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Chapter 11

Horizontal Alignment

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Chapter 11 Amendments - May 2002

Revision Register

Issue/ Rev No.	Reference Section	Description of Revision	Authorised by	Date
1		First Issue.	Steering Committee	Oct 2000
2	11.1	New paragraph re coordination of horizontal and vertical alignment.	Steering Committee	May 2002
	11.2	Last paragraph changed to draw attention to limitations of Equation 11.1. Definition of superelevation added.		
	11.3	New paragraph and Table 11.1B giving maximum side friction factors for trucks. New paragraph and Table 11.1C giving maximum side friction factors for unsealed roads.		
	11.4	Addition of explanation of the purpose of superelevation and effect of adverse superelevation.		
	11.4.1	Retitled and inclusion of guidelines on acceptable increases in side friction. Note re superelevation on bridges added to Table 11.2.		
	11.4.2	Retitled and additional material on adverse superelevation.		
	11.4.3	Additional explanation at start.		
	11.4.5	Revised definition of superelevation development length. Added definitions of superelevation runoff length and tangent runout length. Cross reference to Tables 11.5 and 11.6 added to Fig. 11.1.		
	11.4.7	New paragraph covering when superelevation can be developed entirely on the tangent.		
	11.5	New paragraph on desirable minimum curve length.		
	11.6	Equation for reverse circular curve spacing corrected (Case 4A). Additional cases of reverse curves. New paragraph on design speed for superelevation development when curve speeds vary.		
	Fig. 11.2A & Fig. 11.2B	Fig. 11.2 divided into Fig. 11.2A with 6 cases of reverse curves and Fig. 11.2B for similar (broken back) curves.		
	Figs. 11.4, 11.5 & 11.6	New Figures 11.4 and 11.5. Previous Figure 11.4 now 11.6. Figure 11.6 - case for trucks on right hand curves added.		
	11.8	Additional constraints on the use of compound curves.		
	11.9.2	Additional explanation of the basis of transition curve length.		
	11.10	Table 11.8 - renumbered. Additional note re maintenance of clearances on the straight. Two paragraphs added re application of curve widening on reverse curves.		
	11.12	New section - Curve Perception Issues.		
	11.13	New section - Curves on Steep Downgrades.		
	References	Additional references (McLean, Tziotis and Gunitallake; Rahmann; Kanellaidis).		
	New Section	Relationship to other chapters.		

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Chapter 11
Horizontal Alignment

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11.1 General

The horizontal alignment of a road is usually a series of straights (tangents) and circular curves which may or may not be connected by transition curves. A suitable horizontal alignment is best chosen by the procedure described in Chapter 6 (Speed Parameters). The following sections outline various design criteria which also need to be considered when developing a horizontal alignment.

In addition, the horizontal alignment must be designed in conjunction with the vertical alignment and both properly coordinated in accordance with the principles stated in Chapter 10.

Standard curve notation that is used in various figures in this Chapter is described in Appendix A, Table 11A.1.

11.2 Movement on a Circular Path

As a vehicle traverses a circular curve, it is subject to forces associated with the circular path. According to the principle of inertia, in the absence of forces, a moving body will travel in a straight line. A force must be applied to change direction. For a circular change of direction, the force is called centripetal force and, in road design, this is provided by side friction developed between the tyres and the pavement, and by superelevation.

Superelevation is the crossfall that is provided on the pavement on a horizontal curve in order to assist a vehicle to maintain a circular path.

For normal values of superelevation, side friction and radius, the following formula is accepted:

$$e + f = \frac{v^2}{gR} = \frac{V^2}{127R} \qquad (11.1)$$

where

e = pavement superelevation (m/m or tangent of angle). This is taken as positive if the pavement falls towards the centre of the curve.

f = coefficient of side friction force developed between the vehicle tyres and the road pavement. This is taken as positive if the frictional force on the vehicle acts towards the centre of the curve.

g = acceleration due to gravity = 9.8 m/s²

v = speed of vehicle (m/s)

V = speed of vehicle (km/h)

R = curve radius (m)

Where f equals zero in the formula, all of the centripetal force is provided by the superelevation. This condition can occur on large radius curves with positive superelevation or for slow moving vehicles on curves of any radius. At low speeds, f can be negative, and the curve is then over-superelevated for that speed. Curves are generally designed, however, so that a positive f is required for the range of vehicle speeds likely to occur.

On short length horizontal curves, the radius of the vehicle path can be considerably larger than the centreline or edgeline radius. In these cases, the curve radius 'R' in Equation 11.1 should be made equal to the vehicle path radius in lieu of the horizontal curve radius.

Equation 11.1 can be rearranged as follows:

$$R = \frac{V^2}{127(e+f)} \quad (11.2)$$

It then follows that:

$$R_{min} = \frac{V^2}{127(e_{max} + f_{max})} \quad (11.3)$$

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where

R_{min} = minimum horizontal curve radius (m)

e_{max} = maximum superelevation for the prevailing conditions (see Table 11.2)

f_{max} = maximum coefficient of side frictional force developed between the vehicle tyres and the road pavement. However, the desirable maximum coefficient of side friction (or even a value close to it) should be used in preference to the absolute maximum value whenever possible (see Tables 11.1).

Equation 11.1 is commonly called the 'point mass' formula. This is because the vehicle is considered as a single point (at its centre of gravity) which means all points of the vehicle describe the same radius. This also means that the application of the equation is not relevant for example, for large articulated vehicles turning at intersections.

11.3 Maximum Side Friction (f_{max})

11.3.1 General

The maximum side friction values to be adopted for use in Equations 11.2 and 11.3 are given in Tables 11.1.

The values of f_{max} in Table 11.1A are for passenger cars on sealed pavements. Research has shown that articulated vehicles may roll over at values of side friction in the range 0.2 (even less for some vehicles carrying livestock) to 0.35. The absolute maximum for trucks turning at low speed is usually taken as 0.25 (see McLean, Tziotis and Gunitallake, 2001). Table 11.1B sets out the maximum side friction factors to be used for trucks in situations that require specific checking for truck operation (see Chapter 6, Section 6.3.3).

Table 11.1A Maximum Design Values of Side Friction Demand for Cars on Sealed Pavements

Design Speed km/h	Absolute Maximum Coefficient of Side Friction	Desirable Maximum Coefficient of Side Friction
40	0.35	0.30
50	0.35	0.30
60	0.33	0.24
70	0.31	0.19
75	0.29	0.18
80	0.26	0.16
85	0.22	0.15
90	0.20	0.13
95	0.18	0.13
100	0.16	0.12
105	0.12	0.12
110	0.12	0.12
120	0.11	0.11
130	0.11	0.11

Table 11.1B Maximum Design Values of Side Friction Demand for Trucks on Sealed Pavements

Design Speed (km/h)	Coefficient of Side Friction for Trucks	
	Absolute Maximum	Desirable Maximum
50	0.25	0.21
60	0.24	0.17
70	0.23	0.14
80	0.20	0.13
90	0.15	0.11
100	0.12	0.12
110	0.12	0.12
120	0.11	0.11

Source: McLean, Tziotis and Gunitallake (2001).

Table 11.1C sets out the maximum side friction factors to be used on unsealed road surfaces.

Table 11.1C Maximum Side Friction Factors on Unsealed Road Surfaces

Speed	50	60	70	80	90	100	110
Max. coeff. of side friction	0.12	0.11	0.10	0.10	0.09	0.09	0.08

Source: Unsealed Roads Manual - ARRB 1993.

11.3.2 Pavement Surfacing

Horizontal curves on which high values of side friction are likely to be demanded should be provided with a pavement surfacing capable of providing good skid resistance. This is particularly important on horizontal curves in constrained areas such as mountainous terrain. The following treatments will help decrease the likelihood of skidding:

- Surfacing with materials using aggregates with a high Polished Aggregate Friction Value and Crushing Value;
- Applying larger aggregates with sprayed seals (provides good drainage routes if applied properly (i.e. texture depths up to 2.5mm));
- Using polymer modified surfacings (helps stone retention in asphalt and sprayed seals);
- Applying specially designed surfacings (e.g. open graded asphalts, tar based binders, epoxy resins, slurry treatments);
- Reducing drainage paths by designing appropriate crossfalls and providing drainage inlets/structures; and
- Adopting pavement designs that reduce rutting during the life of the pavement.

11.4 Curve Superelevation

11.4.1 General

It is normal practice for horizontal curves to be superelevated. This allows a component of the vehicle weight to provide some of the centripetal force that is needed for the vehicle to move in a circular path.

If a curve is not superelevated, the curve is said to have adverse or negative superelevation. Therefore, ‘e’ is then expressed as a negative value in equation 11.1. With adverse superelevation, there is a component of the vehicle weight that acts opposite to the centripetal force that is needed for the vehicle to move in a circular path. This in turn requires greater side friction than for a curve of given radius with positive superelevation if the vehicle is to take the curve at the same speed.

The amount of superelevation is chosen primarily on the basis of safety, but other factors are comfort and appearance. The superelevation that is applied to a horizontal curve should take into account the following:

- tendency to increase the tracking of the rear wheels of slow moving vehicles towards the centre;
- stability of high laden commercial vehicles;
- stability of vehicle loads;
- difference between inner and outer formation level, especially in flat country;
- length available to introduce the necessary superelevation;
- the need to avoid major changes in side friction demand between successive horizontal curves;
- the amount of centripetal force provided by superelevation versus that provided by side friction (see 11.4.4); and
- the need to keep superelevation below about 5% on a floodway that is located on a horizontal curve.

The amount of superelevation required for a given radius, speed and coefficient of friction can be calculated by rearranging Equation 11.1 as follows:

$$e = \frac{V^2}{127R} - f \qquad (11.4)$$

The absolute maximum and desirable maximum values of the coefficient of side friction ‘f’ can be obtained from Tables 11.1.

Values of superelevation obtained from Equation 11.4 are subject to the maximum and minimum values given in Sections 11.4.2 and 11.4.3 respectively.

11.4.2 Maximum Values of Superelevation and Increases in Side Friction

Maximum values of superelevation are shown in Table 11.2. These values also take into account the relevant factors that are listed in Section 11.4.1. The maximum values are consistent with practice throughout Australia. The values for rural roads are suitable for road trains carrying livestock. The values for urban roads reflect property boundary controls and traffic operation in congested conditions.

As equation 11.3 shows, the minimum radius horizontal curve (R_{min}) that is suitable for a given speed requires the use of maximum allowable superelevation and maximum allowable coefficient of side friction. With horizontal curves that are larger than R_{min}, it is normal practice to provide the same proportions of superelevation and side friction that apply to R_{min} (see Section 11.4.4). This practice helps (partly but not completely) ensure that any increase in side friction demand between successive horizontal curves is kept within acceptable limits.

Large increases in side friction demand over what drivers have become accustomed to, can lead to a change in vehicle response that is not anticipated by some drivers. Furthermore, these situations may create hazards for motorcyclists because they have difficulty in maintaining control when there is a sudden change in side friction demand. Therefore, the need to limit changes in side friction demand becomes a desirable secondary control for ensuring geometric consistency (see Section 6.6). However, the limits on decrease in speed between horizontal alignment elements that are set out in Section 6.6 are the primary requirement for geometric consistency. The desirable maximum changes in speed will often ensure that changes in side friction demand are acceptable.

Increases in side friction are considered to be a secondary control because drivers select their speed through their perception of the horizontal curvature. In turn, the selected speed reflects driver experience with a number of factors relating to vehicle control, including side friction demand.

It is not possible to prescribe simple limits on the increase in side friction demand from one horizontal curve to another. However, the following guidelines are provided:

- The desirable maximum increase in side friction demand from one horizontal curve to the next is about 25%. This is based on an Austroads (1989) criterion for compound curves and is therefore not a precise value. However, this condition only needs to apply when the side friction factor on the latter curve is greater than about 0.12. This is because 0.12 is a comfortable limit for drivers, even at high operating speeds (see Table 11.1).
- If a curve has a ‘generous’ radius for its design speed (this can still easily occur in a restricted speed environment due to preceding curvature), then a larger increase than the desirable 25% is likely to occur with the next curve. Even when the latter curve has a side friction demand comfortably below the maximum for its design speed, the increase may be greater than 25%. Therefore, an increase of more than 25% is acceptable when a curve follows a ‘generous’ curve. However, it will be necessary to check on any increase in side friction between the latter curve and any curves prior to the 'generous' radius curve. The purpose is to check the side friction demand against the level that drivers have become aware of.
- Sections of existing road or sections of road that involve a reduction in desired speed (see 6.1.3), may involve larger increases in side friction. With the latter, at least, drivers will be alerted to the need for increases in side friction. In these cases, the increase will be acceptable when the

side friction demand does not exceed the desirable maximum for the curve design speed.

- Curves with a side friction demand greater than the desirable maximum for the curve design speed should be preceded by a curve that alerts drivers to the required side friction.
- A higher value of superelevation than that shown for a particular radius/speed combination in Tables 11.5 and 11.6, may be used to limit an increase in side friction.

Table 11.2 Maximum Values of Superelevation

Urban Roads	Rural Roads		
	Desirable Design Speed Range km/h	Number of Heavy Vehicles	
		≥20 veh/day	<20 veh/day
5%	>70	6%	6%
5%	≤70	7%	10%*

* Use with caution.

Notes:

1. Higher values (up to 10%) may be used in special cases such as rural roads in mountainous terrain or the reuse of existing pavement or kerb lines.
2. On bridges, the maximum superelevation should be 5% on urban roads and 6% on rural roads.

11.4.3 Minimum Superelevation and Adverse Superelevation

It is normal practice to superelevate all curves to a value that is at least equal to the normal crossfall on straights. This is normally 3% in order to ensure adequate surface drainage. However, there are situations where the application of superelevation can cause pavement drainage problems. In some situations when the grade is nearly flat, water will not run off the road properly at places where the crossfall is also nearly flat. These areas are also likely to experience maintenance problems. In other situations, drainage paths for the pavement surface will be unacceptably long because the grade will be steep enough to have the water flow being mainly down the grade and switching from one side of the road to the other due to the change in the direction of the crossfall (see also 12.2.4). These situations should always be checked with the aid of pavement surface contours. These contours can be readily generated by road design modelling software.

Problems with drainage of the pavement surface may be overcome by modifying the combination of the grading, the superelevation and the application of the superelevation. If the horizontal curve radius is large enough, there may be scope to leave the curve unsuperelevated (that is, have adverse superelevation). There are also other situations as discussed later that may warrant the use of adverse superelevation on large radius curves. However, the use of adverse superelevation must be considered to be exceptional practice.

In addition to the problems with adverse superelevation that are given in section 11.4.1, further problems with using adverse superelevation are:

- Greater tendency of vehicles to move towards the outside (in terms of radius) of the traffic lane on multilane roads.
- Greater instability of vehicle loads. This is also exacerbated by greater suspension movement due to the weight component acting in the outwards direction.
- On two-lane, two-way roads, the normal crowned pavement structure is carried through the curve. Vehicles on the outer lane will have adverse superelevation and vehicles on the inner lane will have positive superelevation. Hence, a vehicle that crosses from the inner lane to the outer lane will experience a rapid increase in side friction demand with increased probability of the driver losing control.

These problems with adverse superelevation limit its use on rural roads and motorways to:

- Large radius curves that can be regarded as straights and where pavement drainage can be improved. See Table 11.3A.
- Temporary roadways. See Table 11.3B.
- Side tracks. See Table 11.3B.

- Temporary connections. See Table 11.3B.

On urban roads adverse superelevation can be tolerated more than on rural roads. This is mainly due to lower vehicle speeds and driver familiarity with the road. Common urban situations that may require the use of adverse superelevation are:

- Property access controls
- Channel drainage controls
- Grading restrictions
- The need to maintain visibility of the road surface - especially near intersections
- Turns at at-grade intersections
- Roundabouts.

For any road, adverse superelevation should not exceed 3%. In line with common practice throughout the world, the maximum side friction values that can be used in conjunction with adverse superelevation are half the absolute maximum values that are used for curves with positive superelevation. However, this limit on side friction values does not apply to turns at at-grade intersections.

Minimum curve radii for the use of adverse superelevation on urban roads are given in Table 11.3B. This table may also be used for temporary roads, and side tracks and temporary connections on rural roads or motorways.

Table 11.3A Rural Roads - Minimum Horizontal Curve Radii that can have Adverse Crossfall Equal to the Normal Crossfall on Straights

Speed Environment (km/h)	Minimum Radius (m)
70	900
80	1250
90	1500
100	2000
110	3000
120	4000
130	5000

Note: For normal rural roads. Does not apply to intersections where higher *f* demand may be considered (Austroads 1988 - Intersections at Grade - Part 5).

Table 11.3B Minimum Horizontal Curve Radii with Adverse Superelevation for Urban Roads, temporary roads, and side tracks and temporary connections on rural roads or motorways

Curve Design Speed (km/h)	Minimum Radius (m)	Absolute Maximum Coefficient of Side Friction
40	85	0.18
50	130	0.18
60	200	0.17
70	295	0.16
80	505	0.13
90	910	0.10
100	1575	0.08
120	3780	0.06
130	4435	0.06

Based on a maximum adverse superelevation of 3%.

11.4.4 Superelevation on Horizontal Curves with Radius > R_{min}

It is normal practice to have superelevation provide a significant amount of the centripetal force that is needed for a vehicle to move in a circular path. This is because superelevation is a positive and permanent feature, whereas side friction is subject to roadway surface and vehicle variations (Rahmann, 1981). Even so, it is common to have two to three times as much centripetal force provided by side friction as that provided by superelevation.

There are a number of methods to determine the superelevation (and hence resultant side friction) for curves with a radius larger than the minimum radius for a given design speed. Main Roads has used at least two different methods in the past. It must also be reiterated that the length of such curves should be checked to ensure that the length does not cause the operating speed to increase beyond the curve design speed when the design speed is less than 110 km/h.

The method that is currently favoured by Austroads and which will normally be used for new works, is for the superelevation to be varied linearly from 0 for R = infinity to e_{max} for R_{min}. This then means that all curves that are designed for a given speed will have the same proportions of superelevation and side friction demand, not withstanding the following practical considerations:

- For construction expediency, superelevation values are normally rounded (upwards) to a multiple of 1% so that there is a corresponding adjustment of side friction.
- The perceived benefits of uniformity will usually only be possible on high speed rural roads (i.e. where the design or operating speed exceeds 100 km/h) and urban roads with enforced speed limits because curve operating speeds vary on intermediate and low speed rural roads.
- Other methods have been used in the past so that there are likely to be many cases where the reuse of existing pavement will dictate a different superelevation. This is acceptable if the resultant side friction is suitable for the curve design speed and consistent with that for any adjacent curves (see 11.4.2).

With the "linear distribution method", the superelevation 'e' for a curve of radius R that is greater than R_{min} is given by equation 11.5.

$$e = \frac{V^2 e_{max}}{127R(e_{max} + f_{max})} \quad (11.5)$$

Note that f_{max} may be either the absolute maximum value or the desirable maximum value for the design speed V.

The value of 'e' is usually rounded upwards (e.g. 4.0% stays 4.0% but 4.1% becomes 5%) and the corresponding coefficient of side friction is calculated from

$$f = \frac{V^2}{127R} - e_{rounded} \quad (11.6)$$

With different possibilities for e_{max} (see Table 11.2) and f_{max} (absolute maximum vs. desirable maximum) different values of superelevation may be attributed to a given combination of radius and design speed. However, the subjective basis of the "linear distribution method" (and indeed most other methods) and the practice of rounding the superelevation value, allows a practical rationalization to be made. Hence Tables 11.5 and 11.6 cover the majority of rural and urban cases respectively. Tables 11.5 and 11.6 also show corresponding superelevation runoff requirements and transition lengths.

For rural areas, rationalization has been achieved by distributing from the combination of 6% superelevation and the desirable maximum value of coefficient of side friction. This practice is supported by the following points:

- For speeds in the range of 60 to 90 km/h, it is fortuitous that it is possible to distribute 'up' to the combination of 10% superelevation and absolute maximum coefficient of side friction.
- For speeds greater than 100 km/h, the desirable maximum and absolute maximum coefficients of side friction are the same. Hence, curves with superelevation between 6% and 10% use the same coefficient of side friction. This is consistent with long standing practice for these design speeds.
- The maximum superelevation for design speeds of 120 km/h and 130 km/h is 6%.
- Except for large radius curves that are not superelevated (see Section 11.4.3), curves are superelevated at least to a value equal to the normal crossfall on straights. This further limits the range of superelevation that needs to be distributed.
- In areas where 7% maximum superelevation is used, Table 11.5 promotes the use of a coefficient of side friction that is sufficiently close to the desirable maximum in conjunction with 7% superelevation. Given the subjective nature of the desirable maximum coefficients of side friction, this is acceptable.

For urban areas, rationalization has been achieved by distributing from the combination of 5% maximum superelevation and the desirable maximum value of coefficient of side friction. For design speeds less than or equal to 100 km/h, curves with a radius less than that corresponding to 5% superelevation and the desirable maximum coefficient of side friction, maintain the 5% superelevation with increasing side friction until f_{max}. This practice helps ensure that the relatively low maximum superelevation is applied before vehicles have to make use of increasing side friction.

11.4.5 Minimum Length of Superelevation on Horizontal Curves

In constrained situations such as mountainous terrain or urban roads, curves should be fully superelevated even if only instantaneously. But it is desirable that there be at least 30m of fully superelevated curve.

11.4.6 Superelevation Development Length

Superelevation is developed by rotating the roadway cross-section about some axis; most commonly the horizontal control line.

Superelevation development length is defined as the length required to rotate the pavement from the point of normal crossfall on the approach tangent (straight) to the point where the full superelevation for the curve is attained. In turn, this superelevation development length has two components:

- **Superelevation runoff length.** This is the length from the point where the pavement has been rotated to zero crossfall to the point where the full curve superelevation has been attained.
- **Tangent runout.** This is the residual length from the point of normal crossfall to the point of zero crossfall. This component lies on the approach tangent.

There are two criteria that are used to determine the length of superelevation development:

1. Maximum rate of rotation of crossfall.
2. Relative grade between the edge of the carriageway and the control line.

The superelevation development length to be adopted will generally be the longer value calculated for the two criteria above, notwithstanding normal minimum lengths of superelevation runoff. The maximum rate of rotation criterion is a mandatory standard that must be adopted as a minimum. The relative grade criterion is for appearance purposes and should be obtained at all locations unless economic or safety considerations dictate otherwise e.g. between reverse curves on steep grades.

Except for constrained situations such as mountainous areas or urban roads, it is normal practice to recognize minimum lengths for superelevation runoff for both transitioned and untransitioned curves. This is due to the fact that most vehicles describe some transition path when entering or leaving a curve with the minimum length of the transition path being about 30 to 50 m.

This in turn relates to the practices of:

- Basing transition curve lengths on recommended superelevation runoff lengths (see Tables 11.5 and 11.6; see also 11.9.2 for further explanation) and hence, matching the superelevation runoff with the transition (see 11.4.8 Positioning of Superelevation Development); and
- Not providing transitions when there is sufficient room within a traffic lane for vehicles to make their own transition path (see 11.9.2 Use of Transitions).

Therefore, the normal minimum lengths for superelevation runoff for a curve are:

- 40 m for transitioned curves, which ties to a 40 m minimum transition length.
- 50 m for an untransitioned curve when the curve design speed is greater than 80 km/h, with this length normally being equidistant about the curve tangent point.
- 30 m for an untransitioned curve when the curve design speed is less than or equal to 80 km/h, with this length normally being equidistant about the curve tangent point.

Maximum Rate of Rotation of Crossfall

The following maximum rates of rotation are applicable:

- 0.025 radians per second for most road types. In particular, roads with vehicles that carry livestock.
- 0.035 radians per second in low speed areas (≤70km/h). Typically this includes roads in steep terrain and geometry consisting of reverse curves with little or no straight between the curves.
- 0.04 radians per second in low speed areas (<60km/h) with extremely constricted horizontal alignment i.e. sections of roads in steep terrain and geometry consisting of small radius reverse curves with little or no straight between the curves.

The minimum superelevation development length based on the above criteria is given by Equation 11.7.

$$L_e = 0.278V\frac{e_2 - e_1}{r} \qquad (11.7)$$

where

L_e = superelevation development length (m)

V = design speed (km/h)

e_1, e_2 = crossfall or superelevation at ends of development length (m/m); +ve when sloping upwards from the control line; –ve when sloping downwards from the control line

r = rate of rotation of the road crossfall (radians/second)

Note that for the range of crossfalls involved, m/m ≈ radians.

Relative Grade

The relative grade is the percentage difference between the grade of the edges of the carriageway and the grade of the axis of rotation. This difference should be kept below the values shown in Table 11.4 to achieve a reasonably smooth appearance.

Table 11.4 Maximum Relative Grades Between Edge of Carriageway and Control Line in Superelevation Development for Appearance Criterion

Design Speed km/h	Relative Grade - Per cent		
	One Lane[1]	Two Lanes[2]	More than two Lanes[3]
40 or under	0.9	1.3	1.7
60	0.6	1.0	1.3
80	0.5	0.8	1.0
100	0.4	0.7	0.9
120 or over	0.4	0.6	0.8

1 Applies to normal two-lane roadway with control line on centreline.
2. Applies to: two-lane roadway with control along one edge; four-lane roadway with control on centreline; two-lane roadway with climbing lane and control on centreline of basic two lanes.
3. Applies to multilane roadway with more than two lanes between the control and the edge of running lanes.
4. Relative grade values may be interpolated for design speeds not listed in the table.

The minimum superelevation development length based on the criteria in Table 11.4 is given by Equation 11.8.

$$L_e = 100W\frac{e_2 - e_1}{Gr} \qquad (11.8)$$

where

L_e = superelevation development length (m)

W = maximum width from axis of rotation to edge of running lane (m)

e_1, e_2 = crossfall or superelevation at ends of development length (m/m); +ve when sloping upwards from the control line; –ve when sloping downwards from the control line

Gr = allowable relative grade from Table 11.4 (%).

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Table 11.5 Horizontal Curve Design Parameters for Rural Roads

Curve Design Speed km/h	Radius[1,2,3] m	Super-elevation[1,9] %	Friction coefficient[9]	Plan Transition Type & Length[1,5] m	Super. Criteria Satisfied[6,7]	Desirable min. curve length[8] m
50	44	10	0.35	S60	R,g1,g2	140
	45	9	0.35	S60	R,g1,g2,g3	140
	47	7	0.35	S40	R,g1,g2	100
	55	6	0.30	S40	R,g1,g2,g3	80
	65	5	0.25	U30	R,g1,g2	80
	82	4	0.20	U30	R,g1,g2,g3	80
	109	3	0.15	U30	R,g1,g2,g3	80
60	66	10	0.33	S60	R,g1	140
	71	8	0.32	S60	R,g1,g2	140
	81	7	0.28	S60	R,g1,g2,g3	140
	95	6	0.24	S40	R,g1,g2	100
	113	5	0.20	S40	R,g1,g2,g3	100
	142	4	0.16	U30	R,g1,g2,g3	100
	189	3	0.12	U30	R,g1,g2,g3	100
70	94	10	0.31	S80	R,g1,g2	180
	103	9	0.29	S80	R,g1,g2	180
	116	8	0.25	S60	R,g1,g2	140
	132	7	0.22	S60	R,g1,g2	140
	154	6	0.19	S40	R,g1	140
	185	5	0.16	S40	R,g1,g2	140
	222	5	0.13	U40	R,g1,g2,g3	140
	232	4	0.13	U40	R,g1,g2,g3	140
	309	3	0.13	U30	R,g1,g2,g3	140
80	140	10	0.26	S100	R,g1,g2	240
	153	9	0.24	S80	R,g1,g2	200
	172	8	0.21	S80	R,g1,g2	200
	196	7	0.19	S80	R,g1,g2,g3	200
	204	7	0.18	S60	R,g1,g2	180
	230	6	0.16	S60	R,g1,g2	180
	275	5	0.13	S60	R,g1,g2,g3	180
	300	5	0.12	U50	R,g1,g2,g3	180
	344	4	0.11	U50	R,g1,g2,g3	180
	440	4	0.07	U50	R,g1,g2,g3	180
	441	4	0.07	U50	R,g1,g2,g3	180
	458	3	0.08	U30	R,g1,g2,g3	180
90	213	10	0.20	S100	R,g1,g2	230
	224	9	0.20	S100	R,g1,g2,g3	
	252	8	0.17	S80	R,g1,g2	
	288	7	0.15	S80	R,g1,g2,g3	
	336	6	0.13	S60	R,g1,g2	
	400	5	0.11	U50	R,g1,g2	
	440	5	0.09	U50	R,g1,g2	
	441	5	0.09	U50	R,g1,g2	
	500	4	0.09	U50	R,g1,g2,g3	
	671	3	0.07	U50	R,g1,g2,g3	

Table 11.5 Horizontal Curve Design Parameters for Rural Roads (contd.)

Curve Design Speed km/h	Radius[1,2,3] m	Super-elevation[1,9] %	Friction coefficient[9]	Plan Transition Type & Length[1,5] m	Super. Criteria Satisfied[6,7]	Desirable min. curve length[8] m
100	303	10	0.16	S120	R,g1,g2,g3	280
	315	9	0.16	S100	R,g1,g2	
	328	8	0.16	S100	R,g1,g2,g3	
	375	7	0.14	S80	R,g1,g2,g3	
	437	6	0.12	S80	R,g1,g2,g3	
	441	6	0.12	S80	R,g1,g2,g3	
	525	5	0.10	S60	R,g1,g2,g3	
	650	4	0.08	U50	R,g1,g2,g3	
	875	3	0.06	U50	R,g1,g2,g3	
110	433	10	0.12	S120	R,g1,g2,g3	340
	454	9	0.12	S120	R,g1,g2,g3	
	476	8	0.12	S100	R,g1,g2,g3	
	502	7	0.12	S100	R,g1,g2,g3	
	529	6	0.12	S80	R,g1,g2,g3	
	635	5	0.10	U60	R,g1,g2,g3	
	794	4	0.08	U60	R,g1,g2,g3	
	1059	3	0.06	U60	R,g1,g2,g3	
120	667	6	0.11	S80	R,g1,g2,g3	400
	800	5	0.09	S80	R,g1,g2,g3	
	875	5	0.08	S80	R,g1,g2,g3	
	1000	4	0.07	U60	R,g1,g2,g3	
	1334	3	0.06	U60	R,g1,g2,g3	
130	785	6	0.11	S100	R,g1,g2,g3	470
	939	5	0.09	S80	R,g1,g2,g3	
	1174	4	0.07	U60	R,g1,g2,g3	
	1566	3	0.06	U60	R,g1,g2,g3	

Notes:

1. The circular curve radii listed in the table correspond to changepoints in superelevation and/or transition length. In practice, rounded values of curve radius will usually be used and for any radius falling between two radii listed for a given design speed, the superelevation and transition values for the smaller radius will apply.
2. The minimum radius listed for a given design speed is the absolute minimum radius possible, being that which corresponds to the maximum possible superelevation and maximum coefficient of side friction.
3. The radius corresponding to 6% superelevation is the desirable minimum for most design situations, being that which corresponds to a desirable maximum superelevation of 6% and the desirable maximum coefficient of side friction.
4. The shaded rows in the table represent cases that should be avoided where possible.
5. Plan Transition Type: S denotes a Spiral transition is required for the application of superelevation; U denotes that the curve is untransitioned with the superelevation runoff being applied over the given length in accordance with Figure 11.1.
6. Transition Criteria Satisfied: R denotes rate of rotation criterion; g1 denotes relative grade for 1 lane; g2 denotes relative grade for 2 lanes; g3 denotes relative grade for 3 or more lanes. See Table 11.4. For the 50, 60 and 70km/h design speeds, the g1 criterion yields longer transition lengths than the rotation criterion (0.035 rad/s) so that the rounded transition lengths end up being sufficient for a rotation of 0.025 rad/s. 0.025 rad/s is used for all curve design speeds > 70 km/h.
7. If a required transition criterion is not satisfied, use a plan transition length 20m longer than shown in the table.
8. Desirable minimum curve length is for appearance. It includes the length of any spiral transitions.
9. **Other combinations of superelevation and coefficient of side friction are possible for a given radius and curve design speed. Other engineering requirements are drainage of the pavement surface, constraint on change in superelevation between adjoining curves, temporary connections etc. However, the combination should be supported by some engineering requirement. In particular, the need to reuse existing pavement may mandate a different superelevation with the coefficient of side friction being checked against desirable and/or absolute maximum limits and for consistency with any adjacent horizontal curves.**

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Table 11.6 Horizontal Curve Design Parameters for Urban Roads

Curve Design Speed km/h	Radius[1,2,3] m	Super-elevation[1,9] %	Friction coefficient[9]	Plan Transition Type & Length[1,5] m	Super. Criteria Satisfied[6,7]	Desirable min. curve length[8] m
40	32	5	0.35	U25	R,g1,g2	40
	36	5	0.30	U20	R,g1	40
	45	4	0.24	U20	R,g1,g2	40
	60	3	0.18	U15	R,g1,g2	40
50	49	5	0.35	U25	R,g1	50
	56	5	0.30	U30	R,g1,g2	50
	56	5	0.30	U25	R,g1	50
	70	4	0.24	U25	R,g1,g2	70
	94	3	0.18	U20	R,g1,g2,g3	70
60	75	5	0.33	S40	R,g1,g2,g3	100
	98	5	0.24	S40	R,g1,g2,g3	100
	122	4	0.19	U30	R,g1,g2,g3	100
	163	3	0.14	U20	R,g1,g2	100
70	107	5	0.31	S40	R,g1,g2	140
	161	5	0.19	S40	R,g1,g2	140
	200	4	0.15	U30	R,g1,g2	140
	268	3	0.11	U30	R,g1,g2,g3	140
80	163	5	0.26	S60	R,g1,g2,g3	180
	240	5	0.16	S60	R,g1,g2,g3	180
	300	4	0.13	U40	R,g1,g2,g3	180
	400	3	0.10	U30	R,g1,g2,g3	180
	440	3	0.08	U30	R,g1,g2,g3	180
	441	3	0.08	U30	R,g1,g2,g3	180
90	255	5	0.18	S60	R,g1,g2,g3	230
	354	5	0.13	U50	R,g1,g2	230
	440	5	0.10	U50	R,g1,g2,g3	230
	443	4	0.10	U50	R,g1,g2,g3	230
	600	3	0.08	U50	R,g1,g2,g3	230
100	375	5	0.16	S60	R,g1,g2,g3	280
	463	5	0.12	S60	R,g1,g2,g3	280
	579	4	0.10	U50	R,g1,g2,g3	280
	772	3	0.07	U50	R,g1,g2,g3	280
110	560	5	0.12	S80	R,g1,g2,g3	340
	700	4	0.10	U60	R,g1,g2,g3	340
	934	3	0.07	U60	R,g1,g2,g3	340
120	709	5	0.11	S80	R,g1,g2,g3	400
	886	4	0.09	U60	R,g1,g2,g3	400
	1200	3	0.07	U60	R,g1,g2,g3	400

Notes:

1. The circular curve radii listed in the table correspond to changepoints in superelevation and/or transition length. In practice, rounded values of curve radius will usually be used and for any radius falling between two radii listed for a given design speed, the superelevation and transition values for the smaller radius will apply.
2. The minimum radius listed for a given design speed is the absolute minimum radius possible, being that which corresponds to the maximum possible superelevation and maximum coefficient of side friction.

Notes to Table 11.6 (contd):

3. The radius corresponding to 5% superelevation is the desirable minimum for most design situations, being that which corresponds to a desirable maximum superelevation of 5% and the desirable maximum coefficient of side friction.
4. The shaded rows in the table represent cases that should be avoided where possible.
5. Plan Transition Type: S denotes a Spiral transition is required for the application of superelevation; U denotes that the curve is untransitioned with the superelevation runoff being applied over the given length in accordance with Figure 11.1.
6. Transition Criteria Satisfied: R denotes rate of rotation criterion; g1 denotes relative grade for 1 lane; g2 denotes relative grade for 2 lanes; g3 denotes relative grade for 3 or more lanes. See Table 11.4. For the 40, 50, 60 and 70km/h design speeds, the g1 criterion yields longer transition lengths than the rotation criterion (0.035 rad/s) so that the rounded transition lengths end up being sufficient for a rotation of 0.025 rad/s. 0.025 rad/s is used for all curve design speeds > 70 km/h.
7. If a required transition criterion is not satisfied, use a plan transition length 20m longer than shown in the table.
8. Desirable minimum curve length is for appearance. It includes the length of any spiral transitions.
9. **Other combinations of superelevation and coefficient of side friction are possible for a given radius and curve design speed. Other engineering requirements are drainage of the pavement surface, constraint on change in superelevation between adjoining curves, temporary connections etc. However, the combination should be supported by some engineering requirement. In particular, the need to reuse existing pavement may mandate a different superelevation with the coefficient of side friction being checked against desirable and/or absolute maximum limits and for consistency with any adjacent horizontal curves.**

11.4.7 Positioning of the Axis of Rotation

The adopted position of the axis of rotation, i.e. the point about which the crossfall is rotated to develop superelevation, depends on:

- the type of road facility
- total road cross section adopted
- terrain
- the location of the road.

On a two-lane, two-way road, the superelevation is normally developed by rotating each half of the cross section (including shoulders) about the carriageway centreline (axis of rotation).

On divided rural roads where the median is relatively narrow, i.e. less than 5m, the two carriageways may be rotated about the centreline of the median. Where the median is wide, the axis of rotation is usually along the median edge (particularly in flat country).

With the two-lane, two-way road case and the narrow median divided road case, the application of superelevation can result in the profiles at the edges of the carriageways being in opposite vertical directions to each other, or one of the profiles being flat. Where this causes any drainage or appearance problems, the application of the superelevation will need to be modified. The subjective nature of both the rotation and relative grade controls means that there is some scope to modify the application of superelevation.

Any appearance problems are likely to be more noticeable if an edge line is defined by a kerb, guard rail or bridge barrier.

11.4.8 Positioning of Superelevation Development

Transitioned Curves

For curves with transitions, it is normal practice to match superelevation runoff with the transition. That is, the point of 0% crossfall corresponds to the start of the transition (for a vehicle entering the curve) and the full superelevation for the curve (e%) is attained at the end of the transition. The superelevation development is extended back from the start at the same rotation rate to the point of normal crossfall on the approach tangent. Figure 11.1 uses a superelevation diagram to further explain this practice.

The theoretical basis for the uniform development of the superelevation along the length of the transition is that when combined with the uniform increase in curvature provided by the clothoid spiral transition, there will be a uniform attainment of the side friction demand that will be used on the horizontal curve.

Untransitioned Curves

Normal practice for positioning the superelevation development for curves without transitions is as follows:

- ***Tangent to Curve and Vice Versa:*** Positioned such that 50% of the superelevation runoff length (from 0% crossfall to e% for the curve) is located on the tangent and 50% is located on the curve. This positioning is considered to correspond with the transition path that is described by the majority of vehicles entering or leaving the curve. However, in constrained situations such as mountainous areas or urban roads, shorter than desirable arc lengths may force the positioning to be 70% on the tangent and 30% on the curve (see Figure 11.1).

  Development of the superelevation entirely on the tangent for an untransitioned curve is normally not preferred because the superelevation development will not correspond sufficiently with the transition path that is described by the majority of vehicles. Therefore when vehicles approach the curve, there will be a section where a vehicle will still be travelling straight and the crossfall is sufficient to affect tyre slip angles. This will cause drivers to steer slightly in the direction opposite to the pending curve.

  Development of the superelevation entirely on the tangents is not precluded however. There can be exceptional circumstances that favour this practice. The most common will be for exiting from a horizontal curve on a steep downgrade. Application of any curve widening will have to be treated separately in such cases.

- ***Reverse Curves with no intervening straight:*** Zero crossfall is provided at the common tangent point. The full superelevation runoff is then located on each curve. See also Section 11.6.

- ***Compound Curves:*** Where compound curves are provided, the full superelevation on the sharper curve must be retained to the common tangent point. Therefore, the superelevation development length is provided on the larger radius curve.

11.5 Lengths of Horizontal Curves

With deflection angles less than 1 degree, a curve is not required.

At small angles greater than 1 degree, the appearance of a kink must be avoided. This is achieved by the use of long curves as shown in Table 11.7.

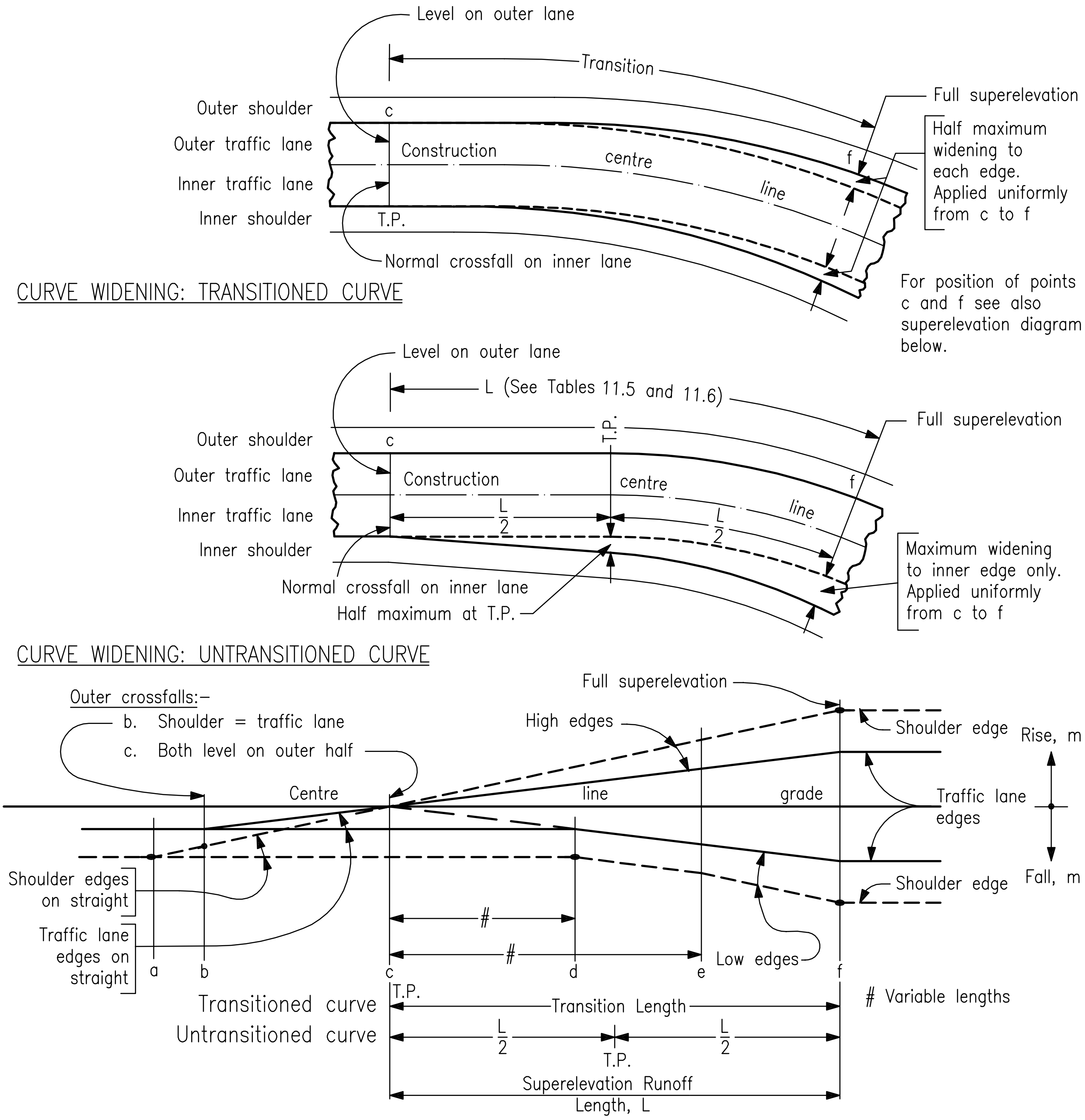
Table 11.7 Curves for Small Deflection Angles

Angle	Min.Lth. of Arc* metres	Radius Metres Calculated Value	Radius Metres Rounded Value	Secant Metres
5°	150	1719	1800	1.715
4°	180	2578	2600	1.585
3°	210	4011	4000	1.371
2°	240	6875	7000	1.066
1°	270	15470	15000	0.572
Less than 1° - No curve required				

* Minimum length of arc to be 150 m for a 5° angle plus an additional 30 m for each degree less than 5°.

The lengths of curves given in Table 11.7 are more applicable to important and high speed roads than to minor roads. The lengths do not apply in difficult locations such as mountainous terrain.

Besides appearance requirements, horizontal curves must be long enough to accommodate the application of superelevation. The desirable minimum lengths given in Tables 11.5 and 11.6 are based on:



This diagram shows that the higher or outer edge of the outer shoulder commences to rise back on the straight at the point a and the higher or outer edge of the outer traffic lane at the point b at which point the rising shoulder crossfall equals the traffic lane crossfall. Both the traffic lane and shoulder on this side are level at c. Point c is the control point for the changing crossfalls. As shown, point c is the T.P. on the transitioned curve and normally L/2 ahead of the T.P. on the untransitioned curve. Between b and f the shoulder and traffic lane crossfalls are equal at all points. The outer shoulder on the curve has the same crossfall as the adjacent traffic lane.

On the inner traffic lane the normal crossfall is not altered until the point d is reached where the outer traffic lane is at the same crossfall; it then changes uniformly to the full superelevation at the point f at the same rate as the outer traffic lane.

On the inner shoulder the steeper shoulder slope on the straight is retained to the point e where the adjacent traffic lane is at the same crossfall. The shoulder and traffic lane then change together uniformly to the full superelevation at point f. If the traffic lane superelevation does not exceed the shoulder crossfall on the straight, the inner shoulder continues unchanged throughout the curve.

Although this diagram addresses the case where the shoulder crossfall on the straight is steeper than the traffic lane crossfall on the straight, it is now common practice for these crossfalls to be the same.

For this diagram TP = curve tangent point
∴ TP = the TS and ST points on a transitioned curve
= the TC and CT points on an untransitioned curve.

Figure 11.1 Standard Methods of Applying Curve Widening and Superelevation

- 30m minimum length of arc with full superelevation (see 11.4.5);
- transition curve lengths.

There is scope to use shorter lengths when the curve is untransitioned because half of the superelevation runoff is applied on the tangent.

11.6 Reverse Curves

Reverse Curves are horizontal curves turning in opposite directions that adjoin (have common tangent points) or have a short length of tangent between the curves. Desirably, reverse curves should not be used unless there is sufficient distance between the curves to introduce the full superelevation required for each of the two curves without exceeding the standard rate of change of crossfall for the particular design speed.

The following cases of reverse curves are acceptable as shown in Figure 11.2A:

- Case 1, reverse **transitioned** curves with a common point of tangency - these can accommodate the required change in superelevation with the point of zero crossfall occurring at the tangent point. See Case 1 in Figure 11.2A. This case requires no change in the method of applying any curve widening.
- Case 2, reverse **transitioned** curves with a sufficient length of intervening tangent to have a section of the tangent with normal crossfall. On a two-lane, two-way road with the control line down the center of the road, a spacing greater than about 0.7V metres (V in km/h) results in a normally crowned length of tangent. See Case 2 in Figure 11.2A. It has been common practice to try to achieve a minimum of 30 metres of normally crowned tangent. It is often possible to increase the length of tangent between the two curves by slightly increasing the spacing between the centres of the two circular arcs. This case requires no change in the method of applying any curve widening.
- Case 3A, reverse **circular** curves with a common point of tangency are possible for operating speeds less than or equal to 80 km/h if the superelevation on each curve does not exceed 3% and a nominal rotation rate of 0.025 rad/s is used. This allows the superelevation to change from zero crossfall at the common tangent point to 3% in 25 metres (or less) in line with established practice of attaining full superelevation 25 metres into an untransitioned curve. See Case 3A in Figure 11.2A. However, the need to switch curve widening from one side of the road to the other will require the treatment shown in Figure 11.5. For lower operating speeds, the acceptable rate of pavement rotation (0.035 rad/s or 0.04 rad/s) may allow more than 3% superelevation to be achieved over the 25m length. Case 3A represents acceptable compromising of the location of the superelevation runoff.
- Case 3B, reverse **circular** curves spaced such that the length of intervening tangent is less than the length in case 4A or case 4B (depending upon the operating speed). As for case 3A, this case is based on attaining full superelevation within a maximum distance of 25 metres into each curve. The pavement is rotated directly between the two curves as shown in Case 3B in Figure 11.2A. Depending upon the spacing between the two curves, this case is slightly less restrictive than Case 3A. The maximum superelevation that is achievable on each curve depends upon the maximum rate of rotation for the operating speed. If the maximum achievable superelevation for a curve is less than that needed for the operating speed, this case is unacceptable. The need to switch curve widening from one side of the road to the other may require the treatment shown in Figure 11.5. Case 3B represents acceptable compromising of the location of the superelevation runoff.
- Case 4A, reverse **circular** curves spaced at about $(e_1 + e_2)V/0.18$ metres (where V is the operating speed in km/h, e_1 and e_2 are the absolute value of the superelevation on each curve in m/m). The rate of rotation of the pavement is nominally 0.025 rad/s. This case will accommodate the standard positioning of the superelevation development for each circular curve (see 11.4.8 and Figure 11.1). The pavement will be rotated directly between the

two curves as shown for Case 4 in Figure 11.2A. This case will also provide scope for switching any curve widening from one side of the road to the other in the normal manner (see Figure 11.1). However, the treatment shown in Figure 11.5 may simplify construction and have better appearance.

- Case 4B, reverse **circular** curves spaced at about $(e_1 + e_2)V/0.25$ metres (where V is the operating speed in km/h, e_1 and e_2 are the absolute value of the superelevation on each curve in m/m). The rate of rotation of the pavement is nominally 0.035 rad/s because the operating speed is less than or equal to 70km/h. This case will accommodate the standard positioning of the superelevation development for each circular curve (see 11.4.8 and Figure 11.1). The pavement will be rotated directly between the two curves as shown for Case 4 in Figure 11.2A. This case will also provide scope for switching any curve widening from one side of the road to the other in the normal manner (see Figure 11.1). However, the treatment shown in Figure 11.5 may simplify construction and have better appearance.

- Case 5, reverse **circular** curves with a sufficient length of intervening tangent to have a section of the tangent with normal crossfall. On a two-lane, two-way road with the control line down the centre of the road, a spacing greater than about 1.5V to 2V metres (V in km/h) results in a normally crowned length of tangent. See Case 5 in Figure 11.2A. It has been common practice to try to achieve a minimum of 30 m of normally crowned tangent. It is often possible to increase the length of tangent between the two curves by slightly increasing the spacing between the centres of the two circular arcs. This case requires no change in the method of applying any curve widening.

- Acceptable cases involving a transitioned curve that is reversed with a circular curve can be inferred from the above cases. For the circular curve, the superelevation runoff length (zero crossfall to full superelevation for the curve) corresponds to the assumed transition path that is described by vehicles.

Most other spacing of reverse curves will require some compromising of the superelevation development method and/or location. Case 6 in Figure 11.2A shows the most common situation. Case 6 occurs when there is a sufficient length of tangent between the two curves to have an acceptable rate of rotation of the pavement but there is no room to have a length of tangent with normal crossfall. The figure shows that the standard superelevation development method and location can be accommodated (see 11.4.8 and Figure 11.1). In fact, the treatment of each traffic lane is no different from that for Cases 2 and 5 above. However, it is usually preferable to directly rotate the pavement between the two curves as also shown in the figure. This alternative treatment simplifies setting out and construction. It also results in a lower rate of rotation of the pavement so that drainage of the pavement surface must be checked. If a circular curve is involved, there is scope to have more superelevation development occur on the tangent if this helps drainage of the pavement surface.

Case 6 requires no change in the method of applying any curve widening. However, if the pavement is directly rotated between the two curves and each curve requires curve widening, the curve widening may be switched in conjunction with the pavement rotation.

Where the spacing between reverse curves will not even permit acceptable compromising of the development of superelevation, the spacing between the curves will need to be adjusted. In many cases, this can be achieved by slightly adjusting the spacing between the centres of the two circular arcs and having a different orientation of the intervening tangent.

Where the operating speed of each curve is different, the speed (V) to be used when designing the superelevation development between the curves depends on the following conditions:

- For cases 2 and 5 above, the curves are spaced sufficiently for the superelevation development to be treated separately for each curve and based on the operating speed for each curve. With Case 1 and where a transitioned curve is involved in Case 6, the transitions and superelevation development will normally be based on the operating speeds of the respective curves.

- For the other cases, the spacing is close enough for one curve to influence the speed towards the end of the other curve. Where a decrease in speed is involved, drivers will have to start slowing towards the end of the leading curve. Given the deceleration lengths (see Table 13.16) and curve perception requirements for a speed change that is in accordance with Section 6.6 or Section 14.10, it is appropriate to base the superelevation development on the mean of the operating speeds for the two curves. On a one-way road where a speed increase is involved, drivers are not likely to increase speed until they are well into the second curve (see Figure 6A.2). Therefore, it is appropriate to base the superelevation development on the operating speed for the leading curve.

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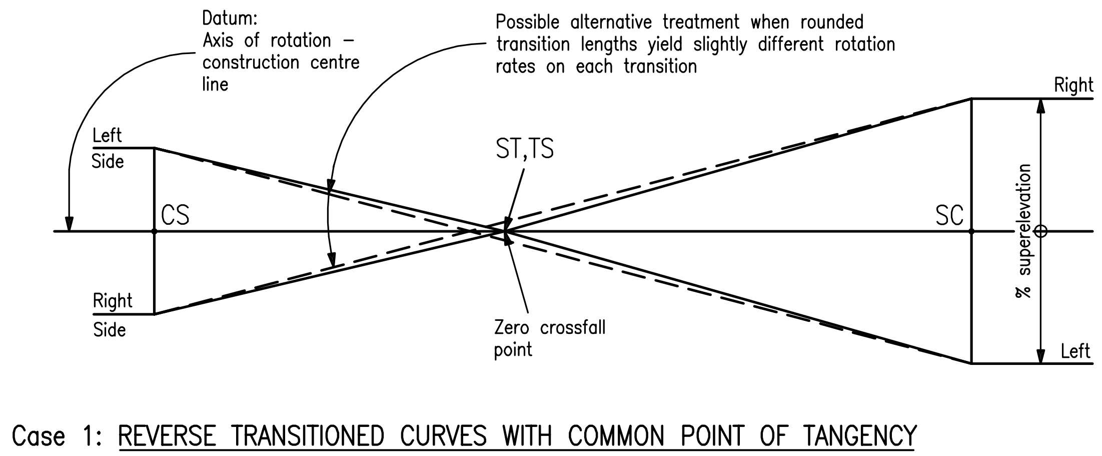
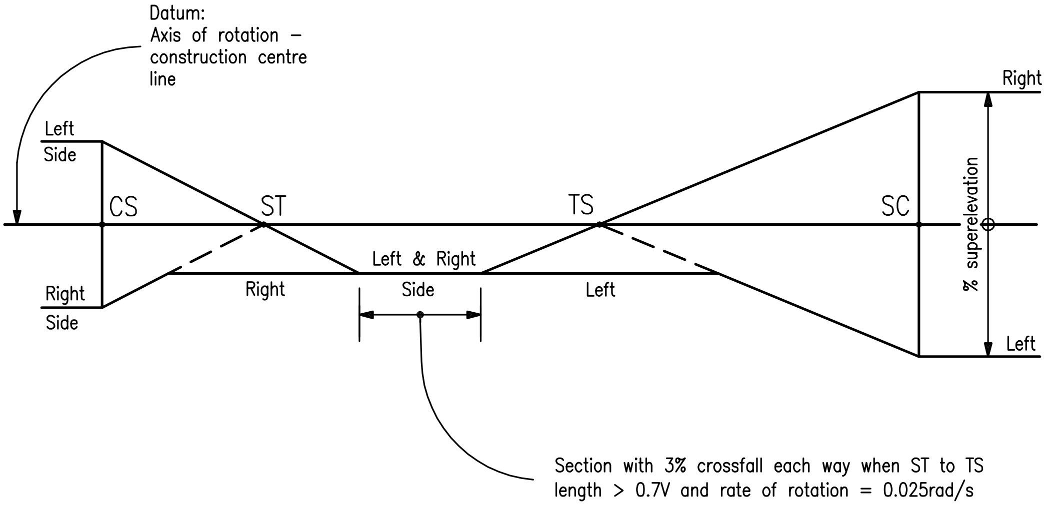
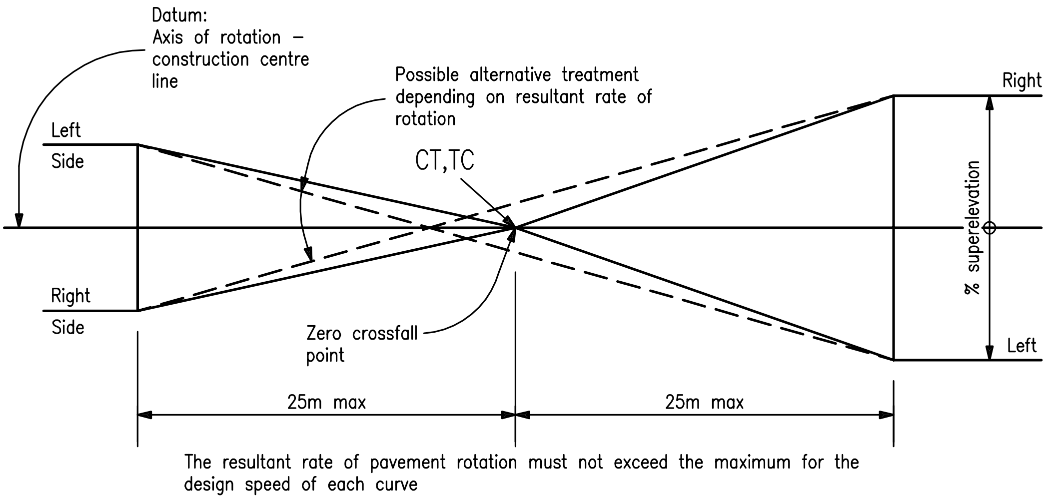


Figure 11.2A Superelevation Development between Reverse Curves (Case 1)

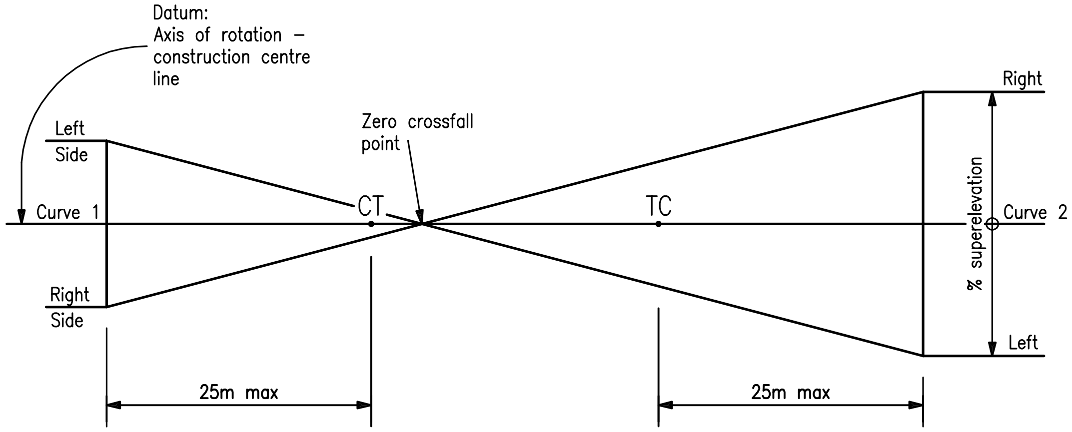


Case 2: REVERSE TRANSITIONED CURVES WITH LENGTH OF INTERVENING TANGENT > 0.7V
(Corresponds to the standard case in Figure 11.1)



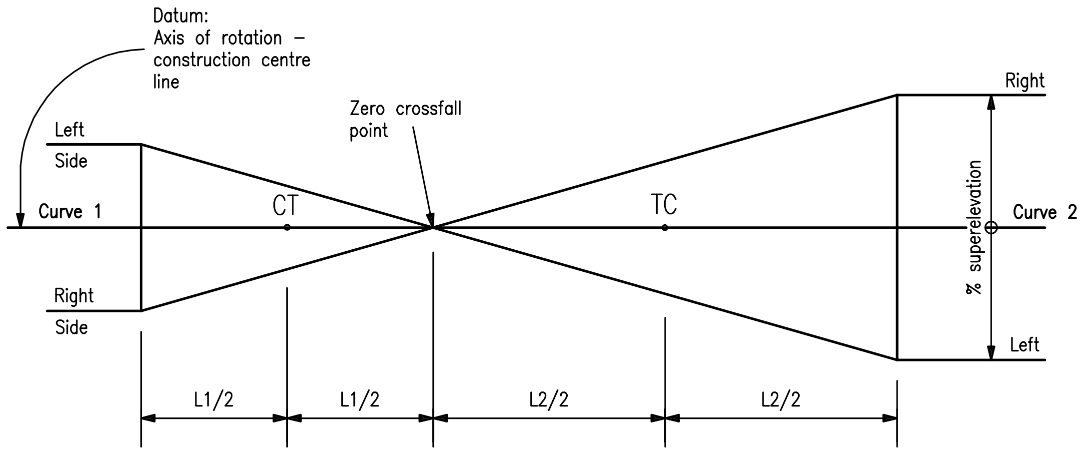
Case 3A: REVERSE CIRCULAR CURVES WITH COMMON TANGENT POINT

Figure 11.2A Superelevation Development between Reverse Curves (Cases 2 and 3A)



The maximum superelevation on each curve is controlled by the resultant rate of rotation of the pavement being less than the maximum for the operating speed. If this superelevation is less than that required for the curve, this case is unacceptable.

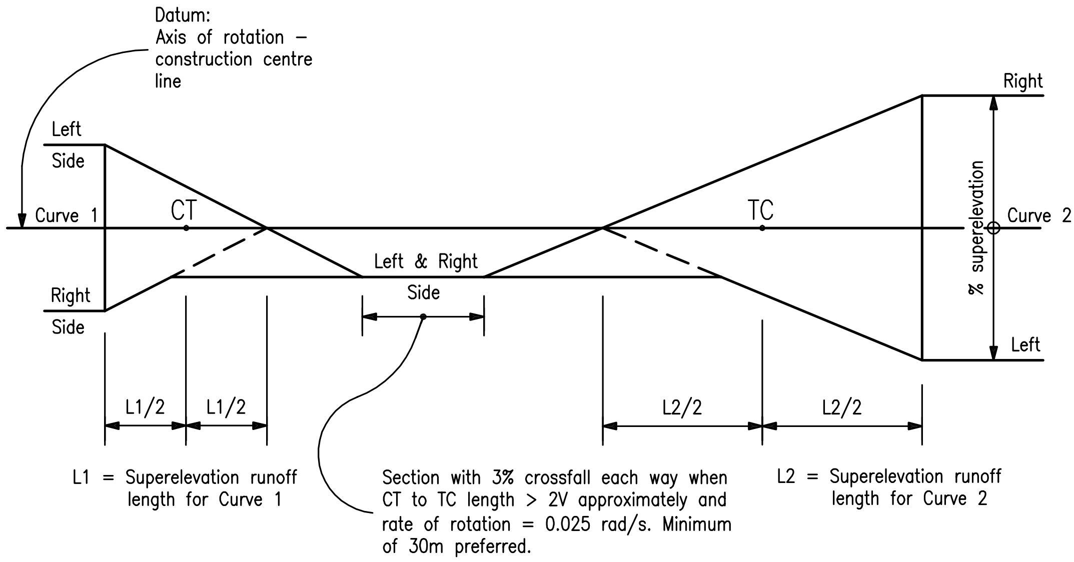
Case 3B: REVERSE CIRCULAR CURVES WITH SPACING < SPACING FOR Case 4



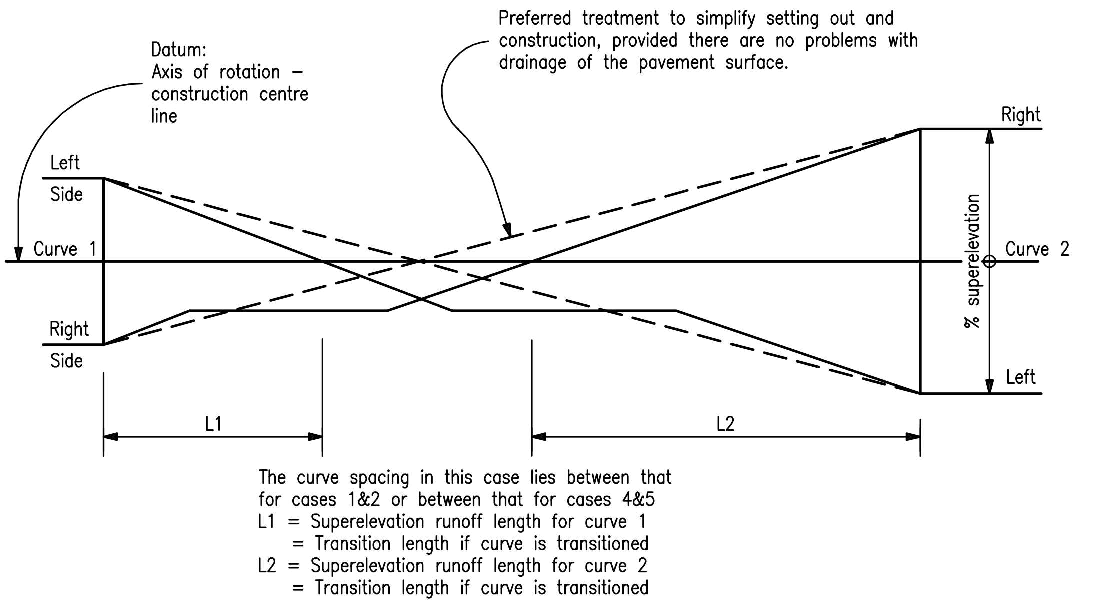
For L1 and L2, see Section 11.4.5
L1 = Superelevation runoff length for Curve 1
L2 = Superelevation runoff length for Curve 2

Case 4 (Covers both Cases 4A and 4B in Section 11.6):
REVERSE CIRCULAR CURVES SPACED SUCH THAT THE PAVEMENT MAY BE ROTATED AT THE NOMINAL MAXIMUM RATE BETWEEN THE CURVES
(Untransitioned equivalent of Case 1)

Figure 11.2A Superelevation Development between Reverse Curves (Cases 3B and 4)



Case 5: REVERSE CIRCULAR CURVES SPACED SO THAT THERE IS A SECTION OF TANGENT WITH NORMAL CROSSFALL
(Corresponds to the standard case in Figure 11.1)



Case 6: REVERSE CURVES SPACED SO THAT THERE IS NO SECTION OF INTERVENING TANGENT WITH NORMAL CROSSFALL
(Derived from the standard case in Figure 11.1)

Figure 11.2A Superelevation Development between Reverse Curves (Cases 5 and 6)

11.7 Broken Back Curves

Broken back curves (also called similar curves) are horizontal curves turning in the same direction joined by a short length of straight or two relatively small unidirectional curves connected by a large radius curve. Broken back curves should be **avoided** if possible.

There are two general cases of broken back curve:

- Where the length of straight is less than about 0.6V metres (based on about 2 seconds travel time with V = operating speed in km/h), the separation of the curves is usually small enough so that there is no visual complication or problems with superelevation. Such curvature may be tolerated in urban areas if there is a need to maintain existing pavement and kerbing. However, it is often possible and preferable to substitute a single curve depending on the difference in the curve radii.

  If a single curve cannot be substituted, the short length of straight and the difference in curve radii will cause the same problems for motorcyclists that compound curves cause.

- Where the length of straight is greater than about 0.6V and less than 2V to 4V, appearance is compromised by there not being sufficient separation of the curves. Within this range of spacing, it is often possible and preferable to substitute a single curve depending on the difference in the curve radii. The length 2V metres may be taken as the absolute minimum with 4V metres the desirable minimum. Even a distance of 4V may be insufficient if both curves are visible at the same time over a long distance. A length of straight greater than about 3V will allow normal crossfall on the straight to be regained for about 4 seconds of travel.

Figure 11.2B shows how superelevation may be treated when a broken back curve can not be avoided. All potential cases of broken back curve should be checked for appearance by perspective views in each direction. These can be readily generated by road design modelling systems.

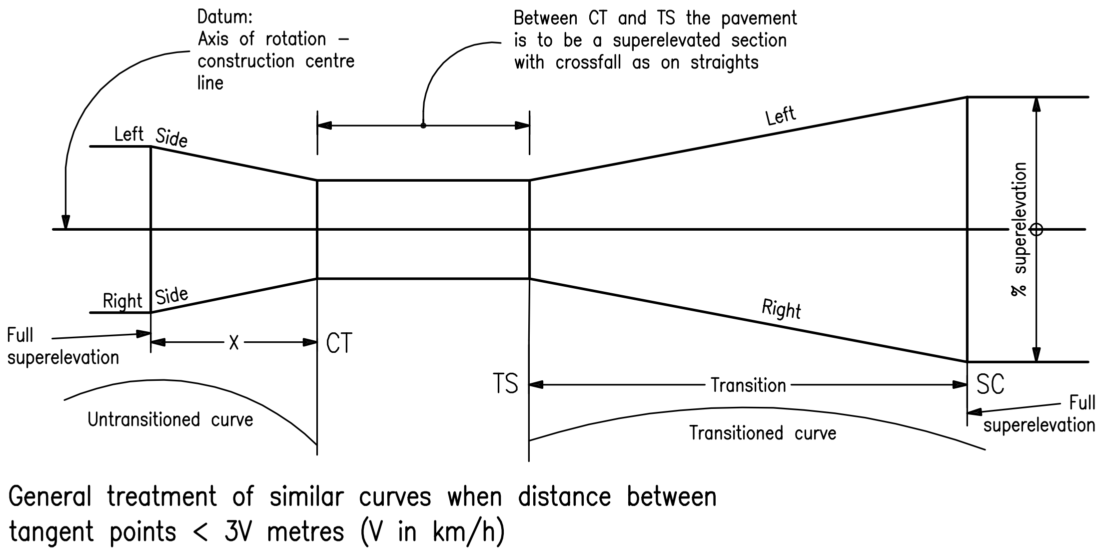


Figure 11.2B Superelevation Development between Similar Curves

11.8 Compound Curves

Compound curves are horizontal curves of different radii turning in the same direction with a common tangent point. When radii less than about 1000m are involved, compound curves may cause operational problems due to problems with drivers not perceiving the change in curvature and drivers not anticipating a change in side friction demand.

Although not conclusive, some literature from throughout the world suggests that a smaller radius curve immediately following a larger radius curve (both turning in the same direction) gives drivers inadequate perception of the smaller radius curve which leads to a higher single vehicle accident rate. This is a particular problem where limited visibility of the smaller radius curve exists; for example, where vegetation or a cut-face on the inside of the larger radius curve limits visibility and driver perception of the smaller radius curve. Furthermore, compound curves involve an undesirable change in side friction demand for motorcycles. Generally, this geometry should be **avoided**.

Where compound curves can not be avoided:

- There should be no more than two curves of diminishing radii. And the radius of the smaller curve should be at least 2/3 of the radius of the larger curve.
- Any change in design speed between the two circular elements must be within the limits given in Table 6.4.
- The change in side friction demand must meet the criteria given in 11.4.2.
- Diminishing radii in the downhill direction should be avoided on steep downgrades.
- On a one-way roadway a smaller curve preceding a larger curve is preferable.

11.9 Transition Curves

11.9.1 General

Any motor vehicle describes a transition path as it changes from a straight to a circular horizontal curve and vice versa, or between the elements of a compound curve. The way drivers, in general, steer a vehicle at speed results in a path that provides a reasonably uniform attainment of centripetal acceleration. For most curves, the average car driver can achieve a suitable transition path within the limits of normal lane width. However, with particular combinations of high speed, heavy vehicles and a large difference in curvature between successive geometric elements, the resultant vehicle transition path can result in encroachment into adjoining lanes. Trucks have more problems because of their greater width, longer wheel base and heavier, less responsive steering. Also, trucks have a greater swept path width on curves as explained in Section 11.10 on Curve Widening.

In cases where the combination of lane width, vehicle speed and curve radius do not allow sufficient space for a vehicle to describe a suitable transition path, it is necessary to provide a transition curve between the tangent and the circular arc. The need for such transition curves was learned from the early days of railway building when problems were encountered with passenger comfort and track wear due to the sudden application of curvature with untransitioned curves. However, the fact that road vehicles are not rigidly confined to a specific path together with the characteristics of road vehicle steering mean that shorter transition lengths are more appropriate than those used for railways. This is why it is current road design practice to base transition lengths on superelevation runoff length (see 11.4.6 and 11.9.2 for further explanation) instead of a comfort criterion that was once used.

Transition curves provide the following advantages:

- A properly designed transition curve allows the vehicle's centripetal acceleration to increase or decrease gradually as the vehicle enters or leaves a circular curve. This transition curve minimizes encroachment on adjoining traffic lanes.

- The transition curve length provides a convenient desirable arrangement for superelevation runoff. The change in the cross fall can be effected along the length of the transition curve in a manner closely fitting the radius-speed relation for the vehicle traversing it.
- A transition facilitates the change in width where the pavement section is to be widened around a circular curve. Use of transitions provides flexibility in the widening on sharp curves.
- The appearance of the highway or street is enhanced by the application of transitions. On multilane divided roads where the curve operating speeds are consistent with the operating speeds on the straights (typically, greater than 100 km/h), there is scope to improve the appearance by using longer transitions.
- Transitioned curves simplify the application of curve widening on closely spaced curves in constrained mountainous situations because they avoid having to switch the widening from one side of the road to the other.

The use of longer transitions than those based on normal superelevation runoff length should be avoided when curve operating speeds are such that drivers have to reduce speed for the curve (see 11.12). Long transitions should only be considered in high speed curvilinear alignments (see Chapter 10) that are designed for a uniform speed of travel.

Effect on Braking

When drivers brake on curves, a combination of forces apply on the tyres, effectively reducing the maximum force that can be developed for braking or cornering. Articulated trucks also have problems with braking on curves because of the tendency of these vehicles to jackknife. On transitioned curves where the operating speed is such that drivers are expected to reduce speed prior to the curve, there is a likelihood of braking on the transition if a long transition is used. This is because the change in curvature is not sufficient to identify the approximate start of the transition.

Effect on Overtaking

Transitions can reduce the length of straights between curves, effectively reducing the length of possible overtaking opportunities.

Effect on Design

Transitions involve additional work both in design and construction. Transitions that are longer than the superelevation runoff length for the curve design speed, but still have the superelevation runoff matched with the longer transition length, are more likely to have problems with drainage of the pavement surface. If this occurs, it will be necessary to compromise the application of the superelevation.

11.9.2 Use of Transitions

It is normal practice for horizontal curves to be transitioned, with the transition length based on the superelevation runoff length for the recommended combination of speed, radius and superelevation in Tables 11.5 and 11.6 (see 11.4.6). This length criterion provides the advantages given in Section 11.9.1 while minimising the negative effects on driver perception, braking and overtaking that are associated with long transitions. It is also normal practice to round the transition length upwards if necessary to the next standard length of: 40, 60, 80, 100, 120 and 140 m. The rounding is primarily in the interests of uniformity and to avoid attributing undue precision to the calculated length. However, in constrained situations, adjacent curves may dictate the use of a transition length rounded upwards if necessary to the next multiple of 5 m.

If circumstances favour a lower superelevation value than that shown in Tables 11.5 and 11.6 (see Note 9 for each table), it is not appropriate to use a shorter transition for the reduced superelevation. This is because the recommended superelevation values in Tables 11.5 and 11.6 yield transition lengths for the various radius/speed combinations that adequately match driver behaviour. It is not a case of transition length being solely related to whatever superelevation is applied (and hence, superelevation runoff length).

Transition curves involve a shift of the circular arc in order to accommodate the transition (see Appendix 11A). When this shift is less than 0.25 to 0.3 m, the transition can be omitted because there is sufficient room for a vehicle to describe a transition path. The test for omitting the transition should be based on the length originally calculated for the superelevation runoff (from 0% crossfall to the rounded superelevation value for the curve) before any rounding of the length is applied.

Due to the characteristics of vehicle steering geometry, there is a transient state (or distance) where it is possible for the front (steered) wheels of a vehicle travelling at slow speed to follow a circular path while the steering angle is changed from straight ahead to the maximum angle needed to describe the turn. This is a major reason why transitions are not needed for intersection turns and most curves with a design speed below 60 km/h.

Tables 11.5 and 11.6 reflect the above criteria with the recommended transition lengths for various combinations of radius and speed. The tables also show when a transition should be omitted together with the superelevation runoff length that is then used. The superelevation runoff lengths for untransitioned curves show the normal rounded values. However, in constrained situations, adjacent curves may dictate the use of superelevation runoff lengths that are rounded upwards if necessary to the next multiple of 5 m instead of the normal 10 m. For untransitioned curves, the superelevation is applied as described in Section 11.4.8.

11.9.3 Types of Transitions

The standard form of transition curve is the clothoid spiral. Other curves have been used in the past such as the cubic parabola, combination of cubic parabola and cubic spiral, lemniscate and a suitably large radius compound curve. All of these alternatives were advocated on the basis that they closely matched the shape of the clothoid spiral but were simpler to calculate. The disadvantage of the first three alternatives was that there were points of discontinuity which were conveniently 'smoothed out' during construction. Furthermore, for simplifying calculations, arc lengths were assumed to equal the tangent length of the transition curve. The combination of cubic parabola and cubic spiral was widely used for transition curves on the Queensland road network prior to the adoption of the clothoid spiral. Many overlay and widening schemes will encounter this form of transition. But it will be possible to fit a control line that contains clothoid spirals instead.

The clothoid spiral was adopted because the simplifications that were assumed for the alternative forms could not be conveniently incorporated into computer programs that came to be used for design calculations. Furthermore, electronic calculators and portable computers made field calculations feasible.

With a defining property of the clothoid spiral being that curvature (reciprocal of the radius) varies uniformly along the spiral (rather than the tangent), other convenient properties can be derived:

- If vehicles travel at a uniform speed and follow the transition curve closely enough, they will experience a uniform change in centripetal acceleration as they enter and exit the circular section of the curve.
- In turn, by matching superelevation runoff to the transition, the uniform rate of application of the superelevation then results in a uniform attainment of side friction force.

The properties of transition curves that are relevant to road design and the derivation of equations for use in design calculations are given in Appendix 11A.

11

11.10 Curve Widening

11.10.1 General

On smaller radius horizontal curves, traffic lanes may need to be widened in order to maintain the lateral clearances that apply to vehicles on straight sections of road. This is due to the following vehicle and driver characteristics:

- The rear wheels of a vehicle offtrack with respect to the front wheels on a curve and this also causes the front overhang of a vehicle to have a lateral component. The offtracking can be either what is commonly called low speed offtracking (or more correctly, offtracking when vehicles have a low side friction demand) or high speed offtracking (or more correctly, offtracking when vehicles have a high side friction demand). With the former form of offtracking, the rear wheels offtrack towards the centre of the curve; with the latter form, the rear wheels offtrack outwards from the centre of the curve.
- Vehicles deviate more from the centreline of a lane when on a curve.

The amount of widening per lane depends on:

- The radius of the curve
- Type (size) of vehicle operating on the road.
- Some allowance for steering variation by different drivers.

However, there is a lower practical limit to widening due to construction feasibility. For a two-lane road, curve widening is omitted when the total widening is less than 0.5m.

Curve widening will be based on the design vehicle for the section of road. This means that a B-Double will be used on B-Double routes and the appropriate type of road train will be used on road train routes. For other roads, the design prime mover & semi-trailer will normally be used given the incidence of these vehicles on state highways and many other main roads (see Cox 1998). On roads where the incidence of prime mover & semi-trailer operation is small (less than about 3% of AADT), or where for some other reason the single unit truck is used as the design vehicle, the curve widening may be based on single unit truck operation; for example, the single unit truck is commonly used as the design vehicle for turns between minor streets and urban sub-arterial roads.

With one possible exception, curve widening is always based on vehicles of the same type meeting or passing each other; even when the incidence of design vehicles such as road trains is low. This is primarily in the interests of safety with the lane marking accommodating the swept path of the design vehicle in each lane. Furthermore, by applying meeting rate equations (see Taneka and Troutbeck 1996) it can be shown that a flow rate of three design vehicles per hour in each direction will result in an average of one meeting per day on a given curve.

The possible exception relates to curve widening on a climbing lane or descending lane (as distinct from overtaking lanes) on a Multi-Combination Vehicle route. In these cases it is acceptable for the curve widening on the climbing lane or descending lane (that is, the outer lane) to suit the Multi-Combination Vehicle and for the curve widening on the adjacent through lane to suit a Single Unit Truck/Bus. In such cases, the lane marking must reflect the design lane widths.

Figure 11.3 shows the road width components when two vehicles meet on a horizontal curve. These components are:

- Horizontal clearances. These are the same as on straights.
- Swept path width of each vehicle; being greater on the curve due to offtracking of the rear wheels and the consequent horizontal component of the front overhang.

Note that there is no additional steering allowance component for difficulty of driving on curves. This has been the Austroads practice since 1979 and has been based on the following assumptions:

- There is less steering variation with the design vehicle since it is a large commercial vehicle that is driven by a professional driver.

- The swept path width of the design vehicle accommodates the swept path width of smaller vehicles. Plus it provides room for steering variation (and driver skill variation) with the smaller vehicles.
- The now common use of full width or part width paved and sealed shoulders compensates for not having a steering allowance component for the design vehicle.

Furthermore, the swept path component is based on low speed/low side friction demand offtracking even though normal operation on some curves may involve high speed / high side friction demand offtracking that will be less than the low speed offtracking. The greater swept path allowance in such cases is then assumed to provide for probable increased steering variation and possible offtracking due to axle misalignment and grade.

Table 11.8 shows the recommended curve widening per lane for a range of curves and design vehicles. The need for widening ceases when the widening is less than 0.25m (to fit the minimum practical widening for a two lane road of 0.5m). Note that the table also indicates a lower limit where the curve should be designed with the aid of turning templates. This is because the angle of turn starts to affect swept path width and the application of the curve widening will have to be checked against vehicle swept path.

Due to the size of the current design vehicles, the curve widening results in wide traffic lanes for curves with a radius less than about 100 metres. However, at traffic flows greater than about 1000 vehicles per hour, cars tend to form two lanes within traffic lanes greater than about 4.6m wide. The width at which two lane operation starts to occur is also dependent upon the horizontal curvature. And this width approaches 10m in the case of a circulating roadway of a roundabout (Austroads 1988, 1993).

Therefore, in practice, potential problems with wide traffic lanes due to curve widening are only likely to occur in urban areas. However, due to right of way constraints in urban areas, it will often not be possible to provide full curve widening for road train operation or B-double operation or even semi-trailer operation. Figure 11.4 shows how large vehicles have to be accommodated in these cases. It can also be seen in the figure that roadway capacity is adversely affected.

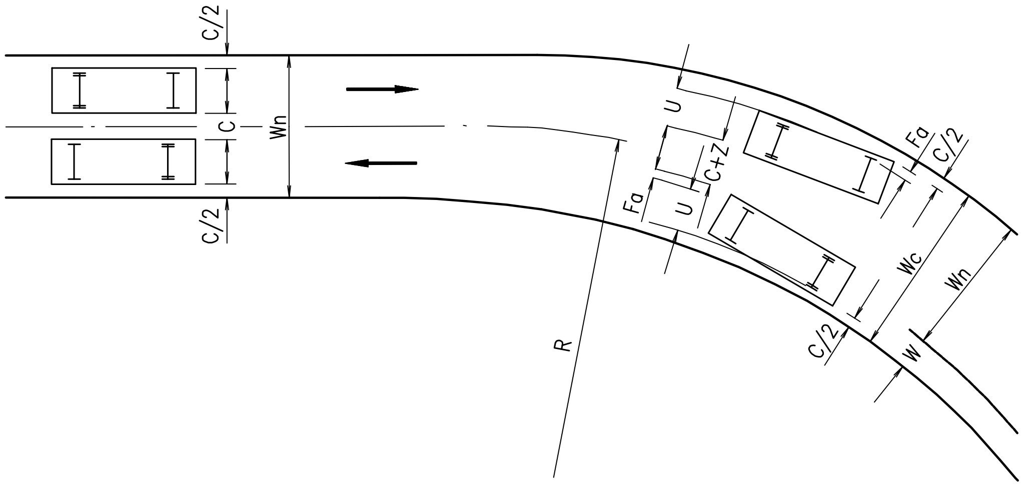
Table 11.8 Curve Widening per Lane for Current Design Vehicles, m

Radius m	SU Truck or Bus	PM & Semi-trailer	B-double	Type 1 Road Train	Type 2 Road Train
30.00		Use			
40.00	1.03				
50.00	0.82			Turning	
60.00	0.71	1.27			
70.00	0.59	1.03	1.31		
80.00	0.52	0.91	1.16	1.62	Templates
90.00	0.46	0.81	1.03	1.44	
100.00	0.41	0.71	0.90	1.26	1.80
120.00	0.36	0.63	0.80	1.13	1.61
140.00	0.32	0.56	0.71	1.00	1.43
160.00	0.28	0.49	0.62	0.87	1.25
180.00	0.24	0.42	0.53	0.74	1.07
200.00		0.35	0.45	0.62	0.89
250.00		0.29	0.37	0.51	0.74
300.00		0.23	0.30	0.41	0.59
350.00			0.26	0.35	0.51
400.00			0.22	0.30	0.44
450.00				0.27	0.39
500.00				0.25	0.35
600.00				0.21	0.30
700.00					0.25
800.00					0.22

Need for using Turning Templates determined by variation in widening due to angle of turn. Need for curve widening ceases when widening per lane < 0.25m.

The curve widening for a given carriageway will be the widening/lane x no. of lanes. The resultant total width of the traffic lanes may then be rounded to the nearest multiple of 0.25m.

Widening per lane is not dependent on the lane width on the straight. The widening is intended to maintain the horizontal clearances used on the straight.



W = widening for two–lane pavement on curve (m)
Z = steering allowance for difficulty for driving on curves – not provided for all vehicles
Wc = width of two–lane pavement on curve (m)
Fa = additional width of front overhang of vehicle on curve (m)
Wn = width of two–lane pavement on tangent (m)
R = radius of centre line of two–lane pavement (m)
U = track width of vehicle, outside to outside of tyres (m)
C = lateral clearance between vehicles in adjacent lanes (m)

Figure 11.3 Lane Width Components on a Horizontal Curve

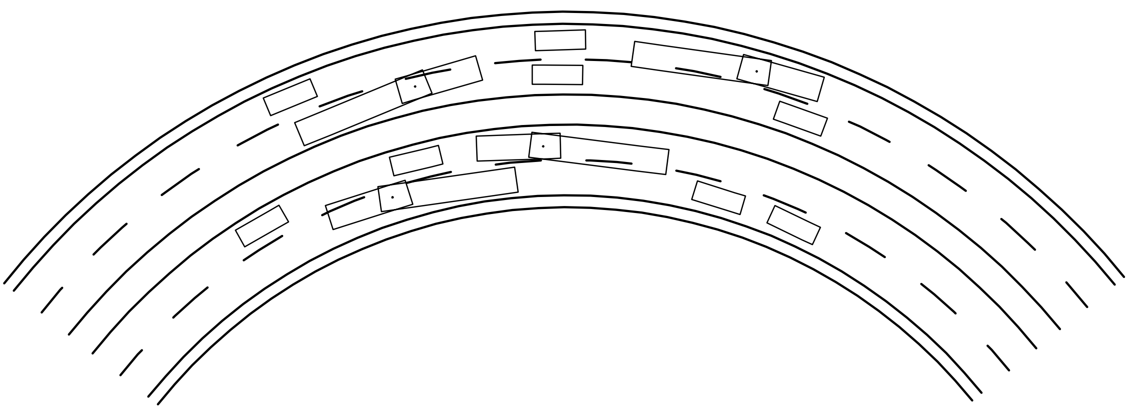


Figure 11.4 Examples of Heavy Vehicle Operation on Multi-lane Roads with Insufficient Curve Widening

11.10.2 Application of Curve Widening

Except for small radius curves where it is necessary to design curve widening and the application of curve widening with the aid of turning templates (see Table 11.8), it has always been AASHTO and Austroads practice to apply curve widening by tapering over the length of the roadway used for superelevation runoff. This is the length where the outer lane goes from level to full superelevation. In the case of a transitioned curve, the superelevation runoff corresponds with the plan transition. In the case of an untransitioned curve, it is usually applied equidistant about the tangent point of the horizontal curve. This adequately matches where drivers make their own transition when entering or leaving an untransitioned horizontal curve.

The basis for tying the application of curve widening to the superelevation runoff is largely one of convenience. Also, the changing curvature over the transition path has some correspondence to the change in swept path width, and hence the change in lane width. Most importantly, this has been found to work in practice. The correspondence to the change in swept path width has been checked for current prime mover & semi-trailer, B-double and road train operation (see Cox 1998).

For transitioned curves, it is normal practice to apply half of the curve widening to each side of the road (see Figure 11.1). However this means that the shift associated with the transition (shift = $L^2/24/R$ approximately) must be greater than the curve widening that is applied to the outer side of the curve so that:

- the design vehicle will make use of the widening; and
- for appearance.

This will usually only be a problem when the curve widening has to suit a road train. And a greater proportion of the total widening will then have to be applied on the inside of the curve. The painted centreline will then be offset from the control line in order to provide equal lane widths.

For untransitioned curves, it is normal practice to apply all of the curve widening to the inside of the curve (see Figure 11.1). The painted centreline is then offset from the control line in order to provide equal lane widths. This practice aids drivers in making their own transition. Drivers steer slightly closer to the centre of the curve when making their own transition.

With closely spaced reverse untransitioned curves (Cases 3A and 3B, and possibly Case 4, in Section 11.6), it will be necessary to modify how the curve widening is applied between the two curves. This is necessary because the widening still has to be fully applied by the normal 15m to 25m distance into the curve in order to match vehicle tracking characteristics. But when the curves are closely spaced, it is necessary to start applying the curve widening for the second curve at a point back on the first curve. Again, this is to match vehicle tracking characteristics.

Figure 11.5 shows how the modified practice is derived from the standard practice shown in Figure 11.1. With both the standard and modified practices, the full curve widening is achieved at the point where full superelevation is attained (denoted as distance L/2 into the curve). With both the standard and modified practices, the curve widening starts to be applied at a point that is distance L/2 prior to the curve tangent point (or longer distance if necessary). Figure 11.5 also shows an alternative treatment. This will often be preferred when it improves appearance and simplifies construction. The alternative treatment also gives drivers more latitude with where they position their vehicle. However, the range of vehicle paths will still adequately match the changing superelevation between the curves. The application of the curve widening should always be checked against vehicle swept paths in these cases.

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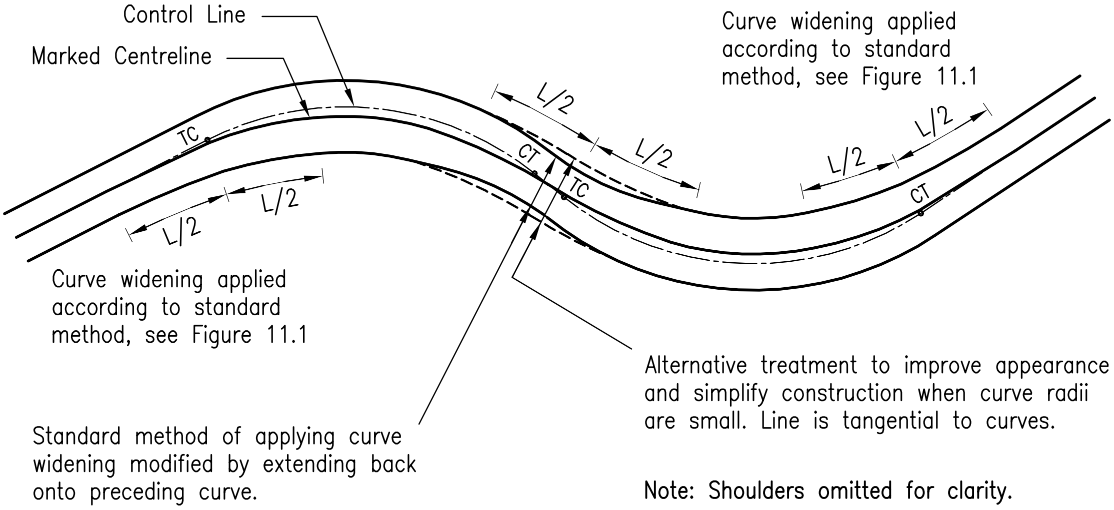


Figure 11.5 Application of Curve Widening on Closely Spaced Reverse Untransitioned Curves

11.11 Radius to Meet Sight Distance Requirements

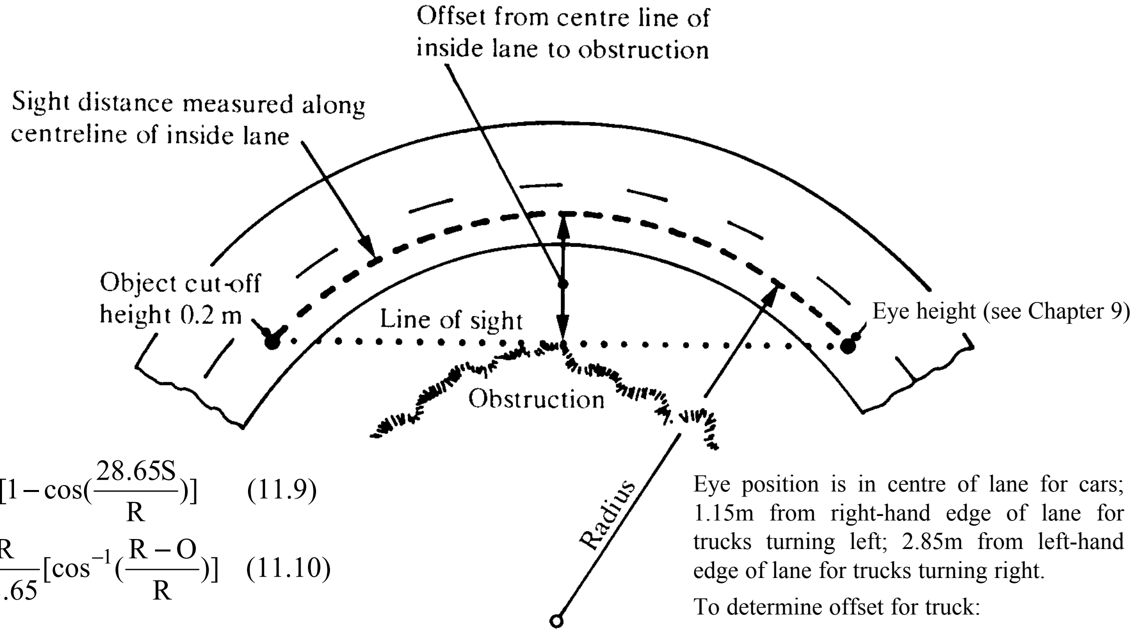
Figure 11.5 shows the relationship between horizontal sight distance, curve radius and lateral clearance to an obstruction. It is valid when the sight distance at the appropriate design speed is not greater than the length of the curve. This relationship assumes that for a car, the driver's eye position and the object are in the centre of the inside lane (in terms of radius for the direction of travel).

For a truck, the object is assumed to be in the centre of the inside lane (in terms of radius for the direction of travel). But the driver's eye position has to be offset from the centre of the lane to better reflect the operation of these vehicles. For a left hand curve, the driver's eye position is assumed to be 1.25m inwards from the right hand edge of the traffic lane. For a right hand curve on a one-way carriageway, the driver's eye position is assumed to be 2.85m from the left-hand edge of the traffic lane. The object is assumed to be in the centre of the lane.

When the design sight distance is greater than the length of curve, the line of sight may be checked on a layout drawing or digital model by using the parameters shown in Figure 11.6.

11.12 Curve Perception Issues

Chapter 6 explains how operating speeds tend to be uniform on rural roads in high speed environments (100km/h and greater) when all of the geometric elements have been designed to a standard equal to or greater than the speed environment. This may also apply to some urban roads and motorways where the speed environment is less than 100km/h. Chapter 6 also explains how operating speeds tend to vary on rural roads and some urban roads in lower speed environments (less than 100km/h). In these cases, drivers will slow down, if necessary, for a curve and increase speed where possible on straights and larger radius horizontal curves. The operating speeds on long straights and large radius curves will match the speed environment (by definition). Chapter 6 further explains that there may also be cases on roads in high speed



$$O = R\left[1 - \cos\left(\frac{28.65S}{R}\right)\right] \quad (11.9)$$

$$S = \frac{R}{28.65}\left[\cos^{-1}\left(\frac{R - O}{R}\right)\right] \quad (11.10)$$

Where:
R = radius in metres
S = sight distance
O = offset in metres
Note: Use this formula only when S ≤ the length of circular curve. Angles in degrees.

Eye position is in centre of lane for cars; 1.15m from right-hand edge of lane for trucks turning left; 2.85m from left-hand edge of lane for trucks turning right.

To determine offset for truck:

1. Use stopping sight distance for truck and radius of centre of lane to calculate offset.
2. Subtract 0.30 from the value obtained in step 1 for truck turning left.
3. Add 0.55 to the value obtained in step 1 for a truck turning right.

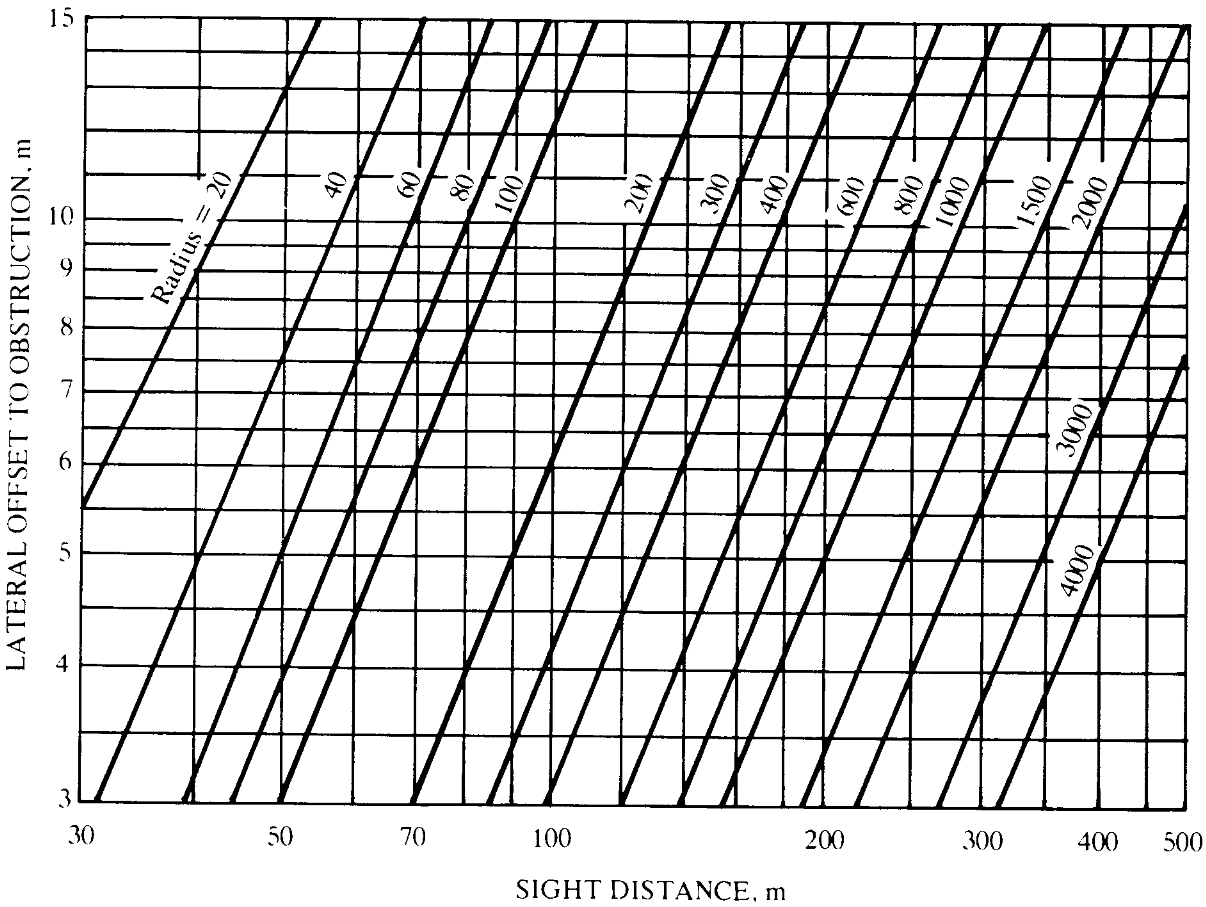


Figure 11.6 Horizontal Stopping Sight Distance for Type F Restriction to Visibility (Reaction Time = 2.5 seconds)

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environments where operating speeds have to vary due to some local constraint or short constrained section of road that could not be designed to a standard equal to the speed environment.

The foregoing explanation of vehicle operating speeds means that many horizontal curves are designed on the basis of the speed that drivers are prepared to slow down to for that curve. When approaching a curve, drivers regulate their speed from the apparent curvature of the road ahead. The amount of superelevation on a curve does not normally influence curve entry speeds (Kanellaidis, 1999) - probably because drivers do not perceive the superelevation.

In practice, there is some variation in curve entry speeds. In turn, this gives rise to three important curve perception issues:

- **Horizontal Curve Perception Distance.** When vehicles have to slow down for a horizontal curve, drivers must see a sufficient amount of the curve in order to perceive its curvature, react and slow down appropriately for the curve. Normally, sufficient sight distance for a horizontal curve is provided through the practice of not having a horizontal curve start over a crest and through the normal principles of alignment coordination (see Chapter 10). However, there are times where this can not be avoided. Chapter 9 sets out the criteria that should be applied in order to check that sufficient visibility is provided for any curve where vehicles have to slow down for it. In some cases, provision of horizontal curve perception distance may require a larger crest vertical curve than is required for stopping sight distance.

- **Horizontal Curves in the Range of 300m to 440m.** New South Wales experience has found that horizontal curves with a radius in the range of 300m to 440m should be avoided for operating speeds greater than 70 km/h. The curves are deceptive to the driver in that they appear to be able to be travelled at higher speeds than is actually possible. As a result, drivers may not slow down appropriately for them. Therefore, these curves should only be used when the predicted curve operating speed meets the following conditions:
  - The curve operating speed is at least 5km/h below the limiting curve speed standard (see 6.1.5);
  - The decrease in operating speed from the preceding horizontal alignment element is less than or equal to 5km/h; or
  - If the decrease in operating speed from the preceding horizontal alignment element is greater than 5km/h but less than or equal to 10km/h, then the decrease in limiting curve speed between the elements must also be less than or equal to 10km/h (see 6.6.3).

  Closely spaced preceding curves help achieve these conditions. In practice, this restriction on the use of curves in the range 300m to 440m limits the range of curves suitable for 80km/h, and particularly, 90km/h.

- **Length of Transition Curves.** The use of longer transitions than those based on the normal superelevation runoff length should be avoided when curve operating speeds are such that drivers have to reduce speed for the curve. In these circumstances, longer transitions may cause drivers to perceive a higher standard of curvature than there is, with consequent increased speed and friction demand on the circular section of the curve. Overseas studies have found that there have been higher accident rates on some curves with a combination of long transition (typically with more than twice the length based on superelevation runoff) and small to medium radius.

11.13 Curves on Steep Down Grades

On steep downgrades there is more chance of some drivers tending to overdrive horizontal curves. Therefore, on steep downgrades, the minimum horizontal curve radius for the design speed should be increased in accordance with the following empirical formula:

$$R_{min\ on\ grade} = R_{abs\ min}\ [1+(G-3)/10] \qquad (11.11)$$

where

$R_{abs\ min}$ = absolute minimum radius for the design speed from Table 11.5 or 11.6 (m). That is, the radius based on absolute maximum side friction coefficient and maximum superelevation.

G = grade (%).

Radii are in metres.

The resulting minimum radius will commonly be of a size where Table 11.5 or 11.6 recommends a lower value of superelevation than the maximum shown in Table 11.2. However, it will usually be better to use the maximum superelevation shown in Table 11.2. The higher superelevation provides an additional margin for drivers who tend to overdrive the curves.

Where it is not possible to increase the minimum radius in accordance with equation 11.11, it will be necessary to try to provide either:

- Preceding curves that limit the speed of vehicles. Using such curves will reinforce the low speed environment that is needed on steep downgrades (see 6.2.1).
- Using curves that require less than the desirable maximum value of side friction. The use of maximum (or close to maximum) superelevation values may also be necessary.

References

AASHO (1954): A Policy on Geometric Design of Rural Highways. American Association of State Highway Officials. Washington, D.C.

AASHTO (1994): A Policy on Geometric Design of Highways and Streets. American Association of State Highway and Transportation Officials. Washington, D.C.

Australian Road Research Board (1993): Unsealed Roads Manual - Guidelines to Good Practice. Melbourne.

Austroads (1989): Rural Road Design – A guide to the Geometric Design of Rural Roads. Sydney.

Austroads (1988): Guide to Traffic Engineering Practice. Part 5 - Intersections at Grade. Sydney.

Austroads (1993): Guide to Traffic Engineering Practice. Part 6 - Roundabouts. Sydney.

Austroads (1999): Guide to Traffic Engineering Practice. Part 15 - Motor Cycle Safety. Sydney.

Austroads (2000): May 2000 draft of the Guide to the Geometric Design of Major Urban Roads. Sydney.

Cox, R.L. (1972): Clothoid (Euler) Spiral Theory as Applied to Transition Curves. Queensland Department of Main Roads.

Cox, R.L. (1998): A Review of Geometric Road Design Standards based on Vehicle Swept Path. Technology Transfer Forum July 1998. Queensland Department of Main Roads.

Kanellaidis, G. (1999): Road Curve Superelevation Design: Current Practices and Proposed Approach. Vol 8, No 2, Jinee 1999, Road and Transport Research.

McLean, J., Tziotis, M. and Gunitallake, T. (2001): Final Draft, Designing for Trucks - When, Where and How? Austroads, Sydney.

Rahmann, W.M. (1981): Design of Horizontal Curves Using the Revised NAASRA Method - Side Friction Factors. Road Design References. Main Roads Department Queensland.

Taneka, D. and Troutbeck, R. (1996): Equations for Meeting and Overtaking Rates. Physical Infrastructure Centre Research Report 96-11. Queensland University of Technology.

Transportation Association of Canada (1999): Geometric Design Guide for Canadian Roads. Ottawa.

Relationship to Other Chapters

- Close relationship with Chapter 6 - Speed Parameters;
- **MUST** be read in conjunction with Chapter 10 (coordination) and Chapter 12 (vertical alignment);

- Sight distance requirements are defined in Chapter 9;
- Elements of Chapters 4, 6, 13, 14, 15, 16, 20 and 22 require information from this Chapter.

Appendix 11A: Clothoid Spiral Transition Curves

11A.1 Properties of the Clothoid Spiral

Basic properties of the Clothoid (or Euler Spiral) are given below. More detailed information, including the derivation of all the equations, is available in Cox, 1972.

Radians - Length Relationships

The clothoid spiral is a curve that is defined by the property that curvature (which is 1/R) varies uniformly along the length of spiral. Therefore, the curvature at the end of the spiral is proportional to the length:

$$\frac{1}{R} \propto L \qquad \text{or} \qquad RL = \text{Constant}$$

where L = length of clothoid
R = radius of clothoid at end

It follows that for any intermediate point P in a clothoid of length L, the following expression applies:

$$\frac{R_i}{R} = \frac{L}{L_i} \qquad (11A.1)$$

where L_i =length of clothoid to point P
R_i = instantaneous radius at point P

Since RL is constant, any particular transition curve can be defined by an “RL value”.

Deflection Angles

From equation 11A.1 it can be derived that the angle between the straight and a line tangential to the clothoid at any point is proportional to the square of the spiral length to that point from the beginning (or origin) of the spiral.

At the end of the transition curve, this angle, called the spiral angle (θ) is given by

$$\theta = \frac{L}{2R} \text{ radians}$$

For any intermediate point on the clothoid, the following expression applies:

$$\frac{\theta_i}{\theta} = \left(\frac{L_i}{L}\right)^2$$

$$\therefore \theta_i = \frac{L_i^2}{2RL} \text{ radians} \qquad (11A.2)$$

where θ_i = angle between straight and tangent to point P.

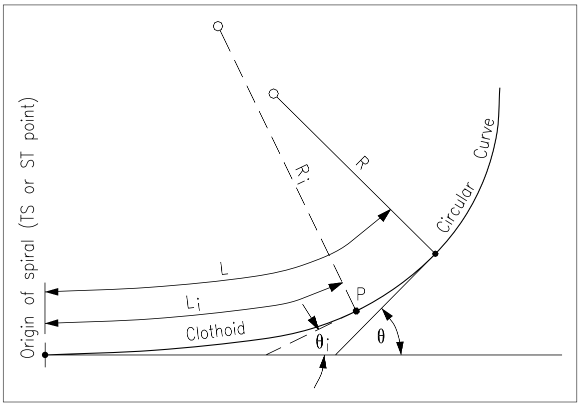

Figure 11A.1 Clothoid Spiral Transition Curve

Coordinates of a Point on the Spiral

A clothoid spiral at any general location and orientation does not lend itself to the direct calculation of the Cartestian coordinates for any point on the spiral. In practice, the most convenient means for setting out points on a clothoid spiral is through the direct or indirect use of distances along the tangent through the spiral origin and corresponding offsets from the tangent. This means that the spiral origin becomes the origin of a local coordinate system and the tangent through the origin becomes the X axis. This practice further suits road construction practice since the tangent line also has to be set out for other purposes. Even so, the calculation of coordinates in the local coordinate system is more complex than for many other curves but are acceptable through the use of computers or calculators. The equations for the X and Y coordinates (and other properties) are in the form of infinite series. The infinite series, however, rapidly converge, and usually only the first two terms of each expression, and occasionally the first three, are significant for the standard transition curves used in road design.

From equation 11A.2 it can be derived that:

$$x = L_i(1 - \frac{\theta_i^2}{5(2!)} + \frac{\theta_i^4}{9(4!)} - \frac{\theta_i^6}{13(6!)} + ...)$$

$$= L_i - \frac{L_i^5}{5 \times 2!(2RL)^2} + \frac{L_i^9}{9 \times 4!(2RL)^4} -$$

$$- \frac{L_i^{13}}{13 \times 6!(2RL)^6} + ...$$

$$y = L_i(\frac{\theta_i}{3} - \frac{\theta_i^3}{7(3!)} + \frac{\theta_i^5}{11(5!)} - \frac{\theta_i^7}{15(7!)} + ...)$$

$$= \frac{L_i^3}{3(2RL)} - \frac{L_i^7}{7.3!(2RL)^3} + \frac{L_i^{11}}{11.5!(2RL)^5} - ...$$

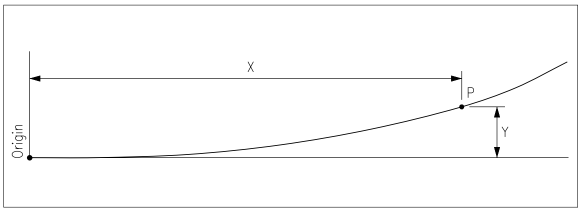

Figure 11A.2

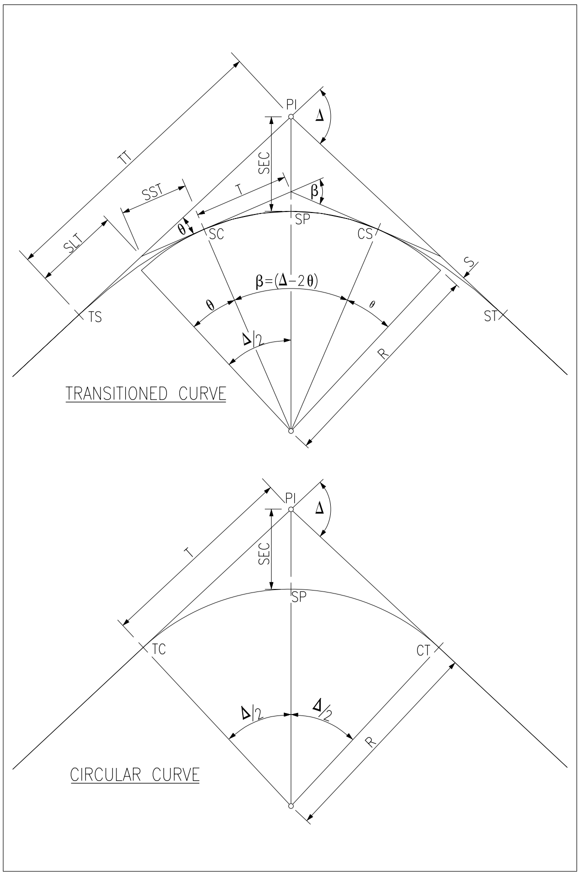

Figure 11A.3

11A.2 Curve Components

Standard curve component descriptions and notation are shown in Figure 11A.3 and Table 11A.1.

Table 11A.1

Q	Alignment number	SLT	Long Tangent of spiral
BgBT	Bearing Back Tangent	SST	Short Tangent of spiral
BgAT	Bearing Ahead Tangent	SEC	Secant length
R	Radius of circular curve	TC	Tangent to Circle
T	Tangent length of circular curve	CT	Circle to Tangent
TT	Total Tangent length	CC	Circle to Circle
Δ	Intersection Angle at point of intersection (in degrees)	SS	Spiral to Spiral
L	Spiral Length	TS	Tangent to Spiral
S	Shift Distance	SC	Spiral to Circle
θ	Spiral Angle	CS	Circle to Spiral
β	Intersection Angle of circular curve (Vertex angle)	ST	Spiral to Tangent
ARC	Total arc length	IP	Intersection Point
	(circular curve plus spirals)	SP	Secant Point

For transitioned curves with unequal spiral lengths, the abbreviations TT, L, S, θ, SLT, SST, are to be suffixed by the letter A (ahead) or B (back), e.g. LA=Spiral Length Ahead.

11A.3 Formulae for Transition Curves

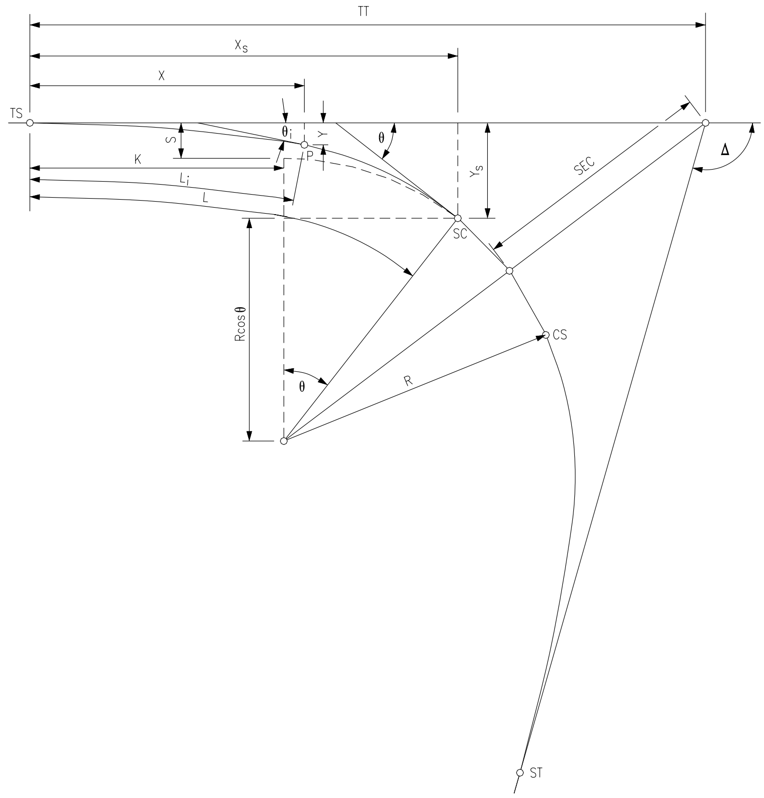

Figure 11A.4

Spiral Angle

$$\theta = \frac{L}{2R} \text{ radians}$$

Offset from tangent to SC

$$Y_s = L\left(\frac{\theta}{3} - \frac{\theta^3}{42} + \frac{\theta^5}{1320} - ...\right)$$ where θ is expressed in radians

$$= \frac{L^2}{6R} - \frac{L^4}{336R^3} + \frac{L^6}{42240R^5} - ...$$

Projection of transition curve on tangent

$$x_s = L\left(1 - \frac{\theta^2}{10} + \frac{\theta^4}{216} - ...\right)$$ where θ is expressed in radians

$$= L - \frac{L^3}{40R^2} + \frac{L^5}{3456R^4} - ...$$

Offset from tangent to transition curve at any intermediate point P

$$y = L_i\left(\frac{\theta_i}{3} - \frac{\theta_i^3}{42} + \frac{\theta_i^5}{1320} - ...\right)$$ where

$$\theta_i = \frac{L_i^2}{2RL} \text{ radians}$$

$$= \frac{L_i^3}{6RL} - \frac{L_i^7}{336R^3L^3} + \frac{L_i^{11}}{42240R^5L^5}$$

Projection of portion of transition curve from any point P to tangent

$$x = L_i\left(1 - \frac{\theta_i^2}{10} + \frac{\theta_i^4}{216} - ...\right)$$ where

$$\theta_i = \frac{L_i^2}{2RL} \text{ radians}$$

$$= L_i - \frac{L_i^5}{40R^2L^2} + \frac{L_i^9}{3456R^4L^4} - ...$$

Shift distance (S)

This distance is the offset from the tangent to the point on the circle at which the radius is normal to it.

$$S = Y_s - R(1-\cos\theta)$$

$$= L\left(\frac{\theta}{12} - \frac{\theta^3}{336} + \frac{\theta^5}{15840} - ...\right)$$ where θ is expressed in radians

$$= \frac{L^2}{24R} - \frac{L^4}{2688R^3} + \frac{L^6}{506880R^5} - ...$$

11

Displacement of the tangent point along the tangent due to the introduction of the transition curve (k)

$$k = x_s - R\sin\theta$$

$$= L\left(\frac{1}{2} - \frac{\theta^2}{60} + \frac{\theta^4}{2160} - ...\right)$$ where θ is expressed in radians

$$= \frac{L}{2} - \frac{L^3}{240R^2} + \frac{L^5}{34560R^4} - ...$$

For a transitioned curve with equal length spirals:

Total arc length (ARC)

$$ARC = R\Delta + L$$ where Δ is expressed in radians.

Total tangent length (TT)

$$TT = (R+S)\tan\frac{\Delta}{2} + k$$

$$= (Y_s + R\cos\theta)\tan\frac{\Delta}{2} + k$$

Secant length

$$SEC = (R+S)\sec\frac{\Delta}{2} - R$$