1.4.3 OPERATING SPEED CONSISTENCY

The safety of a road is closely linked to variations in the speed of vehicles travelling on it. These variations are of two kinds:

1. Individual drivers vary their operating speeds to adjust to features encountered along the road, such as curves, intersections, and accesses in the alignment. The greater and more frequent are the speed variations, the higher is the probability of collision.

2. Drivers travelling substantially slower or faster than the average traffic speed have a higher risk of being involved in collisions.

A designer can therefore enhance the safety of a road by producing a design that encourages operating speed uniformity.

As noted in the discussion of speed profiles in Chapter 1.2, simple application of the design speed concept does not prevent inconsistencies in geometric design. Traditional North American design methods have merely ensured that all design elements meet or exceed minimum standards, but have not necessarily ensured operating speed consistency between elements.

Practices used in Europe and Australia have supplemented the design speed concept with methods of identifying and quantifying geometric inconsistencies in horizontal alignments of rural two-lane highways. In addition, recent research work in Canada and the United States has addressed design consistency for combined (horizontal and vertical) alignments. The focus of this work is on two-lane rural highways. These methods have not been perfected, particularly in predicting the performance of a newly designed road. Their effectiveness is greater in evaluating existing roads and identifying priority improvements to reduce collision rates.

1.4.3.1 Prediction of Operating Speeds

In order to establish consistency of highway alignment design for a proposed new road, it is necessary to predict operating speeds associated with different geometric elements, including isolated horizontal and vertical curves, and horizontal-vertical curve combinations. Limited information for Canadian conditions is generally available to assist designers with predicting operating speed, but the material presented in this chapter may help, noting the limited database used. As an alternative, some jurisdictions have local data on which to base speed predictions.

Two distinct approaches to the assessment of operating speed design consistency are presented here.

- The first – based on research carried out in the US – considers only the horizontal alignment, and as a result, is somewhat simpler to apply, but has been shown to provide consistent and relevant results. This method may have its best application at the planning and preliminary design stages, when the vertical alignment may not be well defined. It may also have good application at the detailed design stage in situations of relatively flat (non-rolling, non-mountainous) terrain. Software is available to assist in this type of design consistency analysis.

- The second – also based on US data – considers both horizontal and vertical alignment. It is somewhat more complex to apply, but has also been shown to provide consistent and relevant results. This approach may be most applicable at the detailed design stage when the vertical alignment is well defined, and where the terrain is rolling or mountainous. In such areas - where elevation changes are substantial - this technique may provide somewhat more realistic predictions of operating speed than the method noted immediately above.

Both methodologies have merit and are deemed to be applicable in the Canadian road design context.

Horizontal alignment approach: Introduction

In this approach, researchers collected data from five US states, measuring 85^{th} percentile operating speeds, under free flowing traffic conditions, on long tangents and horizontal curves on rural two-lane highways. Long tangents (250m or more) are those lengths of straight road on which a driver has time to accelerate to the desired speed before approaching the next curve. The mean 85^{th} percentile speed on long tangents was found to be 99.8 km/h on level terrain and 96.6 km/h in rolling terrain. It was noted that these speeds were probably constrained by the 90 km/h posted speed in force at the time.

On horizontal curves, the research found consistent disparities between 85^{th} percentile speeds, with the greater disparity on tighter radius curves. The 85^{th} percentile speed exceeded the design speed on a majority of curves in each 10 km/h increment of design speed up to a design speed of 100 km/h. At higher design speeds, the 85^{th} percentile speed was lower than the design speed.

Using regression techniques, a relationship was found between 85^{th} percentile speeds and the characteristics of a horizontal curve.

$$V_{85} = 102.45 + 0.0037L - (8995 + 5.73L) / R \qquad (1.4.1)$$

Where V_{85} = 85th percentile speed on curve (km/h)

L = length of curve (m)

R = radius of curvature (m)

Speed Profile Model 1: The technique

The findings outlined in Subsection 1.4.3.1 support the conclusion that there is no strong relationship between design speed and operating speed on horizontal curves. Consistency of horizontal alignment design cannot therefore be assured by using design speed alone. A further check can be carried out by constructing a speed profile model, using predicted 85th percentile operating speeds for new road and measured 85th percentile speeds for existing roads.

For the predictive model, it is necessary to calculate the critical tangent length between curves as follows:

$$TL_c = \frac{2V_f^2 - V85_1^2 - V85_2^2}{25.92\,a} \qquad (1.4.2)$$

Where: TL_C = critical tangent length (m)

V_f = 85th percentile desired speed on long tangents (km/h)

$V85_n$ = 85th percentile speed on curve n (km/h)

a = acceleration/deceleration rate, assumed to be 0.85 m/s^2

The calculation assumes that deceleration begins where required, even if the beginning of the curve is not yet visible.

Each tangent is then classified as one of three cases, as shown on Figure 1.4.3.1, by comparing the actual tangent length (TL) to the critical tangent length (TL_C).

Having found the relationship between each tangent length (TL) and the critical tangent length (TL_C), the equations in Table 1.4.3.1 can then be used, as appropriate, to construct the speed profile model.

The speed profile model is used to estimate the reductions in 85th percentile operating speeds from approach tangents to horizontal curves, or between curves. Designers should note that the research[6,9] on which the model is based dealt only with two-lane rural highways. The same principles, however, can be applied to design of other classes of roads.

Horizontal and vertical alignment approach: Introduction

In this approach, researchers[4] collected data in six states at 176 sites, again measuring

Table 1.4.3.1 Equations for Estimating Operating Speed on Various Types of Speed-Limiting Curves[5]

Type	Alignment Condition	Equation[a]
1	Horizontal curve on grade: $-9\% \leq G < -4\%$	$V_{85} = 102.10 - \frac{3077.13}{R}$
2	Horizontal curve on grade: $-4\% \leq G < 0\%$	$V_{85} = 105.98 - \frac{3709.90}{R}$
3	Horizontal curve on grade: $0\% \leq G < 4\%$	$V_{85} = 104.82 - \frac{3574.51}{R}$
4	Horizontal curve on grade: $4\% \leq G < 9\%$	$V_{85} = 96.91 - \frac{2752.19}{R}$
5	Horizontal curve combined with sag vertical curve	$V_{85} = 105.32 - \frac{3438.19}{R}$
6	Horizontal curve combined with non limited sight distance crest vertical curve	-[b]
7	Horizonal curve combined with limited sight distance crest vertical curve (i.e., $K \leq 43$m/%)[c]	$V_{85} = 103.24 - \frac{3576.51}{R}$
8	Vertical crest curve with limited sight distance (i.e., $K \leq 43$ m/%) on horizontal tangent	$V_{85} = 105.08 - \frac{149.69}{K}$

[a] V85 = 85th percentile speed of passenger cars (km/hr), K = rate of vertical curvature, R = radius of curvature (m), and G = grade (%).
[b] Use lowest speed of the speeds predicted from Type 1 or 2 (for the downgrade) and Type 3 or 4 (for the upgrade).
[c] In addition, check the speed predicted from Type 1 or 2 (for the downgrade) and Type 3 or 4 (for the upgrade) and use the lowest speed. This will ensure that the speed predicted along the combined curve will be higher than that on a horizontal curve alone.

Table 1.4.3.2 Equations for Speed-Profile Model

Case and Condition	**Sub-Condition**	**Equation**[a]
Case 1: $TL \geq TL_c$	N.A.	$X_{1a} = (V_f^2 – V_n^2)/25.92\ a$ $X_{1d} = (V_f^2 – V_{n+1}^2)/25.92\ d$ $TL_c = X_{1a} + X_{1d}$
Case 2: $TL < TL_c$ V_n ³ V_{n+1}	Case 2(a) $TL > X_{2d}$	$X_{2d} = (V_n^2 – V_{n+1}^2)/25.92\ d$ V_t = {Vn2 + 25.92 [ad/(a + d)] (TL – X2d)} $^{1/2}$
	Case 2(b) $TL \leq X_{2d}$	For $TL < X_{2d}$, d' + $(V_n^2 – V_{n+1}^2)$ (25.92 TL)
Case 3: $TL < TL_c$ $V_n \leq V_{n+1}$	Case 3(a) $TL > X_{3a}$	$X_{3a} = (V_{n+1}^2 – V_n^2)/25.92\ a$ $V_t = \{V_{n+1}^2 + 25.92\ [ad/(a + d)]\ (TL – X_{3a})\}^{1/2}$
	Case 3(b) $TL \leq X_{3a}$	For $TL < X_{3a}$, $V_{n+1}^a = [V_n^2 + 25.92\ a\ TL]^{1/2}$

[a] V_n and V_{n+1} = 85[th] percentile speeds for curve n and n+1 (predicted using equations in Table 1.4.2.1)

If A is stated in metres, speed v in kilometres per hour and c in metres per second cubed, the expression becomes:

$$A = \frac{0.1464V^{1.5}}{c^{0.5}} \qquad (2.1.13)$$

Tolerable rate of change of centripetal acceleration varies between drivers. As a basis of design, the value used to provide minimum acceptable comfort is 0.6 m/s^3. The expression then becomes:

$$A = \frac{0.1464V^{1.5}}{0.6^{0.5}} \qquad (2.1.14)$$

$$A = 0.189V^{1.5} \qquad (2.1.15)$$

Using the above expression, the minimum spiral parameter based on comfort can be calculated for each design speed. It may be noted that the spiral parameter is independent of the radius. This is illustrated in Figure 2.1.2.7 by the comfort line parallel to the abscissa.

Spiral Parameter Based on Relative Slope

For superelevation runoff, the relative slope is defined as the slope of the outer edge of a pavement in relation to the profile control line. The maximum permissible value varies with design speed, and is shown in Table 2.1.2.11.

The minimum length of transition, l, is given by the equation

$$l = \frac{100\ we}{2s} \qquad (2.1.16)$$

Where w = the width of pavement in metres

e = the superelevation being developed in metres per metre

s = the relative slope, percentage

l = measured in metres

Table 2.1.2.11 Maximum Relative Slope Between Outer Edge of Pavement and Centreline of Two-Lane Roadways

Design Speed (km/h)	Relative Slope (%)
40	0.70
50	0.65
60	0.60
70	0.55
80	0.51
90	0.47
100	0.44
110	0.41
120	0.38
130	0.36

For a given speed and radius, superelevation and relative slope are known and minimum lengths can be calculated. From minimum length and radius, the minimum spiral parameter can be calculated, using the expression:

$$A^2 = RL \qquad (2.1.17)$$

Spiral Parameter Based on Aesthetics

Short spiral transition curves are visually unpleasant. It is generally accepted that the length of a transition curve should be such that the driving time is at least 2 s. For a given radius and speed, therefore, the minimum length and minimum spiral parameter can be calculated using the expression:

$$A^2 = 0.56RV \qquad (2.1.18)$$

Spiral Parameter: Design Domain Quantitative Aids

Quantitative expressions of the design domain for the spiral parameter are given in Tables 2.1.2.5, 2.1.2.6 and 2.1.2.7 in Subsection 2.1.2.2 for a range of design speeds. Designers should note the following application heuristics in using these tables.

1. For any particular design speed and radius, the highest value of spiral parameter as determined by the methodologies discussed in the previous section are used for these calculations.

2. The design values in the tables are minimum and higher values should be used wherever possible in the interests of safety, comfort and aesthetics.

3. In design it is convenient to select values so that the radius is a standard value and A/R is a rational number as this permits spiral properties to be read directly from tables[33].

Spiral Parameter: Design Domain Application Heuristics

The application of spiral curves in horizontal alignment is a complex design problem that has many variations. A number of design domain application heuristics dealing with some of the more common instances are provided below.

1. A circular curve with simple spirals at both ends each having the same parameter value is referred to as a symmetrical curve. This condition represents the most common practice for spiraled curves.

2. Unsymmetrical curves are common at interchange ramps and loops and represent the case noted in #1 above, but with different parameter values for the spirals at each end.

3. Successive circular curves with different radii but in the same direction are best joined by a spiral curve. Where this occurs, the “joining spiral” is referred to as a segmental spiral. The minimum spiral parameter to be used is found by referring to Tables 2.1.2.5, 2.1.2.6 and 2.1.2.7 and using the smaller of the two radii.

4. In some instances, successive circular curves with different radii may connect directly to one another if the difference in radii is not too large. See Subsection 2.1.2.6 for additional guidance in this regard.

5. Circular, non-successive curves in the same direction joined by a short length of tangent should normally be joined instead by a spiral. The use of tangents to achieve such connections results in what is commonly referred to as a “broken back” curve. It is only justified when some other consideration, for example, property constraints or construction cost, outweighs the visual disadvantages. The term “short tangent” is very subjective and there are several definitions for this term. One suggests that a short tangent length may be regarded as one which allows a driver on the first curve to see at least some part of the following curve. Another definition suggests a broken back curve occurs when the tangent length (m) is less than four times the design speed. If possible a more desirable solution to a broken back curve is to eliminate the short tangent and insert an appropriate circular curve or better yet a segmental spiral curve.

6. A change of direction from one tangent to another may be accomplished by successive spiral curves without a length of circular curve between them. Such a transition curve is referred to as symmetrical where the two spiral parameters are the same, and unsymmetrical where they are different. The minimum permissible spiral parameter to be used in such a situation is the minimum for the design as shown in Tables 2.1.2.5, 2.1.2.6 and 2.1.2.7.

7. A reversal in curvature direction may be accomplished through successive simple spiral curves without a length of tangent between them. The spiral parameters should be at least the minimum for the design speed. However, the alignment will have an improved appearance if the minimum spiral parameter values are exceeded or if a length of tangent is inserted between the two spirals. Superelevation is applied as

Table 2.1.2.14 Pavement Widening Values on Curves for Heavy SU Truck[2]

Radius of Curve (m)	Pavement Width 7.4 m									Pavement Width 7.0 m									Pavement Width 6.6 m								
	Design Speed (km/h)									Design Speed (km/h)									Design Speed (km/h)								
	50	60	70	80	90	100	110	120	130	50	60	70	80	90	100	110	120	130	50	60	70	80	90	100	110	120	130
2000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.1	0.2
1500	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2
1000	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.0	0.0	0.0	0.1	0.1	0.1	0.2	0.2	0.2	0.1	0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.3
750	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.2	0.2	0.0	0.0	0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.1	0.1	0.2	0.2	0.3	0.3	0.3	0.4	0.4
500	0.0	0.1	0.1	0.2	0.2	0.3	0.3	0.4	0.4	0.1	0.2	0.2	0.3	0.3	0.4	0.4	0.5	0.5	0.2	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6
300	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.6		0.3	0.4	0.4	0.5	0.6	0.6	0.7	0.7		0.4	0.5	0.6	0.6	0.7	0.7	0.8	0.9	
200	0.4	0.5	0.6	0.6	0.7	0.8				0.5	0.6	0.7	0.7	0.8	0.9				0.6	0.7	0.8	0.8	0.9	1.0			
150	0.6	0.7	0.8	0.9	1.0					0.7	0.8	0.9	1.0	1.1					0.8	0.9	1.0	1.1	1.2				
125	0.8	0.9	1.0	1.1						0.9	1.0	1.1	1.2						1.0	1.1	1.2	1.3					
100	1.0	1.1	1.3							1.1	1.2	1.3							1.2	1.3	1.5						
80	1.3	1.4	1.5							1.4	1.5	1.6							1.5	1.6	1.7						
60	1.8	1.9								1.9	2.0								2.0	2.1							
40	3.1									3.2									3.3								

Notes:
(1) Widening values, W (m) are based on Heavy SU Truck travelling at the design speed.
(2) The range of values for curve widening extends to curve radii corresponding to approximately 20 km/h less than the design speed indicated.

Table 2.1.2.15 Pavement Widening Values on Curves for WB-20[2]

Radius of Curve (m)	Pavement Width 7.4 m									Pavement Width 7.0 m									Pavement Width 6.6 m								
	Design Speed (km/h)									Design Speed (km/h)									Design Speed (km/h)								
	50	60	70	80	90	100	110	120	130	50	60	70	80	90	100	110	120	130	50	60	70	80	90	100	110	120	130
2000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2
1500	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.1	0.1	0.1	0.1	0.2	0.2	0.3	0.3	0.3
1000	0.0	0.0	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.1	0.1	0.1	0.2	0.2	0.3	0.3	0.3	0.4	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.5
750	0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.4	0.4	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5
500	0.2	0.3	0.3	0.4	0.4	0.5	0.6	0.6	0.6	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8
300	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0		0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1		0.4	0.8	0.9	1.0	1.0	1.1	1.1	1.2	
200	1.0	1.0	1.1	1.2	1.3	1.3				1.1	1.2	1.2	1.3	1.4	1.5				1.2	1.2	1.3	1.4	1.5	1.6			
150	1.4	1.5	1.5	1.6	1.7					1.5	1.6	1.6	1.7	1.8					1.6	1.7	1.7	1.8	1.9				
125	1.7	1.8	1.9	1.9						1.8	1.9	2.0	2.0						1.9	2.0	2.1	2.1					
100	2.1	2.2	2.3							2.2	2.3	2.4							2.3	2.4	2.5						
80	2.8	2.9	3.0							2.9	3.0	3.1							3.0	3.1	3.2						
60	3.6	3.8								3.7	3.9								4.8	5.0							
40	5.5									5.6									5.7								

Notes:
(1) Widening values, W (m) are based on WB-20 truck travelling at the design speed..
(2) The range of values for curve widening extends to curve radii corresponding to approximately 20 km/h less than the design speed indicated.

Table 2.1.2.16 Pavement Widening Values on Curves for B-Train[2]

Pavement Width 7.4 m

Radius of Curve (m)	50	60	70	80	90	100	110	120	130
	Design Speed (km/h)								
2000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1500	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1000	0.0	0.0	0.0	0.1	0.1	0.1	0.2	0.2	0.2
750	0.0	0.1	0.1	0.1	0.2	0.2	0.3	0.3	0.3
500	0.2	0.2	0.3	0.3	0.4	0.4	0.5	0.5	0.6
300	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	
200	0.9	0.9	1.0	1.1	1.2	1.2			
150	1.2	1.3	1.4	1.5	1.5				
125	1.5	1.6	1.7	1.8					
100	1.9	1.9	2.1						
80	2.4	2.5	2.6						
60	3.2	3.4							
40	5.2								

Pavement Width 7.0 m

Radius of Curve (m)	50	60	70	80	90	100	110	120	130
	Design Speed (km/h)								
2000	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.1	0.1
1500	0.0	0.0	0.0	0.0	0.1	0.1	0.1	0.2	0.2
1000	0.1	0.1	0.1	0.1	0.2	0.2	0.3	0.3	0.3
750	0.1	0.2	0.2	0.2	0.3	0.3	0.4	0.4	0.4
500	0.3	0.4	0.4	0.4	0.5	0.5	0.6	0.6	0.7
300	0.6	0.7	0.7	0.8	0.8	0.9	1.0	1.0	
200	1.0	1.0	1.1	1.2	1.3	1.3			
150	1.3	1.4	1.5	1.6	1.6				
125	1.6	1.7	1.8	1.9					
100	2.0	2.1	2.2						
80	2.5	2.6	2.7						
60	3.3	3.5							
40	5.4								

Pavement Width 6.6 m

Radius of Curve (m)	50	60	70	80	90	100	110	120	130
	Design Speed (km/h)								
2000	0.0	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.2
1500	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.3	0.3
1000	0.1	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4
750	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.7
500	0.4	0.5	0.5	0.5	0.6	0.6	0.7	0.7	0.8
300	0.7	0.8	0.8	0.9	0.9	1.0	1.1	1.1	
200	1.1	1.1	1.2	1.3	1.4	1.4			
150	1.4	1.5	1.6	1.7	1.7				
125	1.7	1.8	1.9	2.0					
100	2.1	2.2	2.3						
80	2.6	2.7	2.8						
60	3.4	3.6							
40	5.5								

Notes:
1. Widening values, W (m) are based on B Train travelling at the design speed.
2. The range of values for curve widening extends to curve radii corresponding to approximately 20 km/h less than the design speed indicated.

Crest Vertical Curves: Technical Foundation Element

Crest vertical curves have to be flat enough to provide the required sight distances (various sight distances are outlined in Chapter 1.2). The most common sight distances that have to be considered in the design of vertical curves are:

- stopping sight distance
- passing sight distance
- decision sight distance
- non-striping sight distance[34]

At the instant an object comes into view on a crest curve, (the height of object depends on the type of sight distance under consideration) the line of sight from the driver's eye to the top of the object is tangential to the curve. To ensure that the required sight distance is provided, the curve should be sufficiently flat so that the distance from the driver to the object is at least equal to the required sight distance. Figure 2.1.3.1 illustrates the relationship between the sight distance (stopping, passing, or decision), length of vertical curve, height of object, height of eye, and the algebraic difference in grades.

For curves where the length of curve exceeds the sight distance, K is given by the expression:

$$K = \frac{S^2}{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2} \qquad (2.1.24)$$

Where: S = sight distance (either stopping, passing, or decision sight distance) (m)

h_1 = height of driver's eye (m)

h_2 = height of object (m)

For curves where the length of curve is less than the sight distance, K is given by the expression:

$$K = \frac{2S}{A} - \left[\frac{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}{A^2}\right] \qquad (2.1.25)$$

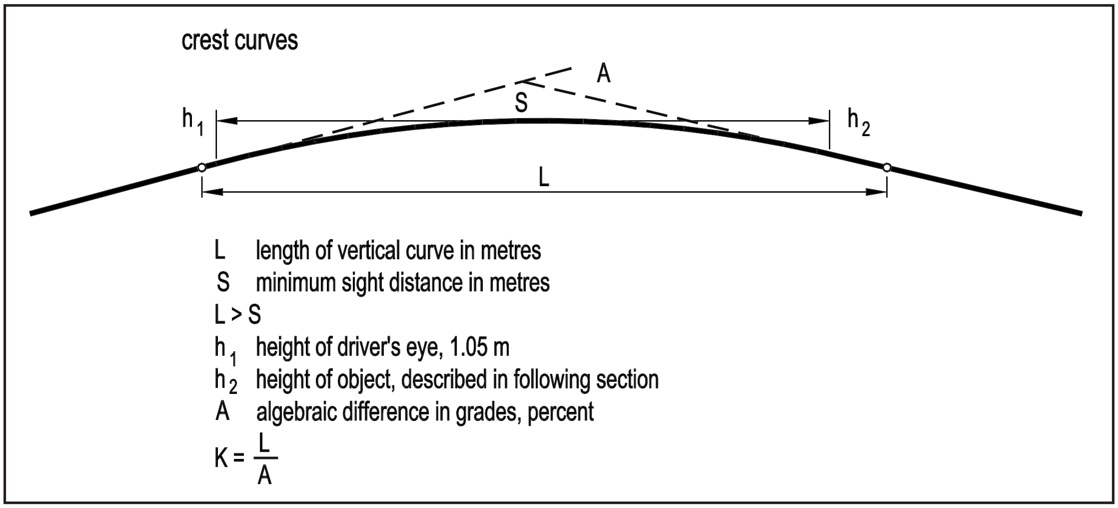
Where: S = sight distance (either stopping, passing, or decision sight distance) (m)

h_1 = height of driver's eye (m)

h_2 = height of object (m)

A = algebraic difference in grades

Figure 2.1.3.1 Sight Distance on Crest Vertical Curve[1]



In calculating K values for various sight distances, the height of driver's eye is 1.05 m, and the height of object is as outlined following, and discussed in more detail in Chapter 1.2.

- For stopping sight distance the most common object a vehicle has to stop for is another vehicle ahead on the road, the height of tail light is used. The legislated minimum is 0.38 m and is adopted for design. Other heights of objects can be used if necessary.
- For decision sight distance the more common height of object is 0.15 m, although other heights, such as zero for pavement markings, are not uncommon.
- For passing sight distance the height of object is 1.30 m, which represents the height of the opposing vehicle.

Crest Vertical Curves: Design Domain Quantitative Aid

Based on the above most commonly used heights of object, and on sight distances from Tables 1.2.5.3 and 1.2.5.5, the K values for stopping sight distance are provided in Table 2.1.3.2 and for passing sight distance the K values are provided in Table 2.1.3.3. The decision sight distance K values are not included because the vertical curvature depends on the height of object which is variable (depending on what the driver has to see).

The calculated K values are based on the length of curve exceeding the sight distance and they can be used without significant error when the length of curve is less than the sight distance. Appreciable differences occur only where A is small and little or no additional cost is involved in obtaining longer vertical curves.

On undivided roads non-striping sight distance is used to determine when no-passing pavement markings are required[7]. It is desirable to provide passing sight distance wherever possible but non-striping sight distance is generally adequate for safe passing manoeuvres.

Non-striping sight distance is less than passing sight distance, at each design speed. Passing manoeuvres can be completed in less than the full passing sight distance because of the timing of oncoming vehicles.

Table 2.1.3.2 K Factors to Provide Stopping Sight Distance on Crest Vertical Curves

Design Speed (km/h)	Assumed Operating Speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature (K)	
			Computed	Rounded
30	30	29.6	1.6	2
40	40	44.4	3.7	4
50	47-50	57.4-62.8	6.1-7.3	6-7
60	55-60	74.3-84.6	10.2-13.3	10-13
70	63-70	99.1-110.8	16.4-22.8	16-23
80	70-80	112.8-139.4	23.6-36.1	24-36
90	77-90	131.2-168.7	32.0-52.8	32-53
100	85-100	157.0-205.0	45.8-78.0	45-80
110	91-110	179.5-246.4	59.8-112.7	60-110
120	98-120	202.9-285.6	76.4-151.4	75-150
130	105-130	227.9-327.9	96.4-199.6	95-200

Note: The above are minimum values, use higher K factors whenever possible.