CHAPTER 5

HORIZONTAL ALIGNMENT

INTRODUCTION

Horizontal and vertical alignments establish the general character of a rural highway, perhaps more than any other design consideration. The configuration of line and grade affects safe operating speeds, sight distances, and opportunities for passing and highway capacity. Decisions on alignment have a significant impact on construction costs.

The continuous line of a highway is formed longitudinally by its "alignment" in the horizontal and vertical planes. In combination with the cross-sectional element, the highway then -- in three dimensions -- becomes functional and operative. The elements for purposes of geometric design are first treated separately and finally combined and coordinated to form the whole facility. The assembly of these units into a continuous whole establishes the alignment of the road. The components of the horizontal alignment include tangents (segments of straight lines), circular curves and, in some cases, spiral transition curves. The manner in which these components are assembled into a horizontal alignment will significantly affect the safety, operational efficiency, and aesthetics of the highway.

FACTORS INFLUENCING THE SELECTION OF HORIZONTAL ALIGNMENT

Some of the factors that influence the location and configuration of the horizontal alignment include:

1. Physical controls - topography, watercourses, geophysical conditions, land use, and man-made features.

2. Environmental considerations - affect on adjacent land use, community impacts, ecologically sensitive areas.

3. Economics - construction costs, right-of-way costs, utility impacts, operating and maintenance costs.

4. Safety - sight distance, consistency of alignment, human factor considerations.

5. Highway classification and design policies - functional classification, level of service, design speed, design standards.

Although the designer must attempt to optimize the horizontal alignment with respect to these factors, the alignment cannot be finalized until it has been compared and coordinated with the vertical and cross-sectional features of the highway.

GENERAL CRITERIA

Design speed is the principal factor controlling the design of horizontal alignment. Several geometric standards related to design speed are very specific. Other criteria cannot be defined as specifically and require that judgments be made by designers in consideration of local conditions. Guidelines for some of these decisions are outlined below. Additional, general discussion on Design Speed can be found in Chapter 2 of the current AASHTO publication *A Policy on Geometric Design of Highways and Streets*.

In general, above-minimum design criteria for specific design elements should be used, where practical, particularly on high-speed facilities. In contrast, the designer is cautioned that on lower speed facilities, use of above-minimum design criteria may encourage travel at speeds higher than the design speed.

Consistency

Alignment should be consistent and as directional as possible. A smooth flowing alignment is desirable on a rural arterial. Changes in alignment, both horizontal and vertical, should be sufficiently gradual to avoid surprising the driver. Frequent changes in alignment with many short curves should be avoided except in cases of very low design speed and low traffic volume in rough terrain. Sharp and / or short curves should not be introduced at the ends of long tangents. Sudden changes from areas of easy curvature to sharp curvature should be avoided. Where sharp curvature must be introduced, it should be gradually approached by successively sharper curves from the generally easy curvature.

Length of Curves

Avoid the use of the maximum permissible degree of curvature except where absolutely necessary. Generally, curves should be as flat as practicable for the conditions. On the other hand, avoid the use of excessively long, flat curves where there is need to provide considerable passing opportunities. Drivers are reluctant to pass on a curve, even though the sight distance may be adequate. It may be better to use a shorter curve, thus lengthening the adjacent tangent section and increasing the passing opportunity.

A horizontal curve is not required for deflection angles of 15 minutes or less. For small deflection angles greater than 15 minutes, curves should be long enough to avoid the appearance of a "kink". Curves should be at least 500 feet (150 m) long for a deflection angle of 5 degrees, and the minimum length should be increased 100 feet (30 m) for each 1 degree decrease in deflection angle. (Note – In the superelevation tables, the minimum curve radii for each design speed without superelevation is listed. This is a separate, different criteria from that noted above.)

The minimum length of horizontal curves on main highways should be about ($L = 15 \times V_d$) in (ft) or ($L = 3 \times V_d$) in (m). (V_d = Design Speed in mph or kph & L = Length of Curve in ft or m.) On high-speed controlled access highways, a desirable minimum length of curve would be about ($L = 30 \times V_d$) in (ft) or ($L = 6 \times V_d$) in (m).

Adjacent Curves

Care should be used in the design of compound curves. Preferably, their use should be avoided where the curves are sharp. Compound curves with large differences in curvature introduce the same problems that arise with a tangent approach to a circular curve. Where compound curves must be used, the radius of the flatter curve should not be more than 50 percent greater than the radius of the sharper curve.

Avoid abrupt reversals in alignment (S-curves) by providing enough tangent distance between the curves to ensure adequate superelevation transition length for both curves and sufficient distance for adequate signing.

Avoid short tangent segments between horizontal curves (with no superelevation). Instead, lengthen one or both of the curves to tie the PC and PT of the curves together to create a reverse curve.

Avoid "broken-back" curves (short tangent sections between two curves in the same direction). Use of spiral transitions, compound curves or a single longer curve is preferable because they provide some degree of continuous superelevation. When broken-back curves are necessary, there should be about 1500 feet (460 m) of tangent distance between the curves.

High Fills

On long, high fills, tangents should be used wherever feasible. If curvature is needed, it should be kept as flat as possible. Without visual "cues" such as cut slopes or shrubs and trees above the roadway, it is difficult for drivers to perceive the extent of curvature and adjust their operation to the conditions.

Bridges

It is desirable that bridges be located on tangent positions of the alignment. But sometimes it is necessary to locate a bridge on a curve. When this happens, care should be used to avoid beginning or ending a curve on the bridge. This can be hazardous under slippery surface conditions -- and also adds complications to bridge design and construction. When curves are necessary on road sections near bridge ends, the beginnings and endings of curves should be located so that no portion of the superelevation transition extends onto the bridge.

Stationing

New contiguous stationing is established for construction/reconstruction projects. The direction of stationing is set to follow the direction of increasing MRM (Increase from west to east for even numbered highways and south to north for odd numbered highways). Existing grading plans stationing is used for resurfacing projects with equations as needed. If a project is a combination project, work with the resurfacing designer to determine if existing and/or new stationing should be used. If it is not a construction/reconstruction project the standard is to retain existing stationing.

The factors which influence this decision are:

A) Completeness of survey – a construction/reconstruction survey for the whole project would allow new stationing.
B) Direction of existing surveys – if existing alignments are not all stationed in the same direction, new stationing is desirable.
C) Multiple existing surfacing typical sections (different width and/or thickness) – existing stationing is more convenient for the resurfacing design.

When using equations refer to the stationing as 10+00, a 10+00, b 10+00, c 10+00, etc. This corresponds to the format utilized by the current SDDOT Road Design software. Coordinate with the resurfacing designer on this procedure as well.

Miscellaneous

Following is a list of basic tips to use as guidance when setting a horizontal alignment. It is recommended that these miscellaneous criteria be followed unless there is an engineering design reason that overrides the criteria.

A) When the highway alignment follows section line ROW and horizontal alignment PI's are set very near section corners, place the PI's directly on the section corners instead of inches or a few feet away from them.
B) Minimize or avoid unnecessary horizontal PI's in low speed urban design. The alignment typically does not need to exactly follow the existing highway centerline.
C) Avoid unnecessary horizontal PI's (deflection with no curve) adjacent to a horizontal curve.
D) Avoid negative horizontal alignment stationing.

SIGHT DISTANCE

Sight distance across the inside of curves must be considered. Sight obstructions such as walls, cut slopes and buildings may limit sight distance on curves. Where these obstructions can not be removed, adjustment in the normal cross section or a change in alignment may be required to provide adequate sight distance.

When checking a curve for sight distance, the sight line is a chord of the curve, and the applicable sight distance is measured along the centerline of the inside lane around the curve. Figure 5-1 provides formulas and a design aid for evaluating sight distances on horizontal curves. The formulas give the designer the option to compute the actual resulting sight distance for the given geometrics or to design for a middle ordinate distance that will satisfy the required site distance.

To provide for desirable passing site distance, clear sight areas on the insides of curves (middle ordinate distance) must be considerably wider. Often this is not practical and it becomes necessary to use lower values that may result in no-passing zones.

Geometric design standards (including stopping, decision and passing sight distances) are presented in Chapter 3 of the current AASHTO publication *A Policy on Geometric Design of Highways and Streets*. These design values are applicable to all projects regardless of functional classification, type of improvement or traffic volume group. They are based on a maximum permissible superelevation of 6 %, a 3.5 ft (1070 mm) height of eye and 2.0 ft (610 mm) height of object for stopping site distance and decision sight distance or a 3.5 ft (1070 mm) height of object for passing site distance.

For information on intersection sight distance see Road Design Manual Chapter 12 – Intersections.

Stopping Sight Distance

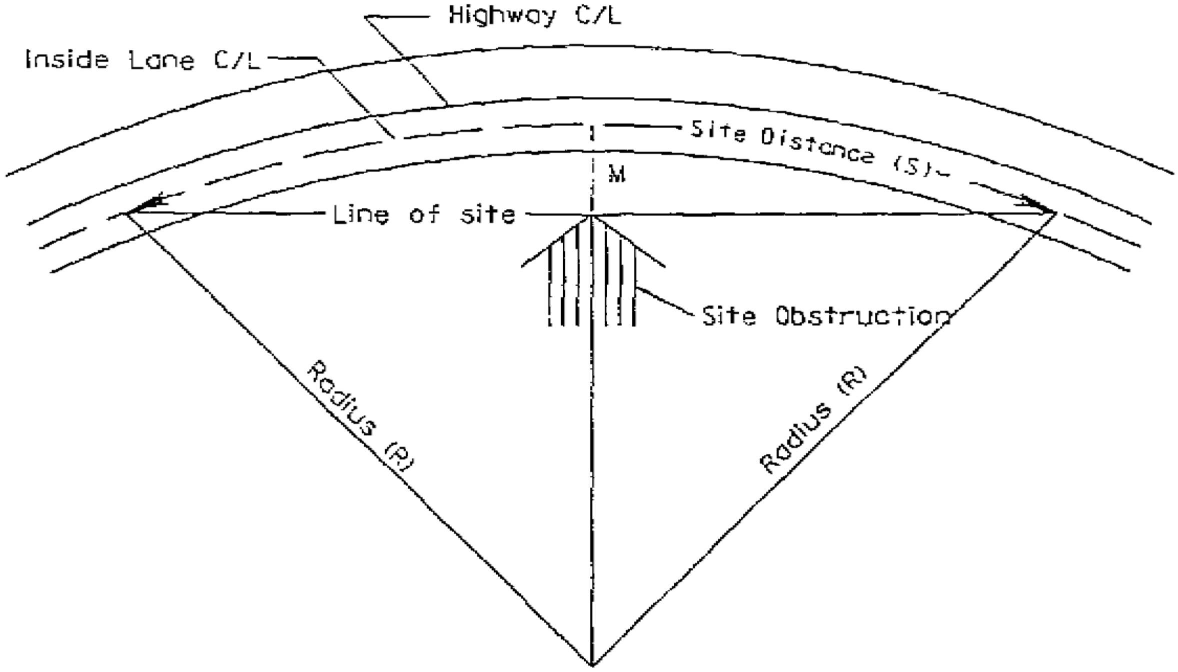
Stopping sight distance (SSD) is the sum of two distances: The distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied and the distance required to stop the vehicle from the instant brake application begins. (SSD = BRD + BD)

SSD - Stopping Sight Distance (ft)
BRD - Brake Reaction Distance (ft)
BD - Braking Distance (ft)

SSD is computed from the following formulas:

English	Metric
$SSD = 1.47x(V)x(t) + (1.075 \times (V^2 / a))$	$SSD = 0.278x(V)x(t) + (0.039 \times (V^2 / a))$

SSD - Stopping Sight Distance (ft) or (m)
V - Initial Speed (mph) or (kph)
a - deceleration rate (11.2 ft / s^2 or 3.4 m / s^2)
t - AASHTO assumes a brake reaction time of 2.5 seconds.



$$S = \frac{R \times \cos^{-1}\left(1 - \frac{M}{R}\right)}{28.65}$$

$$M = R \times \left(1 - \cos\left(\frac{28.65 \times S}{R}\right)\right)$$

$$R = \frac{5730}{D}$$

Where:

S = Stopping Site Distance (Feet) or (Meters)

D = Degree of Curve

R = Radius (Feet) or (Meters)

M - Middle Ordinate (Feet) or (Meters)

Figure 5-1 Sight Distance Measurements for Horizontal Curves

Stopping Sight Distance (Continued)

The computed distances are based upon wet pavements. Vertical downgrades have a negative effect (lengthen) and vertical upgrades have a positive effect (shorten) SSD lengths. See “Sight Distance” in Chapter 3 of the current AASHTO publication *A Policy on Geometric Design of Highways and Streets* for more information on the effects of vertical grades on SSD.

Stopping and passing sight distance tables can be found in Road Design Manual Chapter 6 – Vertical Alignment.

Decision Sight Distance

Decision sight distance is the distance needed for a driver to detect an unexpected or otherwise difficult to perceive information source or condition in a roadway environment that may be visually cluttered, recognize the condition or its potential threat, select an appropriate speed and path, and initiate and complete the maneuver safely and efficiently. Decision sight distance values are substantially greater than stopping sight distance.

Examples of critical locations where it is desirable to meet decision sight distance criteria include interchanges and intersections where unusual or unexpected maneuvers are required, changes in cross section such as lane drops and areas of concentrated demand for the driver’s attention (urban locations) such as roadway elements, high traffic volumes, traffic control devices and advertising signs.

Additional information (including decision sight distance design criteria) can be found in “Sight Distance” in Chapter 3 of the current AASHTO publication *A Policy on Geometric Design of Highways and Streets*.

HORIZONTAL & VERTICAL ALIGNMENT COORDINATION

The horizontal alignment can not be finalized until it has been compared and coordinated with the vertical and cross-sectional features of the highway.

Horizontal curvature and vertical grades should be in proper balance. Emphasis on the horizontal tangent alignment is not desirable when it results in extremely steep or long vertical grades. Neither is emphasis on flat vertical grades when it results in excessive horizontal curvature. A compromise between the two extremes is the best approach. Several general criteria should be kept in mind:

1. Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. This condition makes it difficult for drivers to perceive the horizontal change in alignment, especially at night.

2. Sharp horizontal curvature should not be introduced at or near the low point of a pronounced sag vertical curve. This is aesthetically undesirable and can be hazardous since vehicle operation speeds, particularly trucks, are often higher at the bottom of vertical grades.

3. On two-lane roads and streets with considerable traffic volume, safe passing sections must be provided at frequent intervals and for an appreciable percentage of the length of the roadway. In these cases it is necessary to work toward long horizontal tangent sections to secure sufficient passing sight distance rather than the more economical combination of vertical and horizontal alignment.

4. Both horizontal curvature and the vertical profile should be as flat as feasible at intersections where sight distances along both roads and streets is important and vehicles may have to slow or stop.

5. For locations with vertical grades of 5% or greater, some adjustment to superelevation rates for horizontal curves should be considered. This is most important on facilities with high truck volumes and on low speed urban facilities with intermediate curves using high levels of side friction demand. To accommodate these situations, a slightly higher design speed of +5 mph (or higher, if deemed appropriate) can be assumed with the corresponding higher superelevation rate. For divided highways with each roadway independently superelevated, the higher assumed design speed is only necessary for the downgrade roadway.

6. When Superelevation Transitions occur at the crest or sag of a long vertical curve and / or along roadway segments with minimal longitudinal grade (especially where the transition gradient is similar to but opposite in sign to the longitudinal grade), the Designer should check for flat spots in these locations. Poor drainage may result from this situation due to the combination of no cross slope and / or no vertical slope. Two techniques can be used to alleviate the potential for these drainage problems:

A) Maintain minimum profile grade of 0.5 percent through the transition section.

B) Maintain minimum edge of pavement (depending upon pivot point, this could be centerline) grade of 0.2 percent (0.5 percent for curbed streets) through the transition section.

Drawing centerline and edge of driving lane profiles and / or subgrade contours can be helpful aids to review the smoothness of edge profiles as well as potential areas of poor drainage. This is most important for divided highways that typically carry higher traffic volumes.

Some examples of poor and good horizontal and vertical coordination in design can be found in the section "Combinations of Horizontal and Vertical Alignment" in Chapter 3 of the current AASHTO publication *A Policy on Geometric Design of Highways and Streets*.

STRIPING NO-PASSING ZONES

The striping for passing / no passing will be the responsibility of the Region Office after construction, because the Office of Road Design does not have sufficient information for topographical obstacles to properly design the striping. The Office of Road Design will do sight distance checks, but actual measurements are performed in the field.

Perhaps the simplest way to measure passing sight distance for objects in the horizontal plane, is directly from the plans, using a straight edge. Potential obstructions are plotted on the plans. In the case of cut slopes, a dotted line is plotted for the horizontal distance from the centerline of the inside lane to a point on the cut slope 3.5 ft (1070 mm) above the traffic lane. The straight edge is placed along the edge of the obstruction (or dotted line) and the intercepts with the centerline identify the sight distance. Vertical curve geometrics often dictate the majority of the passing / no passing zones. The striping diagram can not be considered complete until both the horizontal and vertical curve geometrics have been evaluated. See "Striping Diagram (Title Sheet)" in Chapter 6 – Vertical Alignment of the Road Design Manual.

INTERSECTING ROADS

The Secondary Road Plan, an agreement between the FHWA and the SDDOT, establishes design standards for county on and off system roads, is available in the Secondary Roads Office.

More detailed explanation and design controls for the design of intersecting roads are presented in Road Design Manual Chapter 12 - Intersections.

UTILITIES

To ensure that reconstruction projects that may involve new alignments are constructed in the most economical manner, it is important that alternate locations be considered relative to existing utilities, real and personal property.

Procedures shall be as follows:

1) When utilities parallel the alignment and require relocation as a result of reconstruction, cost of relocation shall be a consideration in the final route selection. The alignment may be shifted off the existing alignment to avoid relocation and reduce cost while considering Right-of-Way impact of line shift.

2) Alignment shifts to reduce impact on real / personal property shall be considered in conjunction with current design standards.

3) Alignments off the existing route corridor shall be selected on the basis of Right-of-Way impact, utility relocation cost and adherence to current design standards.

TRANSITION (SPIRAL) CURVES

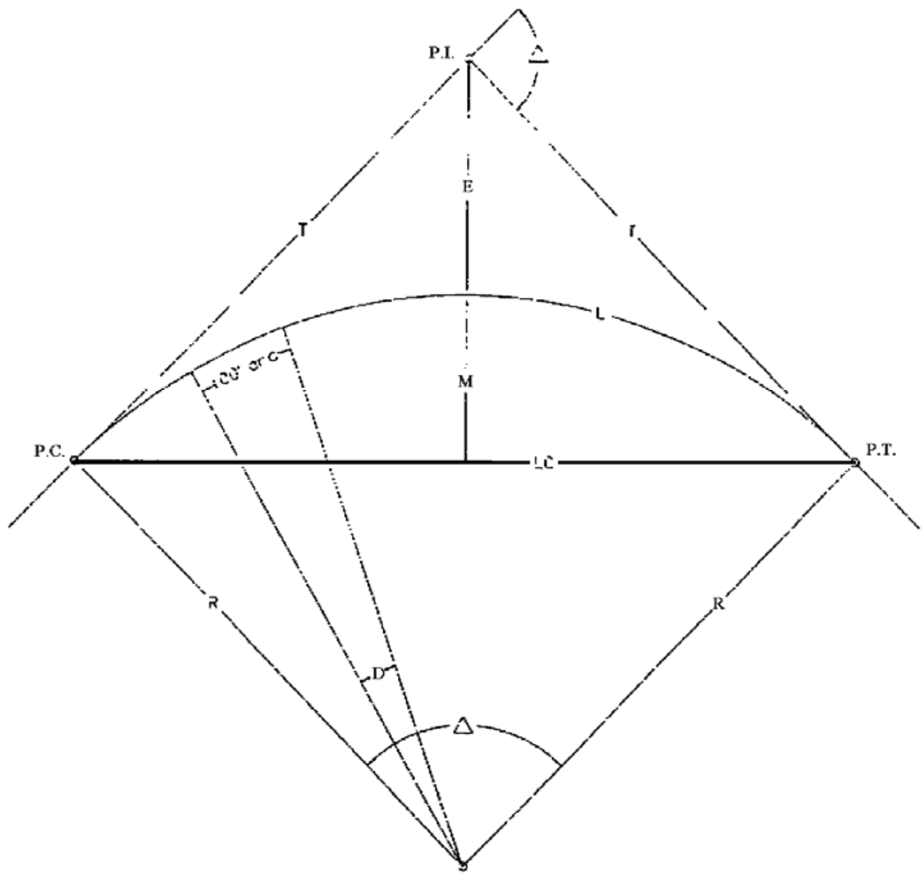
Although typically not used in new design, certain applications may require the use of spiral curves. The use of spiral curves generally is limited to areas of rough terrain where curves often approach the maximum degree of curvature for the particular design speed.

The spiral curve, as one of the alignment components, is used to allow for a transitional path from tangent to circular curve, from circular curve to tangent, or from one curve to another which have substantially different radii. The spiral provides for ease in operation and comfort, allowing for easy-to-follow natural superelevated transitional paths and promote uniformity in speed and increased safety. The use of a Spiral may enhance highway aesthetics. (For more information on spirals, see the current AASHTO publication *A Policy on Geometric Design of Highways and Streets*.)

CIRCULAR CURVES

Generally, circular curves should be used on most projects. The fundamental properties of the circle, as utilized in highway engineering, consist of interrelated elements as shown by Figure 5-2. (English)

The radius for horizontal curves is measured to the horizontal control line which is typically the centerline of the alignment. For wide roadways with sharp horizontal curvature, it may be appropriate to measure the radius to the inside edge of the inner most travel lane. For example, with a 5-lane undivided highway in an urban situation, the radius at the inside edge of the inner most travel lane would be 30 feet smaller than the radius at the centerline of the alignment. This could result in a significant difference in superelevation rate selected for this scenario. (See Table 5-2 Low Speed Urban)



P.C. = Point of Curvature

P.I. = Point of Intersection

P.T. = Point of Tangency

Δ = Central or Deflection Angle

R = Radius of Curve

D = Degree of Curve = $\frac{5729.57795}{R}$

LC = Long Cord = $2 \times R \times \sin\left(\frac{\Delta}{2}\right)$

T = Tangent of Curve = $R \times \tan\left(\frac{\Delta}{2}\right)$

E = External of Curve = $T \times \tan\left(\frac{\Delta}{4}\right)$

M = Middle Ordinate = $R \times \left(1 - \cos\left(\frac{\Delta}{2}\right)\right)$

L = Length of Curve = $\frac{\Delta \times R}{57.2958} = \frac{100 \times \Delta}{D}$

Figure 5-2 Circular Curves

SUPERELEVATION

Introducing superelevation permits a vehicle to travel through a curve more safely and at a higher speed than would be possible with a normal crown section. The rate of superelevation generally increases with speed and sharper curvature.

Axis of Rotation

In the design of superelevation, it is necessary to select a point on the cross section around which the cross slope will be rotated to gradually change to the specified superelevation slope. The location of this point varies with the basic characteristics of the typical section.

Based on 12 ft (3.6 m) standard lanes, South Dakota has adopted the centerline as the rotation point for all undivided roadway sections. (For three or five lane roadways, the rotation point should be the same as the crown point.) For divided highways, the axis of rotation should typically be at the edge of the traffic lane nearest the median.

The designer should be aware of the impact superelevation has on ditch grades and the pavement cross slope in flat terrain. You could possibly end up with ponded areas on the inside of the curve and / or extended stretches of little or no cross slope on the outside driving lane of the curve. If this situation arises, the designer should review the vertical gradeline or the horizontal alignment to see if revisions to either could alleviate the problem. The superelevation Total Transition Lengths (TTL's) that are used by the SDDOT are from the current AASHTO publication *A Policy on Geometric Design of Highways and Streets*. They are based upon the maximum relative gradient which corresponds to the minimum recommended values for transition lengths. Thus, these values should not be reduced.

Development

Because of South Dakota's weather conditions, the maximum permissible rate of superelevation is 6%. The minimum permissible radius of curvature was established for each design speed based on this 6% maximum rate of superelevation. South Dakota has adopted a superelevation method based upon the current AASHTO publication *A Policy on Geometric Design of Highways and Streets* Method 5 for all rural roadways (including Interstate Ramps / Turning Roadways) and the current AASHTO publication *A Policy on Geometric Design of Highways and Streets* Method 2 for urban roadways (<u>for speeds of 45 mph (70kmh) or less</u>). Method 5 introduces superelevation (e) gradually as the friction factor (f) increases until both reach their maximum values at the minimum design curve radius for each design speed. In contrast, Method 2 utilizes all of the available (f) before utilizing any superelevation (e). Once the maximum (f) is reached, (e) is increased until it also reaches its maximum value at the minimum design curve radius for each design speed. The difference between the two methods can be seen by comparing Table 5-1 and Table 5-2. Both methods utilize a straight-line superelevation rate of change.

Rates of Superelevation

For low speed rural & urban designs and high speed designs, Table 5-1 Rural Low Speed, Table 5-2 Urban Low Speed and Table 5-3 High Speed provide the basic design criteria for rates of superelevation (**e**) (measured in %) and Total Transition Length **(TTL)** (measured in feet or meters) as related to a selected design speed and curve radius. The minimum radius for each design speed is also shown. These tables are based on a maximum superelevation rate of 6%. In unusual situations, a superelevation rate less than the recommended value may be necessary for a given design speed. For these instances, the following formulas can be used to determine the Minimum Radius:

English
$R_{min} = V^2 / (15 \text{ x } ((e_{max} \text{x} 0.01) + f_{max}))$

Metric
$R_{min} = V^2 / (127 \text{ x } ((e_{max} \text{x} 0.01) + f_{max}))$

Rmin - curve radius (ft or m)
e_{max} - superelevation rate (%)
f_{max} - friction factor (dimensionless)
V^2 - vehicle speed (mph) (kph)

The current AASHTO publication *A Policy on Geometric Design of Highways and Streets* provides recommendations on values to use for the side friction factor.

Superelevation Transition

Total Transition Length (TTL) is the general term denoting the cross-slope transition from a section with normal crown to a fully superelevated section. The TTL requires careful design to ensure a safe, comfortable and aesthetic product. To meet the requirements of comfort and safety, superelevation should be introduced and removed uniformly over a length adequate for the proposed travel speeds.

TTL = TR + L (See Figures 5-3 & 5-4)

TTL	- Total Transition Length	L	- Runoff
TR	- Tangent Runout	e	- Superelevation Rate

For the following discussion, a -2% (-0.02 ft / ft) cross slope refers to centerline sloping downward to the edge of pavement. A +2% (+0.02 ft / ft) would refer to the opposite condition, the edge of pavement sloping downward to the centerline.

TR is the distance required to remove the crown slope of one side of the roadway to a level cross slope, while the opposite side maintains the normal cross slope. (Example: If the curve is to the right, the left side of the roadway would transition from a -2% cross slope to a level cross slope, while the right side would maintain the -2% cross slope.)

The length TR occurs within the tangent section of the roadway.

L is the distance required to transition the roadway section from the position of crown removed (where it is after TR) to full superelevation.

Over the distance L, full superelevation is attained in two parts. First, the level roadway section attained in TR, rotates to a cross slope of +2% to match the opposite side of the road. (This point is referred to as Remove Adverse Crown - (RC)) (Example: If the curve is to the right, the left side of the roadway will transition from a level cross slope to a +2% slope, while the right side would maintain the -2% cross slope.) Second, the cross slope for both sides of the roadway will transition from the +2% (or -2%) cross slope to the full superelevation rate for the curve. (See Figure 5-4)

The TTL is applied 20% on the curve itself and 80% on the tangent section adjacent to the curve. This is a compromise between placing all of the TTL on the tangent section (where superelevation is not needed) and placing all of the TTL on the curve (where full superelevation is needed). Thus, full superelevation is not reached until past the P.C. and starts to reduce before reaching the P.T. Based upon Design Speed, the Curve Radius and project characteristics (rural or urban), the length TTL can be selected from Table 5-1, 5-2 or 5-3.

Very flat curves not requiring superelevation will use a normal crown (NC) section through the curve. Slightly sharper curves may need to remove the adverse crown on the outside lane(s) and be superelevated for the full width at the normal crown slope. These curves correspond to the NC and RC rows in the superelevation tables, respectively. (For South Dakota, RC is the same as a 2% superelevation rate.) For all curve radii less than the NC row, but greater than the RC row in each table a 2.0% (RC) superelevation rate should be used.

When spiral transition curves are used, the superelevation transition is usually coincident with the spiral length (T.S. to S.C. or C.S. to S.T.) and the designated full superelevation is provided between the S.C. and the C.S. The geometrics for spiral curves provide for a natural introduction of superelevation without the compromise necessary for circular curves. However, if the spiral length is less than the needed superelevation transition length, some of the superelevation may need to be accomplished either on the tangent section or on the simple curve to allow for the needed superelevation transition length. The application of superelevation for spirals is illustrated in Figure 5-3. (See the current AASHTO publication *A Policy on Geometric Design of Highways and Streets* for more information on Spiral Curves.)

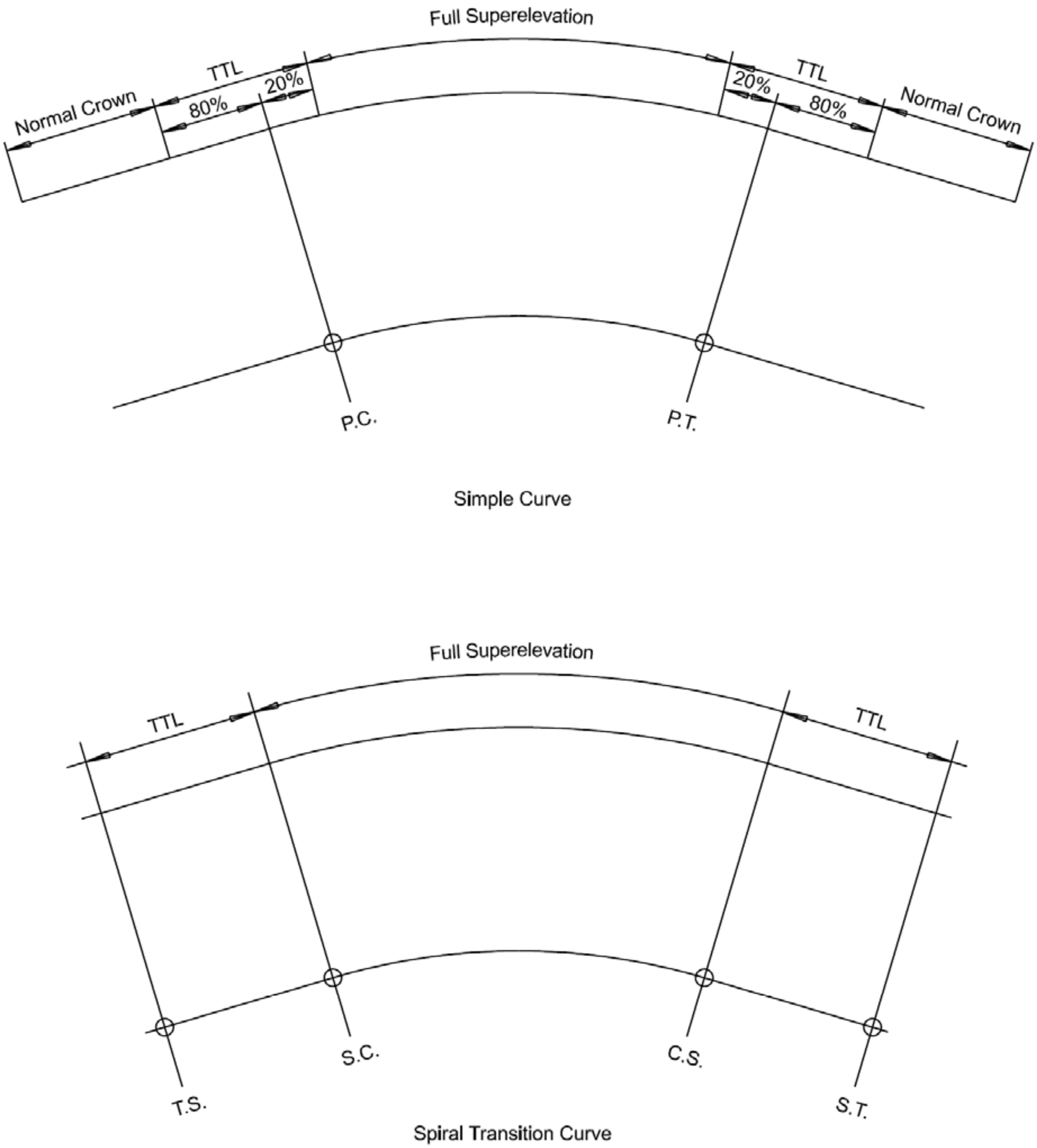


Figure 5-3 Superelevation Transitions

Superelevation Sample Calculation

Below is a sample calculation of how the SDDOT calculates superelevation for a simple horizontal curve.

KNOWN

- Vd = 70 MPH	- PC = 311+31.80	- R = 2864.79
- 2 Lane Roadway	- PT = 325+20.34	- Delta = 27°46’15” R

Vd - Design Speed (MPH)
R - Curve Radius (ft)
Delta - Deflection Angle / Direction

- From Table 5-3 High Speed: e = 5.6% (0.056 ft / ft) and TTL = 228 ft

Based Upon (20%) TTL on curve & (80%) TTL on tangent:

Begin Transition	(BT)	= PC – (0.80)(228 ft) = 311+31.80 – 182.4	= 309+49.40
Begin Full Superelevation	(BFS)	= PC + (0.20)(228 ft) = 311+31.80 + 45.6	= 311+77.40
End Full Superelevation	(EFS)	= PT – (0.20)(228 ft) = 325+20.34 – 45.6	= 324+74.74
End Transition	(ET)	= PT + (0.80)(228 ft) = 325+20.34 + 182.4	= 327+02.74

Thus, between Sta. 311+77.40 & Sta. 324+74.74, the curve is at the full superelevation rate of 0.056.

Though not typically necessary or important, if you want to calculate the location of the end of TR or RC, the following formulas can be used:

TR Location = (0.02 / (0.02 +e)) x TTL + BT
= (0.02 / (0.02 + 0.056)) x 228 + 309+49.40 = 310+09.40

or

TR Location = ET - (0.02 / (0.02 +e)) x TTL
= 327+02.74 - (0.02 / (0.02 + 0.056)) x 228 = 326+42.74

For this example, at these stations, the left side of the roadway would be at a level cross slope while the right side of the roadway would still be at the normal 0.02 cross slope.

RC Location = (0.04 / (0.02 +e)) x TTL + BT
= (0.04 / (0.02 + 0.056)) x 228 + 309+49.40 = 310+69.40

or

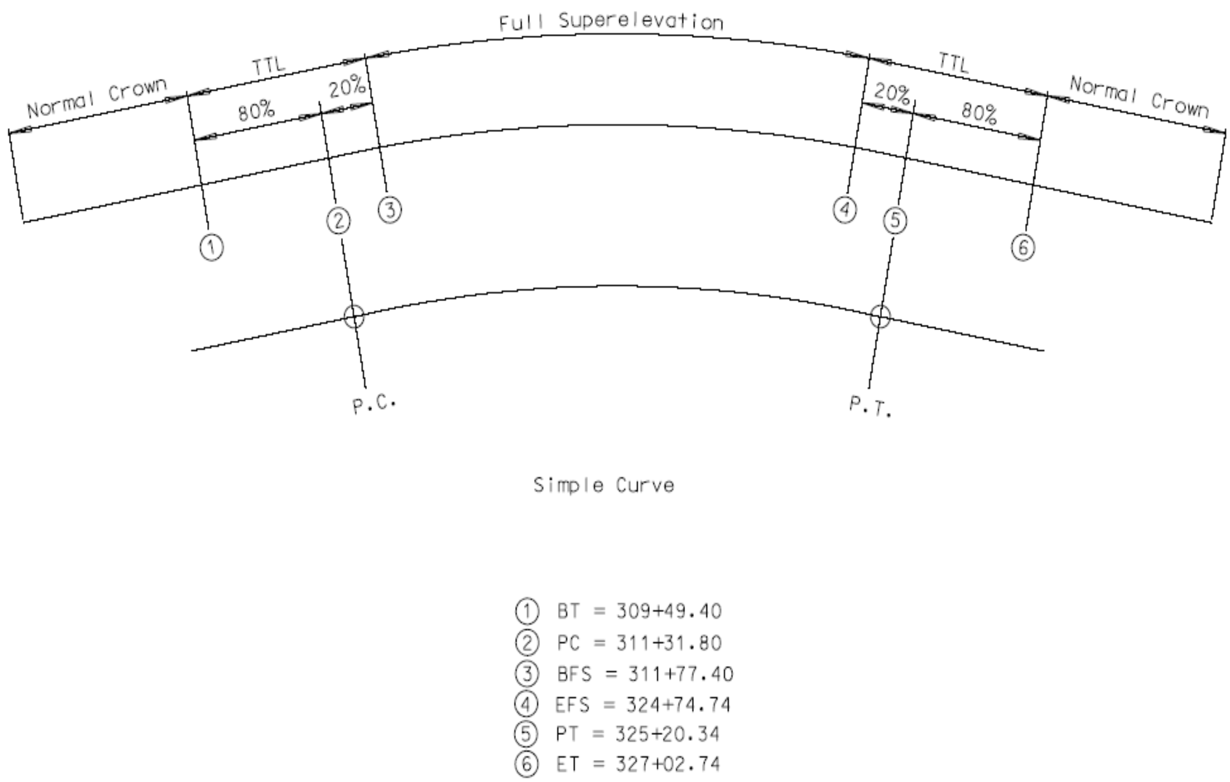
RC Location = ET - (0.04 / (0.02 +e)) x TTL

= 327+02.74 - (0.04 / (0.02 + 0.056)) x 228 = 325+82.74

For this example, at these stations, the left side of the roadway would be at a 0.02 cross slope to match the right side of the roadway. (At this point, we have a 0.02 superelevation.)

Note: Since the axis of rotation is the centerline for a 2 lane roadway, the vertical elevations shown on the vertical profile will be correct.

Figure 5-4 Sample Calculation



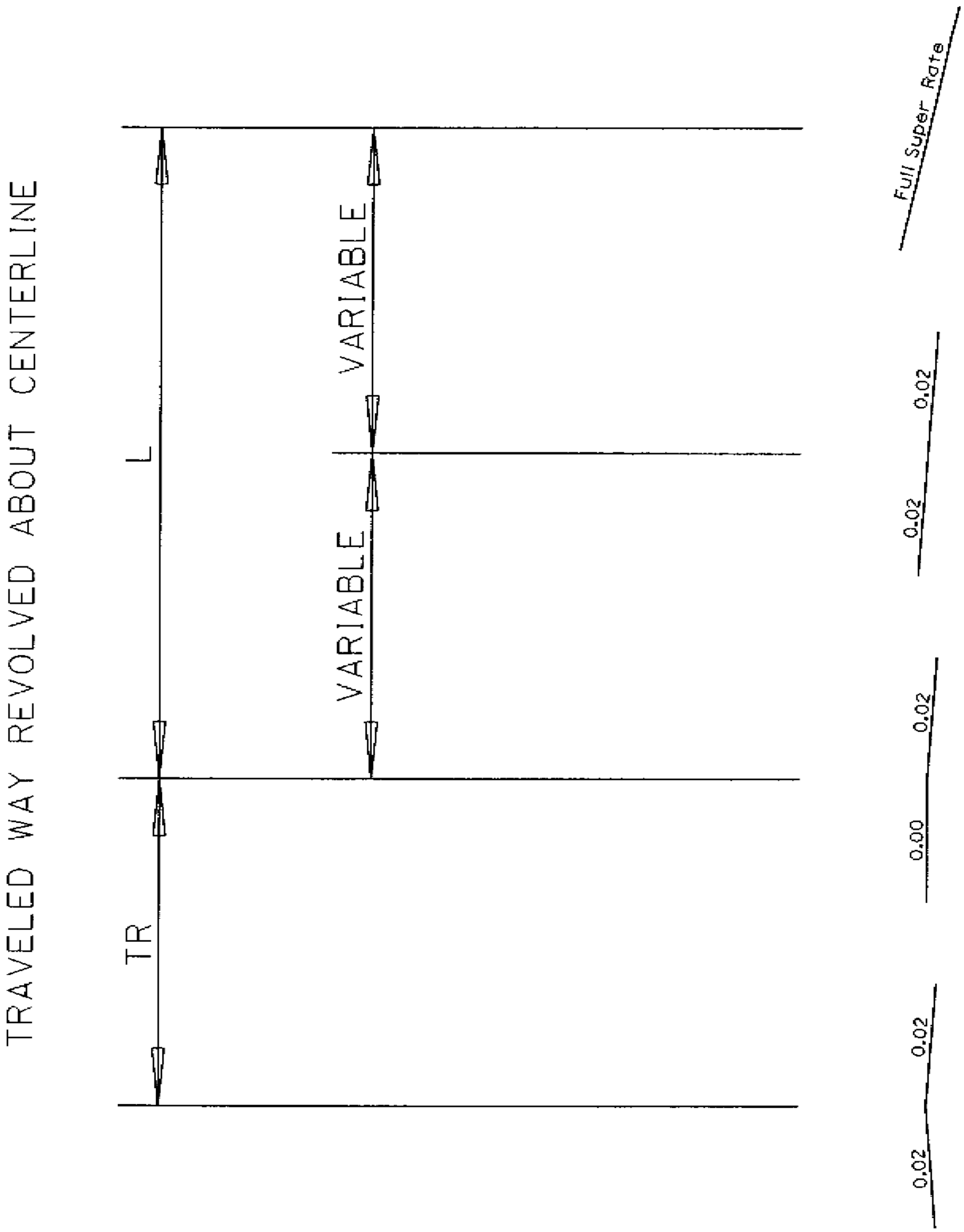


Figure 5-5 Superelevation Transition for 2 Lane Highway

<u>Design Speed (MPH) / Total Transition Length (2 Lane Roadways)</u>

e	25	TTL	30	TTL	35	TTL	40	TTL	45	TTL
(%)	Rad. (ft)	(ft)	Rad. (ft)	(ft)	Rad. (ft)	(ft)	Rad. (ft)	(ft)	Rad. (ft)	(ft)
NC	2290	0	3130	0	4100	0	5230	0	6480	0
RC	1630	68	2240	72	2950	78	3770	82	4680	88
2.2	1450	72	2000	76	2630	82	3370	87	4190	93
2.4	1300	75	1790	80	2360	85	3030	91	3770	97
2.6	1170	79	1610	83	2130	89	2740	95	3420	102
2.8	1050	82	1460	87	1930	93	2490	99	3110	106
3.0	944	85	1320	91	1760	97	2270	103	2840	111
3.2	850	89	1200	94	1600	101	2080	107	2600	115
3.4	761	92	1080	98	1460	105	1900	111	2390	120
3.6	673	96	972	101	1320	109	1740	115	2190	124
3.8	583	99	864	105	1190	113	1590	120	2010	128
4.0	511	103	766	109	1070	116	1440	124	1840	133
4.2	452	106	684	112	960	120	1310	128	1680	137
4.4	402	109	615	116	868	124	1190	132	1540	142
4.6	360	113	555	120	788	128	1090	136	1410	146
4.8	324	116	502	123	718	132	995	140	1300	151
5.0	292	120	456	127	654	136	911	144	1190	155
5.2	264	123	413	131	595	140	833	149	1090	160
5.4	237	127	373	134	540	144	759	153	995	164
5.6	212	130	335	138	487	147	687	157	903	168
5.8	186	133	296	141	431	151	611	161	806	173
6.0	144	137	231	145	340	155	525	165	643	177

Table 5-1 Rural Low Speed

(English) Minimum Radii for Design Superelevation Rates, Design Speeds and e_{max} = 6%

Notes –

- The top row identified with NC (Normal Crown) is the minimum curve radius allowed for each design speed without superelevation.
- The second row identified with RC (Remove Adverse Crown) is the minimum curve radius allowed for each design speed with a 2.0% rate of superelevation.
- The bottom row identified with e=6.0% is the minimum curve radius allowed for each design speed with the maximum 6% superelevation rate.
- Total Transition Lengths (TTL's) listed are for typical two lane roadways rotated around the centerline. See "Multi-Lane Superelevation Transition Lengths" for information on transition lengths for roadways with more than one lane rotated around the pivot point.

Design Speed (MPH) / Total Transition Length (2 Lane Roadways)

e	25	TTL	30	TTL	35	TTL	40	TTL	45	TTL
(%)	Rad. (ft)	(ft)	Rad. (ft)	(ft)	Rad. (ft)	(ft)	Rad. (ft)	(ft)	Rad. (ft)	(ft)
NC	198	0	333	0	510	0	762	0	1039	0
RC	167	68	273	72	408	78	593	82	794	88
2.2	165	72	270	76	404	82	586	87	785	93
2.4	164	75	268	80	400	85	580	91	776	97
2.6	163	79	265	83	396	89	573	95	767	102
2.8	161	82	263	87	393	93	567	99	758	106
3.0	160	85	261	91	389	97	561	103	750	111
3.2	159	89	259	94	385	101	556	107	742	115
3.4	158	92	256	98	382	105	550	111	734	120
3.6	157	96	254	101	378	109	544	115	726	124
3.8	155	99	252	105	375	113	539	120	718	128
4.0	154	103	250	109	371	116	533	124	711	133
xxxxx	xxxxx	xxxxx	xxxxx	xxxxx	xxxxx	xxxxx	xxxxx	xxxxx	xxxxx	Xxxxx
4.2	153	106	248	112	368	120	528	128	703	137
4.4	152	109	246	116	365	124	523	132	696	142
4.6	151	113	244	120	361	128	518	136	689	146
4.8	150	116	242	123	358	132	513	140	682	151
5.0	149	120	240	127	355	136	508	144	675	155
5.2	148	123	238	131	352	140	503	149	668	160
5.4	147	127	236	134	349	144	498	153	662	164
5.6	146	130	234	138	346	147	494	157	655	168
5.8	145	133	233	141	343	151	489	161	649	173
6.0	144	137	231	145	340	155	485	165	643	177

Table 5-2 Urban Low Speed

(English) Minimum Radii for Design Superelevation Rates, Design Speeds and e_{max} = 6%

Notes –

- Though allowed based upon the table above, superelevation rates greater than 4% should be avoided, if possible, and for most situations a 2% maximum rate is preferred.
- The top row identified with NC (Normal Crown) is the minimum curve radius allowed for each design speed without superelevation.
- The second row identified with RC (Remove Adverse Crown) is the minimum curve radius allowed for each design speed with a 2.0% rate of superelevation.
- The bottom row identified with e=6.0% is the minimum curve radius allowed for each design speed with the maximum 6% superelevation rate.
- Total Transition Lengths (TTL's) listed are for typical two lane roadways rotated around the centerline. See "Multi-Lane Superelevation Transition Lengths" for information on transition lengths for roadways with more than one lane rotated around the pivot point.

Design Speed (MPH) / Total Transition Length (2 Lane Roadways)

e	50	TTL	55	TTL	60	TTL	65	TTL	70	TTL	75	TTL	80	TTL
(%)	Rad. (ft)	(ft)	Rad. (ft)	(ft)	Rad. (ft)	(ft)	Rad. (ft)	(ft)	Rad. (ft)	(ft)	Rad. (ft)	(ft)	Rad. (ft)	(ft)
NC	7870	0	9410	0	11100	0	12600	0	14100	0	15700	0	17400	0
RC	5700	96	6820	102	8060	106	9130	112	10300	120	11500	126	12900	138
2.2	5100	101	6110	107	7230	112	8200	117	9240	126	10400	132	11600	144
2.4	4600	106	5520	112	6540	117	7430	123	8380	132	9420	139	10600	151
2.6	4170	110	5020	117	5950	122	6770	129	7660	138	8620	145	9670	158
2.8	3800	115	4580	122	5440	128	6200	134	7030	144	7930	151	8910	165
3.0	3480	120	4200	128	4990	133	5710	140	6490	150	7330	158	8260	172
3.2	3200	125	3860	133	4600	138	5280	145	6010	156	6810	164	7680	179
3.4	2940	130	3560	138	4250	144	4890	151	5580	162	6340	170	7180	186
3.6	2710	134	3290	143	3940	149	4540	156	5210	168	5930	177	6720	192
3.8	2490	139	3040	148	3650	154	4230	162	4860	174	5560	183	6320	199
4.0	2300	144	2810	153	3390	160	3950	168	4550	180	5220	189	5950	206
4.2	2110	149	2590	158	3140	165	3680	173	4270	186	4910	196	5620	213
4.4	1940	154	2400	163	2920	170	3440	179	4010	192	4630	202	5320	220
4.6	1780	158	2210	168	2710	176	3220	184	3770	198	4380	208	5040	227
4.8	1640	163	2050	174	2510	181	3000	190	3550	204	4140	215	4790	234
5.0	1510	168	1890	179	2330	186	2800	196	3330	210	3910	221	4550	240
5.2	1390	173	1750	184	2160	192	2610	201	3120	216	3690	227	4320	247
5.4	1280	178	1610	189	1990	197	2420	207	2910	222	3460	234	4090	254
5.6	1160	182	1470	194	1830	202	2230	212	2700	228	3230	240	3840	261
5.8	1040	187	1320	199	1650	208	2020	218	2460	234	2970	246	3560	268
6.0	833	192	1060	204	1330	213	1660	223	2040	240	2500	252	3050	275

Table 5-3 High Speed

(English) Minimum Radii for Design Superelevation Rates, Design Speeds and $e_{max} = 6\%$

Notes –

- The top row identified with NC (Normal Crown) is the minimum curve radius allowed for each design speed without superelevation.
- The second row identified with RC (Remove Adverse Crown) is the minimum curve radius allowed for each design speed with a 2.0% rate of superelevation.
- The bottom row identified with e=6.0% is the minimum curve radius allowed for each design speed with the maximum 6% superelevation rate.
- Total Transition Lengths (TTL's) listed are for typical two lane roadways rotated around the centerline. See "Multi-Lane Superelevation Transition Lengths" for information on transition lengths for roadways with more than one lane rotated around the pivot point.

3R Projects

The maximum superelevation rate for 3R projects is 7.0% for roadways classified as "Non-Interstate National Highway System" and for roadways that fall within the "Other Than National Highway System" classification.

Urban Superelevation

Several methods exist to distribute superelevation and side friction in determining the minimum radius of curve. A different method (Method 2) is used for low-speed roads located in urban areas, than for low-speed rural or high-speed roads (Method 5). The practical benefit of the different distribution methods is that superelevation in built-up urban areas is minimized, allowing for a smooth intersection design and accommodation of adjacent property entrances. On highways with design speeds of 50 mph or higher, this rationale is not applicable, and urban highways of this design speed should be superelevated in the same manner as rural highways.

The choice to incorporate urban (based upon Method 2) or rural (based upon Method 5) superelevation distribution methods shall be made on a project by project basis by the Designer and as approved by their Supervisor. The decision should be based upon the characteristics of the adjacent land (development) and design constraints of the project site.

The rationale for the distinction between the two methods is that drivers, through conditioning, have developed a higher threshold of discomfort on low-speed urban streets when reacting to the outward pull of centrifugal force on horizontal curves than on high-speed roads.

The different distribution for low-speed urban highways (design speed of 45 mph and below) is based upon a different distribution of friction and superelevation and is briefly described as follows:

Side friction is such that a vehicle traveling at design speed has all centrifugal force counteracted by side friction on curves up to that requiring f_{max}. For sharper curves, f remains at f_{max} and e is used in direct proportion to the continued increase in curvature until e reaches e_{max}.

Because greater driver discomfort is allowed with this superelevation distribution, the minimum radius allowed for each of these design speeds in an urban situation with a Normal Crown is much less than the values for rural locations. (30 mph – 333' vs. 3130', 35 mph – 510' vs. 4100', 40 mph – 762' vs. 5230') But, it is also noted that the minimum radii for each design speed at the maximum superelevation rate for both rural and urban designs are equivalent.

Though allowed based upon Table 5-2 Urban Low Speed, superelevation rates greater than 0.04 ft / ft should be avoided, if possible, and for most situations a 0.02 ft / ft maximum rate is preferred.

Thus, rates in gray shown below the “xxxxx” area within Table 5-2 (rates greater than 4.0%) should typically not be used. The Designer shall only use rates greater than 4% in an urban situation with approval from the Chief Road Design Engineer.
The designer should note the differences between the rural and urban superelevation rates shown in Tables 5-1 Rural Low Speed and 5-2 Urban Low Speed, respectively. For example, for a design speed of 40 mph and a design radius of 600 ft, the rural chart shows “6.0” and the urban chart shows “2.0”.

Though Table 5-2 Urban Low Speed is recommended for urban situations, the designer may have a situation that merits consideration of a superelevation rate that is greater than the value shown in the urban table. A common example would be for the 40 mph design speed with a 600 ft radius curve discussed in the previous paragraph. Instead of using a 2.0% superelevation rate as recommended by the urban table, for a specific situation, a 4.0% rate may be more appropriate.

If a rate other than the value recommended in Table 5-2 Urban Low Speed is used for an urban situation, the designer shall provide documentation for their decision. The rate may not exceed the value shown in Table 5-1 Rural Low Speed.

For some existing sharp radii curves that do not meet the proposed design speed, reconstruction may not be feasible due to design constraints such as surrounding development. In situations such as these, a design exception will be necessary. These curves would be tested by Region Traffic with a ball bank indicator and may result in posting of advisory speed plates.

Design Speeds of Less than 30 MPH

For urban streets with design speeds of less than 30 mph, it is rarely necessary to introduce superelevation. Most of the time, the side friction factor (f) is adequate. If the maximum allowable f is exceeded, the following factors must be considered before adding superelevation:

1. Superelevation is not used on curves if it would increase drainage problems.

2. Grades of the surrounding properties, entrances and cross streets must be met without introducing grades on the main roadway which exceed the maximum longitudinal grade.

3. There must be sufficient area to properly transition to and from the desired superelevation.

4. The use of superelevation and the attainable design speed must be weighed against the construction effort and the local municipal practice. Curves that do not meet design speed criteria should be investigated.

   For example, instead of introducing a 6% superelevation rate at a curve, a reduced rate or no superelevation with a speed advisory sign may be more appropriate for some situations.

<u>Multi-Lane Superelevation Transition Lengths</u>

The indicated superelevation rates in Tables 5-1, 5-2 and 5-3 are applicable regardless of the number of lanes. But, multi-lane roadways require longer TTL's than for 2-lane highways. Based upon maximum relative gradients, the TTL's for 4-lane and 6-lane roadways would be 2 and 3 times, respectively, those for a 2-lane roadway. Though desirable, it is frequently not feasible to provide transition lengths of this magnitude.

On a purely empirical basis, the current AASHTO publication *A Policy on Geometric Design of Highways and Streets* recommends that the minimum runoff length L for undivided multi-lane pavements should be adjusted / increased based upon the table shown below. (For South Dakota design practices, these factors should be applied to the TTL.) Example – For a 5-lane undivided pavement with the pivot point offset 6 feet from centerline, 3.0 lanes are rotated on one side. Thus, the recommended TTL would be twice that of a 2-lane pavement.

Number of Lanes Rotated	TTL Length Increase *
1.0	1
1.5	1.25
2.0	1.50
2.5	1.75
3.0	2.0

* Relative to 1 Lane Rotated.

<u>Proper design attention to obtain smooth edge profiles and to avoid distorted appearances may suggest lengths greater than these values.</u>

For divided 4-lane roadways (with typical centerline to centerline widths of 80' to 96'), the axis of rotation used shall typically be the 12 ft (3.6 m) inside edge of the driving lane nearest to the median for both sets of lanes. Due to this, 4-lane TTL's are recommended.

With medians narrower than those described above, problems may arise with the ability to maintain a functional median ditch section. In these situations, different rotation points may be considered. Instead of using the inside edge of the driving lane as described above, alternatives could be to use the inside finished shoulder points or the inside subgrade shoulder points. Using either of these alternatives for the rotation points should only be used in special situations and with the approval of the Chief Road Design Engineer.

Because a wider pavement width would be rotated with either of these situations, the profile gradient (in association with the recommended increase for relative number of lanes rotated) must also be checked to ensure that the superelevation transition lengths are sufficient. For example, if the rotation point is the inside shoulder point (6' width), then an equivalent of 2.5 lanes would be rotated. The corresponding recommended TTL would be 1.75 times that of one lane rotated.

There are three general cases for axis of rotation with a Median:

Case 1 – Narrow Width Medians up to 20 ft (6.1 m) with rotation about the centerline (or crown point offset 6'). Common examples – 12' (3.6 m) Flush Median (Dual Center Turn Lane) or 20 ft (6.1 m) raised curb & gutter median.

Case 2 – Intermediate Width Medians of 20 ft to 72 ft (6.1 m to 22.0 m) with rotation about the centerline or inside edge of the driving lane nearest the median. For the lower part of the range, rotation about the centerline would be most common. For the upper part of the range, rotation about the inside edge of the driving lane nearest the median would be most common. Common examples – 20 ft (6.1 m) to 32' (9.8 m) raised curb & gutter median (centerline rotation) and 60 ft to 72 ft (18.3 to 22.0 m) median width Interstate Highways / Expressways with a depressed grass median (rotation about the inside edge of the driving lane nearest the median).

> For Case 2, the selection of rotation point is dependent upon median width & type and should be made on a project specific basis. Example – For wide pavement sections (typically greater than two thru lanes in each direction) with a barrier curb & gutter median of 20' to 32', rotation about the centerline can lead to relatively long superelevation transitions (and the possibility of poor pavement drainage) as well as a significant elevation difference between pavement edges. Conversely, with the same situation, rotation about the median edges can lead to a significant elevation difference within the median in left turn lane locations and would create problems at intersections. In this example, if rotation about the median edges is used, a slightly wider median may be appropriate to improve the median slope.

Case 3 – Wide Medians greater than 72 ft (22.0 m) with independent roadways and rotation about the centerline of each roadway. Common example – Interstate Highways / Expressways with separated alignments due to terrain.

In comparison to the current AASHTO publication *A Policy on Geometric Design of Highways and Streets*, the SDDOT has slightly modified the median width ranges and case descriptions that are noted above. The intent of these Case definitions is to provide general guidance for the selection of rotation point. For example, utilizing Case 2 for a median width of 84 ft (within the Case 3 range) may be deemed appropriate for an Interstate or Expressway project.

Note: Because the axis of rotation for 4-lane divided roadways is typically the inside edge of the driving lane nearest to the median for both sets of lanes, the centerline elevations in superelevated curves will be higher than those shown on the vertical profile for the outside lanes and lower for the inside lanes.

Figure 5-6 depicts a superelevation transition with the rotation point located at the median edges <u>and both lanes sloping outward</u>. For additional examples, see the section "Transition Design Controls" in Chapter 3 of the current AASHTO publication *A Policy on Geometric Design of Highways and Streets.*

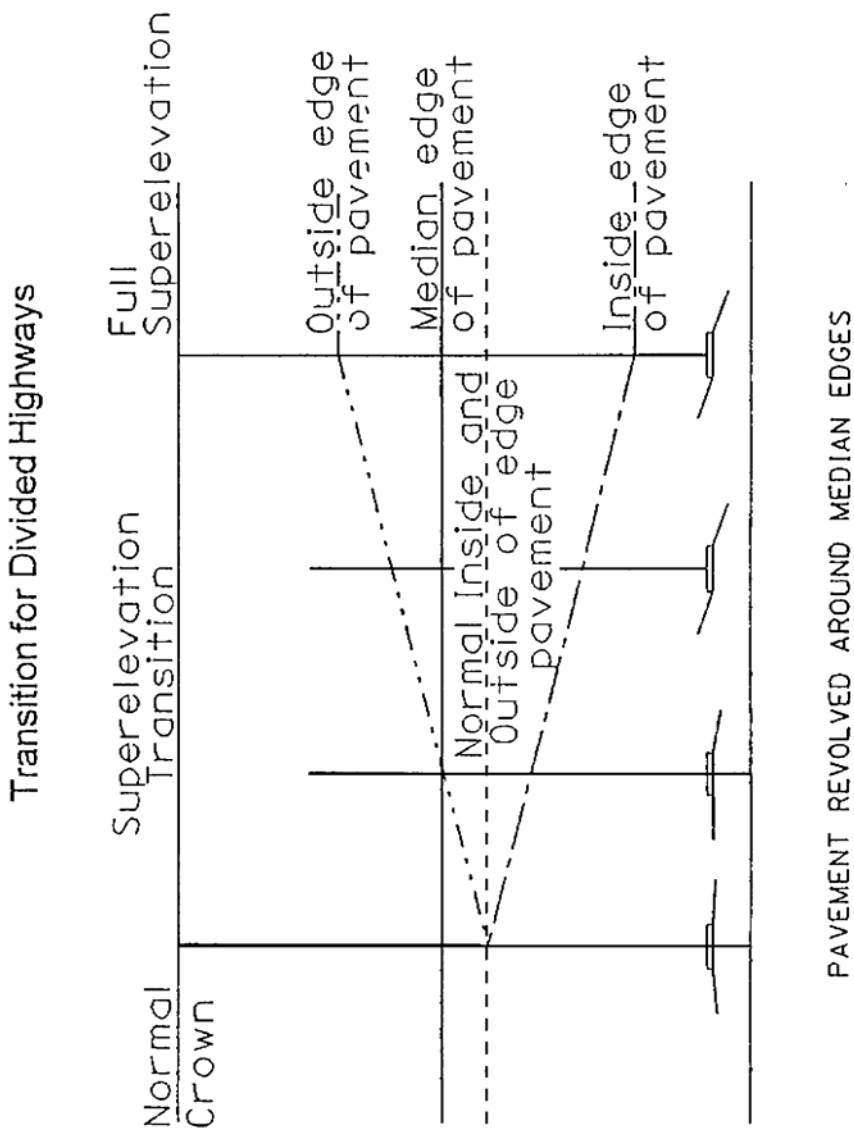


Figure 5-6 Superelevation Transition for Divided Highways

<u>Minimal Tangent Distance Between Curves (Mountainous Terrain)</u>

There may be instances when short tangency between horizontal curves doesn't allow for the typical design application of placing 80% of the TTL on the tangent. Separation distances less than 300 ft (90 m) between the P.C. and P.T. of two adjacent curves will frequently require something other than the normal superelevation transitions that are done when there is a long tangent separation distance. Curves with a separation of 300 ft to 500 ft (90 m to 150 m) should also be reviewed to determine if the normal transition is appropriate.

Below are some options available to the designer to accommodate such conditions.

A decision to use any of these options should be made on a case by case basis and with the approval of the Chief Road Design Engineer. The distance ranges listed both above and below are approximate and intended as guidelines only. Design speed and superelevation rate have a direct impact upon the required superelevation transition length, and subsequently, the transition option that is selected.

1. Place a higher percentage of the TTL onto the curve, but ensuring that 50% of the curve length remains in full super.

2. Maintain a planar section between curves and don't return to normal crown.

3. Move the horizontal PI's or sharpen the curves.

If "broken-back" curves (short tangent sections between two curves in the same direction) are unavoidable, the designer may employ one of the following options:

A. Direct transition from the full superelevation of the 1st curve to the full superelevation of the 2nd curve. This should be considered with separation distances that are less than 200 ft (60 m).

B. Transition of the 1st curve from full superelevation to an 0.02 ft / ft superelevation, hold that rate for 100 ft to 300 ft (30 m to 90 m) and then transition to the full superelevation of the 2nd curve. This should be considered with separation distances that are between 200 ft (60 m) and 400 ft (120 m).

C. Transition of the 1st curve from full superelevation to a normal crown and then back to full superelevation for the 2nd curve. This should be considered with separation distances that are greater than 400 ft (120 m).

The designer should also be aware of and consider Options 1, 2 and 3 listed above when working with Options A, B and C for "broken-back" curves. For example, Option 1 may be used in conjunction with Options B and C for "broken-back" curves.

Every effort should be made to provide the prescribed superelevation rate on curves. When this is not practicable, advisory speed signs should be provided indicating the maximum safe speed for the curve with existing superelevation. The change in design speed must be handled with a "Design Exception" and must be approved by the Chief Road Design Engineer.

As noted previously, drawing centerline and edge of driving lane profiles and / or subgrade contours can be helpful aids to review the smoothness of edge profiles as well as potential areas of poor drainage.

<u>TTL Sample Calculation (2 Lane)</u> –

50 MPH Design Speed
Curve = Minimum Radius for Design Speed = 835'
0.06 Superelevation Rate
Net Cross Slope Change = 0.02 + 0.06 = 0.08
From Table 5-4 – Gradient = .50% = .005

TTL = (lane width x net cross slope change) / Gradient

TTL = (12 x 0.08) / .005 = 192 Feet

This value can be found in Table 5-3 High Speed.

Table 5-4
Maximum Relative Gradient

Design Speed Vd (MPH)	Maximum Relative Gradients (and Equivalent Maximum Relative Slopes) for Profiles Between the Edge of Two-Lane Pavement and the Centerline (Percent)
30	0.66 (1:152)
35	0.62 (1:161)
40	0.58 (1:172)
45	0.54 (1:185)
50	0.50 (1:200)
55	0.47 (1:213)
60	0.45 (1:222)
65	0.43 (1:233)
70	0.40 (1:250)
75	0.38 (1:263)
80	0.35 (1:286)

Traffic Diversion Alignment

See Chapter 16 – Miscellaneous for information on horizontal alignment for traffic diversions.