speed different than posted speed should be considered in determining the proper speed reduction for detour speeds. When other geometric or operational factors affect possible detour speeds, the above reductions may be increased upon approval of the Roadway Engineering Manager. Where adjacent sections keep prevailing speeds low, detour speed reductions need not be as large as shown above. Detour speeds should never be less than 30 km/h.

5.7 CLEARANCES

5.7.1 VERTICAL CLEARANCE - HIGHWAYS

All new construction and reconstruction structures on State Highways shall be designed and constructed with a vertical clearance of 5.2 m from the top of the pavement to the bottom of beam as shown in Figure 5-12. The minimum vertical clearance includes the entire roadway width, including the shoulder area. The lateral clearances shown are to the face of rail and assume the barrier is warranted. In dealing with vertical clearances the Motor Carrier Transportation Division (MCTD) should either be notified or consulted with depending on the type of project as noted below. For new construction, where 5.2 m of clearance is required, the designer should notify the MCTD.

On projects utilizing ODOT 3-R standards (Resurfacing, Restoration, and Rehabilitation), the clear height of structures shall not be less than 4.9 m over the entire roadway width for both rural and urban sections. In urban areas, the 4.9 m clearance shall apply to a single routing. If there is additional routing, other routes shall have a vertical clearance of not less than 4.3 m. The designer should allow for future resurfacing in determining the minimum vertical clearance. The vertical clearance to sign trusses and pedestrians overpasses shall be 5.2 m. The vertical clearance from the deck to the cross bracing on the through truss structures shall also be a minimum for 5.2 m. For 3R or preservation projects the MCTD should be contacted as follows.

- Maintain existing vertical clearance over roadways wherever possible
- 4.9 minimum clearance required
  - If maintaining existing vertical clearance (between 4.9 and 5.2 m) notify the MCTD
  - If proposing to decrease vertical clearance but end result is still greater than 5.2 m-notify MCTD
  - If proposing to decrease vertical clearance below 5.2 m- consult the MCTD

In addition to ODOT vertical clearance standards, the FHWA has agreed that all exceptions to the 4.9 m (AASHTO) vertical clearance standard for the rural Interstate and the single routing in urban areas will be coordinated with the Military Traffic Management Command Transportation Engineering Agency (MTMCTEA) of the Department of Defense. Regardless of funding, this agreement applies whether it is a new construction project, a project that does not provide for correction of an existing substandard condition, or a project which creates a substandard condition at an existing structure. This standard applies to the full roadway width including shoulders for the through lanes, and to ramps and collector-distributor roadways in Interstate-to-Interstate interchanges.

Clearance requirements for transmission and communication lines vary considerably and must comply with the National Electrical Safety Code. Clearance information should be obtained from the Railroad/Utilities Engineer.

5.7.2 VERTICAL CLEARANCE - RAILROADS

The minimum railroad clearance to be provided on crossings shall conform to Oregon Administrative Rule (OAR) 741 and as shown in Figure 5-13. Additional clearance may be required and should be determined individually for each crossing. Information regarding additional clearance shall be obtained from the Railroad/Utility Engineer.

5.7.3 CLEAR ZONE

The *2002 AASHTO "Roadside Design Guide"* is the most recent publication written to provide guidance in roadway design regarding roadside clearances. The 2002 *"Roadside Design Guide"* gives procedures and tables to determine the correct clear zone distance for use in the placement of barrier, sign installation, guard rails, ditch location, and other roadside appurtenances. It provides the criteria for the placement or removal of any object which may influence the trajectory of a vehicle which has left the travel lanes, either in a controlled or uncontrolled situation.

The clear zone is determined by several factors, including design speed, ADT, horizontal curvature, and embankment slope. Using tables given on the following pages and these four criteria for determination of clear zone distance, the distance required for vehicular recovery can be found. These distances are not absolute and the design options selected to mitigate the effect of roadside hazards require good engineering judgment in order to balance cost effectiveness with the expected increase in safety.

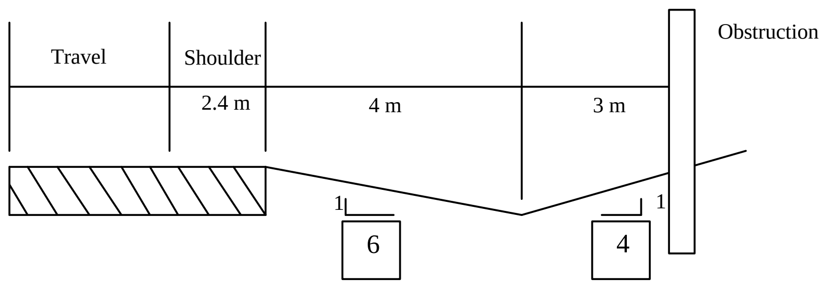
Keeping that in mind, the *2002 AASHTO "Roadside Design Guide"* suggests the following options to be considered when evaluating a roadside hazard:

- Removing or redesigning the obstacle
- Relocating the obstacle
- Reduce impact severity by breakaway devices
- Redirection of vehicle by installation of barrier device
- Delineation of object

The clear zone distance can be determined by using Figures 5-14 and 5-15 shown at the end of this section. These figures were taken from the *2002 AASHTO "Roadside Design Guide"*. They are provided as a quick reference source for the experienced designer who is already familiar with the determination process. Figure 5-14 is used to determine general clear zone distance. Figure 5-15 is used for horizontal curve adjustments.

Care must be taken in arriving at the proper clear zone requirements. Figure 5-14 lists the different clear zone requirements for cut and fill slopes. Many times multiple slopes have to be used to determine the appropriate clear zone distance. At times the roadway typical section will have both a front slope and back slope. When this occurs the procedure for determining the proper clear zone requires more than pulling a number from Figure 5-14. Following is an example of the proper procedure for determining clear zone requirements for a typical section that includes both a front slope and a back slope.

Example:
Design ADT: 7000
Design Speed: 100 km/h
Recommended clear zone for 1:6 slope (fill): 9.0 to 10.0 m from Figure 5-14
Recommended clear zone for 1:4 slope (cut): 7.5 to 8.0 m form Figure 5-14



Discussion: Since the example is within the preferred channel cross section, the table on Figure 5-14 can be used to determine the clear zone. When multiple slopes are used the clear zone distance can be determined as follows:

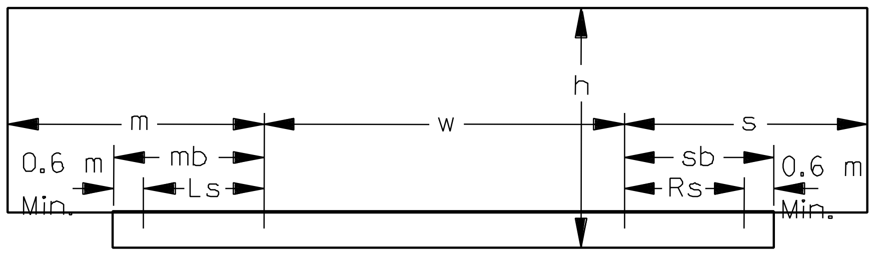
Calculate the percentage of clear zone range for the fill slope: 6.4 m /9 m = 71% and 6.4 m/10 m= 64%
Subtract the percentage of fill slope from 100% and multiple the difference by the recommended clear zones for the backslope: (100% - 71%) = 29% x 7.5 = 2.2 m and (100% - 64%) = 36% x 8 = 2.9 m. The required clear zone on the backslope ranges from 2.2 m to 2.9 m.
Add the two recommended clear zone distances to the available clear zone distance on the foreslope to determine the adjusted clear zone: 6.4 m + 2.2 m = 8.6 m and 6.4 m + 2.9 m = 9.3 m.

Since the obstacle is located outside of the recommended clear zone distances of 8.6 to 9.3 m, removal of the obstacle is not required.

For further information and more detailed procedures it is recommended all designers read the *ODOT Highway Design Manual* Section 3.3 relating to roadside inventory procedures, and the *2002 AASHTO "Roadside Design Guide"*.

FREEWAY AND HIGHWAY CLEARANCES

(to be adjusted to provide minimum stopping sight distance when necessary)



h Minimum Vertical Clearance includes shoulder area
m Distance from Edge of Traveled Way to Obstacle on Left
mb* Distance from Edge of Traveled Way to Barrier on Left
Ls Left Shoulder
w Width of Traveled Way
s Distance from Edge of Traveled Way to Obstacle on Right
sb Distance from Edge of Traveled Way to Barrier on Right
Rs Right Shoulder

NOTE: On two-lane, two-way highways, s and sb apply to shoulders on both sides of highway.
* When barrier is warranted.

	FREEWAY	FREEWAY RAMPS		ALL OTHER HIGHWAYS
		ONE LANE	TWO LANES	
h	5.2 m	5.2 m	5.2 m	5.2 m ③
w	3.6 m Lanes	4.8 m	7.2 m	3.6 m Lanes
m	See The 1996 AASHTO Roadside Design Guide			
s				
Ls	Full Shoulder	1.2 m	1.8 m	Full Shoulder
Rs	Full Shoulder ①	1.8 m ②	3.0 m ①	Full Shoulder

① Where curb is introduced intermittently for drainage on interstate it shall set back 0.6 m from edge of shoulder at guardrail locations.

② 3.0 m on freeway to freeway ramps.

③ 4.9 m may be used under special cases with approval of the Roadway Engineering Manager.

State of Oregon
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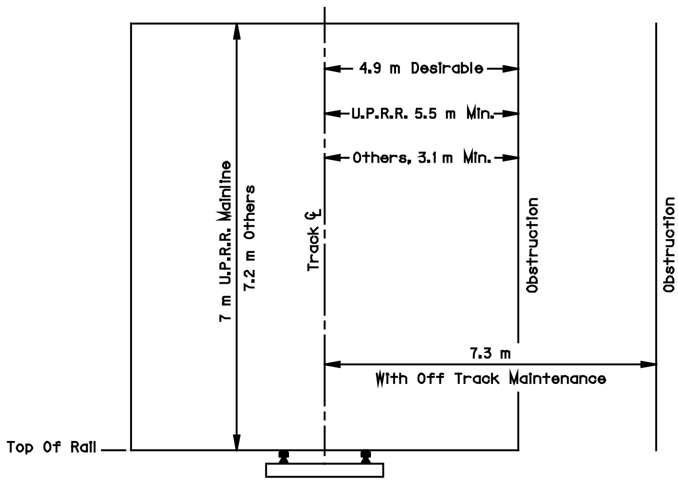
HIGHWAY DESIGN MANUAL

ROADWAY CLEARANCES

Figure: 5-12 2002

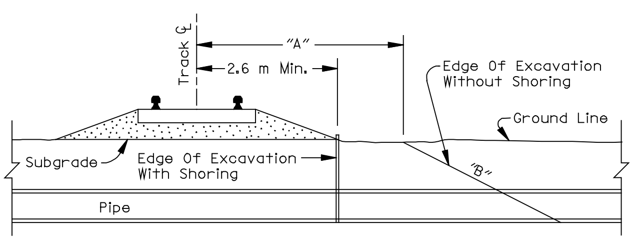
Figure 5-12
Freeway & Highway Clearances

RAILROAD CLEARANCES



On curved tracks, clearance is determined by adding to the proper tangent clearance, 15 m for each degree of curve plus the tilt of the railroad car itself.

EXCAVATION CLEARANCES AT RAILROAD



RAILROAD	A (m)	Slope "B"
Union Pacific	3.1	1:1.5
Burlington Northern & Santa Fe	3.1	1:1.5
All Others	3.1	1:2

ON ALL PROJECTS:

Railroad clearances shall be checked by Railroad/Utilities Engineer.
Structure clearances shall be checked by Bridge Section.
Clearance for transmission & communication lines shall be obtained from Railroad/Utilities Engineer.

State of Oregon
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HIGHWAY DESIGN MANUAL
RAILROAD CLEARANCES

Figure: 5-13 2002

Figure 5-13
Railroad Clearances

CLEAR ZONE DISTANCES

DESIGN SPEED	DESIGN ADT	FILL SLOPES			CUT SLOPES		
		1V:6h or FLATTER	1V:5H To 1V:4H	1V:3H	1V:3H	1V:5H To 1V:4H	1V:6h or FLATTER
60 km/h or LESS	UNDER 750	2.0 - 3.0	2.0 - 3.0	* *	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0
	750-1500	3.0 - 3.5	3.5 - 4.5	* *	3.0 - 3.5	3.0 - 3.5	3.0 - 3.5
	1500-6000	3.5 - 4.5	4.5 - 5.0	* *	3.5 - 4.5	3.5 - 4.5	3.5 - 4.5
	OVER 6000	4.5 - 5.0	5.0 - 5.5	* *	4.5 - 5.0	4.5 - 5.0	4.5 - 5.0
70 - 80 km/h	UNDER 750	3.0 - 3.5	3.5 - 4.5	* *	2.5 - 3.0	2.5 - 3.0	3.0 - 3.5
	750-1500	4.5 - 5.0	5.0 - 6.0	* *	3.0 - 3.5	3.5 - 4.5	4.5 - 5.0
	1500-6000	5.0 - 5.5	6.0 - 8.0	* *	3.5 - 4.5	4.5 - 5.0	5.0 - 5.5
	OVER 6000	6.0 - 6.5	7.5 - 8.5	* *	4.5 - 5.0	5.5 - 6.0	6.0 - 6.5
90 km/h	UNDER 750	3.5 - 4.5	4.5 - 5.5	* *	2.5 - 3.0	3.0 - 3.5	3.5 - 4.5
	750-1500	5.0 - 5.5	6.0 - 7.5	* *	3.0 - 3.5	4.5 - 5.0	5.0 - 5.5
	1500-6000	6.0 - 6.5	7.5 - 9.0	* *	4.5 - 5.0	5.0 - 5.5	6.0 - 6.5
	OVER 6000	6.5 - 7.5	8.0 - 10.0*	* *	5.0 - 5.5	6.0 - 6.5	6.5 - 7.5
100 km/h	UNDER 750	5.0 - 5.5	6.0 - 7.5	* *	3.0 - 3.5	3.0 - 4.5	4.5 - 5.0
	750-1500	6.0 - 7.5	8.0 - 10.0*	* *	3.5 - 4.5	5.0 - 5.5	6.0 - 6.5
	1500-6000	8.0 - 9.0	10.0 - 12.0*	* *	4.5 - 5.5	5.5 - 6.5	7.5 - 8.0
	OVER 6000	9.0 - 10.0*	11.0 - 13.5*	* *	6.0 - 6.5	7.5 - 8.0	8.0 - 8.5
110 km/h	UNDER 750	5.5 - 6.0	6.0 - 8.0	* *	3.0 - 3.5	4.5 - 5.0	4.5 - 5.0
	750-1500	7.5 - 8.0	8.5 - 11.0*	* *	3.5 - 5.0	5.5 - 6.0	6.0 - 6.5
	1500-6000	8.5 - 10.0*	10.5 - 13.0*	* *	5.0 - 6.0	6.5 - 7.5	8.0 - 8.5
	OVER 6000	9.0 - 10.5*	11.5 - 14.0*	* *	6.5 - 7.5	8.0 - 9.0	8.5 - 9.0

Ref. AASHTO Roadside Guide, Table 3.1

* Where A Site Specific Investigation Indicates A High Probability Of Continuing Accidents, Or Such Occurances Are Indicated By Accident History,The Designer May Provide Clear Zone Distance Greater Than 9 Meters As Indicated.Clear Zones May Be Limited To 9 Meters For Practicality And To Provide A Consistent Roadway Template If Previous Experience With Similar Projects Or Design Indicates Satisfactory Performance.

* * Since Recovery Is Less Likely On The Unshielded, Traversable 1v:3H Slopes, Fixed Objects Should Not Be Present In The Vicinity Of The Toe Of These Slopes. Recovery Of High-speed Vehicles That Encroach Beyond The Edge Of The Shoulder May Be Expected To Occur Beyond The Toe Of Slope. Determination Of The Width Of The Recovery Area At The Toe Of Slope Should Take Into Consideration Right-Of-Way Availablity, Environmental Concerns, Economic Factors, Safety Needs, And Crash Histories. Also, The Distance Between The Edge Of The Through Traveled Lane And The Begining Of The 1v:3H Slope Should Influence The Recovery Area Privided At The Toe Of Slope. While The Application May Be Limited By Several Factors, The Foreslope Parameters Which May Enter Into Determining A Maximum Desirable Recovery Area Are Illustrated In Figure 3.2.

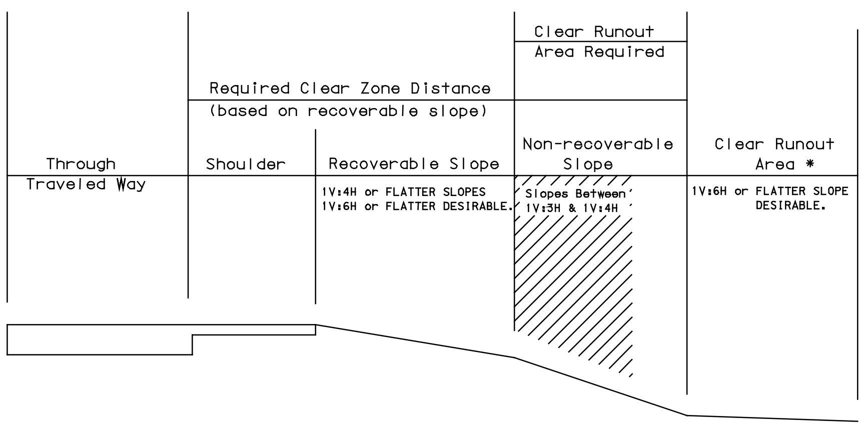
Figure 5-14
Clear Zone Distances

HORIZONTAL CURVE ADJUSTMENTS
Kcz (Curve Correction Factor)

RADIUS (m)	DESIGN SPEED (km/h)					
	60	70	80	90	100	110
900	1.1	1.1	1.1	1.2	1.2	1.2
700	1.1	1.1	1.2	1.2	1.2	1.3
600	1.1	1.2	1.2	1.2	1.3	1.4
500	1.1	1.2	1.2	1.3	1.3	1.4
450	1.2	1.2	1.3	1.3	1.4	1.5
400	1.2	1.2	1.3	1.3	1.4	
350	1.2	1.2	1.3	1.4	1.5	
300	1.2	1.3	1.4	1.5	1.5	
250	1.3	1.3	1.4	1.5		
200	1.3	1.4	1.5			
150	1.4	1.5				
100	1.5					

CZc = Lc x Kcz
where CZc = clear zone on outside of curvature, meter
Lc = clear zone distance, meter. AASHTO Roadside Design Guide, Figure 3.1b or Table 3.1

Kcz = Curve correction factor
Note: Clear zone correction factor is applied to outside of curves only. Curves flatter than 900 m do not require an adjusted clear zone.



Ref. AASHTO Roadside Guide, Figure 3.2 or Table 3.1

* The Clear Runout Area Is Additional Clear-zone Space That Is Needed Because A Portion Of The Required Clear Zone (shaded area) Falls On A Non-recoverable Slope. The Width Of The Clear Runout Area Is Equal To That Portion Of The Clear Zone Distance That Is Located On The Non-recoverable Slope.

Figure 5-15
Horizontal Curve Adjustments

5.8 GUARDRAIL AND CONCRETE BARRIER

- **General**

This section provides information to the designer concerning guardrail and concrete barrier. Information on offsets, single slope barrier, cast in place, and slip form barrier is provided. The *2002 AASHTO "Roadside Design Guide"*, shall be used to determine guardrail and concrete barrier locations. Exceptions to this guide are to be approved by the Roadway Engineering Manager. Standard Drawings RD400 - RD470 deal with guardrail while Standard Drawings RD500 - RD580 deal with concrete barrier. Barrier treatment in rural areas should consider impacts to animal crossings and the designer should contact the region environmental representative for assistance.

- **Concrete Barrier and Bridge Columns**

There are a couple of treatments for bridge column protection depending upon available shoulder width. When the design shoulder width is not encroached upon by placement of the concrete barrier, the concrete barrier should be placed as shown in Figure 5-17. For existing structures, the minimum clearance between the bridge column and the barrier is 3 inches. For new structures, the normal clearance between the bridge column and barrier is 2 feet. The designer should consult with the bridge designer to determine the appropriate clearances.

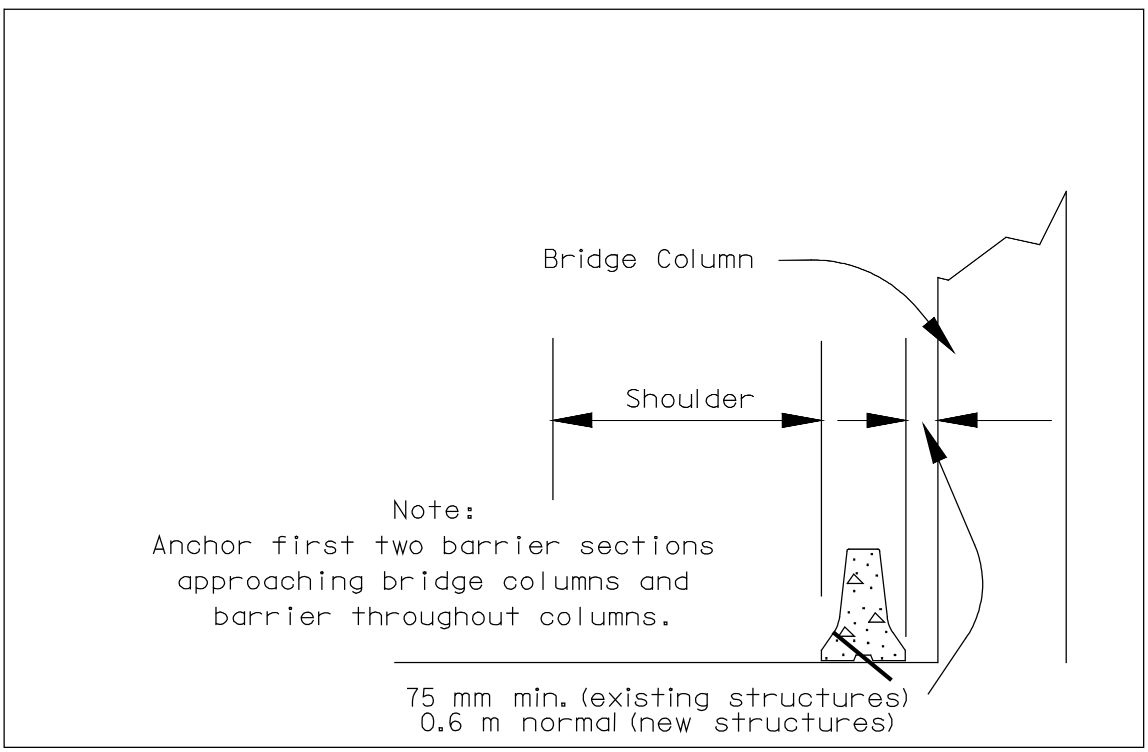


Figure 5-17
Concrete Barrier Placement at Bridge Column

When the design shoulder width is encroached upon by the placement of the concrete barrier, the designer should consult with Bridge Section personnel to develop the best solution to protect the bridge columns.

- **Tall "F" Shape Precast Concrete Barrier**

The single slope barrier designed in 1992 as an innovative alternate barrier system has been replaced with the tall "F" shape precast barrier. This 1065 mm high safety shape is available only as precast, with segments 3.81 m long, matching the length of ODOT's standard precast barrier. The tall barrier does not replace the standard, but it is to be used in the medians of interstates and on the State Highway Freight System where median barrier is justified or where existing barrier is to be replaced. Use the tall barrier on shoulders of any highway system as needed where adverse geometrics may occur such as curvature with radii smaller than specified in Tables 6-1 and 7-1 herein, or where severe consequences at specific locations might occur with penetration of a barrier by a heavy vehicle.

- **Overlays and Concrete Median Barrier Vertical Face**

For relatively straight forward overlay projects, the 75 mm vertical face on concrete median and shoulder barrier may be utilized without adjustment of the barrier. In no instance should the overlay exceed the vertical face height.

Tapering of an overlay so the vertical face height will not be exceeded must be investigated to insure that recommended slopes adjacent to the median barriers are not exceeded. Chapter 6 in the *"Roadside Design Guide"* provides additional information on terrain effect and barrier placement.

- **Barrier End Treatment**

Any barrier end exposed to the flow of traffic must be protected in some manner. Impact attenuators are recommended by AASHTO. Burying ends in embankment is another approved method. Sloped ends may be used, but only when the design speed is less than 70 km/h and the end is outside of the clear zone. In light of recent crash tests indicating potential launching hazards, earth mounds are not in favor with AASHTO.

- **Guardrail Upgrades**

On projects where any portion of an existing run of guardrail is being reconstructed to current safety standards, the entire run of guard rail shall be brought up to current safety standards. This includes transitions to bridge rail, longitudinal runs of guardrail, and guardrail end terminals. Exception to these requirements is required by the Roadway Engineering Manager.

- **Guardrail and Length of Need**

On any project where guardrail or barrier is being proposed, the length of need calculation is required. This will assure that the fixed objects within the clear zone are shielded as intended. Chapter 5 in AASHTO's *"Roadside Design Guide"* contains information and details on length of need calculations.

- **Guardrail Terminals**

Guardrail terminals are protective systems that prevent errant vehicles from impacting hazards, by either gradually decelerating the vehicle to a stop when the terminal is hit head-on, or by redirecting the vehicle away from the hazard when struck on the side. These systems are connected to the ends of runs of guardrail and work in concert with the guardrail run to shield rigid objects or hazardous conditions that cannot be removed, or relocated, or break away.

Some terminals utilize W-Beam rail and breakaway timber posts, which are set in two steel foundation tubes for ease of replacement. Some end terminals utilize hinged breakaway steel posts. The rest of the breakaway posts are drilled. All systems establish the third post from the end as length-of-need point, referred to in the AASHTO *"Roadside Design Guide"*.

Approved end terminals are listed in the Qualified Products List (QPL). Also available are terminals that are designed for a lower speed impact (under 70 km/h) that are called Test Level 2 terminals. They are shortened versions of the standard terminals. With the competition as it is, all products undergo routine adjustments to design that make it impractical to list current models. The designer should refer to the QPL, as the QPL stays abreast with all changes and regularly posts updates.

  - Design Criteria

  The current line of available terminals shows a common trait. They are all classified as gating terminals, meaning that, if hit other than head-on (angular impact) in the vicinity of the first two posts, the vehicle likely will gate (or break) through the device. The systems are more collapsible than the devices ODOT used prior to NCHRP 350 testing. One concern expressed was that with the new systems in place, the angle of impact is more critical now. The longtime favorite of ODOT designers, the 2.49 m flare has, by nature of its design, a fairly high angle of impact near the head. The 2.49 m flare was developed when it was considered better to impact the run of rail rather than impact the head, thus the high offset. The recommendation now is to reduce the flare so that the impact angle is reduced. With the "soft" heads it is preferred that the vehicle hit the terminal head-on or at a shallower angle, if impacted downstream of the head. An additional advantage of smaller offsets is the lesser impact to right-of-way and/or wetlands, so paved terminal flares can be made smaller, with less footprint intrusion.

  If a new guardrail terminal is under consideration, the first preference is to use the "non-flared" or parallel terminal. To get the edge of the extruder head completely off the normal edge of pavement, call for a 0.3 m offset. Avoid using a non-flared terminal where there is less than 1.8 m of shoulder available, where snow poles are commonly used, or where snow pack routinely obscures the guardrail section. Remember that non-flared terminals require more rail to satisfy length of need than a flared terminal. There also are terminals with offset available from 0.76 m to 1.22 m. It is the designer's choice of offset depending on local conditions. Lastly, if a terminal is being replaced and there exists a paved flare, it is acceptable to use the existing footprint of the paved flare, even if it is a 2.49 m flare, as long as Length of Need is adequate according to the latest publication of the Roadside Design Guide.

5.8 DRAINAGE

- **General**

Drainage facilities enable the carrying of water across the highway right of way and also provide a mechanism for removing storm water from the roadway itself. There are many type of drainage facilities including channels, bridges, culverts, curbs, gutters, and a variety of drains. Typically the roadway designer designs roadside ditches, cut-off ditches, inlet spacing and locations, drainage systems for storm sewers pipes, 600 mm or less, culverts 1200 mm or less, and outlet protection. The designer should work with the Hydraulics Unit in determining drainage needs for projects with systems larger than described above, or when flood plains, bridge hydraulics, scour or bank protection, fish passage, detention, water quality, or temporary erosion control are involved. More discussion is provided on hydraulic issues in Sections 10.1 and 10.2. The Hydraulics Manual should be referred to when performing hydraulic designs.

- **Wearing surface drains for open grade mix Asphalt Concrete (AC)**

Wearing surface drains for open grade mix AC were developed by Pavement Design in order to solve potential freezing problems occurring where "F" mix AC abuts a bridge or dense graded pavements. Wearing surface drains should be installed using the following general guidelines:

1. The drains should be considered for all projects that will have a open grade AC wearing surface
2. The drains should be placed at bridge ends where the profile grade is sloping towards the bridge (water flowing towards the bridge). The drains are not necessary if the open grade mix is being paved over on the bridge deck.
3. The drains should be placed at project ends where the profile grade is sloping out of the project, and the adjacent pavement is Portland Cement Concrete (PCC) or dense graded AC. The drains are not necessary if the adjacent pavement has an open-graded wearing surface.
4. Additional locations as suggested by Region or Pavement Design.

- **Longitudinal Slope**

Experience has shown that the recommended minimum values of roadway longitudinal slope will provide safe, acceptable pavement drainage. A minimum longitudinal gradient can be more important for a curbed pavement than for an uncurbed pavement since the water is constrained by the curb. However, flat gradients on uncurbed pavements can lead to drainage problems if vegetation is allowed to build up along the pavement edge. Desirable gutter grades should not be less than 0.5 percent for curbed pavements with an absolute minimum of 0.3 percent. The designer should contact the Hydraulics Unit for potential solutions to flat longitudinal grades. Superelevation and/or widening transitions can create a gutter profile different from centerline profile. The design should carefully examine the gutter profile to prevent the formation of ponds potentially created by superelevation and widening transitions.

- **Selection of inlets**

The performance of inlets and cross slope has an impact on hydraulic capacity. In a past study, the performance of the CG-3 inlet was compared to the standard grated inlets. The efforts of the study provided the following results. The CG-3 inlet outperformed the CG-1 and G-1 inlets when the gutter grade were less than 1%. The CG-3 inlet provided about the same performance as the CG-2 and G-2 inlets when the gutter grade was less than 0.8%. When the gutter grade exceeded 1%, bypass became a problem with CG-3 inlets and required close inlet spacing to control the bypass flow. In summary the study concluded that the CG-3 inlets are cost effective when the gutter grade is less than 1%.

- **Water Quality**

Many urban projects and some rural projects must address water quality issues. The ODOT Hydraulics Manual and the ODOT Geo/Hydro Section can provide more guidance in the design of water quality treatment opportunities. State Highways may also consider guidance contained in the Metro's *"Green Streets"* on a project by project basis in urban environments, where right of way is available and the design is cost effective.

5.10 MISCELLANEOUS

5.10.1 CHAIN LINK FENCE

The installation of chain link fence, located in clear zones, should be done without the use of the top rail. FHWA has reviewed the use of top rail installations and considers the use of top rail or pipe rail hazardous. They do not recommend using this type of support for chain link fences or pedestrian hand rails where they can be struck by an errant vehicle. In the event of a crash, the rails can penetrate the passenger compartment of vehicles. Chain link fences with top rails are particularly poor as vehicle impact on the fabric tends to pull the rail down onto the hood of the vehicle and into the windshield. Top rails, or other rigid horizontal rails or members, metal or wood, should not be used within the clear zone on projects.

5.10.2 PASSING LANES

Passing lanes should be considered on two-lane arterials where it is not practical to achieve adequate passing sight distance or where increased traffic volumes have an adverse impact on the desired Level of Service. Ideally, passing lanes should be considered only in areas where the roadway can be widened on both sides to provide simultaneous passing opportunities for both directions.

The standard travel lane for a passing lane section is 3.6 m. The desirable shoulder width should be 1.8 m with a minimum of 1.2 m. If the roadway has substantial bike use, consult the ODOT Bicycle-Pedestrian Program Manager for input on shoulder width. The minimum median width in a passing lane section (three or four lanes) shall be 0.6 m.

If at all possible, passing lanes should be located where there are no approaches. If there are existing approaches, the type of approach is critical.   Consideration of closing the approach should be given. It may be possible to allow a passing lane where there are single residential approaches or possible forest service type roads, but the approach to public/county roads and approaches that serve multiple trip generation opportunities are not favorable in a passing lane section. Other areas that are poor locations for passing lanes include ending the passing lane at the crest of a hill, on a curve, or where there is potential for left turns at the end of the passing lane.

Passing lanes should be clearly identified to prevent motorists from thinking they are entering a four-lane section of roadway. The minimum length of a passing lane should be 380 m, plus tapers. The taper section at the end of a climbing lane should be computed by the following formula L=0.6WS (L=Length in meters, W=Width in meters, S=Speed in km/h). The recommended length for the lane addition taper is half to two-thirds of the lane drop length. Optimum passing length is 2 km. It is very important to have passing lanes long enough to allow the passing of vehicles but not too long as to make the added passing lane seem like an additional travel lane. The Transportation Planning Analysis Unit (TPAU) should be contacted to determine the appropriate length of passing lane.

Design considerations for providing passing lanes on two-lane highways are as follows:

1) Horizontal and vertical alignment should be designed to provide as much length as feasible with sight distance for safe passing.

2) To maximize safe operations, drivers should be able to clearly recognize both lane additions and lane drops.

3) For volumes approaching design capacity, the effect of lack of passing lanes in reducing capacity should be considered.

4) Where the traffic is slowed or capacity reduced because of trucks climbing long grades, construction of climbing lanes should be considered.

5) Where the passing opportunities provided by application of Items 1 and 4 are still inadequate, the construction of a four-lane highway should be considered. Inability to economically justify climbing lanes or multilanes may require that the roadway be designed for the minimum acceptable level of service.

6) Consider providing extensions to the passing lane section to allow slower vehicles the opportunity to attain free flow speed prior to merging. This reduces the speed differential between vehicles at the merge, improving safety and operations.

5.10.3 CLIMBING LANES

Climbing lanes are normally provided to prevent unreasonable reductions in operating speeds. Normally the combination of heavily loaded vehicles operating on long uphill grades results in the need for climbing lanes. A climbing lane section is not considered a three lane section but a two lane section with an additional lane for uphill slow moving vehicles. (See AASHTO's *"A Policy on Geometric Design of Highways and Streets – 2001"*)

Where climbing lanes are required as specified in Table 7-2 ODOT 4R/New Standards, the location of the beginning and the end of the lane can be determined by the chart, "Trucks Speed-Distance Curves", Figure 5-18. In using this chart for design purposes, vertical curves are not considered, and the speeds are taken from the chart assuming that the vehicle travels in a straight line from one point of grade intersection to the next. Vertical curves can be broken up into straight-line segments if additional accuracy is desired. The taper section added at the beginning of a climbing lane should have a 25:1 ratio desirably, but not less than 50 m in length. The taper section added at the end of a climbing lane should have a 50:1 ratio desirably, but not less than 90 m in length.

Whenever climbing lanes are warranted, the feasibility of supplemental downhill passing lanes should be investigated. Both climbing lanes and downhill passing lanes shall be the same width as the travel lanes used for normal construction. The desirable adjacent shoulder width is 1.8 m with a minimum of 1.2 m. If the roadway has substantial bike use, consult the ODOT Bicycle-Pedestrian Program Manager for input on shoulder width. When climbing lanes are supplemented with downhill passing lanes, a 0.6 m wide median shall be introduced. Four-lane construction with appropriate shoulder and median widths should be substituted for climbing lanes wherever traffic is likely to approach or exceed capacity.

5.10.4 STOPPING LANES AT RR CROSSINGS

Additional stopping lanes for vehicles that must come to a stop at railroad at-grade crossings were formerly added routinely. In some cases stopping lanes are not justified. The following procedure is established to determine whether additional stopping lanes are justified.

The Project Leader is responsible to determine that an at-grade railroad crossing will exist within the project limits. The Project Leader notifies Region and requests an investigation. Region will proceed with the investigation per the procedure outlined in the Traffic Management Section Manual.

Additional design guidance for Railroad Grade Crossings can be found on ODOT Standard Drawing No. RD445 for use when stopping lanes have been justified.

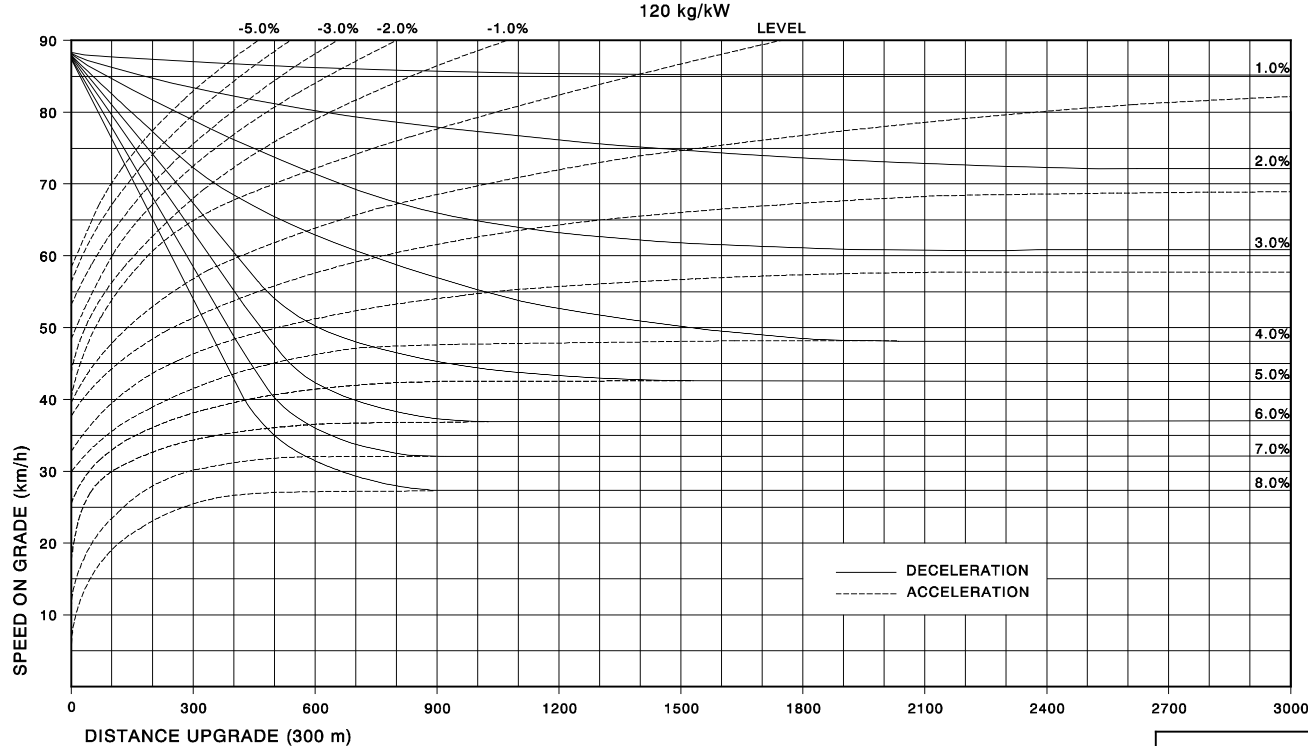
5.10.5 STOCK AND EQUIPMENT PASSES

The standard stock pass shall be a pipe 2250 mm in diameter or a box culvert with inside dimensions of 1800 mm wide by 2400 mm high. If the length is over 45 m, a box culvert with inside dimensions of 2400 mm by 2400 mm shall be used. In some cases, smaller sizes may be feasible and may be used with the approval of the Roadway Engineering Manager. In no case shall a stock pass be smaller than a pipe with 2100 mm inside diameter. When a pipe is used for a stock pass, the invert should be paved. However, the pipe should not be asphalt coated. (The asphalt drips in hot weather and cattle will not use it.) Stock passes are to be at locations free from flow of surface water in order to comply with the DEQ regulations on water quality.

Various dimensions may be appropriate for equipment passes. A reinforced box culvert with inside dimensions of 3000 mm by 3000 mm will accommodate small farm machinery and small trucks. It may not be feasible to provide equipment passes for larger farm equipment.

MULTILANE HIGHWAYS

120 kg/kW



TRUCKS
SPEED-DISTANCE CURVES
FROM ROAD TEST OF
A TYPICAL HEAVY TRUCK
OPERATING ON VARIOUS GRADES

Effect of length and grade on speed of average trucks on modern multilane highways. Based on trucks having a weight-power ratio of 120 kg/kW.

Note: AASHTO Fig. III-25(A) & III-25(B) use w-p ratio of 180 kg/kW.

Note: Refer to chapter on trucks, page 242 & 254, AASHTO Geometric Design of Highways & Streets 1994

State of Oregon
Department of Transportation

HIGHWAY DESIGN MANUAL
TRUCK SPEED/DISTANCE CURVES

Figure: 5-18 2002

Figure 5-18
Truck Speed Distance Curves

5.11 ACCESS MANAGEMENT

5.11.1 INTRODUCTION

Access management is a tool available to designers, planners, and other transportation professionals to improve traffic safety, capacity, and efficiency. The benefits of managing access to highways are well documented. Good access management techniques and strategies when designed properly along state highways will reduce the overall number of accidents and increase the highway's capacity. This chapter is not an exhaustive description of all the rules, laws, and techniques related to access management. This chapter will outline some of the basic concepts, definitions, and appropriate tools for use on Oregon State Highways.

There are several documents that designers, planners, and field staff are encouraged to review to get a big picture understanding of access management. These include:

- OAR 734 Division 51 - This is the access management rule that the Department must comply with when approving approach road connections to a state highway.
- Access Management Manual - This document prepared by the Transportation Development Division is an exhaustive manual on access management. It covers legal issues and policy issues, as well as design tools and alternatives for use on state highways.
- Office of Project Delivery, Project Delivery Leadership Team, Operational Notice PD-03, "Project Development Access Management Sub-teams". This document describes the Department's policy on implementing access management during project development.

This chapter is not intended to be a detailed discussion of approach road design. For more detail on approach road or median design refer to Sections 9.2 and 5.5 respectively.

5.11.2 DESIGN TOOLS

- **Right In – Right Out Only**

The most common design tool is restricting an approach road to right turns in and out only. The preferred design alternative to accomplish this is installation of a non-traversable median. In urban environments this median should be a raised curb style. In more rural environments the median could be raised curb, median barrier, or depressed median. Controlling the median with a non-traversable design is the only design that provides a positive reinforcement of the turn restrictions. For more information on median design, refer to Section 5.5. For more information on approach road design, refer to Section 9.2. Figures 5-19 and 5-20 show some examples of median designs limiting approach roads to right turns only. Figure 5-21 shows the benefits of median control involving pedestrians.

Another design option that may be considered in some situations is the use of a “pork chop” design. A pork chop design consists of a channelization island, usually raised curb, that directs traffic in the intended direction. The channelization island tries to discourage turn movements by angling the entry and exit so that left turn movements are uncomfortable. The problem with the pork chop design is that passenger vehicles are still physically able to make left turn movements. Most pork chop designs that do not include a non-traversable median design have a very high rate of non-compliance for the restricted movements. Therefore, a pork chop design should still include a non-traversable median design as well. Where a non-traversable median is not practical or is unacceptable, the designer should attempt to maximize the entry and exit angles to make left turn movements as difficult as possible. Figure 5-22 shows a pork chop design concept with median control.

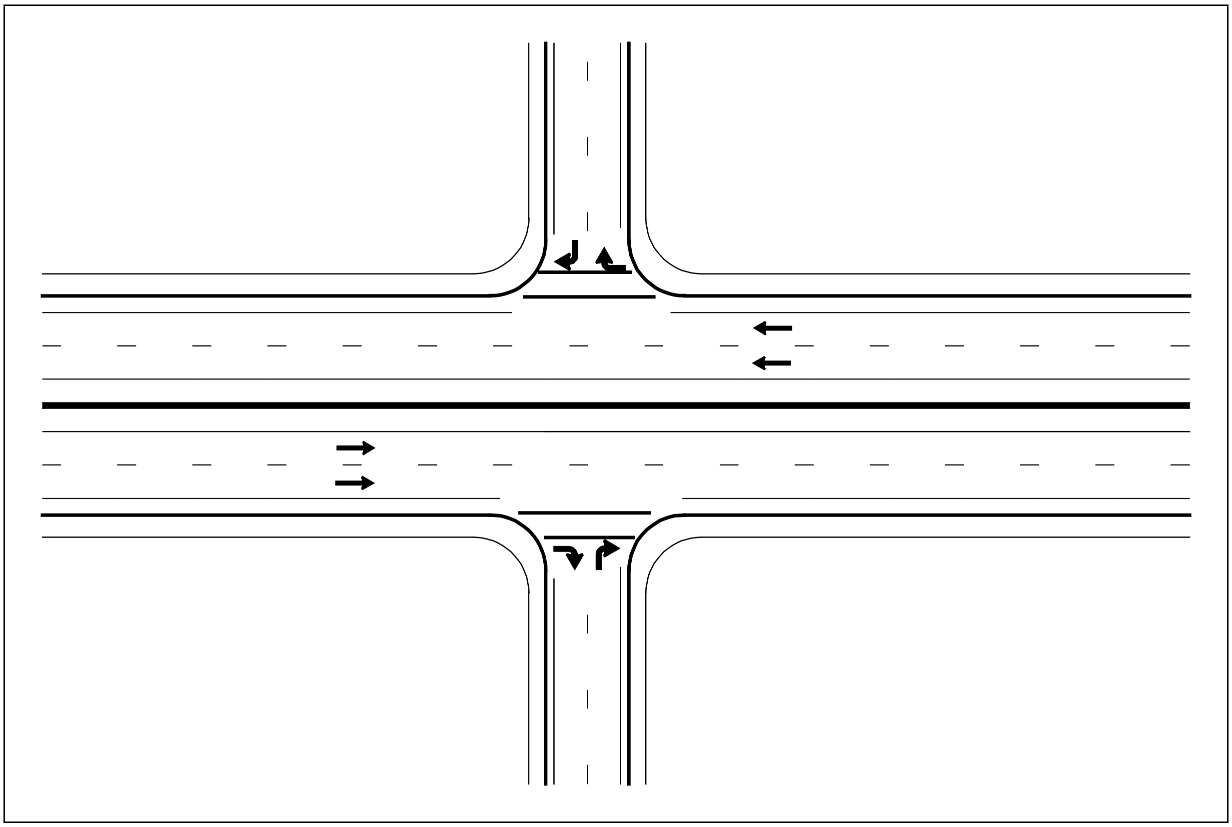


Figure 5-19
Median Detail with Right In/Out

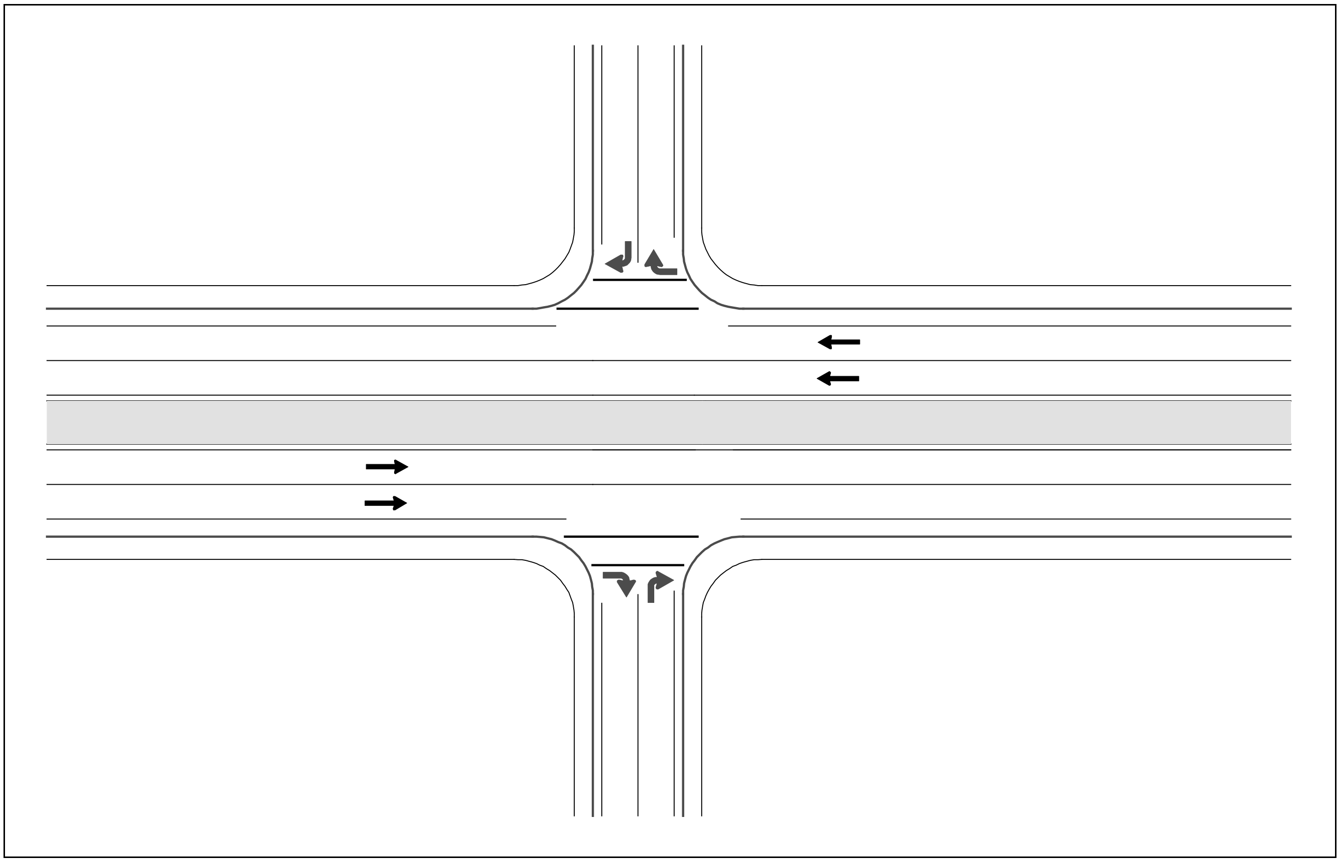


Figure 5-20
Raised Median Detail – Right In Right Out

- **Right In – Right Out with Left In**

From a traffic analysis perspective, the left turn out movement from approach roads usually operates worse than all other movements. This is because in the hierarchy of turn movements, the left turn out from an approach road is the last priority. In addition, the left turns from an approach road usually experience a higher number of accidents than the other movements. Because of these factors, there are several situations where eliminating a left turn out movement from an approach road is the preferred design solution. The only effective design option for this technique is a non-traversible median. Generally the preferred median style is raised curb. Median barrier is not applicable to this design technique. When designing this type of median it is critical to physically exclude the left turn out movement. The basic concept of this design is to extend a traffic separator along the right edge of the left turn entering traffic. This separator should extend back away from the approach road far enough so that passenger vehicles cannot physically turn left from the approach road. The design still must accommodate the appropriate design vehicle. Figure 5-23 illustrates this design concept.

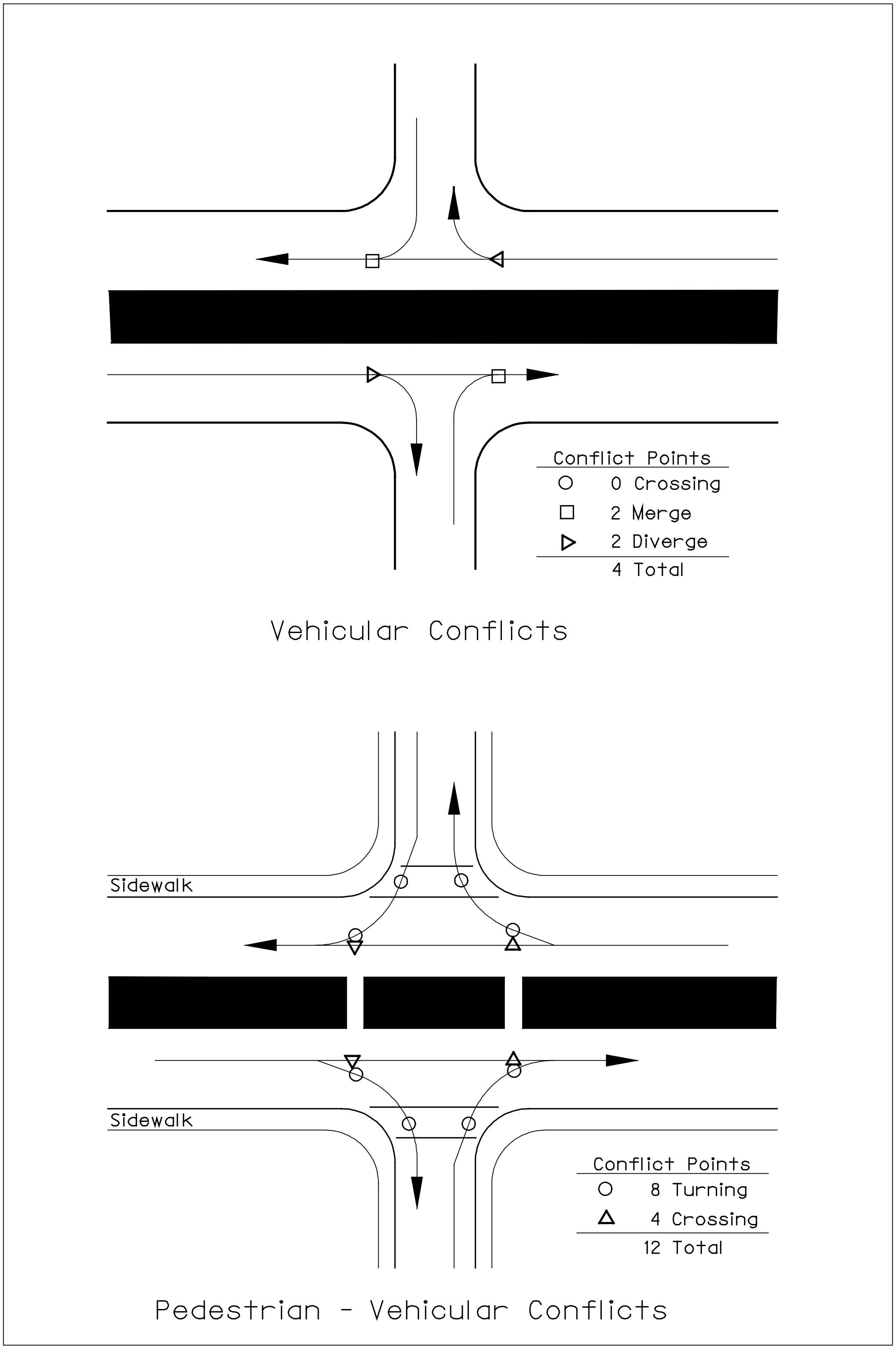


Figure 5-21
Vehicle-Pedestrian Conflict

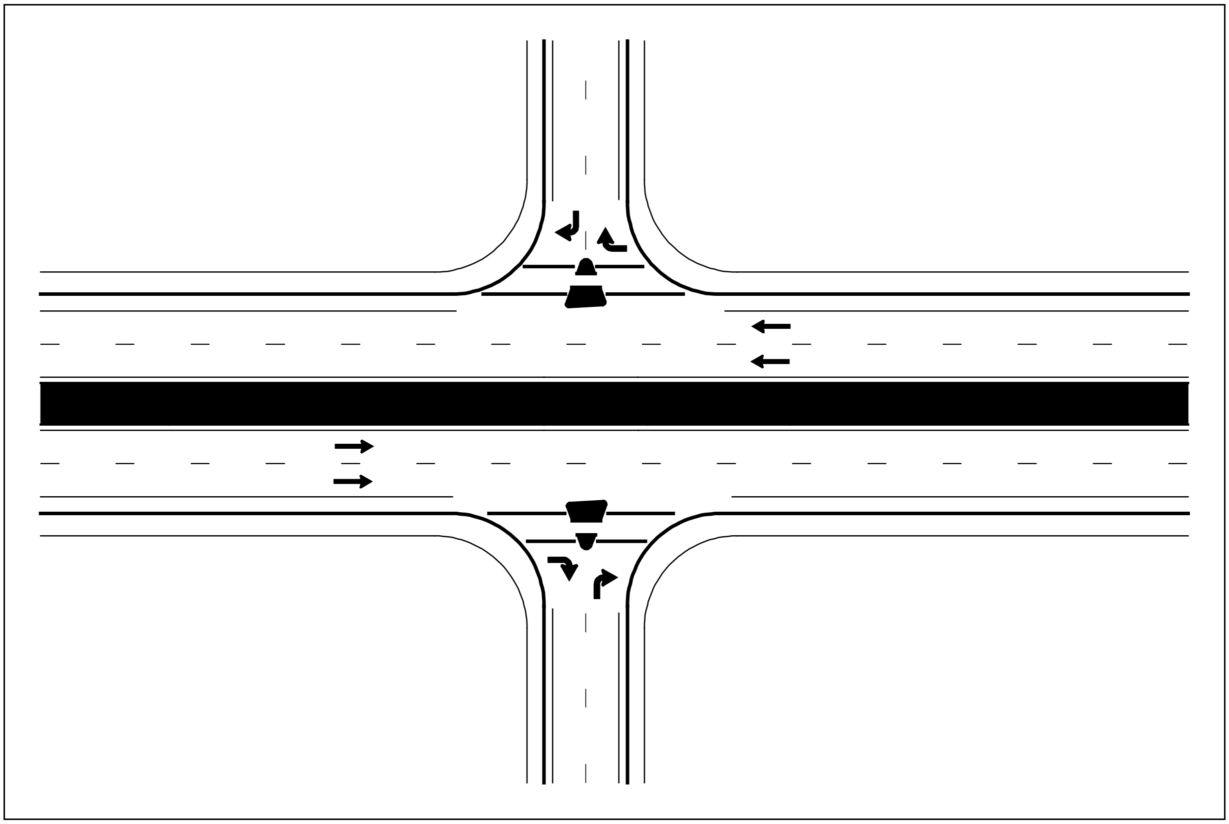


Figure 5-22
Pork Chop With Non-Traversable Median

- **Opposing approaches with Left In**

In many urban environments, approach roads will be directly opposite from each other. In some situations, eliminating left turns out of the approaches is desired. In these cases, the appropriate design is very similar to the design described in “Right Out with Left In” for a single approach restricting left turns out. The difference is the median design now accommodates opposing left turn traffic. The concept remains the same however, physically eliminate the ability for passenger vehicles to make a left turn out movement. The difference is the traffic separator must now “snake” through the intersection transitioning from one side of the median to the other using reversing curves. The curvature is determined by the design vehicle. It is preferred with this technique to obtain additional width of the traffic separator in the middle of the median. This will provide additional visual guidance through the intersection. Figure 5-24 illustrates the use of this design concept.

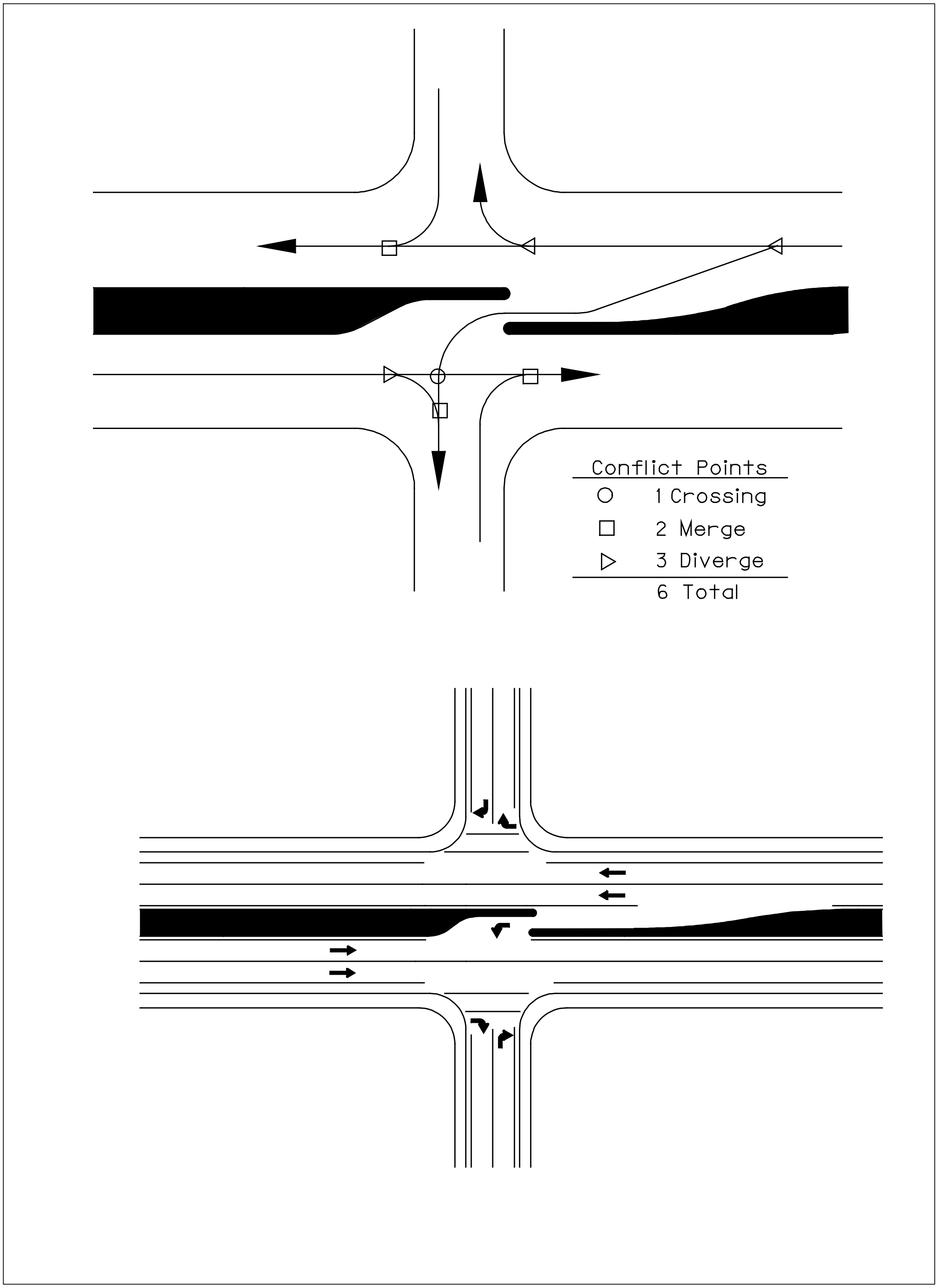


Figure 5-23
Left Ingress from One Direction Only

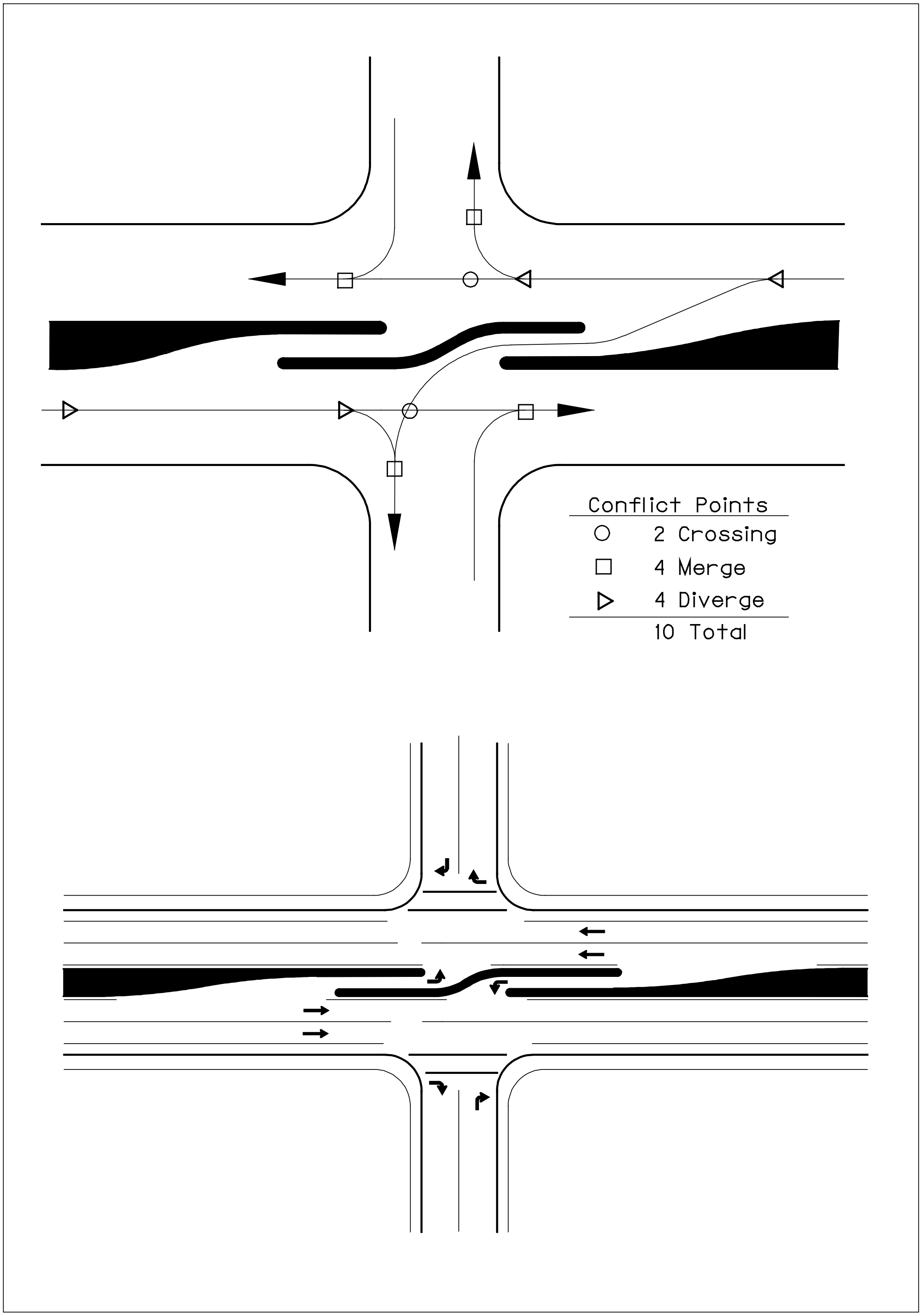


Figure 5-24
Left Ingress from Both Directions

- **Offset Approaches**

Primarily, this design option is used in rural or fringe areas where spacing between approach points is large. This design tool is implemented where a four-leg intersection is experiencing significant operational and safety problems. By separating the intersection into two individual intersections, the number of conflicts is reduced which should improve the safety of the intersections. If this design option is chosen, the intersection needs to be split in the correct direction. The approaches should be offset to the right in order to eliminate the back to back left turn queue conflict. The amount of the offset will vary depending upon the highway volume, approach road volume, surrounding land uses, speed of the highway, and direction of the offset. The designer needs to consider the functional area of each intersection and the amount of weaving traffic. In addition, the Region Access Management Engineer and Traffic Management Section should be contacted when considering offset approaches. For more information on offset intersections refer to Section 9.1.

- **Frontage Roads**

Frontage roads are a very useful design to eliminate or restrict direct highway access from a section of highway. The frontage road needs to be designed to accommodate the volume and type of traffic anticipated. Two of the most important elements of the frontage road design are the connection to the highway and turning roadway. The connection needs to be designed to accommodate the allowable turning movements for the appropriate design vehicle. If trucks are to use the frontage road, they must be considered in the design. Secondly, the design of the connection to the frontage road is critical. Usually, this connection is a turning roadway, but may be an intersection. The connection needs to provide off-tracking room for trucks using the frontage road. The design needs to consider the roadway alignment and width to make sure trucks can physically make the turns required. Finally, frontage roads should be offset from the highway so as to not interfere with highway operations. The frontage road must be physically separated from the highway by use of barriers, fencing, or ditches. The separation between the highway and frontage road edges of pavement must be at least 12 m, but preferably 15 m or more. The design also needs to consider clear zone requirements and the effect of headlight glare on both roadways.

Another option involving the location of the frontage road is to locate the frontage road on the back side of the adjacent properties. This option may be more appealing from a visual standpoint allowing the properties to front the mainline roadway while the parking lot and frontage road are located further away from the mainline roadway. This option may also provide for better mainline/frontage road traffic operations. See Figure 5-25 for frontage road examples.

- **U-Turns**

Where a section of highway contains a non-traversable median for an extended length, there may be a need to accommodate U-Turning traffic. There are several design techniques available to accommodate U-Turns. The first option is at an intersection without a jug handle. This design option generally requires widening the highway in one quadrant of the intersection to accommodate the required turning space of vehicles. Designs need to consider the type of vehicle using the U-Turn. In many situations, trucks will be prohibited from using this style of U-Turn. The widening can make use of a far side bus stop, or can be tapered. All U-Turns using this type of design technique at a signalized intersection must have the approval of the State Traffic Engineer.

A second design option for accommodating U-Turning traffic is the use of a jug-handle. There are two options for jug-handle U-Turn designs. One option is the left side jug-handle. The left side jug-handle is a turning roadway alignment located on the left side of a highway. U-Turning traffic makes a left turn from the highway into the jug-handle. The jug-handle circulates the traffic back to the highway where vehicles re-enter the traffic stream as right turns through normal gaps in traffic flow. This style of jug- handle can be used at an existing "T" intersection or mid-block. The jug-handle is only compatible with a right side "T" intersection, which may or may not be signalized.

The other jug-handle design option is the right side jug-handle. The right side jug-handle is located on the right side of the highway. U-Turning traffic makes a right turn off the highway into the jug-handle, then loops around to the left. The vehicles then make a left turn across the highway. This movement may or may not be signalized. As with the left side jug-handle, the right side jug-handle is only compatible with a "T" intersection. In this case, however, the intersection is only on the left side of the highway. Additionally, this type of jug-handle can be used at a mid-block location. The major disadvantage of this style is traffic must make a left turn across both directions of highway traffic and is therefore less efficient and may also have additional safety risks. See Figures 5-26 and 5-27 for U-Turn treatments.

- **Indirect Left Turns**

One tool available is indirect left turns at intersections. In some situations for capacity or safety reasons, it may be desirable to remove left turning traffic. The left turns are accomplished by other connections. The first option available is the use of a right side jug-handle just like the one described for U-Turns above. Vehicles wishing to turn left actually leave the highway on the right side then cross the highway. Generally these designs are signalized to facilitate the crossing movement. Again this particular type of jug-handle is only compatible with a left side "T" intersection.

A different type of indirect left turn design uses connecting roadways. This design concept is similar to the jug-handles described in the U-Turn section. Within this type of design are several options. These include the single quadrant and double quadrant. The single quadrant design provides one connecting roadway that provides for two way traffic operation. Location of the connecting roadway is dependent upon traffic flow characteristics, adjacent roadside development, need for intersection spacing, and signalization needs. The concept of the single quadrant design is to remove all left turning traffic from a specific intersection. The traffic uses the connecting roadway to gain access to the particular street. Location of the connecting roadway is critical to the operation on the highway, particularly if both intersections are to be signalized. The designer should make sure the Traffic Management Section and the Transportation Planning and Analysis Unit (TPAU) have reviewed and approved this design concept prior to actual installation.

As mentioned previously, another option is the double quadrant design. This design is very similar to a jug-handle style interchange, except that the intersecting roadways are not grade separated. Again, turning traffic, generally left turns, use the connecting roadways. The roadways may provide for all movements or may be right in/out only depending upon traffic capacity and safety needs. Again, the Traffic Management Section and TPAU should review and approve this type of design prior to installation. In addition, there may be access management issues on these connecting roadways. The Region Access Management Engineer should be consulted to identify and address these issues. In many situations, these last two design alternatives may be a phased approach towards grade separation in the future.

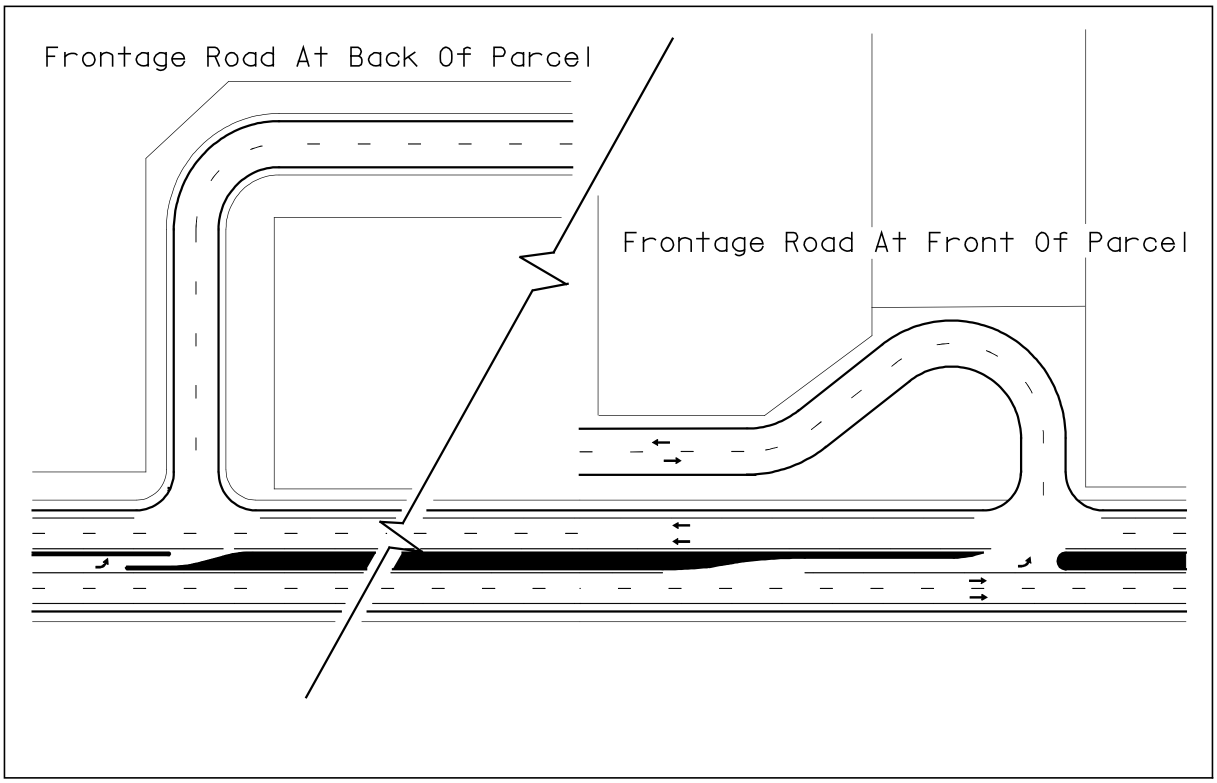


Figure 5-25
Example of Frontage Road Locations

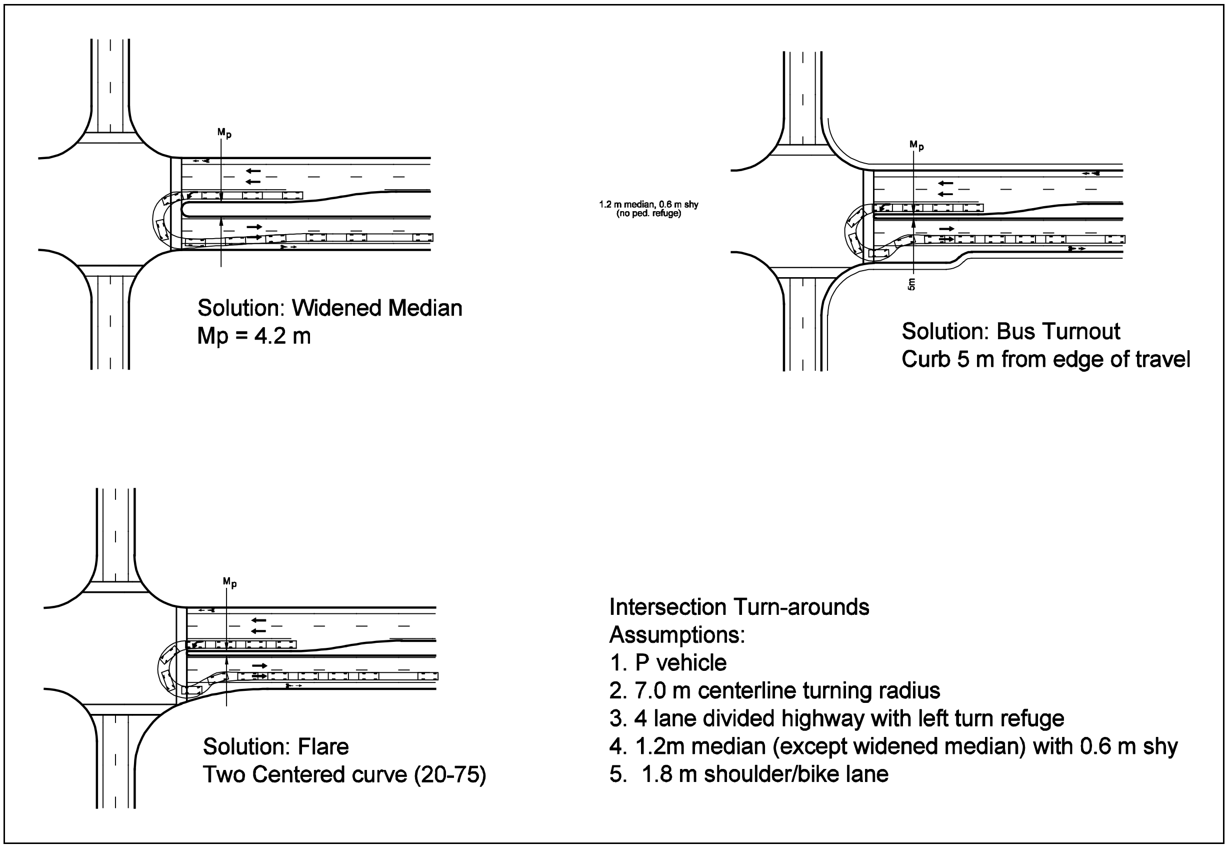


Figure 5-26
U-Turn at Intersection

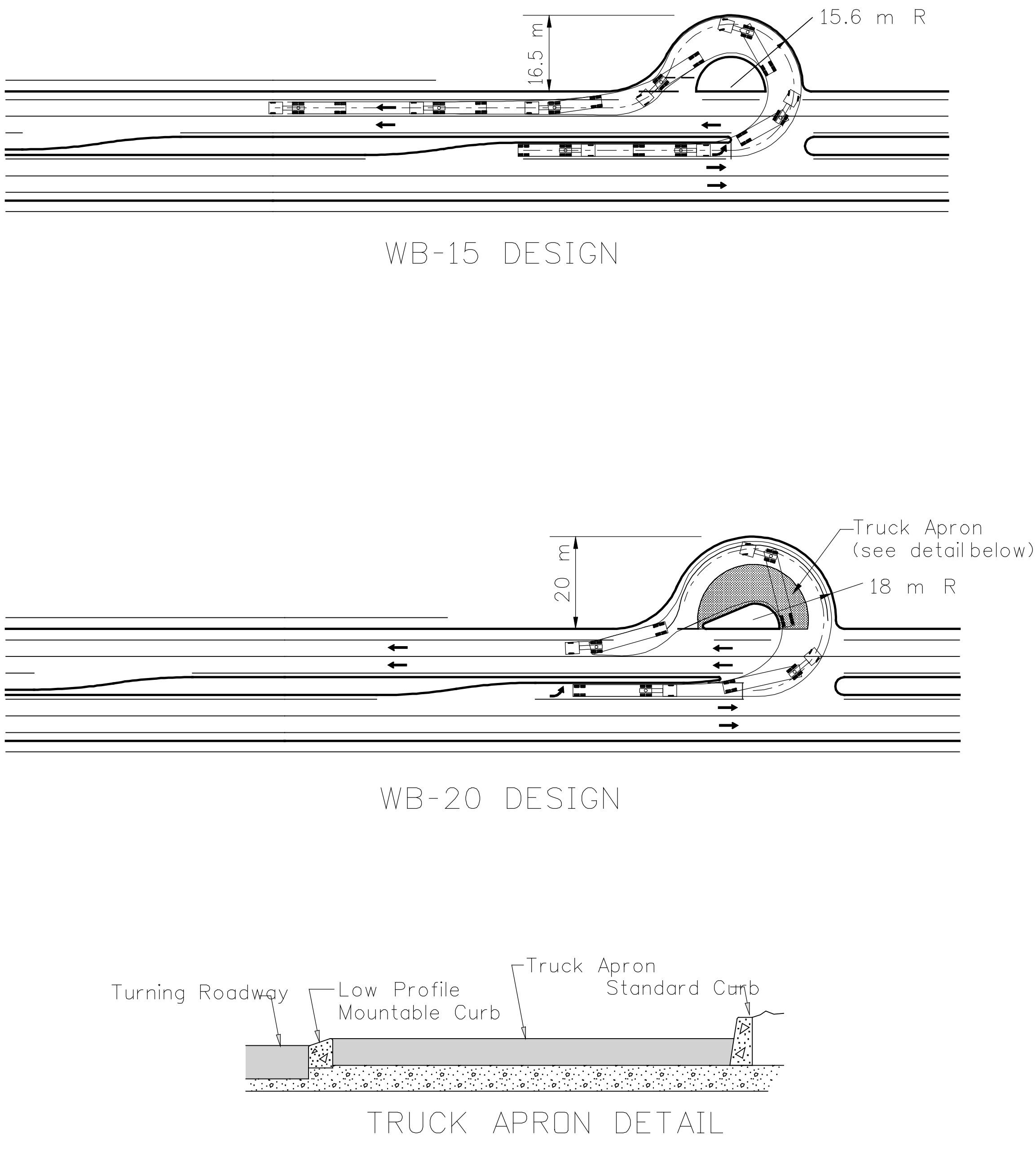


Figure 5-27
U-turn at Midblock

5.11.3 MANAGEMENT TOOLS

- **Access Control**

Acquiring the access rights from properties abutting a state highway provides a high level of protection to the highway. However, acquiring access control is not justifiable in all conditions. The Department has developed guidelines for access management decisions during project development. These guidelines are contained in Transportation Operations Bulletin PD-03. They attempt to focus the Department's limited resources for projects that really need access control. The guidelines can be found in the Oregon Highway Plan, Appendix C.

- **Grants of Access**

A Grant of Access is a transfer of a property right to a property owner for a right of access at a particular location. The Department must follow the requirements of OAR 734 Div. 51 when issuing Grants of Access. Obtaining a Grant of Access can be a complex process. Before even considering a Grant of Access as part of a project, the designer should contact the Region Access Management Engineer.

- **Access Management Plans**

An access management plan is a useful management tool. An access management plan can be done as part of an ODOT STIP project or during a coordinated planning study. Access management plans developed in a coordinated planning process establish a plan for accessing properties in the future. An access management plan essentially is a detailed plan outlining how adjoining properties are to be accessed during project development.

- **ODOT Permit Process**

The ODOT Permit Process is also outlined in OAR 734 Div. 51. All approaches to a state highway built after 1949 must have an Approach Road permit to be considered legal. Through the permitting process ODOT can negotiate access designs, approach configurations, turn movement restrictions, and even shared approaches. Properties with multiple approaches can be modified to provide the minimum number needed. Again, designers should work closely with the Region Access Management Engineers when making approach permit type of decisions. The authority for issuing permits resides with the Region Manager or designee.