5 Vertical Alignment

5.1 General

The vertical alignment of a road is known as the *grade line*, the *profile*, the *longitudinal section* or more simply the *long section*. *Profile* is the term used to identify vertical alignment in this manual.

The profile consists of straights, or grades, joined by curves. These curves, known as vertical curves to distinguish them from horizontal curves, are defined by two parameters, which are:

- a comfort factor which provides for a smooth passage from one grade to another, and
- a safety factor which ensures drivers have a safe sight distance over the full length of the vertical curve.

The profile is therefore the reference line by which the elevation of the pavement and other features of the road are established. Factors to be considered in the development of a profile include:

- topography
- road type
- horizontal alignment
- sight distance
- drainage
- heavy vehicle operational characteristics
- appearance.
- land purchase and construction costs
- cultural developments

The profile must ensure that all relevant design speed sight distance requirements are met at every point on the road alignment. It is also good design practice to make the vertical alignment design speed 10 to 15 km/h greater than the horizontal alignment design speed, to provide an additional safety margin.

In flat terrain the profile is often determined by drainage considerations.

In rolling terrain some undulation in the profile can allow a more economic construction. The appearance of the road must, however, be carefully considered in these cases, eg. a straight horizontal alignment which has a series of humps which are visible for some distance ahead should be avoided whenever possible.

In hilly and mountainous terrain the profile is usually closely dependent upon the physical controls.

The profile should normally coincide with the axis of rotation for superelevation development and be located in respect to the road cross-section in the following manner:

(a) Two Lane Roads

The profile should coincide with the road pavement centreline.

(b) Ramps and Motorway to Motorway Connections

The profile may be positioned at either edge of the travelled way, or centreline if a multilane road.

(c) Dual Carriageway Roads

The profile may be positioned at either the centreline of the median or at the ultimate median edge of the travelled way. The former case is appropriate for paved medians 4.0 m wide or less. The latter case is appropriate when:

(i) the median edges of the travelled way of the two roadways are at the same elevation,
(ii) the two roadways are at different elevations, or
(iii) the median width varies.

The vertical alignment of a road affects travel speed, safety and its appearance. The effect of grades can usually be ignored however when assessing the likely travel speeds of light vehicles because these are governed more by the effects of the horizontal alignment than by the vertical alignment. However, the reverse is the case for heavy vehicles. See Section 5.2 for details of grade limits for roads.

Vertical curves are described as crest or sag curves depending on their orientation. Six possible variations of vertical curve are possible and are illustrated in Figure 5.1.

Visibility and comfort are the most important factors in vertical curve design. Crest vertical curves *must* provide stopping sight distance for the design speed of the horizontal alignment and, where possible, sight distances should be increased to provide passing opportunities. Sag vertical curves *must* ensure vehicle occupant comfort, ie. the rate of vertical acceleration, and headlight performance criteria *must* be met. Other factors which must also be considered in sag vertical curve design are drainage requirements and sight line restrictions caused by overhead structures.

During all stages of design the form of the road must be considered as a three-dimensional structure which should not only be safe, functional and economical but also aesthetically pleasing. The co-ordination of horizontal and vertical alignment is described in more detail in Section 3 of this manual.
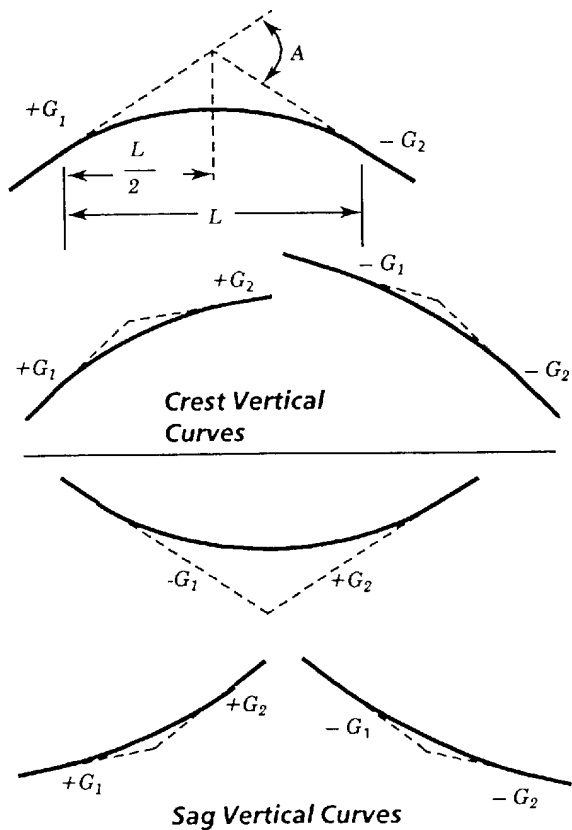
Where:

G_1 and G_2 = tangent grades (%)
A = algebraic difference in grade (%)
L = length of vertical curve (m)

Source: RTA

Figure 5.1:   Types of Vertical Curves

5.2   Grades

5.2.1   General

Road grades are normally expressed as percentages. A rising grade is a positive grade and a negative grade is a falling grade.

5.2.2   Minimum Grades

Minimum state highway grades are set by the need to provide adequate drainage along the road of water from the road surface and are shown in Table 5.1.

Road Type	Grade (%)
Roads with Kerb and Channel	0.3
Un-kerbed roads in cuttings	0.5
Un-kerbed roads in all other situations (See Note 1)	0

Table 5.1   Absolute Minimum Road Grades

NOTES:

1. *A level grade can provide acceptable drainage conditions on un-kerbed roads in flat terrain provided the roadway is adequately crowned and there are no problems in removing the pavement surface water runoff by table drains or ditches.*

2. *On curved roads in flat terrain, drainage problems can arise on the inside of curves and also in the vicinity of zero pavement crossfall, which occurs within superelevation development lengths. In these areas some longitudinal grade must be provided to maintain the water depths within acceptable limits and long plan transition spiral curves should be avoided to reduce the extent of flat crossfall. The amount of grade required depends on the width of pavement, the location of the grading point and whether or not kerb and channel is being used. As a general rule longitudinal grades on superelevation development sections need to be at least 1% and whenever grades are less than 1.5%, the depth of surface water flows should be checked to ensure that aquaplaning will not occur.*

3. *The minimum gradient for kerbed roads is suitable for most normal conditions of rainfall intensity, pavement surface texture, and drainage outlet spacing. In special cases, hydraulic analysis is required to determine the extent of water spread on the adjacent on the adjacent travel lane.*

4. *Intersections can contain large paved surface areas. It is important to avoid operational problems related to surface water ponding by ensuring that the combination of crossfall and longitudinal gradient is at least 2.5% for a chip seal surface and at least 2.0% for a smooth asphalt surface.*

5. *Flat longitudinal grades also occur at the apex of crest and sag vertical curves and adequate crossfall must be provided at these locations.*

6. *In very flat urban areas, it is preferable to use near level grades by providing additional sumps or other drainage outlets rather than introducing artificial undulations in order to provide self-draining kerb and channel. Artificial undulations can result in an unsightly appearance which suggests faulty construction, particularly where the horizontal alignment is straight or nearly straight. Alternatively, the false grading technique, where the pavement grade is not parallel to the top of the kerb and channel, can be used to ensure that minimum grades are provided for kerbs and channels.*

5.2.3   Maximum Grades

Maximum road grades are determined by vehicle performance, particularly heavy vehicles, and level of service criteria. In some cases approval may be given to increase the maximum grade permitted on a section of a state highway after due consideration of the type of terrain, adjacent land use, financial conditions, relative importance of the road, etc.

On high-speed roads, grades up to 3% provide road users with a good level of service and minimise the adverse effects of speed variance between different types of vehicle.

On roads with more modest operating speeds, grades up to about 6% do not usually cause noticeable problems with speed variance.

Grades steeper than 10% often cause speed variance problems. The main problem is the very slow uphill speeds of heavy vehicles but there is also the potential for high downhill speeds on steep grades and the safety problems associated with these. In both cases design options include flattening grades and the provision of auxiliary lanes and/or special facilities for safely controlling runaway vehicles on downgrades. In general, a flatter grade makes fewer demands on both vehicles and drivers.

Table 5.2 indicates the general maximum grades that should be used on sealed state highways in relation to terrain type and design speed.

Design Speed (km/.h)	Terrain		
	Flat	Rolling	Mountainous
60	6 - 8	7 - 9	9 - 10
80	4 - 6	5 - 7	7 - 9
100	3 - 5	4 - 6	6 - 8
120	3 - 5	4 - 6	-

Table 5.2: Maximum Grades for Roads with Sealed Pavements (%)

NOTES:

1. Grades less than the lower limits shown in Table 5.2 should normally be used on state highways, and grades steeper than 10% should be avoided.

2. For roads with unsealed surfaces reduce the values in Table 5.2 by 1%.

3. The use of maximum grades may have other design consequences, eg. whether or not a truck climbing lane, or an escape lane, is required.

4. Many drivers are unable to judge the increase or decrease in stopping distances, or the increase in acceleration distances, required on steep grades. Therefore, the desirable maximum grades on the approaches to, and within, an intersection is 3% but 6% may have to be accepted in difficult terrain. A 1% minimum gradient is also desirable to allow for a reduction in crossfall within the intersection area without impairing drainage.

5. Long lengths of grade steeper than 3% will cause heavy vehicles to slow down. See Figure 5.12 for details of truck performance on grades.

6. In areas subject to frost or snow, grades should be kept as low as possible. The desirable maximum grade in these conditions is 3% and the absolute maximum is 6%.

7. In areas of zero crossfall, and within superelevation development areas, surface water flowing longitudinally along the road can increase in depth to the point where aquaplaning occurs. Surface water flow depths should therefore always be checked on grades steeper than 6%.

5.2.4 Steep Grades

Although the general maximum grade for all state highways is 10% there may be circumstances where steeper grades are unavoidable, eg. in mountainous terrain. The use of grades steeper than 10% requires the approval of the Highways Strategy and Standards Manager.

As a guide for use in these situations most cars can climb a grade of 30% on a sealed surface but trucks are generally limited to approximately 15%.

The following situations are examples where the use of grades steeper than the general maximum may be warranted:

- a comparatively short section of steeper grade which will allow significant construction cost savings,
- difficult terrain where achieving the general maximum grade is not practical,
- where very few heavy , orcles use the road.
- local roads where the costs of achieving high standards are less able to be justified.

5.2.5 Grading at Intersections

Refer to Section 5.2.4: Note 4
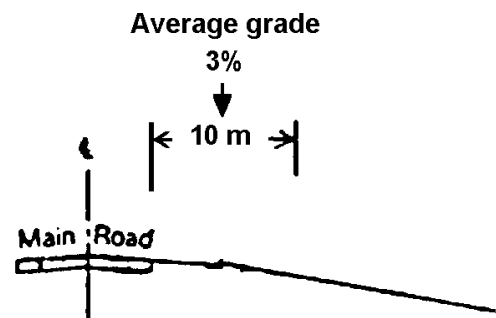
Figure 5.2: Typical Uphill Side Road Approach to an Intersection
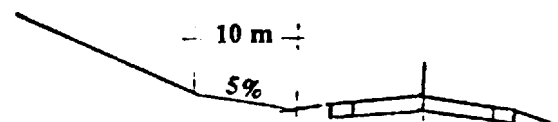
Figure 5.3: Typical Downhill Side Road Approach to an Intersection

(a) Adverse Crossfall

The grades within intersections can create problems for turning vehicles, particularly trucks, because the combined effect of grade and crossfall can create an adverse crossfall which exceeds the limits of stability for high vehicles.

Where it is not possible to achieve the desirable 3% maximum longitudinal grade the adverse crossfall developed on turning vehicle paths must be checked.

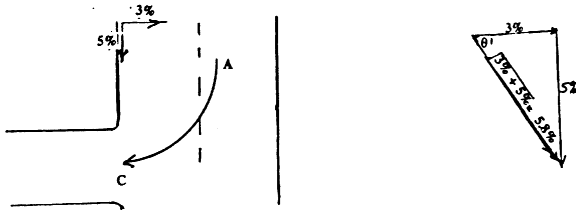A method for checking the resultant crossfall for a vehicle turning path is illustrated in Figure 5.4.

Figure 5.4:   **Resultant Crossfall on a Vehicle Turning Path at an Intersection**

The maximum adverse crossfall experienced by turning vehicles should not exceed 4%, and *must never* exceed 6%.

(b)   Ride Comfort

The grading of all roads through an intersection, particularly the side and/or cross roads, must be carefully checked for vehicle occupant ride comfort, especially where vehicle movements can take place at speed. Vehicle occupant ride comfort is measured in terms of vertical acceleration and vertical curves should be provided at each change of grade within an intersection wherever possible. This is very important for channelised intersections and at-grade intersections on dual carriageway roads where opposing directions of travel may be well separated. Refer to Section 5.6. for more details on the lengths of vertical curves necessary to provide acceptable levels of vehicle occupant ride comfort.

Care must be also taken with sag curves at intersections to avoid ponding at the low point and water flowing across the traffic lanes.

(c)   Roundabouts

Trucks with high loads, and travelling at 20 to 25 km/h on a roundabout, operate close to their rollover thresholds. This problem is aggravated by the adverse crossfall which is commonly used to make the central circular island more visible on the approaches to the roundabout, and to aid surface water drainage.

Care *must* be taken to avoid features which could initiate truck instability on roundabouts. Some guidelines to help minimise the problem are:

- Avoid rapid changes in crossfall.
- In flat terrain the preferred design for a roundabout is in the form of an inverted plate. On single lane roundabouts 2% adverse crossfall should be used and 2.5% on multi-lane roundabouts.

- In sloping terrain, a roundabout can be designed as a tilted plane surface. In these situations the crossfall will vary around the roundabout with the maximum being developed on the low side. To minimise this effect the lower edge of the roundabout can be tilted upwards.
- The maximum grade of the overall plane of the roundabout should not exceed 3%. If the terrain is steeper than 3%, it should be flattened to provide a grade of 3% or less and care must be taken to ensure that the roundabout is clearly visible on all approaches.
- Roundabouts with a fold or ridge through the centre, as shown in Figure 5.5, can create instability problems for trucks. To minimise the problem, the maximum change of grade over the ridge should be limited to 4% and the vertical curve joining the two grades should be as long as possible.
- Where a roundabout is used as an interchange ramp terminal, the general slope of the roundabout can be graded towards the off ramp. This provides better sight distance for traffic on the off ramp approach to the roundabout.
- Pavement surfaces must be maintained in good condition free from loose gravel, potholes, corrugation or rutting. For these reasons, it is good practice to use concrete or full depth asphalt for the pavement within the roundabout.
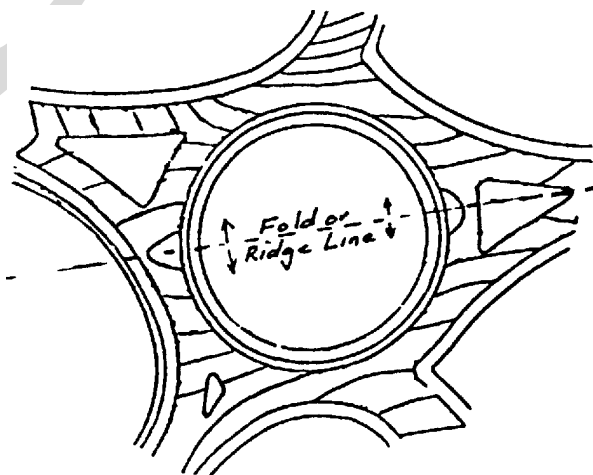

Figure 5.5:   **Example of a Grade Change within a Roundabout**

5.3 Vertical Curves

5.3.1   General

Various curve forms are suitable for use as vertical curves but the parabola has been traditionally used because it gives a constant rate of change of grade which maintains a constant driver sight distance, and also because of its ease of manual calculation. Other forms of curves, particularly those more suited to computer calculation, are equally satisfactory.

For a simple parabola the rate of change of grade per unit length is a constant Q and can be expressed as:

$$Q = \frac{A}{L}$$

The reciprocal K is defined as the horizontal distance in metres which results in a 1% change in grade:

$$K = \frac{L}{A}$$

Where:

L	=	Length of vertical curve (measured horizontally)
A	=	Algebraic difference of grades (%)
K	=	Length of vertical curve (m) for a 1% change of grade (m/unit %)
Q	=	Rate of change of grade per unit length (% / m)

The K value concept is a simple and convenient method for designating the size of a vertical curve. For each design speed and sight distance configuration a single value of K defines the length of the curve L for all values of A.

Also, for design purposes a vertical curve can be plotted using a circular curve of radius R which approximates very closely a parabolic curve by the relationship:

$$R = 100\,K$$

Figure 5.6 illustrates the K value concept.
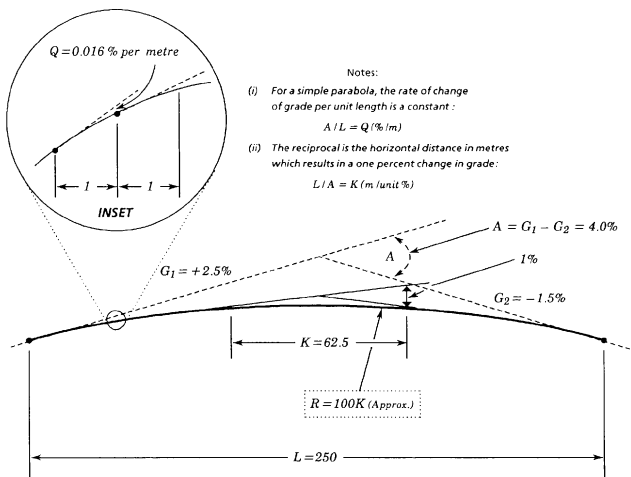
**Figure 5.6:   Parabolic Vertical Curves
                       K Value Concept and Details**

Figure 5.7 illustrates the components of a parabolic vertical curve and the formulae used to calculate the middle ordinate, intermediate ordinates and the distance from either end of the curve to the low point on a sag curve or high point on a crest curve.
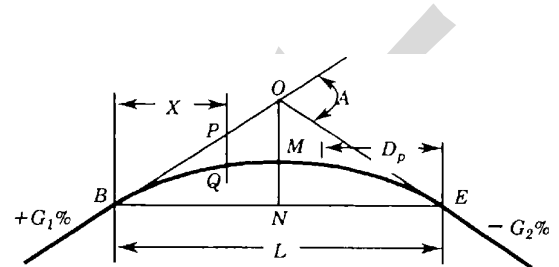
Calculation for Middle Ordinate. (OM)

$$OM = \frac{L\,(G_1 - G_2)}{800} = \frac{LA}{800}$$

Calculation for Intermediate Ordinate. (PQ)

$$PQ = \frac{X^2}{\left(\frac{L}{2}\right)^2}\,OM = \frac{4X^2}{L^2}\,OM$$

*Calculation for distance of curve high / low
point from curve extremity, B or E. (D_p)*

$$D_p = \frac{L \times G_1 \text{ or } G_2}{G_1 - G_2} = G_1 \text{ or } G_2 \times K$$

Where:

A	=	*Algebraic grade difference of vertical grades (%).*
$B\&E$	=	*Beginning and End of vertical curve. (in relation to stationing).*
D_p	=	*Distance of curve High/Low point from curve extremity B or E.*
$G_1\,G_2$	=	*Percent of intersecting grades.*
L	=	*Length of vertical curve ($K \times A$)*
M	=	*Middle point on curve between B-E and middle point between O-N*
N	=	*Middle point on chord between B-E.*
O	=	*Intersection point (I.P.) of the two grades.*
OM	=	*Middle Ordinate (m).*
PQ	=	*Intermediate Ordinate (m).*
X	=	*Distance of intermediate ordinate from curve extremity B or E.*
K	=	*Measure of vertical curvature*

Figure 5.7:   Parabolic Vertical Curve Details

5.3.2 Vertical Curve Length Controls

(a) Appearance Requirements

For very small changes of grade, vertical curves have little effect on the appearance of the road's profile and may usually be omitted.

Short vertical curves can, however, have a significant effect on the appearance of a road's profile Therefore, vertical curves for small changes of grade should have K values significantly greater than those needed for minimum sight distance reasons. This is particularly important on high standard roads, especially for sag curves.

Table 5.3 shows the maximum grade change which may be used without a vertical curve and the minimum length of vertical curve necessary to give a satisfactory appearance. Longer curves are preferred where they can be achieved without conflict with other design requirements, such as drainage.

The values in Table 5.3 are subjective and the lack of precision is intentional. The general ranges are, however, relevant.

Design Speed (km/h)	Maximum Change of Grade without a Vertical Curve (%)	Minimum length of Vertical Curve for Satisfactory Appearance (m)
40	1.0	20 - 30
60	0.8	40 - 50
80	0.5	60 - 80
100	0.4	80 - 100
120	0.2	100 - 150

Source: Austroads

Table 5.3: Vertical Curve Appearance Criteria

(b) Comfort Requirements

A human being subjected to rapid changes in vertical acceleration feels discomfort. However, vertical acceleration only becomes critical in the design of sharp sag curves.

For normal road design purposes the vertical acceleration generated when passing from one grade to another is limited to a maximum of 0.05g, where g is the acceleration due to gravity (9.8 m/sec[2]).

On low standard roads, and at intersections, a vertical acceleration of 0.10g may, however, be used where necessary.

The vertical component of acceleration normal to the curve, when traversing the path of a parabolic vertical curve at uniform speed is given by:

$$a = \frac{V^2}{12960\ K}$$

Where:

a	=	vertical component of radial acceleration (m/sec[2])
V	=	speed (km/h)
K	=	a measure of vertical curvature (m / 1% change of grade)

K values for specific design speeds and vertical accelerations are shown in Table 5.4. These are subjective criterion and the values have been rounded.

Design Speed (km/h)	K values	
	a = 0.05g	a = 0.10g
40	3	1.5
50	4	2
60	6	3
70	8	4
80	10	5
90	13	7
100	16	8
110	19	10
120	23	12

Table 5.4: Sag Vertical Curve Comfort Criteria

(c) Sight Distance Requirements

The length of a vertical curve for a given sight distance is given by the following expressions:

(i): Where length of curve is less than the sight distance:

$$L = 2D_s - \frac{C}{A} \quad \dots\dots\dots\dots\dots\dots \textbf{(1)}$$

(ii): Where length of curve is greater than the sight distance:

$$L = \frac{(D_s^2 \times A)}{C} \quad \dots\dots\dots\dots\dots \textbf{(2)}$$

Where:

L	=	vertical curve length (m)
D_s	=	sight distance (m)
A	=	algebraic difference of vertical grading (%)
C	=	sight line constant.

Substituting the vertical curve parameter K for $\frac{L}{A}$ in equation **(2)** gives:

$$K = \frac{D_s^2}{C} \quad \dots\dots\dots\dots\dots\dots \textbf{(3)}$$

K is therefore a constant for a given sight distance and method of defining the sight line.

Calculated vertical curve lengths should be rounded, and may need to be modified to comply with the subjective appearance and comfort criteria described in Sections 5.3.2 (a) and (b).

While equation *(1)* will always result in a lower *K* value than equation *(2)*, it is often convenient to use the expression *L = KA* to determine the length of curve in all situations, with *K* derived from equation *(3)*. This is not an appropriate practice in the case of overtaking sight distance where the long sight distances involved may extend over more than one vertical curve.

5.3.3 Crest Vertical Curves

(a) Sight Line Constants

The crest curve sight line constant for use in equations *(1)*, *(2)* and *(3)* is given by:

$$C = 200 \left[\sqrt{h_1} + \sqrt{h_2} \right]^2$$

Where:

h_1	=	driver's eye height above the road surface (m)
h_2	=	height of object on road surface (m)

Refer to Section 2.9.2 for full details of driver eye height and object height. The C values shown in Table 5.5 are for a driver's eye height of 1.05 m and selected values of object height.

Driver eye height - h_1 (m)	1.05	1.05	1.05
Object height - h_2 (m)	0 (pavement surface)	0.2	1.15
Sight line constant - C	210	433	879

Table 5.5: Sight Line Constants for Crest Vertical Curves

(b) Crest Vertical Curve Length

(i) General Minimum Curve Length

The minimum length of a crest vertical curve is usually governed by sight distance requirements. The absolute minimum sight distance which *must* be provided on crest vertical curves on all state highways is stopping sight distance from driver eye height to a 200 mm high object on the road surface, at the design speed of the associated horizontal alignment. Longer than minimum length vertical curves should, however, be used wherever possible:

- as a matter of good design practice
- where they can be used economically
- where they give a better fit to the topography.

(ii) Minimum Curve Length on Two-way Two-lane State Highways

Refer to Section 2.9.8.

The provision of crest vertical curves on two-lane two-way roads to give overtaking sight distance is usually impractical, and not always effective. Extremely long crest curves, ie. more than 750 m long, should be avoided because many drivers refuse to pass on such curves despite adequate sight distance, and for drainage reasons. It is often more economical to use an auxiliary lane and a short vertical curve than to provide overtaking sight distance by the use of a long vertical curve.

Intermediate sight distance is the desirable minimum sight distance and it should be provided on two-lane two-way roads wherever possible. This sight distance will usually provide reasonably safe overtaking opportunities for many drivers. Refer to Section 2.9.5 for more information on intermediate sight distance.

(ii) Minimum Curve Length on Dual Carriageway State Highways

Refer to Section 2.9.9.

In addition to the minimum sight distance requirement specified in Section 5.3.3 (b) (i) above, three extra sight distance requirements apply on a one-way multilane carriageway of a dual carriageway state highway. These are:

1. The provision of overtaking sight distance is not necessary.

2. A minimum distance of 1.4 times the design speed stopping sight distance, as measured between two points 1.05 and 1.15 m above the road, ie. driver eye height to vehicle height, *must* be provided. This distance is needed to ensure drivers have time to recognise that a vehicle ahead of them has stopped and for them to bring their vehicle to a stop in a safe and controlled manner.

   The application of this sight distance requirement often means that greater lateral clearances to obstructions on horizontal curves may be needed, otherwise it is not normally as critical as stopping sight distance.

3. On the approaches to off ramps drivers in the left lane of the through carriageway *must* have stopping sight distance as measured from a point 1.05 m above the road to the pavement surface, ie. driver eye height to zero object height. This provides 1.4 times the normal design speed stopping sight distance and it is considered necessary to ensure drivers have the best possible visibility conditions on the approach to the off ramp, and early identification of the off ramp alignment and its road markings.

(iii) Curve Design Data

Sight distance criteria for the design of crest vertical curves is shown in Table 5.6.

Design Speed (km/h)	Stopping Requirements					Minimum Overtaking Requirements	
	Stopping Sight Distance (m)	K $h_1 = 1.05\ m$ $h_2 = 0.20\ m$ $C = 433$	K $h_1 = 1.05\ m$ $h_2 = 0\ m$ $C = 210$	1.4 x Stopping Sight Distance (m)	K $h_1 = 1.05\ m$ $h_2 = 1.15\ m$ $C = 879$	Intermediate Sight Distance (2 x SSD) (m)	K $h_1 = 1.05\ m$ $h_2 = 1.15\ m$ $C = 879$
40	40	4	8	56	4	80	7
50	55	7	14	77	7	110	14
60	75	13	27	105	13	150	26
70	95	21	43	133	20	190	41
80	115	31	63	161	29	250	60
90	140	45	93	196	44	280	89
100	170	67	138	238	64	340	132
110	210	102	210	294	98	420	201
120	250	144	298	350	139	500	284

Table 5.6:   K Values for Crest Vertical Curve Design
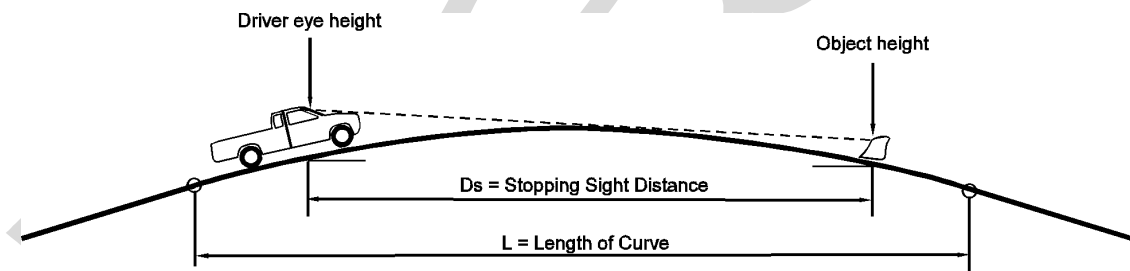

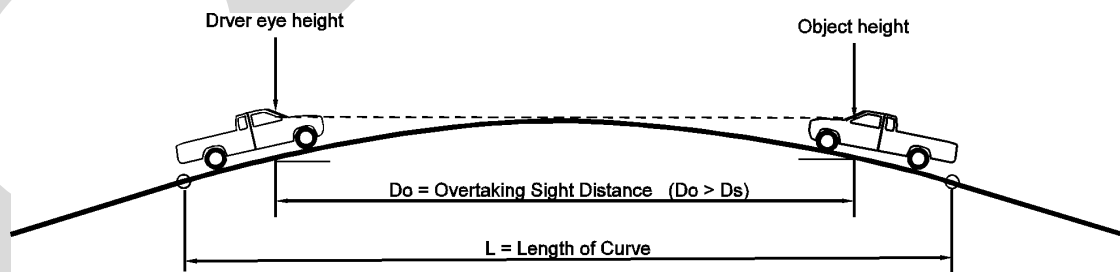**Figure 5.8:   Crest Vertical Curve - Stopping Sight Distance**

Figure 5.9:   Crest Vertical Curve - Overtaking Sight Distance

5.3.4 Sag Vertical Curves

(a) General

During daylight hours, and at night on roads with full standard street lighting, sight distances on sag vertical curves are not restricted by the vertical alignment unless an overhead obstruction is present. At night, vehicle headlight performance limits the effective sight distance to between 120 and 150 m on unlit roads. This is suitable for safe operating speeds up to about 90 km/h.

On dual carriageway state highways not likely to have street lighting, a headlight sight distance of 150 m *must* be provided on all sag vertical curves. The adjoining crest curves *must* also have a similar headlight sight distance so that the resulting vertical alignment will provide near consistent driving conditions night and day.

On state highways with street lighting, a 150 m headlight sight distance should be provided on all sag and crest vertical curves.

Where a sag curve is combined with horizontal curvature which would cause the headlight beam to shine off the pavement, assuming a 3 m lateral spread each way, very little is gained by flattening the sag vertical curve.

When sag vertical curves cannot be flattened to provide headlight stopping distance, they *must* provide an adequate level ride comfort. The vertical acceleration felt by vehicle occupants should normally be limited to no more than $0.05g$ although $0.10g$ may have to be accepted in difficult terrain.

(b) Sight Line Constant

The sag vertical curve sight line constant *C* for use with the equations in Section 5.3.2 (c) is given by:

$$C = 200 (h + D_s \, Tan \, q)$$

Where:

h = headlight mounting height (m)
Ds = stopping sight distance (max. 150 m)
q = elevation angle of headlight beam (+ is upwards)

A mounting height of 0.75 m and zero elevation gives:

$$C = 150$$

(c) Sag Vertical Curve Length

The length of a sag vertical curve should normally be determined by headlight sight distance requirements. When these cannot be met, sag vertical curve length *must* be determined by vehicle occupant ride comfort critera.

Sight distance and comfort criteria for the design of sag vertical curves are shown in Table 5.7.
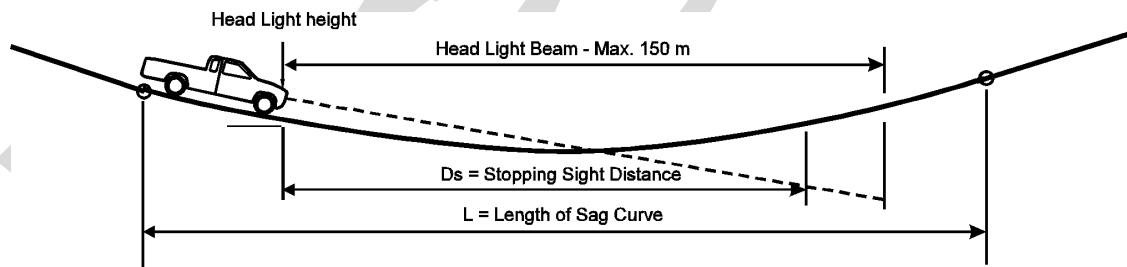
Figure 5.10: Sag Vertical Curve - Headlight Sight Distance

Design Speed	Headlight Sight Distance Control		Vehicle Occupant Ride Comfort Controls	
	Sight Distance	*C = 150*	Normal Design Situations *(a = 0.05g)*	Special Design Situations *(a = 0.10g)*
(km/k)	(m)	K	K	K
40	40	11	3	1.5
50	55	20	4	2
60	75	38	6	3
70	95	60	8	4
80	115	88	10	5
90	140	131	13	6
> 90	150	150	≥ 15	≥ 8

Table 5.7: K Values for Sag Vertical Curve Design

(d) Overhead Obstructions at Sag Curves

Overhead obstructions, such as road or rail overpasses, sign gantries, overhanging trees, etc. may limit driver sight distance on sag vertical curves, see Figure 5.11.

The sight line constant, *C*, for this situation is given by:

$$C = 200 \left[\sqrt{H - h_1} + \sqrt{H - h_2} \right]^2$$

Where

H = height of overhead obstruction
h_1 = eye height
h_2 = object cut off height.

An eye height of 1.8 m and an object height of 0.6 m, ie. heavy vehicle driver eye height to vehicle tail-light height, should be used to calculate the sight line constants and corresponding sight distances for a range of overhead obstruction vertical clearances.

Obstruction Height H (m)	Sight Line Constant C
4	2200
4.5	2600
5	3000
5.5	3400
6	3800

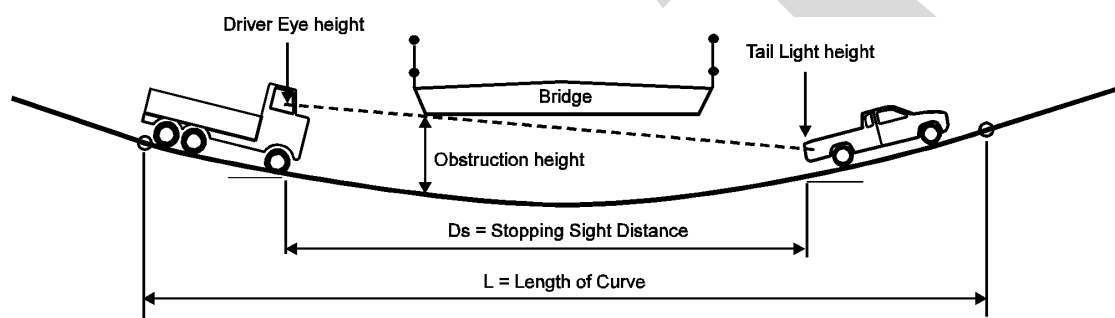**Table 5.8: Sight Line Constants for Overhead Obstructions**

Figure 5.11: Sag Vertical Curves - Overhead Obstruction

5.4 Vertical Alignment Controls

5.4.1 General

The vertical alignment of a road is usually affected by a physical control, and more often than not, several controls with competing demands, including:

- structures crossing the road. Future road widening requirement affects the type of structure to be used and the shape of the cross section can influence both the type of construction and the appearance of bridge soffits.
- ground water tables and flood levels. Particular care must be taken with superelevated sections of road because the low point of the pavement can be significantly lower than the profile level.
- clearances to culverts and other underpass structures which cross the road.
- minimising hard rock excavation.

5.4.2 Clearance to Overhead Structures

The minimum vertical clearances for structures crossing state highways are specified in Section 2.10 of this manual, and also in *Appendix A* of Transit's *Bridge Manual*. These are summarised as follows:

(i) The minimum vertical clearance over a traffic lane, at any time, *must never* be less than 4.9 m. The general minimum for design purposes is 5.0 m and this allows for up to 100 mm of pavement overlay during the design life of the road..

(ii) The minimum vertical clearance over the road shoulder is 4.5 m.

(iii) The minimum vertical clearance to the underside of an overhead mounted traffic sign or traffic signal is 5.5 m.

(iv) The minimum vertical clearance of any part of a pedestrian bridge crossing a state highway is:

1. at least 200 mm more than adjacent traffic bridges, and not less than 5.1 m, and

2. where there are no adjacent traffic bridges, at least 5.5 m.

Because of the high cost of structures, and the complex layouts at many state highway crossings, care must be taken to ensure that all factors have been considered and that critical vertical clearance points have been identified.

5.4.2 Clearance to Water Tables and Flood Levels

On dual carriageways and other high standard state highways the vertical alignment *must* be located above the natural ground level. A clearance of 500 to 1000 mm *must* be

maintained between the water table and the lowest point of the road pavement.

For economic reasons, it is generally not practical to provide these clearances on lower standard roads. The critical control in these cases is to ensure that the pavement is located above the water table.

Flood levels also need to be checked in flood prone areas, to ensure that at least the traffic lanes remain flood free.

Culverts are also critical profile controls and clearances to these also need to be checked at an early stage in road design.

5.4.3 Clearance to Underground Services

All underground services in the vicinity of the roadworks *must* be identified and located to ensure that minimum clearance requirements are satisfied. Services commonly involved include:

• gas and water mains
• stormwater and sewer drains
• underground telecommunication and power cables

In each case, the minimum clearances allowed *must* be obtained from the owners of these services.

Where it is difficult to achieve the minimum clearance it is sometimes possible to obtain agreement to the use of a reduced clearance together with some form of special protection for the service. This could involve relocating it into a duct, encasing it in concrete, the provision of a concrete slab over the service, etc.

5.4.4 Other Controls

The minimum vertical clearance for electric power transmission cables and telecommunication lines over a road should be checked and confirmed with owners of these services.

When a state highway is constructed near an airport, the minimum vertical clearances to airways specified by the agency responsible for the airport *must* be maintained. Street lighting poles *must* also be contained within the minimum clearance envelope.

Access to properties abutting the state highway *must* always be considered.

5.5 Passing Opportunities

5.5.1 General

The demand for passing opportunities on two-lane two-way roads increases with traffic volume and when there are differences in individual vehicle speeds. The typical slow vehicles are trucks, and these usually slow down on upgrades.

Slow vehicles cause a deterioration of the traffic flow and the general level of service on a road. One method of improving both of these is to provide passing opportunities by the addition of auxiliary, relatively short passing lanes. The warrants for determining the need for passing lanes and their design criteria are discussed in following sections.

5.5.1 Slow Vehicle Bays

(a) General

Slow vehicle bays are sometimes referred to as *'passing bays'* or *'turnouts'* in international literature. Slow vehicle bays are the formalised use of very short lengths of widened, unobstructed sealed shoulder on two-lane two-way roads, to allow slow moving vehicles to pull out of a traffic lane and give following vehicles an opportunity to pass.

Slow vehicle bays should usually be less than 300 m in length, as compared to passing lanes which should be at least 800 m in length. *Slow vehicle bays are not short passing lanes* but they can provide some of the benefits of passing lanes in certain circumstances.

(b) Application / Location

Slow vehicle bays are rarely used by trucks, except on very steep grades where the trucks are reduced to absolute crawl speeds. Slow vehicle bays are more suited to recreational and/or tourist routes where drivers of slow vehicles are usually more willing to let faster vehicles past them.

The provision of slow vehicle bays should only be considered on winding two-lane two-way rural roads in mountainous, coastal and scenic areas where *ALL* of the following conditions are met:

(i) long platoons of vehicles are rare, and
(ii) normal traffic flows are low, ie. AADT is less than 2000 vpd, and
(iii) traffic flows contain a high proportion of slow-moving vehicles, ie. at least 10% of AADT, and
(iv) the slow-moving vehicles are mainly recreational/tourist vehicles such as campervans, cars towing caravans and/or boats, etc, and
(v) passing opportunities are limited.

Proposals to provide slow vehicle bays *must* include careful consideration of traffic flows and their composition, the need to minimise queue lengths, minimum sight distance requirements and the relative costs/benefits of providing other types of passing opportunities.

Slow vehicle bays should be located so they end at the crests, or on the down sides of hills. It is essential that they do not end abruptly just prior to, or just beyond blind horizontal curves or just over the crests of hills, ie. where drivers do not have an adequate view of a slow-moving vehicle re-entering the traffic lane immediately ahead of them.

Similarly, drivers of vehicles exiting from slow vehicle bays should have a clear view of vehicles approaching from behind.

Where practicable, a series of slow vehicle bays should be provided, to give regular passing opportunities.

Where slow vehicle bays are located on a sustained grade the bays should be sufficiently long, or located frequently enough, to reduce the probability of slow moving vehicles having to stop completely to allow following queued vehicles to pass.

There are advantages in locating slow vehicle bays on right hand horizontal curves, when sight distance requirements can be met, because passing vehicles have shorter distances to travel and better inter-visibility between vehicles.

It is not advisable to locate slow vehicle bays on left hand horizontal curves because passing vehicles will have longer distances to travel and poor inter-visibility between vehicles.

Slow vehicle bays should not be located in the vicinity of parking areas, rest areas and scenic outlooks. When they are, great care must be taken with signs and markings to ensure that the slow vehicle bay is clearly distinguished from the other trafficable areas.

(c)   Slow Vehicle Bay Design

The geometric design of slow vehicle bays should be related to the mean traffic speed on the section of road under consideration and the length and location of each bay should bedetermined in respect to horizontal and/or vertical curve sight distance limitations.

(i)   Bay Length

Slow vehicle bays should be kept short, to ensure that approaching drivers will have a clear view of the full length of each bay.

Normally, a slow vehicle bay should be:

* not less than 60 m long, even on very low speed roads, and
* not more than 300 m long, because drivers may treat it as a conventional passing lane.

The minimum lengths recommended for slow vehicle bays, in relation to mean traffic speed on the road in the vicinity of the bay and exclusive of entry/exit tapers, are given in Table 5.9.

(ii)   Sight Distance Requirements

Slow vehicle bays *must* not be located where horizontal and/or vertical curves limit safe stopping sight distance, particularly in the exit taper merge areas.

Sight distance on the approach to a slow vehicle bay should be at least equal to the stopping distance required for the 85th percentile operating speed of traffic on the road. Drivers approaching a slow vehicle bay *must* also have a clear view through the full length of the bay, so they can determine whether the bay is available for use or a vehicle in the bay is about to re-enter the traffic lane.

Slow vehicle bays that cannot be clearly seen by approaching drivers are unlikely to be well used.

Mean Traffic Speed (km/h)	Minimum Length of Slow Vehicle Bay * *(excluding entry and exit tapers)*
30	60
40	60
50	70
60	80
70	100
80	135
90	175

* Minimum bay length is based on the assumption that a vehicle will enter a slow vehicle bay travelling at least 8 km/h slower than the mean speed of traffic on that section of road and it will be able to stop, if necessary, within in half the length of the bay while using a deceleration rate not exceeding 3 m/sec².

Table 5.9:   Minimum Lengths for Slow Vehicles Bays

(iii)   Signs

The correct signing is necessary to maximise the use of slow vehicle bays, and to ensure safe and efficient traffic operations. The signs used for slow vehicle bays are shown in the *Manual of Traffic Signs and Pavement Markings, Part 1: Section 10*.

5.5.3   Climbing Lanes

(a)   General

The effect of grades on heavy vehicle speeds is much more noticeable than on cars. Just limiting the maximum grade is not a complete design control, it is also necessary to consider the length of grade in relation to desirable vehicle operation. The term *'critical length of grade'* is used to indicate the maximum length of a grade on which a designated loaded heavy vehicle can operate without unreasonable reduction in speed. The amount of speed reduction depends on the combination of grade, length of grade and the mass power ratio of the vehicle.

On steeper grades, the reduction in speeds of heavy vehicles and the inability of drivers of other vehicles to complete overtaking manoeuvres in a safe manner at a comfortable overtaking speed, warrants design consideration of means to improve traffic flows. The horizontal alignment of the road cannot usually be altered and, where the grade exceeds the critical length, two improvement alternatives should be considered:

* adjust the vertical alignment until the grade is no longer critical, and
* add a passing, or climbing, lane for slow vehicles.

(a)   Warrants for Climbing Lanes

The power/mass ratio of a vehicle determines how it will operate on different grades. A vehicle having a 5kw/ton is considered to represent an appropriate lower level of performance for design purposes.

The power/mass ratio of a vehicle determines how it will operate on different grades. A vehicle having a 5kw/tone is considered to represent an appropriate lower level of performance for design purposes.

When a steep grade causes heavy vehicle speeds to fall well below those of other vehicles, a climbing lane may be appropriate at lower traffic volumes. Studies show that regardless of the average speed, the greater a vehicle deviates from this average speed the greater its chance of being involved in an accident. The accident involvement rate increases more rapidly when the vehicle speed reduction is greater than 15km/h. From a safety point of view, a 15km/h speed reduction should be used as the criteria for a minimum tolerable speed at which slow vehicles can operate on a grade without a climbing lane.

A 25 km/h speed reduction criteria may be used for multilane roads when speed differentials are significant and are considered to be contributing to safety concerns. The need for passing opportunities is not common on multilane roads because there are normally fewer problems associated with the passing of slow moving vehicles.

Steep downgrades can also have a detrimental effect on the capacity and safety of a road with high traffic volumes and numerous heavy vehicles. Heavy vehicles descending steep downgrades in low gear have the same effect as they do on the equivalent upgrades. Although no passing lane criteria have been established for these conditions, providing passing opportunities is generally appropriate when delays caused by slow moving heavy vehicles on downgrades are seen as a problem, particularly where the horizontal alignment contains tight curves.

(b)   Lengths of Climbing Lanes

The length of a climbing lane is based on a speed differential of 15 km/h or greater. A minimum length of 800 m is recommended, including tapers, in order to allow for the completion of overtaking manoeuvres. 500 m may be adequate on low volume roads in hilly terrain.

A climbing lane should normally be introduced at the point where the design vehicle is expected to experience a 15 km/h speed reduction below the design speed of the section of road. The lane should end where the design vehicle is anticipated to regain a speed equal to, or higher than, the speed at which the climbing lane was initiated. In general, a climbing lane should end beyond the crest of the vertical curve where there is enough sight distance for a driver of an overtaking vehicle to decide whether to complete or abandon the overtaking manoeuvre. Where sight distance is not adequate, the climbing lane *must* be extended to a point where intermediate sight distance is available.

An example of a climbing lane length calculation, based on a 15 km/h speed reduction and grade performance curves for a 5 kw/ton design vehicle, is illustrated in Figure 5.12.

Where the algebraic difference of the tangent grades is less than 4% the intersection point of the grades is used in the analysis of climbing lane length. Where the algebraic difference is greater than 4 %, grades on vertical curves are approximated by the use of average grades which connect the quarter points of the vertical curves. An approach speed of 90 km/h should be used because it is the legal maximum speed for trucks on New Zealand roads. The maximum speed limit for trucks with trailers is 80 km/h.

NOTE:

> *When consecutive climbing lanes occur at close spacing they should be joined together to form one continuous climbing lane.*

5.5.3   Passing Lanes

(a)   General

Overtaking and passing of slow moving vehicles on two-lane rural roads can be restricted even on relatively flat sections of road because:

- restricted horizontal and vertical sight distance reduces passing opportunities, and/or
- high traffic volumes, particularly in the opposing direction, do not provide sufficient gaps for passing manoeuvres to be performed safely.

It is generally desirable to provide for overtaking or intermediate sight distances in order to allow faster vehicles to overtake and to delay the need for passing lanes. However, as traffic volumes increase on roads with adequate sight distances, safe passing opportunities decrease. Consequently, if passing lanes are not provided, queues build up, driver frustration increases and unsafe passing manoeuvres often result. The provision of passing opportunities also improves the traffic operation and delays the need for widening the road to four lanes.

Driver frustration can be reduced by signing which informs them of the next passing opportunity, eg. the next passing lane is 5 km ahead. Refer to the *Manual of Traffic Signs and Pavement Markings, Part 1: Section 10* for details of these signs.

(b)   Warrants for Passing Lanes

Some drivers may not overtake other vehicles even when sight distances are adequate. Therefore, passing opportunities should be provided at intervals of about 10 minutes of driving distance, to improve operating conditions over a long section of road.

A passing lane is generally considered necessary when the Level of Service falls below C for that section of road. A level of service measure based on bunching, ie. percent of following vehicles with headways less than 4 seconds, may be used for a scheme plan analysis rather than the HCM level of service criteria. Simulation models and traffic recording equipment can be used to measure the existing traffic flows and predict the level of service based on bunching.
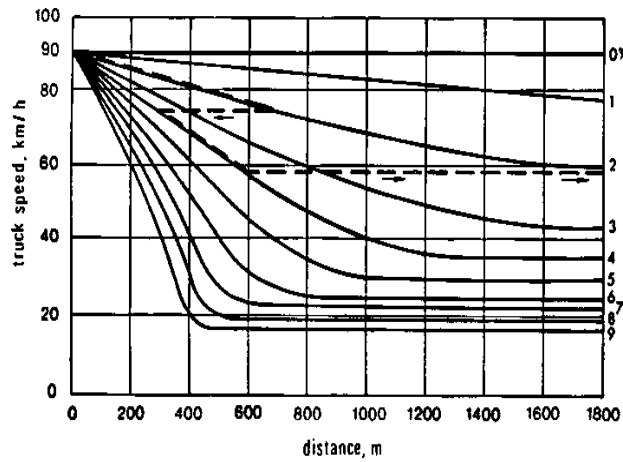
Performance Curves for 5 kw/Tone Trucks on Upgrades
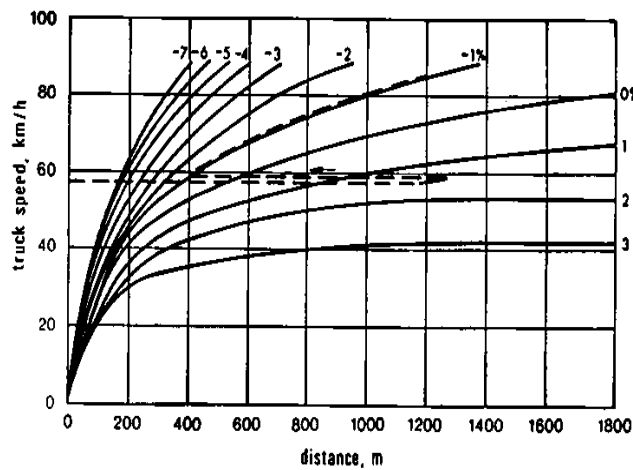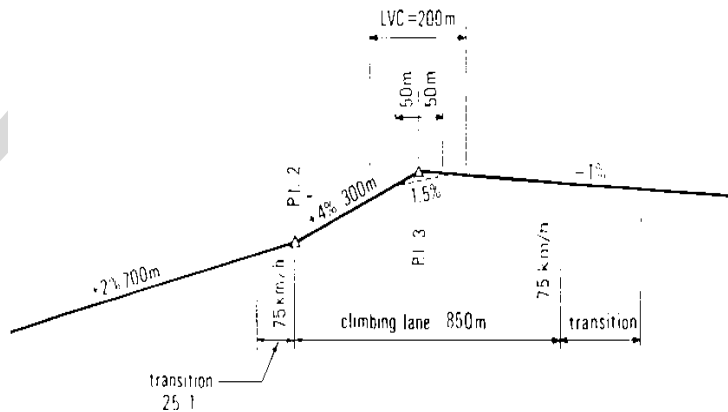

Performance Curves for 5 kw/Tone Trucks on Downgrades

Source: ???

Figure 5.12:   Climbing Lane Length - Example Calculation

Mathematical and simulation models such as TRARR may be used to evaluate the effectiveness and benefits of passing lanes. Transfund's *Project Evaluation Manual* describes a simplified method for assessing the benefits associated with providing passing lanes.

Table 5.10 also gives a guide, based on current year AADT volumes, as to when a passing lane is generally justified.

Overtaking Opportunities Over Preceding 5 km (a)		Current Year Design Volume (AADT)		
Description	Percent Length Providing Overtaking (b)	Percentage of Slow Vehicles (c)		
		5	10	20
Excellent	70 - 100	5670	5000	4330
Good	30 - 70	4330	3670	3330
Moderate	10 - 30	3130	2800	2470
Occasional	5 - 10	2270	2000	1730
Restricted	0 - 5	1530	1330	1130
Very Restricted (d)	0	930	800	670

(a) Depending on road length being evaluated, this distance could range from 3 to 10 km.
(b) Including light trucks and cars towing trailers, caravans and boats.
(c) No overtaking for 3km in each direction

Source: Austroads

Table 5.10: Guidelines for the Provision of Passing Lanes

(c) Spacings and Lengths of Passing Lanes

Driver frustrations and economic constraints influence the frequency of passing lanes. In general, a spacing of 10 to 15 km, ie. approximately 10 minutes travel time, provides an adequate level of passing opportunity on low volume roads. Additional passing opportunities may be constructed between these as the traffic volume increases and in the long term passing lanes may be located as close as 3 to 5 km apart on some sections of road. It may not always be possible to maintain the desirable spacing, however, because of substandard road geometry and roadside developments.

The minimum length of a passing lane is 800 m. Short passing lanes do not allow multiple vehicle overtaking, or overtaking when there is only a small difference in vehicle speeds. Passing lane lengths in the order of 1.5 km to 2.0 km will provide sufficient opportunity for most queues formed behind a slow vehicle to overtake it and disperse. Where very long queues form it is generally preferable to provide several relatively close passing lanes rather than a single long one, because the queue dispersion rate is high at the start of a passing lane and reduces along it.

5.6 Emergency Escape Lanes or Arrester Beds

On steep down grades where there is a history of runaway truck accidents, escape lanes may be considered to bring these vehicles to a safe and controlled stop. Escape lanes are very effective in minimising accident injury and preventing gross property damage.

Escape lanes usually take the form of an unsurfaced, steep, uphill ramp. Deceleration is provided by the retardation component of gravity on an upgrade and drag/friction of the vehicle's tyres running on the gravel or earth surface. The lanes are usually located adjacent to the road shoulder on the left hand side of the road.

Escape lanes must be sufficiently long and there may be difficulties in providing them because of property acquisition costs or terrain conditions. In such cases, a gravel bed arrester system might be appropriate.

A gravel bed arrester consists of an escape lane with a regulated depth of sized loose gravel contained within a formed bed. The typical depth of the loose gravel bed is 450 mm and its width 6.0 m.

The length of the escape lane is dependent on the design vehicle, the grade of the road and the grade of the arrester bed lane. For a level escape lane the length required for the worst case, ie. a travel speed of 160 km/h, is 300 m. This is reduced to about 180 m for an upgrade of 20%.

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