Chapter Five

Alignment and Superelevation

A facility's horizontal and vertical alignments establish the general character of a roadway, perhaps more than any other design consideration. The configuration of line and grades affects safe operating speeds, sight distances, opportunities for passing, and highway capacity. Decisions on alignment have a significant impact on construction costs, and social and environmental issues. Alignment is defined by several factors, including: the length of tangent sections; the transition into horizontal curves; the degree of curvature (radius) for horizontal curves; the transition out of the curves; the rate of superelevation applied to the horizontal curves; and the rate of grade change for any vertical curves.

Basic design controls and standards related to alignment were presented in Chapters Two and Three, including criteria for design speed, maximum curvature, superelevation and sight distances. This chapter provides more detailed explanations and discusses practical application of the criteria.

5.1 HORIZONTAL ALIGNMENT

Horizontal alignment of a roadway is defined graphically using a series of straight-line tangents with transition sections into and out of horizontal curves. Many factors, including terrain conditions, physical features and right-of-way considerations, affect the design of tangent and curve sections. The two types of curves used in designing horizontal alignments are simple circular curves (a single constant radius) and compound curves (a series of symmetrical or asymmetrical radii). The two methods of transitioning from a tangent section to full curvature are tangent-to-curve and spiral

For most horizontal alignment designs, the Department has adopted the use of simple circular curves and the tangent-to-curve transition method. Placement of the superelevation runoff length is an important design consideration when using the tangent-to-curve transition method. The location where the runoff length begins and ends has an effect on a vehicle's lateral velocity and motion. The result can be operational problems due to driver tendency to over steer to compensate for the increase in side friction; see Section 5.3.3 for more detailed discussion.

5.1.1 GENERAL CRITERIA

Design speed is the principal factor controlling horizontal alignment design. Several geometric standards related to design speed are very specific. Other criteria cannot be defined as specifically and require that judgments be made in consideration of local conditions. The following guidelines outline some of these decisions.

- Alignment should be consistent and as directional as possible. 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 curves should not be introduced at the ends of long tangents or flat curves. Sudden changes from areas of slight curvature to sharp curvature should be avoided. Where sharp curvature must be introduced, it should be gradually approached by successively sharper curves.

- Avoid the use of the minimum curve radii shown in Figures 5-1 and 5-2. Generally, curves should be as flat as practicable for the conditions. On the other hand, avoid the use of excessively long, flat curves on two-lane highways where considerable passing opportunities are needed. Many 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 radii of deflection angle of 0.25^{o} or less. Curves with a small deflection angle should be long enough to avoid the appearance of a "kink."
- The minimum length of horizontal curves on primary roadways should be about 15 times the design speed in mph [3 times the design speed in km/h]. On high-speed controlled access highways, a desirable minimum length of curve would be about 30 times the design speed in mph [6 times the design speed in km/h].
- 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 reverse curve alignments by providing enough tangent distance between the curves to ensure adequate superelevation transition for both curves and sufficient distance for adequate signing.
- 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 a tangent distance 500 to 1500 ft [150 to 450 m] between the curves, depending on the design speed.
- 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, shrubs and trees above the roadway, it is difficult for drivers to perceive the extent of curvature and adjust their operation to the conditions.

It is desirable that bridges be located on tangent sections of the alignment. When it is necessary to locate a bridge on a curve, 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. Where 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.

5.1.2 CONTROL LINE LOCATIONS

The relationships between the construction baseline and the profile grade must be clearly identified on the typical section sheets. The following general criteria should apply:

1. For two-lane roads and undivided multi-lane roads, the horizontal and vertical controls should be located at the centerline of the typical section.
2. For multi-lane divided highways with relatively narrow medians, the horizontal control should be at the centerline of the median, and a single profile grade should be located at the median edge of pavements for both traveled ways.

Figure 5-1
Minimum Radius for
Open Highway Conditions and
Superelevation Rate of 4%

US Customary		Metric	
Design Speed (mph)	Radius (ft)	Design Speed [km/h]	Radius [m]
15	42	20	8
20	86	30	22
25	154	40	47
30	250	50	86
35	371	60	135
40	533	70	203
45	711	80	280
50	926	90	375
55	1190	100	492
60	1500		

Figure 5-2
Minimum Radius for
Open Highway Conditions and
Superelevation Rate of 6%

US Customary		Metric	
Design Speed (mph)	Radius (ft)	Design Speed [km/h]	Radius [m]
15	39	20	8
20	81	30	21
25	144	40	43
30	231	50	79
35	340	60	123
40	485	70	184
45	643	80	252
50	833	90	336
55	1060	100	437
60	1330	110	560
65	1660	120	756
70	2040	130	951

3. For multi-lane divided highways with independent roadways or relatively wide medians, independent horizontal and vertical controls are established at the centerline of each roadway.

The relationships between these control line locations and the pivot points for superelevation of horizontal curvature are described in Section 5.3.

5.1.3 TYPES OF CURVES

The types of curves used in designing horizontal curvature may be simple circular curves, spiral transition curves or compound curves. Circular curves use a uniform radius for the entire distance between adjacent tangent sections. Spiral transition sections more closely replicate the vehicle and driver's behavior when entering a curve. They are introduced at each end of the circular curve to gradually ease the driver into and out of curves without a sharp break at the tangent sections. This is particularly noticeable with relatively sharp curves and higher vehicle operating speeds. Compound curves are most commonly used for turning roadways where it is necessary to fit the curve to the inside edge of the design vehicle's swept path. When the design speed of a turning roadway is 45 mph [70 km/h] or less, compound curvature can be used to form the entire alignment of the turning roadway. However, the exclusive use of compound curves can increase the right-of-way impacts.

Although circular curves are normally used in the design of Delaware roadways, using spiral transitions may be considered as described in the Green Book, pages 184 to 192.

Where spiral transition curves are to be used, right-of-way lines should not be defined as a spiral curve paralleling the centerline. Instead, the right-of-way should be described with a circular curve or compound circular curve of a similar shape. A practical guide for the length of a spiral is the length required for superelevation runoff.

5.1.4 SIGHT DISTANCE ON HORIZONTAL CURVES

An important element in ensuring driver safety and maintaining a roadway's operational efficiency is providing adequate sight distance—the length of roadway ahead visible to the driver. Sight distance applies to four conditions that may arise when setting a project's horizontal alignment:

(1) Is adequate distance available to stop?

(2) Is there adequate opportunity and length available for passing on two-lane roadways?

(3) Is there adequate distance for drivers to react when approaching complex decision points?

(4) Has the selected criteria for measuring these distances been applied to the selected design?

Providing adequate sight distance is also important in the design of intersections, in particular, those in rural areas. These locations tend to be less safe than urban ones, primarily because of higher speeds and lack of driver awareness. Providing at least the minimum sight distance will play an important role in reducing these occurrences.

5.1.4.1 STOPPING SIGHT DISTANCE

The designer must check sight distance across the inside of horizontal curves. Sight obstructions such as walls, concrete safety barriers, bridge parapets, cut slopes, vegetation and buildings may limit sight distance on curves. Where these obstructions cannot be removed or permanently controlled, adjustment in the normal cross section or a change in alignment may be required to provide and assure continuation of adequate sight distance. For areas within a project that may cause confusion or delay a driver's reaction time i.e. multiple decision points, it may be necessary to check the decision sight distance also.

Minimum stopping sight distance for each design speed is shown in Chapter Three-Design Standards. The sight line is a chord of the curve. The applicable stopping sight distance is checked by measuring along the centerline of the inside lane around the curve. See the Green Book, pages 224-228 for the design and evaluation of stopping sight distances on horizontal curves. Horizontal sight distance is based on the formula:

$$HSO = R\left[\left(1 - \cos\frac{28.65\,S}{R}\right)\right]$$

Where:

S = Stopping sight distance, ft [m]
R = Radius of curve, ft [m]
HSO = Horizontal sightline offset, ft [m]

Where the obstruction is a cut slope on the inside of the curve, it is necessary to know the critical height of vegetation on the slope for measuring the middle ordinate distance. Because the height criteria for stopping sight distance are 3.5 ft [1,080 mm] for the eye and 2 ft [600 mm] for the object, a height of 2.75 ft [840 mm] may be assumed as the midpoint of the line of sight where the cut slope usually obstructs sight. In some cases, retaining walls, concrete median safety barriers, and other similar features constructed on the inside of curves may be sight obstructions and need to be checked for stopping sight distance.

Solutions to sight distance problems on horizontal curves might be removal of obstructions, flattening the curves and flattening or benching cut slopes. It should be kept in mind that stopping sight distances greater than the minimum should be used for design. Minimum stopping sight distance values may be used only if greater values cannot be obtained without undue costs. On new construction, the stopping sight distance at any location shall never be less than the minimum standard for stopping sight distance for the selected design speed. Designs for new construction and reconstruction projects that do not meet these standards must have a design exception approved by the Chief Engineer.

5.1.4.2 PASSING SIGHT DISTANCE

The minimum passing sight distance for a two-lane road is about four times greater than the minimum stopping sight distance at the same design speed. To provide the greater passing sight distance, clear sight areas on the insides of curves must be considerably wider. Often this is not practicable. It is necessary to acknowledge and accept no-passing zones.

Passing sight distance depends on the eye height of 3.5 ft [1,080 mm] and object height of 3.5 ft [1,080 mm]. The sight line to the center of the area inside a curve is about 0.75 ft [240 mm] higher than the stopping sight distance.

Perhaps the simplest way to measure passing sight distance is directly from the plans, using a straightedge. 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 4 ft [1.2 m] above the traffic lane. Because vegetation also blocks vision, its anticipated height must be included in the 4 ft [1.2 m]. The straight edge is placed along the edge of the obstruction (or dotted line), and the intersection with the centerline identifies the sight distance.

Where horizontal curves and vertical curves occur at the same general location, the sight distances for each must be considered together. At least the minimum stopping sight distance must be provided for each. Efforts to provide passing sight distance for one might be completely negated by a no-passing zone situation for the other.

For more information see the Green Book, pages 118 to 131.

5.1.4.3 DECISION SIGHT DISTANCE

Drivers frequently are called on to make decisions concerning vehicle operations. Occasionally, the characteristics of the horizontal alignment can adversely affect the ability to make these decisions. Examples of this include:

- ***Proximity to a Curve***. It is important that the driver has a complete or partial view of the curve ahead to indicate the direction of curvature. With some combinations of vertical and horizontal curvature, the curve may come as a surprise and the driver may have difficulty reacting properly.
- ***Curve Signing***. To be effective, curve signing must be located a considerable distance ahead of the curve. The use of short tangents between curves results in inadequate length for proper signing. Where the design speed of the curve is equal to or greater than the legal posted speed, the length of the tangent should be at least 300 ft [90 m] plus the required distance for superelevation transition.
- ***Route Continuity***. When a driver approaches a diverging roadway situation, such as a Y intersection, an exit ramp on a curve, or a flat-angle intersection, the main route should be distinctly emphasized with sufficient sight distance to eliminate any uncertainty on the part of the driver.

The Green Book, pages 115-117, provides more details and tables of calculated values for checking decision sight distance.

5.1.5 COORDINATION WITH VERTICAL ALIGNMENT

Curvature and grades should be in proper balance. Emphasis on a tangent alignment is not desirable when it results in extremely steep or long grades. An emphasis on flat grades is not desirable when it results in excessive curvature. A compromise between the two extremes is the best approach.

Several general criteria should be kept in mind:

- Sharp horizontal curvature should not be introduced near or just beyond 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.
- 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 because vehicle-operating speeds, particularly for trucks, often are higher at the bottoms of grades.
- 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 roadway. In these cases, it is necessary to work toward long tangent sections to secure sufficient passing sight distance rather than the more economical combination of vertical and horizontal alignment.
- Both horizontal curvature and the 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.

5.2 VERTICAL ALIGNMENT

The profile grade line defines the vertical alignment for construction in terms of straight grades and parabolic curves. This section provides guidelines and criteria for the design of profile grades.

5.2.1 GENERAL CRITERIA

As with other design elements, the characteristics of vertical alignment are influenced greatly by basic controls related to design speed, traffic volumes, functional classification and terrain conditions. Within these basic controls, several generally accepted criteria must be considered.

- Use a smooth grade line with gradual changes, consistent with the type of highway and character of terrain, rather than a line with numerous breaks and short lengths of tangent grades.
- The “roller coaster” or “hidden dip” type of profile should be avoided. Often they are proposed in the interest of economy, but they are aesthetically undesirable and extremely hazardous. A driver cannot avoid or compensate for a hazard that cannot be seen.
- Avoid “broken back” grade lines (two crest or sag vertical curves separated by a short tangent). One long vertical curve is more desirable.
- Avoid very long crest vertical curves if passing sight distance cannot reasonably be attained. A shorter vertical curve may permit more passing opportunity on adjacent tangent grades.
- On a long grade it is preferable to place the steepest grade at the bottom and flatten the grade near the top.
- Maintain moderate grades through intersections to facilitate turning movements. For the design of vertical alignment through intersections, refer to Chapter Seven-Intersections.
- Consider auxiliary lanes where passing opportunities are limited and it is probable that slow–moving vehicles will affect operating speeds and the desired level of service.

5.2.2 MAXIMUM GRADES

Criteria for maximum grades are based mainly on studies of the operating characteristics of typical heavy trucks. Although design values have been determined and agreed upon for many highway features, few conclusions have been reached on roadway grades in relation to design speed.

Refer to the Green Book for the maximum grades permitted for various combinations of functional classification, traffic volume and terrain. The maximum grades should be used only where absolutely necessary. Grades much flatter than maximum normally should be used.

For short grades less than about 500 ft [150 m] in length, the maximum gradient may be one percent steeper than the values shown in the tables.

5.2.3 MINIMUM GRADES

Minimum grades are primarily related to the need for adequate drainage. For uncurbed pavements that are adequately crowned to drain laterally, relatively flat or even level profile grades may be used. With curbed pavement, the minimum longitudinal grade in usual cases should be 0.5 percent. With a high-type pavement accurately crowned on a firm subgrade, a longitudinal grade of about 0.35 percent may be used. Even on uncurbed pavements, it is desirable to provide a minimum of about 0.35 percent longitudinal grade because the lateral crown slope originally constructed may subsequently be reduced as a result of irregular swell, pavement structure consolidation, maintenance operations or resurfacing. Use of flatter grades may be justified in special cases.

5.2.4 MINIMUM DITCH GRADES

Special attention should be directed to minimum ditch grades. Any ponding of water in the side ditches, particularly on expansive soils, has a very detrimental effect on the subgrade. To ensure continuing flow, ditch grades should be sloped at least 0.5 percent–preferably steeper. This may require some special warping of ditch grades where the roadway profile cannot be adjusted accordingly. A minimum depth of ditch has been established at 2.5 ft [800 mm] below the elevation of the hinge point between the shoulder and frontslope to assure proper drainage of pavement base and subgrade. In superelevated sections both the ditch grade and bottom width may have to be adjusted in order to prevent water ponding onto the shoulder or traveled way.

5.2.5 CRITICAL LENGTH OF GRADE

From the standpoint of vehicle operating characteristics and the effect on highway capacity, the steepness of the grade is not the only factor to be considered. The length of the grade can become a critical factor and must also be considered.

The term “critical length of grade” is used to indicate the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed. For a given grade, lengths less than critical ones result in acceptable operation in the desired range of speeds. If the desired freedom of operation is to be maintained on grades longer than critical ones, design adjustments such as change in location to reduce grades or addition of extra lanes should be made. It is recommended that a 10 mph [15 km/h] speed reduction be used as the general guide for determining critical lengths of grades. The Green Book, pages 242 and 243, provides curves showing the critical lengths of grade resulting from various combinations of percent upgrade and designated speed reductions.

On roads with moderate to heavy traffic volumes, where critical lengths are approached or exceeded, and passing opportunities are limited, long lines of smaller vehicles will accumulate behind the slower vehicles. This reduces both the operating speed and highway capacity and, consequently, the level of service. Consideration should be given to providing climbing lanes. A capacity analysis should be conducted to determine whether the addition of a climbing lane is warranted. Procedures for such an analysis are shown in Chapter Ten of the *Highway Capacity Manual*. Factors considered in the analysis include:

- Desired level of service,
- Lane widths and lateral clearance,
- Percent of trucks and buses,
- Passing sight distance,

- Steepness and length of grades,
- Volume/capacity ratio, and
- Service volume.

5.2.6 CLIMBING LANE CRITERIA

The need for climbing lanes in Delaware is seldom warranted. The Green Book pages 241 to 250 gives a through explanation for the design of these lanes.

5.2.7 VERTICAL CURVES

Vertical curves are used to effect gradual changes between tangent grades at their point of intersection. They have the properties of a simple parabolic curve. The vertical offsets from the tangent grade vary with the square of the horizontal distance from the curve end (point of tangency).

Vertical curves that are offset below the tangent are termed crest vertical curves. Those that are offset above the tangent are termed sag vertical curves. Examples of each curve type are shown in Figure 5-4.

The minimum lengths of crest vertical curves are determined mainly by sight distance requirements. These lengths generally are satisfactory from the standpoint of safety, comfort and convenience. An exception may be at decision areas, such as intersections and approaches to ramp exit gores, where adequate sight distance requires longer lengths.

Passing sight distance seldom can be attained on a crest vertical curve simply by lengthening the curve. Excessively long vertical curves often reduce the length of passing opportunities on the adjacent tangent sections on either side of the crest. They also can adversely impact roadway and roadside ditch drainage systems.

Sag vertical curves use four different criteria for determining their lengths:

(1) headlight sight distance,

(2) passenger comfort,

(3) drainage control, and

(4) general appearance.

The primary control used in design is headlight sight distance.

5.2.8 VERTICAL CURVE DESIGN

The principal concern in designing vertical curves is to ensure that at least the minimum stopping sight distance is provided. The values set forth in the design standards for stopping sight distance are also applied to vertical curves. Refer to the Green Book pages 265 to 280 for more design detail.

For crest vertical curves, the design eye height is 3.5 feet [1,080 mm] and the object height is 2.0 ft [600 mm]. The crest of the curve should not obstruct the line of sight.

Nighttime driving conditions govern sag vertical curves. The sight distance control is the height of headlight and the distance illuminated to an object rather than driver eye height. The distance illuminated is that of a headlight beam with an assumed upward divergence of 1 degree and headlight mounting height of 2 ft [600 mm]. Equations found in the Green Book are used to determine these values for various design speeds. For overall safety, a sag vertical curve should be long enough that the light beam distance is nearly the same as the stopping sight distance. The values in Figure 5-6 were developed using the design stopping sight distance as the light beam distance.

For passing sight distance, the controls are different than for stopping sight distance. The design height of the eye remains at 3.5 ft [1,080 mm], but the height of the object (oncoming car) is increased to 3.5 ft [1,080 mm].

By analyzing the requirements relating to sight distances and the characteristics of the curve, determinations can be made as to the minimum permissible length of curve for particular situations. A ride control criterion for vertical curve length of not less than three times the design speed in mph [0.6 times the design speed in km/h] is recommended for comfort.

The minimum length of a vertical curve is computed by the following formula:

$$L = KA$$

Where

L = minimum length of vertical curve in feet [meters],

A = algebraic difference in grades in percent, and

K = a constant value for the design speed.

The rate of change of grade along a vertical curve is constant, and is measured by dividing the algebraic difference between the grades by the length of the curve in feet [meters]. This value gives the percent change in grade per horizontal feet [meters] of curve.

The reciprocal of this value represents the horizontal distance required to effect a one percent change in the gradient along the curve. The expression L/A is termed K, and is useful for determining minimum lengths of vertical curves. Based on the geometrics of each sight distance condition and assumption, formulas are used to compute values of K for each design speed.

Established values of K are shown in Figure 5-5 for the calculated minimum stopping sight distance. Similar K values are shown as minimum criteria for passing sight distance. Curve lengths should be based on greater than minimum stopping sight distance wherever possible. Usually the length is rounded off to the next multiple of 100 ft [30 m].

Generally, crest vertical curves in rural areas should be longer than the minimum to avoid the appearance of an angular break in the alignment, even though sight distance criteria may permit a shorter curve. On long sag vertical curves, special attention may be required for drainage features.

5.2.9 PASSING SIGHT DISTANCE MEASUREMENT

Vertical curves designed in accordance with the criteria in the preceding section will, in all cases, provide adequate stopping sight distance. But often it is not practicable to provide passing sight distance. Passing sight distances are 4 to 5 times the corresponding length for stopping sight distance. On some projects with relatively high traffic volumes, the lack of sufficient passing opportunities can cause problems with traffic operations and levels of service. A key factor in identifying these potential conditions is the percentage of passing sight distance 1,500 ft [450 m] or greater available within the limits of the roadway section.

A practical approach for this determination is to measure the length of all “non-passing” locations (sight distances less than 1,500 ft [450 m]) and subtract this length from the total length of the roadway section. The remainder is available for passing and is the basis for computing the percent of passing sight distance. Separate calculations are needed to check both directions where roadways carry two-way traffic.

Measurements of vertical curve non-passing lengths can be made directly from the profile using a straightedge and the prescribed criteria of 3.5 ft [1,080 mm] for the height of the eye and 3.5 ft [1,080 mm] for the height of the object. These measurements should be coordinated with similar measurements of horizontal sight distance restrictions before computing the percent available passing sight distance. As a general guide, the restricted passing lengths (less than 1,500 ft [450 m] sight distance) should not exceed the values shown in Figure 5-3 or the level of service will drop below an acceptable level.

Figure 5-3
Restricted Passing Sight Distance Criteria

ADT	Percent Restricted Passing
Under 250	70
250 - 500	60
500 – 1000	40
1000 – 1500	20
Over 1500	10

5.2.10 GRADELINE ELEVATIONS

For many reconstruction projects in urbanized areas there is little opportunity for any major variation in gradeline elevations. The elevation controls usually are fairly well fixed by the existing facility and any adjacent roadside development.

However, for new construction or major reconstruction in rural areas there may be greater opportunity to adjust grades, particularly in relatively low areas with level terrain. Structural problems often occur where the pavement structure or upper portions of the embankment become saturated with water. Extremely high water may encroach on the roadway surface or shoulders.

Generally, the finished grade elevation should be set above the surrounding terrain to minimize the chance of ponding water or a natural high water table from saturating the pavement box. Evidence of occasional standing water in adjacent ditches or fields is a good indication that the gradeline should be high enough to assure drainage of the pavement structure at high-water levels.

Adjusting the profile grade up or down in rolling terrain is one way of balancing the earthwork so excavation within the roadway prism will be adequate to construct the designed embankments. In some instances, this is a practical approach as long as the previously mentioned criteria for gradeline elevations are not compromised. It is generally desirable to have a substantial portion of the roadway designed as an embankment, with fewer excavation areas and longer balance points. Consideration should be given to deeper cuts, wider side ditches, and daylight in cut areas where feasible. It may be more economical to use borrow material than to attempt to balance a project within the roadway prism.

Figure 5-4
Types of Vertical Curves

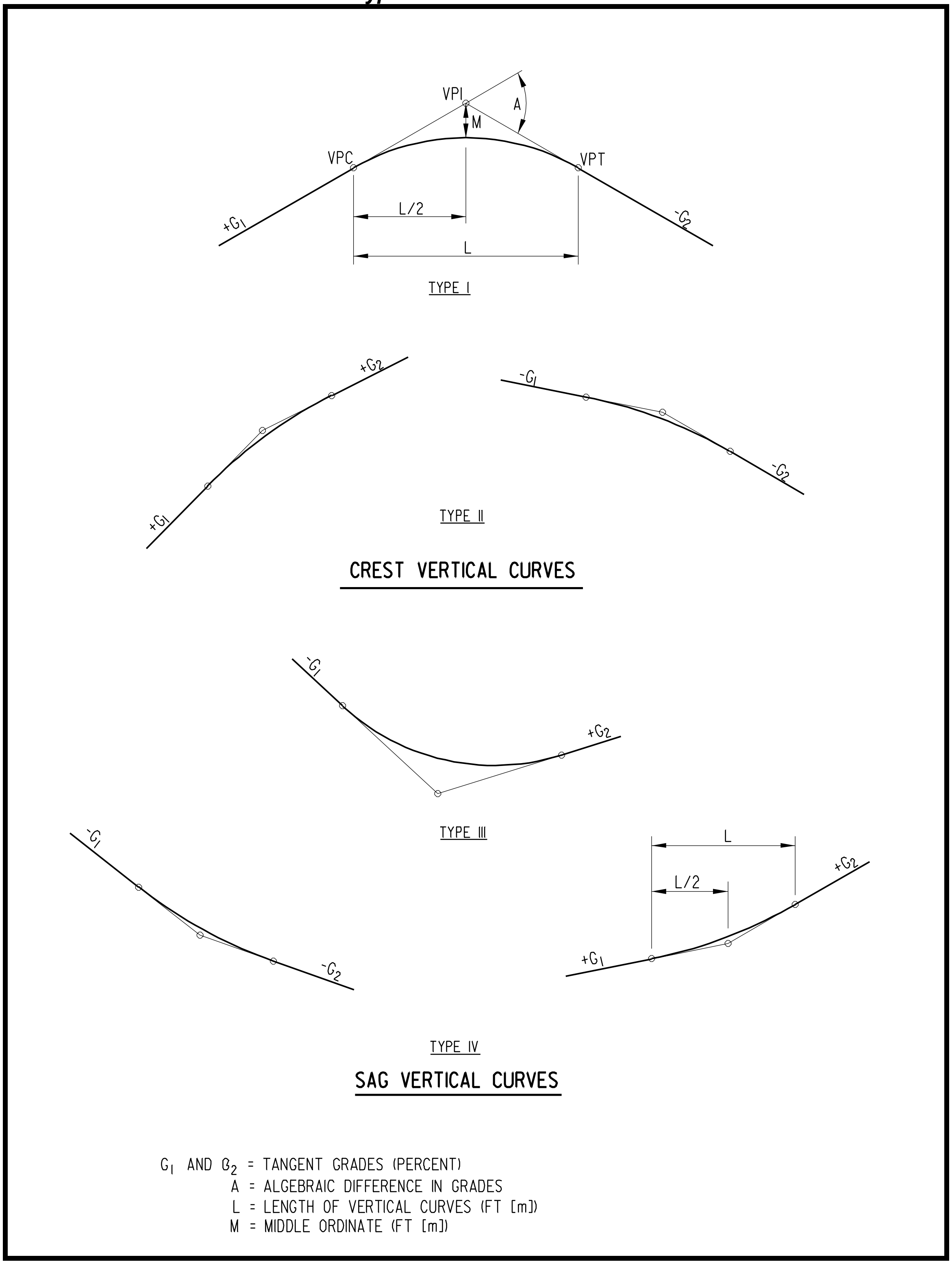


Figure 5-5
Criteria for Crest Vertical Curve Design
U S Customary [Metric]

Design Speed mph [km/h]		30 [50]	35	40 [60]	45 [70]	50 [80]	55 [90]	60 [100]	65	70 [110]
Based on Minimum Stopping Sight Distance	**Distance in ft [m]**	200 [65]	250	305 [85]	360 [105]	425 [130]	495 [160]	570 [185]	645	730 [220]
	K Value	19 [7]	29	44 [11]	61 [17]	84 [26]	114 [39]	151 [52]	193	247 [74]
Based on Minimum Passing Sight Distance	**Length of curve ft [m]**	1090 [345]	1280	1470 [410]	1625 [485]	1835 [540]	1985 [615]	2135 [670]	2285	2480 [730]
	K Value	424 [138]	585	772 [195]	943 [272]	1203 [338]	1407 [438]	1628 [520]	1865	2197 [617]

Note: Length of minimum crest vertical curve in feet [m] required to meet criteria is computed by multiplying the algebraic difference in grades by the value of the coefficient "K." Longer curves are desirable. Normally the length should be rounded to the next multiple of 100 feet [30 m]. When K>167 [51], drainage should be more carefully designed.

Figure 5-6
Criteria for Sag Vertical Curve Design
U S Customary [Metric]

Design Speed mph [km/h]		30 [50]	35	40 [60]	45 [70]	50 [80]	55 [90]	60 [100]	65	70 [110]
Based on Minimum Stopping Sight Distance	**Distance in ft [m]**	200 [65]	250	305 [85]	360 [105]	425 [130]	495 [160]	570 [185]	645	730 [220]
	K Value	37 [13]	49	64 [18]	79 [23]	96 [30]	115 [38]	136 [45]	157	181 [55]

Note: Length of minimum sag vertical curve in feet [m] required to meet criteria is computed by multiplying the algebraic difference in grades by the value of the coefficient "K." Longer curves are desirable. Normally the length should be rounded to the next multiple of 100 feet [30 m]. When K>167 [51], drainage should be more carefully designed.

5.2.11 URBAN GRADE DESIGN

For most projects in urbanized areas there is little opportunity for any major variation in gradeline elevations. The elevation controls usually are fairly well fixed by the existing facility and/or adjacent roadside development.

The design of vertical alignment on urban projects frequently involves consideration of special problems such as existing street intersections and adjacent property development.

Vertical curves are not required when the algebraic difference in grades is less than 0.5 percent. Because long vertical curves tend to create drainage "flat spots," the vertical curves on street sections usually are considerably shorter than for comparable locations on rural roads.

Where controlling factors are not severe, the normal practice of carrying the profile grade on the centerline or on the median edges of pavement will work satisfactorily. Where outside controls are significant, it may be necessary to supplement the main profile with other elevation controls, such as gutter-line profiles or top-of-curb profiles. Where this is necessary, the supplemental controls should be clearly shown on the typical sections, profiles, and grades and geometrics sheets.

Special attention must be given to existing features when designing urban grades. This is particularly true in the case of private driveways when a street is being widened. With even moderate driveway grades, up or down, angular breaks must be kept flat enough with adequate clearance so that the undercarriage or bumpers of vehicles will not drag. Reference should be made to the Department's publication *DelDOT Entrance Manual*.

Where roadside development is extensive and the general elevation is higher on one side than on the other, an unsymmetrical section may be required. The crown point (and profile grade) may be offset from the centerline so the total drop from the crown line to the gutter line will be more than normal on one side and less than normal on the other. However the location of the crown point must be at the edge of the travel lane.

5.3 SUPERELEVATION

The transitional rate of applying superelevation into and out of curves is influenced by design speed, degree of curvature and number of lanes. 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. For a given degree of curvature, a steeper superelevation is required for a higher design speed than is needed for a lower design speed. For a given design speed more superelevation is needed through sharp curves than for relatively flat curves.

The maximum rates of superelevation used on roadways are controlled by four factors:

(1) Climate conditions (i.e. frequency of ice and snow);

(2) Terrain conditions (i.e. flat or rolling);

(3) Type of area (i.e. rural or urban); and

(4) Frequency of slow-moving vehicles.

Basic design controls for superelevation are presented in Chapter Three. Rural roadways are usually designed with a maximum superelevation rate of 6 percent but it may be appropriate to use a rate of 8 percent. Urban roadways are normally to be designed with a superelevation rate of 4 percent. Superelevation may be omitted on low-speed urban streets subjected to severe constraints. The selected superelevation rate establishes the minimum permissible radius of curve based on a project’s design speed.

This section discusses practical application of superelevation criteria, with particular attention to:

- The rates of superelevation to be used for various combinations of design speed and curve radius,
- The manner of transition of slope between normal tangent sections and superelevated sections on curves, and
- Special criteria for superelevation of shoulders and auxiliary lanes.

5.3.1 RATES OF SUPERELEVATION

The Green Book, pages 165 to 174, sets forth the basic design criteria based on design speeds for the normal design superelevation rates of e_{max} = 4 and 6 percent as well as other values ranging up to 12 percent. The criteria shown includes the minimum radius of curvature, crown treatment and superelevation runoff lengths (L), all of which are related to the number of lanes to be rotated. The minimum rate of cross slope for a traveled lane is determined by drainage requirements.

The preferred cross slope (normal crown line) for high type pavement surfaces is 2 percent. The data in the Green Book exhibits are based on this cross slope. Very flat curves will not require superelevation. In the table, these are identified with the symbol “NC” meaning that the normal crown slopes used on tangent sections can be carried through the curve.

For slightly sharper curves, some superelevation is required and the normal practice is to remove the adverse crown on the outside lanes and carry the normal crown slope across all lanes. In the table, this is identified with the symbol “RC” meaning removal of the normal crown.

The indicated superelevation rates are applicable regardless of the number of lanes. The runoff lengths are for two-lane and four-lane highways. For three-lane undivided pavements, the runoff length should be 1.25 times the length shown for two-lane pavements.

On new construction and reconstruction projects, every effort shall be made to provide the prescribed superelevation rate of curves. Where this is not practicable, advisory speed signs shall be provided indicating the maximum safe speed for the curve at the existing superelevation.

5.3.2 SUPERELEVATION TRANSITION

Two terms related to superelevation transition are defined below.

- ***Superelevation runoff*** is the length of roadway needed to transition the outside-lane cross slope from zero (flat) to full superelevation, or vice versa.
- ***Tangent runout*** is the length of roadway needed to transition the outside-lane cross slope from the normal cross rate to zero (flat), or vice versa.

These two elements are applicable to superelevation on both simple circular curves and spiral transition curves, but the manner of application is somewhat different for each. General criteria for application of runoff and terminology for both types of curves are shown in Figure 5-8.

In order to achieve driver comfort and safety in operating a vehicle into, through and out of a curve, superelevation should be introduced and removed uniformly over a length adequate for the anticipated travel speeds. Figure 5-7 shows AASHTO’s recommended desirable proportion of runoff on the tangent to minimize lateral acceleration and a vehicle’s lateral motion. AASHTO does allow agencies to adopt a single value for all design speeds and rotated widths. For simplicity, DelDOT has adopted a runoff proportion of two-thirds in the tangent section and one-third into the curve. However, where conditions permit, the desirable values shown in Figure 5-7 should be used.

In the case of simple curves, the superelevation runoff distance (from Figures 5-9 and 5-10) is applied with one-third on the curve itself and two-thirds (or preferably as per Figure 5-7) on the tangent adjacent to the curve. This is a compromise between placing all the transition on the tangent section (where superelevation is not needed) and placing the transition on the curve (where full superelevation is needed throughout the entire length). Thus, full superelevation is not reached until slightly past the P.C. and starts to reduce shortly before reaching the P.T.

Where spiral transition curves are used, the superelevation runoff is always 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.

In the case of both simple curves and spiral curves, the tangent runout is placed outside the superelevation runoff sections. The tangent runout length is not a critical factor. The only criterion is that the longitudinal slope of the outside edge of the traffic lanes (compared to the profile grade) should not exceed a rate of about 1:200 during the tangent runout.

Where more than two lanes are to be rotated, the runout distance should be lengthened so the 1:200 longitudinal slope limitation is not exceeded.

Figure 5-7
Runoff Locations that Minimize Vehicle Lateral Motion

Design Speed (mph) [km/h]	Portion of runoff located prior to the curve in percent		
	No. of lanes rotated		
	1.0	1.5	2.0-2.5
15-45 [20-70]	80	85	90
50-80 [80-130]	70	75	80

5.3.3 AXIS OF ROTATION

In the design of superelevation, it is necessary to select a point (axis of rotation) 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 and the characteristics of the project area such as drainage, adjacent land elevations, and roadside development.

The available options for rotating traveled ways to attain superelevation are graphically shown in Figures 5-11 to 5-13. These are:

1. Revolving a traveled way with normal cross slopes about the centerline profile.
2. Revolving a traveled way with normal cross slope about the inside-edge profile.
3. Revolving a traveled way with normal cross slopes about the outside-edge profile.
4. Revolving a straight cross-slope traveled way about the outside-edge profile.

The most commonly used method for transitioning into a superelevated section is rotating the traveled way about the centerline as shown in Figure 5-11. This method provides a more uniform appearance with a less distorted edge of traveled way.

Figure 5-12 shows the transition method most commonly used for two-lane one-way roadways where the axis of rotation coincides with the edge of the traveled way adjacent to the highway median.

There are many possible profile arrangements. Selecting the most appropriate is based on drainage requirements, avoidance of critical grades, aesthetics and making adjustments to fit the roadway section into the adjacent topography.

The methods of attaining superelevation shown result in angular breaks in the profiles of the pavement edges. For appearance and safety, these breaks should be rounded in final design by using vertical curves. The simplest method is by graphically plotting the center line or edge profile on a vertical scale. Then by means of a spline (a flexible straight edge), ship curve, curve template, or circular curve, draw smooth-flowing lines to approximate the straight-line controls. Once the edge profiles are drawn in the proper relationship, elevations can be read as needed from the plot. Vertical curves can also be used. The minimum length of vertical curve in feet [meters] can be used as numerically equal to the design speed in mph [km/h]. The angular break for transitioning about the centerline is less pronounced than for other transitions and the vertical curve may be adjusted accordingly.

Figure 5-8
Superelevation Runoff Elements

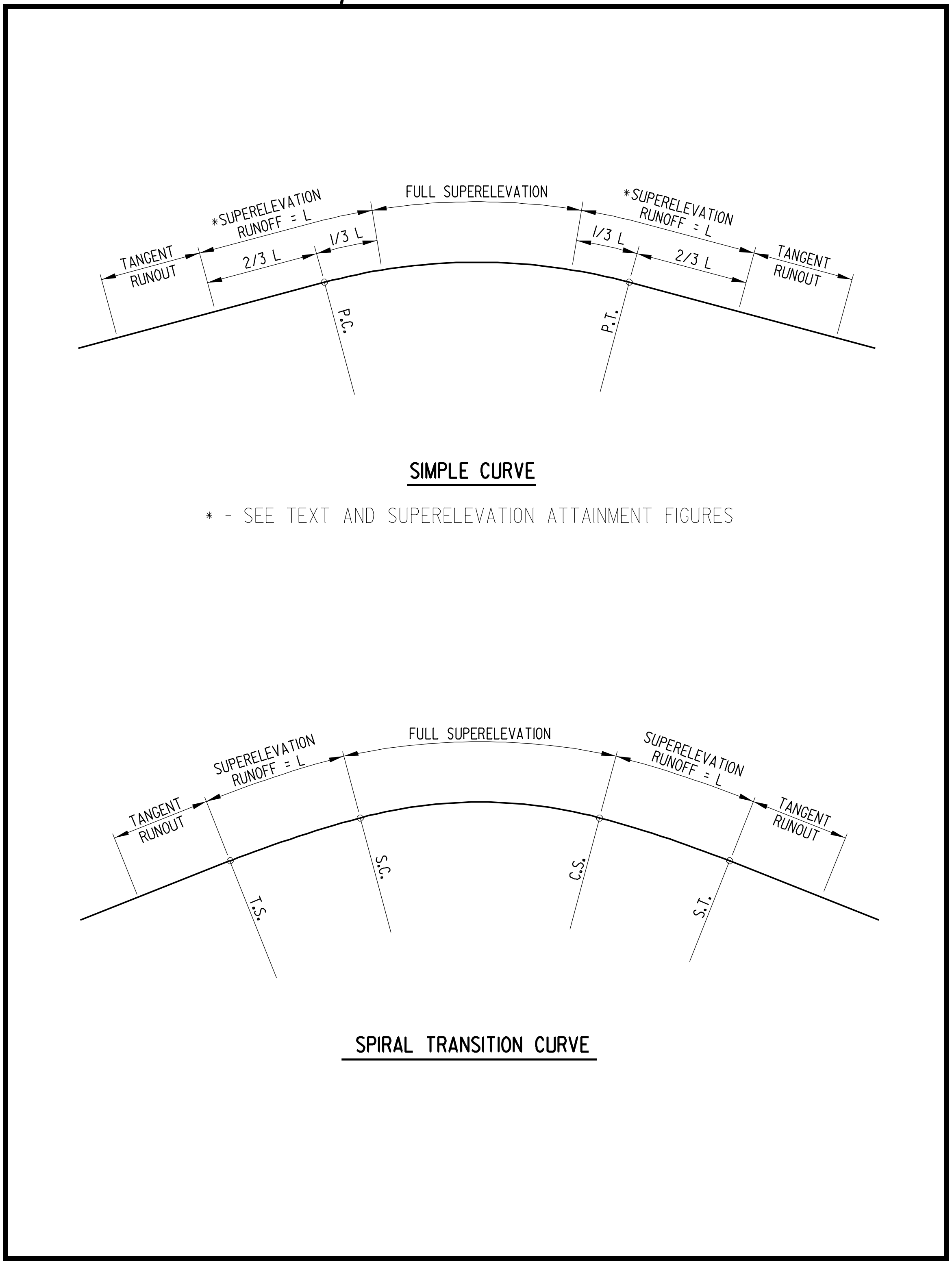


Figure 5-9*
Minimum Superelevation Runoff and Tangent Runout Lengths (US Customary)

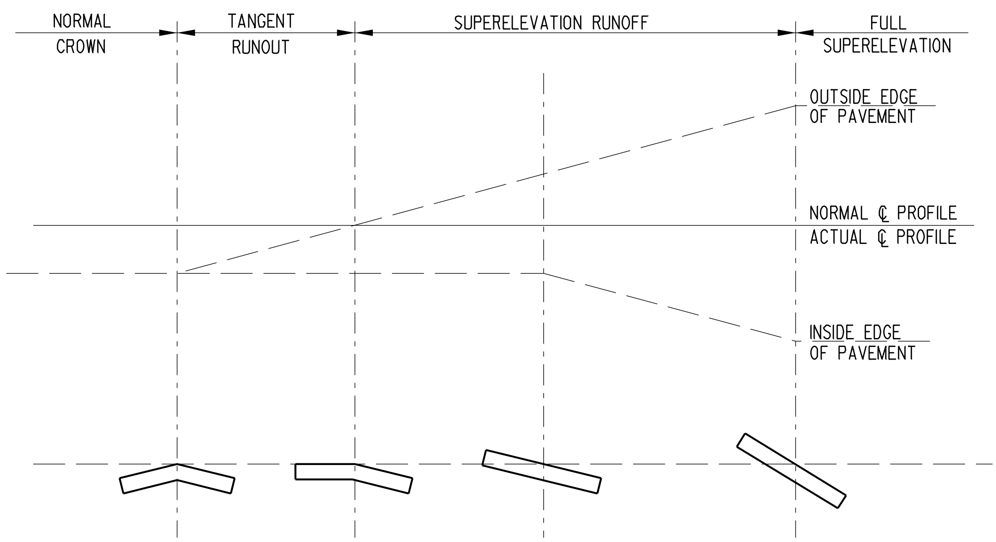
Minimum runoff and runout length (ft)				
Design speed (mph)	**Runoff**			**Runout**
	Superelevation Rate			
	2 %	**4%**	**6%**	**Any**
One lane rotated				
25	34	69	103	34
30	36	73	109	36
35	39	77	116	39
40	41	83	124	41
45	44	89	133	44
50	48	96	144	48
55	51	102	153	51
60	53	107	160	53
65	56	112	167	56
70	60	120	180	60
Two lanes rotated				
25	51	103	154	51
30	55	109	164	55
35	58	116	174	58
40	62	124	186	62
45	67	133	200	67
50	72	144	216	72
55	77	153	230	77
60	80	160	240	80
65	84	167	251	84
70	90	180	270	90

Figure 5-10*
Minimum Superelevation Runoff and Tangent Runout Lengths [Metric]

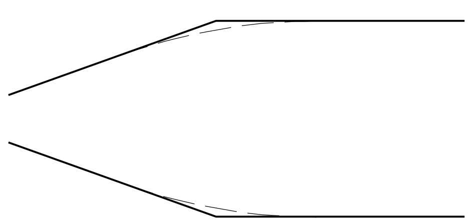
Minimum runoff and runout length [m]				
Design speed [km/h]	**Runoff**			**Runout**
	Superelevation Rate			
	2 %	**4%**	**6%**	**Any**
One lane rotated				
20	9	18	27	9
30	10	19	29	10
40	10	21	31	10
50	11	22	33	11
60	12	24	36	12
70	13	26	39	13
80	14	29	43	14
90	15	31	46	15
100	16	33	49	16
110	18	35	53	18
Two lanes rotated				
20	14	27	41	14
30	14	29	43	14
40	15	31	46	15
50	17	33	50	17
60	18	36	54	18
70	20	39	59	20
80	22	43	65	22
90	23	46	69	23
100	25	49	74	25
110	26	53	79	26

*Note: Figures 5-9 and 5-10 are based on 12-ft [3.6 m] lanes and 2.0% normal cross slope

Figure 5-11
Superelevation Attainment
Traveled Way Rotated about Centerline



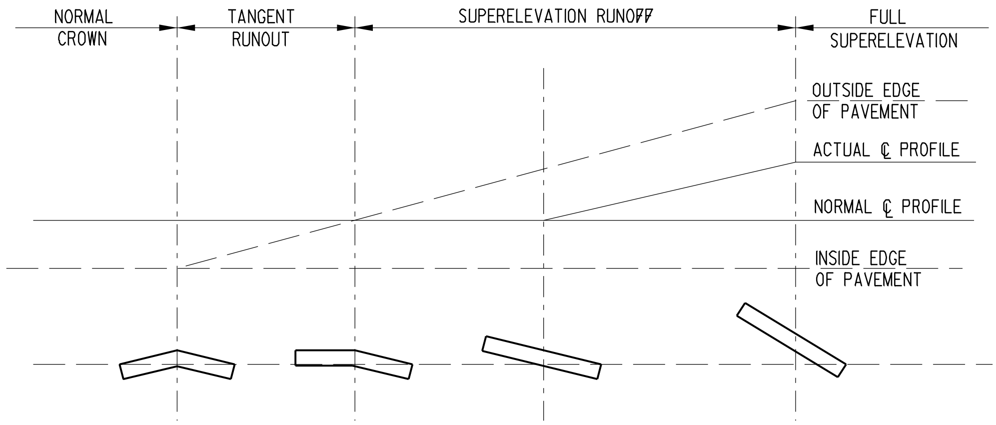
TRANSITION A - PAVEMENT REVOLVED ABOUT CENTERLINE



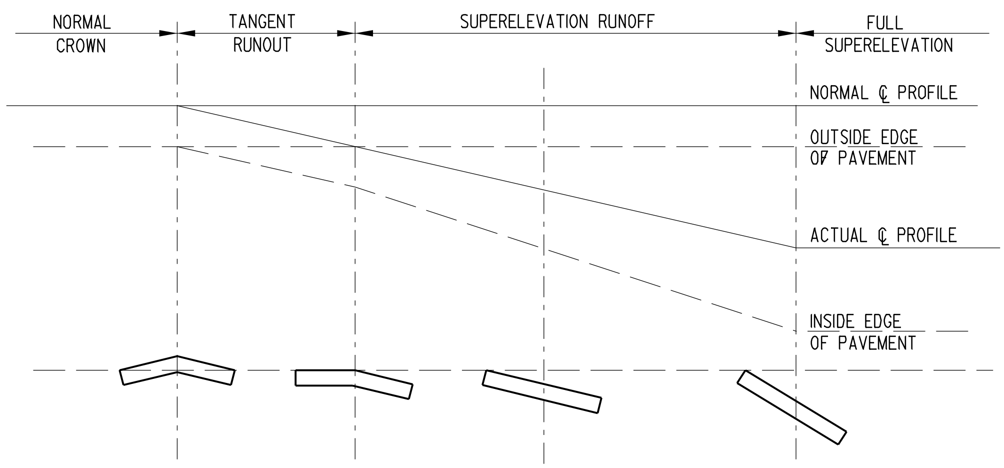
NOTE : ANGULAR BREAKS TO BE APPROPRIATELY
ROUNDED AS SHOWN BY DASHED LINE.

ROUNDING

Figure 5-12
Superelevation Attainment
Traveled Way Rotated about Inside and Outside Edge



TRANSITION B – PAVEMENT REVOLVED ABOUT INSIDE EDGE



TRANSITION C – PAVEMENT REVOLVED ABOUT OUTSIDE EDGE

NOTE : USE TRANSITION A FOR ALL UNDIVIDED HIGHWAYS.
FOR DIVIDED HIGHWAYS WITH MEDIAN, USE TRANSITION A FOR TRAFFIC LANES ON OUTSIDE OF CURVE AND TRANSITION B FOR TRAFFIC LANE ON INSIDE OF CURVE.

Figure 5-13
Superelevation Attainment
Traveled Way with Straight Cross Slope