CHAPTER B
ALIGNMENT ELEMENTS

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CHAPTER B ALIGNMENT ELEMENTS

B.1 INTRODUCTION

Roads are traditionally designed in three views: plan, profile and cross-section. The highway design engineer will often design each view independently, perhaps including a sight distance calculation. Drivers, however, have a different appreciation for road appearance since they see the road from various angles. Some road features will be in view from all points for a considerable length, while other features will be in view momentarily.

Generally, when topography is rugged or rolling, considerable construction cost savings can be made when the design speed is lowered at selected locations. In rugged or rolling terrain, when reductions in design speed are considered, the horizontal alignment standard is generally more critical than the vertical alignment standard. The posted highway speed is not frequently lowered due to vertical alignment deficiencies unless they occur at hazardous locations such as intersections. However, horizontal alignment deficiencies may necessitate lower posted speed. Consequently, it is important to attach a high priority to meeting horizontal alignment standards while exercising more flexibility with vertical alignments. The examination of different alternative vertical alignments is encouraged to maximize the benefits considering construction costs, road user costs and safety. Notwithstanding the above, vertical alignment standards should not be compromised near intersections due to the importance of intersection sight distance.

Certain combinations of horizontal and vertical curves can result in an apparent distortion in the alignment or grade although the horizontal and vertical curves comply to the design standards outlined in Table A.7.

Although this guide does not identify all concepts of good design form, Figures B-3.2a through B-3.2l illustrate various bad design forms that are aesthetically displeasing to drivers and should be avoided.

B.2 SIGHT DISTANCE

B.2.1 General Considerations

The ability to see ahead is of utmost importance in the safe and efficient operation of a vehicle on a highway. The path and speed of motor vehicles on highways and streets are subject to the control of drivers whose ability, training and experience are quite varied. For highway safety, sufficient sight distance must be provided so that drivers can avoid striking unexpected objects on the roadway surface. Certain two-lane highways should also have sufficient sight distance to enable drivers to safely occupy the opposing traffic lane during passing manoeuvres. Two-lane rural highways should generally provide such passing sight distance at frequent intervals and for substantial portions of their length. Conversely, it is normally impractical to provide passing sight distance on two-lane urban streets or arterials. The length and interval of passing sight distance should be compatible with highway function. Generally, for two-lane undivided rural arterial highways, it is desirable to provide passing sight distance over at least 70 percent of the length.

Sight distance is discussed in five steps:

1. Stopping sight distances (the distances required for stopping, applicable on all highways).

2. Passing sight distances (the distances required for passing manoeuvres, applicable only on two-lane highways).

3. Non-striping sight distances (the distances required for the pavement markings to allow passing) on two-lane highways.

4. Decision sight distances (the distances needed for decisions at complex locations).

5. The criteria for measuring these distances for use in design.

The design of alignment and profile to provide these distances and to meet these criteria are described later in this chapter. The special conditions related to sight distances at intersections are discussed in Chapter D.

B.2.2 Criteria for Measuring Sight Distance

Height of Driver's Eye

For passenger vehicle sight distance calculations, the height of the driver's eye is considered to be 1.05m above the road surface. This height is based on surveys of actual vehicles and drivers and will accommodate the vast majority of vehicle/driver combinations. This height of eye is used in measuring stopping, passing and decision sight distances.

An eye height of 1.8m is adopted for single unit vehicle (SU) and the BUS design vehicle.

An eye height of 2.1m is adopted for all large trucks or truck-trailer combinations based on the height of typical highway tractors.

For pavement marking purposes (non-striping sight distance), a height of eye = 1.15m and height of object = 1.15m is adopted for simplicity.

Height of Object

For stopping sight distance an object height of 0.38m is used. This value is based on the legal minimum height for a vehicle tail light in Canada.

This height was adopted based on the rationale that a driver may not make the decision to stop for an object less than 0.38m. from the road surface. In previous years, the height of 0.15m was the standard, but it was determined that drivers may not stop for objects of that height. A driver will, however, stop for an object that is 0.38m in height.

The average tail light height typically seen on Alberta roads is considerably greater than 0.38m. However, 0.38m is used to ensure that the worst scenario is accommodated by the stopping sight distance model.

The height of object used for passing sight distance and intersectional sight distance is 1.3m. This represents the full height from road surface to roof of a design passenger vehicle.

The height of object used in non-striping sight distance is 1.15m.

The height of object used in decision sight distance is selected based on circumstances. For example, the object height is zero if the driver needs to see the road surface.

B.2.3 Stopping Sight Distance

Sight distance is the length of roadway ahead visible to the driver. The minimum sight distance available on a roadway should allow a vehicle, travelling at an assumed running speed (which is based on the design speed), to stop before reaching a stationary object in its path. Although greater length is desirable, sight distance at every point along the highway should be at least the distance required for a vehicle to stop.

Stopping sight distance is the distance a vehicle travels from the instant the driver sights an object and decides to stop, to the instant the vehicle comes to a complete stop after applying the brakes. This distance depends upon perception-reaction time, height of the driver's eye, height of object, coefficient of friction for the highway surface and the initial vehicle speed. For design purposes, the following criteria have been adopted by Alberta Infrastructure to determine these minimum values:

- A fixed perception-reaction time of 2.5 seconds
- Eye height of 1.05m
- Object height of 0.38m
- Coefficient of friction for wet pavement (based on American Association of State Highway and Transportation Officials 1990 and Transportation Association of Canada 1986 values)
- Assumed running speed equals design speed up to 100km/h. For design speeds of 110 km/h and higher, the assumed running speeds are based on the 85th percentile running speeds recorded on Alberta highways in good conditions.

These criteria and the resulting minimum values are tabulated in Table B.2.3.

The derived minimum stopping sight distances directly reflect passenger car operation and might be questioned for use in design for truck operation. Trucks as a whole, especially the larger and heavier units, require longer stopping distances than passenger cars. This is assuming that the braking capability of the vehicles is the limiting factor in determining the stopping distance.

There is a factor that tends to balance the additional braking lengths for given truck speeds with those for passenger cars. The truck operator is able to see the vertical features of the obstruction from farther away because of the higher position of the truck seat. Separate stopping sight distances for trucks and passenger cars, therefore, are not used in highway design standards.

There are several situations that should be treated with caution. Every effort should be made to provide stopping sight distances greater than the minimum design value when horizontal sight restrictions occur on downgrades, particularly at the ends of long downgrades. Even when the horizontal sight obstruction is a cut slope, the truck operator's greater height of eye is of little value on long downgrades. Truck speeds may closely approach or exceed those of passenger cars. Although the average truck operator tends to be more experienced and quicker to recognize hazards than the average passenger car operator, under conditions of restricted horizontal sight lines it is best to supply a stopping sight distance that exceeds the values in Table B.2.3.

Another situation where trucks may have difficulty stopping is in the vicinity of underpasses due to railway grade separations, interchanges etc. The structure may restrict the sight lines for traffic (particularly trucks) on the lower elevation roadway. Additional sight distance should be provided where possible to avoid problems in these areas.

Minimum vertical crest and sag curvatures which satisfy stopping sight distance criteria are given in Section B.4.4.

Table B.2.3 - Minimum Stopping Sight Distance (SSD)

Design Speed (km/h)	Assumed Running Speed (km/h)	Perception-Reaction		Friction Factor	Braking Distance (m)	Computed Value (m)	Min. SSD for Design (Rounded) (m)
		Time (s)	Distance (m)				
40	40	2.5	27.8	0.38	16.6	44.4	45
50	50	2.5	34.7	0.36	27.3	62.0	65
60	60	2.5	41.7	0.34	41.7	83.4	85
70	70	2.5	48.6	0.32	60.2	108.8	110
80	80	2.5	55.6	0.31	81.2	136.8	140
90	90	2.5	62.5	0.30	106.2	168.7	170
100	100	2.5	69.4	0.30	131.1	200.5	200
110	108	2.5	75.0	0.29	158.2	233.2	235
120	115	2.5	79.9	0.28	185.8	265.7	270
130	115	2.5	79.9	0.27	192.7	272.6	275

Minimum stopping sight distance (SSD):

$$\text{SSD} = \text{perception / reaction distance} + \text{braking distance}$$

$$= \frac{Vt}{3.6} + \frac{V^2}{254f}$$

Where V is the assumed running speed, (km/h)
t is the perception/reaction time (sec)
f is the coefficient of longitudinal friction

Braking distance is derived from the general expressions:

$$u^2 = v^2 - 2ad \quad \text{and} \quad a = gf$$

Where u is the final speed
v is the initial speed
a is acceleration
d is distance travelled during acceleration
g is acceleration due to gravity

In this case u = 0, therefore:

$$0 = v^2 - 2gfd$$

$$d = \frac{v^2}{2gf} = \frac{v^2}{\left(\frac{3600}{1000}\right)^2 2(9.81)f} = \frac{v^2}{254f}$$

Where d is the braking distance (m)
v is the initial speed (km/h)
g is 9.81 m/s^2

B.2.4 Passing Sight Distance

Most rural highways are two-lane and two-way, so drivers must use the opposing traffic lane to pass slower vehicles. To pass safely, the driver should be able to see that the opposing traffic lane is clear for a sufficient distance ahead. A driver should have time to complete or terminate the manoeuvre without interfering with the smooth flow of traffic in either direction.

Many passes are accomplished without the driver seeing a safe passing section ahead, but design based on such manoeuvres does not have the desired safety factor. Because many cautious drivers would not attempt to pass under such conditions, design on this basis would reduce highway usefulness.

Passing sight distance for use in design should be determined on the basis of the length needed to complete a safe passing manoeuvre. While there may be occasions to consider multiple passes, where two or more vehicles pass or are passed, it is not practical to assume such conditions in developing minimum design criteria. Instead, sight distance is determined for a single vehicle passing a single vehicle. Longer sight distances occur naturally in design and these locations can accommodate an occasional multiple passing.

Certain assumptions for traffic behavior are necessary when computing minimum passing sight distances on two-lane highways for design use and some offer a wide choice. The assumed control for driver behavior should be that practised by a high percentage of drivers, rather than the average driver. Such assumptions follow:

1. The overtaken vehicle travels at uniform speed.

2. The passing vehicle has reduced speed and trails the overtaken vehicle as it enters a passing section.

3. When the passing section is reached, the driver requires a short period of time to perceive the clear passing section and start the manoeuvre.

4. Passing is accomplished under what may be termed a delayed start and a hurried return in the face of opposing traffic. The passing vehicle accelerates during the manoeuvre, and its average speed during occupancy of the left lane is 16 km/h higher than that of the overtaken vehicle.

5. When the passing vehicle returns to its lane, there is a suitable clearance length between it and oncoming vehicles.

Some drivers accelerate at the beginning of a passing manoeuvre to an appreciably higher speed and then continue at a uniform speed until passing is complete. Many drivers accelerate at a fairly high rate until just beyond the vehicle being passed and then complete the manoeuvre either without further acceleration or at reduced speed. For simplicity, extraordinary manoeuvres are ignored and passing distances are developed with the use of observed speeds and times that fit the practices of a high percentage of drivers.

The minimum passing sight distance for two-lane highways is determined as the sum of the four distances (shown in the diagram at the bottom of Figure B-2.4):

d_1 - Distance traversed during perception and reaction time and during the initial acceleration to the point of encroachment on the left lane

d_2 - Distance travelled while the passing vehicle occupies the left lane

d_3 - Distance between the passing vehicle at the end of its manoeuvre and the opposing vehicle

d_4 - Distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane, or 2/3 of d_2 above.

The minimum passing sight distances given in Table B.2.4 have been developed based on extensive field observations of driver behavior during passing manoeuvres. The average speed differential of 16 km/h between overtaken vehicles and passing vehicles is based on these observations.

Initial manoeuvre distance (d_1). The initial manoeuvre period has two components, a time for perception and reaction, and an interval during which the driver brings the vehicle from the trailing speed to the point of encroachment on the left or passing lane. To a great extent the two overlap. As a passing section of highway comes into view, a driver desiring to pass may begin to accelerate and manoeuvre the vehicle toward the centreline of the highway while deciding whether or not to pass. Studies show that the average passing vehicle accelerates at less than its maximum potential, indicating that the initial manoeuvre period contains an element of time for perception and reaction. However, some drivers may remain in normal lane position while deciding to pass. The exact position of the vehicle during initial manoeuvre is unimportant because differences in resulting passing distances are insignificant.

Distance while passing vehicle occupies left lane (d_2) In studies, passing vehicles have been found to occupy the left lane from 9.3 to 10.4 seconds. These studies involved extensive field observations of driver behavior during passing manoeuvres, documented by Prisk[3] during the years 1938 to 1941. The distance d_2 travelled in the left lane by the passing vehicle is computed by the following formula:

$$d_2 = t_2 \frac{v}{3.6}$$

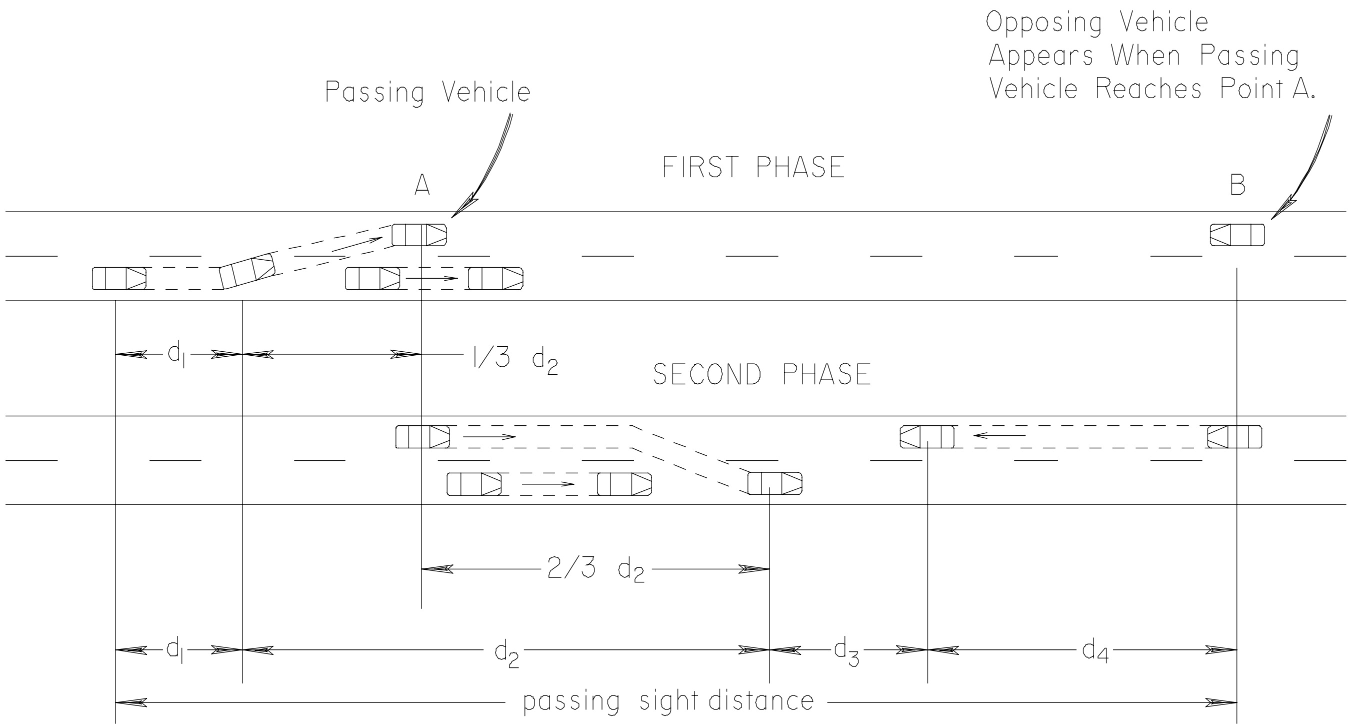
Where t_2 = time passing vehicle occupies the left lane (seconds)
v = average speed of passing vehicle (km/h)

Clearance length (d_3) The clearance length between the opposing and passing vehicles at the end of the manoeuvres found in the passing study varied from 33m to 92m. This length increases with increased design speed.

Distance traversed by an opposing vehicle (d_4) Passing sight distance includes the distance traversed by an opposing vehicle during the passing manoeuvre to minimize the chance of a passing vehicle meeting an opposing vehicle while in the left lane. Conservatively, this should be the distance traversed by an opposing vehicle during the time it takes to complete a pass, or the time that the passing vehicle occupies the left lane. Such a distance is questionably long. During the first phase of the passing manoeuvre, the passing vehicle has not yet pulled alongside the vehicle being passed. Even though the passing vehicle occupies the left lane, its driver can return to the right lane if an opposing vehicle is seen. It is unnecessary to include this trailing time interval in computing the distance traversed by an opposing vehicle. This time interval, which can be computed from the relative positions of passing and passed vehicle, is about one-third the time the passing vehicle occupies the left lane. Therefore, the passing sight distance element for the opposing vehicle is the distance it traverses during two-thirds of the time the passenger vehicle occupies the left lane. The opposing vehicle is assumed to be travelling at the same speed as the passing vehicle, so $d_4 = 2d_2/3$.

[3] Prisk, C.W., “Passing Practices on Rural Highways”, HCB Vol. 21, Washington, D.C., Highway Research Board, 1941.

Figure B-2.4 Elements of and Total Passing Sight Distance - Two-Lane Highways (Source: A Policy on Geometric Design of Highways & Streets, AASHTO 1994)



The design values for minimum passing sight distance for each design speed are listed in Table B.2.4.

Table B.2.4
Minimum Passing Sight Distance

Design Speed (km/h)	Minimum Passing Sight Distance (m)
40	275
50	340
60	420
70	480
80	560
90	620
100	680
110	740
120	800
130	860

Note: Passing Sight Distance is based on height of driver's eye at 1.05m and height of opposing vehicle at 1.30m.

These maximum passing sight distances for design should not be confused with other distances used as warrants for placing no-passing zone pavement markings on paved highways. Such values, described in Section B.2.5, are substantially less than design distances and are derived for traffic operating-control needs that are based on different assumptions from those for highway design.

Minimum vertical curvatures which satisfy passing sight distance criteria are given in Section B.4.4.

B.2.5 Non-Striping Sight Distance

Non-striping sight distance is the limiting value used to determine when no-passing pavement markings (barrier lines) are required. Although passing sight distance is a desirable condition on two-lane highways, non-striping sight distance is still adequate for safe passing manoeuvres.

The non-striping sight distance for each design speed is substantially less than the passing sight distance. The principal reason for the difference is that many drivers consider roadways marked according to passing sight distance requirements to be too restrictive. A safe passing manoeuvre can often be executed where full passing sight distance is not available depending on the timing of oncoming vehicles.

To explain the difference between passing sight distance and non-striping sight distance, reference is made to the passing model described in Section B.2.4 for passing sight distance. In the case of passing sight distance, if an oncoming vehicle comes into view at the critical moment, that is at the end of the first phase, the driver has sufficient time to complete the pass safely. In the case of non-striping sight distance, if an oncoming vehicle appears at the critical moment, there is only sufficient time available to safely abort the pass.

The non-striping sight distance value chosen for design in Alberta is based on the Uniform Traffic Control Devices for Canada (UTCDC) manual prepared by TAC, and the current Alberta standards for pavement marking. Both Alberta and TAC use an eye height and object height of 1.15m in their model.

Alberta currently installs a barrier line where the sight distance is less than 425m. TAC suggests 475m for a speed of 110 km/h. Speed is defined as the higher of the 85th percentile speed and the posted speed limit. Alberta's daytime 85th percentile running speed for passenger cars on two lane highways is currently approximately 110 km/h, and has been rising slightly over the last few years. Because of the high running speeds, it is best to use a non-striping sight distance that allows for a 110 km/h speed for design purposes.

Efforts should be made to achieve at least minimum non-striping sight distance on the flatter crest curves to maximize passing opportunities and consequently improve the level of service. This is especially important on higher volume highways in rolling terrain.

In general, designers should strive to achieve at least 75 percent of the length of a highway as barrier-free. Higher percentages are desirable on higher volume roads. Information obtained from videolog shows that approximately 73 percent of Alberta's paved two-lane highways are presently barrier free. Bear in mind that barrier lines are used at intersections and climbing/passing lanes even where there is no sight-line restriction.

For design purposes, a sight distance of 480m is used for non-striping to allow for a running speed of 110 km/h. This speed is commonly encountered on two-lane undivided rural highways in Alberta which are posted for 100 km/h. If 480m is used, rather than 475m, the resulting K values will be a convenient number (250) while offering an additional factor of safety. K values are explained in detail in Section B.4.4.

Distances immediately above the minimum non-striping sight distance values may cause a false feeling of safety for passing because of the absence of barrier lines. Also, frequent barrier lines are likely to appear unreasonable to the driver. It is, therefore, important that the sight distances only slightly greater than the non-striping values be increased as much as economically possible.

If it is not feasible to provide the non-striping values, it may be desirable to reduce the length of vertical curve to approach the stopping sight distance. This accomplishes three things: it shortens the total length of the no-passing zone, it may make the restrictive marking appear more reasonable to the driver, and it may provide a more economical design.

Minimum crest vertical curvatures that satisfy non-striping sight distance criteria are given in Section B.4.4.

B.2.6 Decision Sight Distance

Minimum stopping sight distance is usually sufficient to allow reasonably competent and alert drivers to come to a hurried stop under ordinary circumstances. However, this distance is often inadequate when drivers must make complex or instantaneous decisions, when information is difficult to perceive, or when unexpected or unusual manoeuvres are required. Limiting sight distance to that provided for stopping may also preclude drivers from performing evasive manoeuvres that are often less hazardous and otherwise preferable to stopping. Even with an appropriate complement of standard traffic control devices, stopping sight distance might not provide sufficient visibility distance for drivers to corroborate advance warnings and to perform the necessary manoeuvres. It is evident that there are many locations where it would be prudent to provide longer sight distance. In these circumstances, the use of decision sight distance instead of minimum stopping sight distance provides the greater length that drivers need.

Decision sight distance is the distance required for a driver to:

- detect an information source or hazard which is difficult to perceive in a roadway environment that might be visually cluttered
- recognize the hazard or its threat potential
- select appropriate action
- complete the manoeuvre safely and efficiently.

Because decision sight distance gives drivers additional margin for error and affords them sufficient length to manoeuvre their vehicles at the same or reduced speed, rather than simply to stop, it is substantially greater than minimum stopping sight distance.

Drivers need decision sight distance whenever there is likelihood for error in either information reception, decision making, or control actions. Examples of critical locations where these kinds of errors are likely to occur, and where it is desirable to provide decision sight distance are:

- interchanges and intersections
- locations where unusual or unexpected manoeuvres are required
- changes in cross section such as at rest areas and lane drops
- areas of concentrated demand where sources of information compete; for example, from roadway elements, traffic, traffic control devices, and advertising signs.

The decision sight distances in Table B.2.6 are used for appropriate sight distance at critical locations and serve as criteria in evaluating suitability of the sight distance lengths at these locations. Because of the additional safety and manoeuvrability these lengths yield, decision sight distances instead of minimum stopping sight distances are provided at critical locations. If it is not feasible to provide these distances because of horizontal or vertical curvature, special attention should be given to the use of suitable traffic control devices for providing advance warning of conditions likely to be encountered.

A range of decision sight distance values applicable to most situations has been developed. The range recognizes the variation in complexity that occurs at various sites. For less complex situations, values toward the lower end of the range are appropriate and for more complexity, values at the upper end are used. The calculations for decision sight distance are given in Table B.2.6.

Decision sight distance should be considered for crests near major intersections and for ramp exits. Each major intersection or ramp exit should be checked on a site specific basis, and analyzed individually to determine if decision sight distance is achieved. Other sight distance requirements must also be met.

For measuring decision sight distance, the height of eye of 1.05m is used together with an appropriate height of object depending on the anticipated prevailing conditions. In some circumstances, the driver needs to see the road surface, in which case the object height is zero.

Because of the variation in height of eye, minimum crest vertical curves which satisfy decision sight distance requirements are not given.

Table B.2.6 Calculation of Decision Sight Distance for Design

Design Speed (km/h)	Time				Decision Sight Distance	
	Pre-manoeuvre Time		Manoeuvre (lane change) (s)	Total (s)	Calculated (m)	Rounded for Design (m)
	Detection and Recognition (s)	Decision Response and Initiation (s)				
40	1.5-3.0	4.2-6.5	4.5	10.2-14.0	113-155	110-160
50	1.5-3.0	4.2-6.5	4.5	10.2-14.0	141-194	140-190
60	1.5-3.0	4.2-6.5	4.5	10.2-14.0	170-233	170-230
70	1.5-3.0	4.2-6.5	4.5	10.2-14.0	198-272	200-270
80	1.5-3.0	4.2-6.5	4.5	10.2-14.0	226-311	230-310
90	2.0-3.0	4.7-7.0	4.5	11.2-14.5	280-362	280-360
100	2.0-3.0	4.7-7.0	4.0	10.7-14.0	297-389	300-390
110	2.0-3.0	4.7-7.0	4.0	10.7-14.0	327-427	330-430
120	2.0-3.0	4.7-7.0	4.0	10.7-14.0	357-466	360-470
130	2.0-3.0	4.7-7.0	4.0	10.7-14.0	386-505	390-500

B.3 HORIZONTAL ALIGNMENT

B.3.1 Introduction

The horizontal alignment of a road is usually a series of tangents and transition spirals. Where a transition curve is omitted, the tangents connect with a circular curve. Curvilinear alignment is horizontal alignment in which long flat curves are connected by long transitions, generally without connecting tangents.

B.3.2 General Controls

The following are general controls and considerations for horizontal alignment. These controls are not subject to empirical or formula derivation, but they are important for attainment of safe, smooth-flowing and aesthetically pleasing highways.

Note: Specific controls for horizontal alignment are discussed in the following sections.

1. Alignment should be as directional as possible and consistent with topography. Effort should be made to preserve developed properties and community values. The use of winding alignment, composed of short curves, should be avoided since it tends to cause erratic operation and accidents. While the aesthetic qualities of curvilinear alignment are important, passing sight distance requirements on two-lane highways necessitate long tangents to provide passing opportunities.

2. The use of the minimum radius of curvature should be avoided, if possible, to establish an alignment based on the selected design speed. Use flat curvature generally, reserving the minimum radius for critical locations.

3. Consistent alignment should be provided. Sharp curves should not be introduced at the ends of long tangents or at other locations where high approach speeds are anticipated. Where physical restrictions dictate curvature of a lower standard than the project design speed, the critical curve should be approached by successively sharper curves. In this way erratic operation and accidents can be minimized because the driver will not be surprised by a sudden need to slow down.

4. For small deflection angles, curves should be long enough to avoid the appearance of a kink. Curves should generally be long enough to provide an aesthetically pleasing alignment (refer to Figure B-3.2e**)**. A deflection angle of 30 minutes requires a curve; smaller deflections do not. For smaller deflection curves (between 30’ and 1^0), that occur on rural roads in open country, a minimum curve length of 350m should be used to maintain a pleasing appearance. For the purpose of determining curve length, where spiral curves are applied, 50 percent of the spiral length is regarded as part of the curve.

The longer the distance a curve is viewed from, the more kinky its appearance, and in these cases there is a greater need to lengthen the curve. Curves that do not require any superelevation, i.e. normal crown curves are very desirable for small deflections.

5. Sharp curves should not be introduced on steep hills. With the absence of physical objects above the roadways, a driver may have difficulty estimating the radius and may fail to adjust to the conditions.

6. A broken back curve consists of two curves in the same direction joined by a short tangent. This type of alignment appears unpleasant and is potentially hazardous to drivers who do not expect successive curves in the same direction. The use of spiral transition curves, which provide some degree of continuous superelevation, is preferable. The term broken back is usually applied when the length in metres of the connecting tangent is less than four times the design speed in kilometres per hour.

7. Long spirals should be used whenever possible rather than compounding circular curves. If it is necessary to compound circular curves without a spiral between them, the ratio of the longer radius to the shorter radius should not exceed 1.5.

8. Abrupt alignment reversal must be avoided. When reverse curves are too close it is difficult to superelevate them adequately, resulting in hazardous and erratic vehicle operation. Alignment reversal can be suitably designed by including back-to-back spirals of sufficient length for the applicable design speed with enough tangent length between the spiral curves to allow for tangent runout.

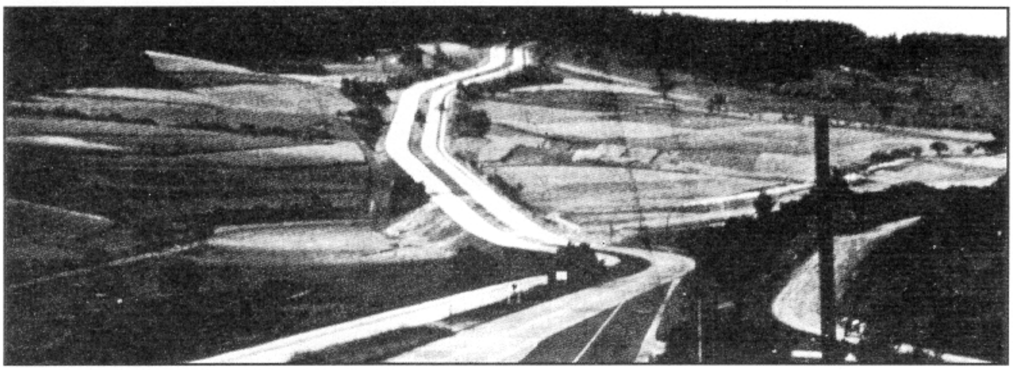
9. Where feasible, a curve beginning or ending near a bridge should be located so the superelevation transition does not occur on the structure.

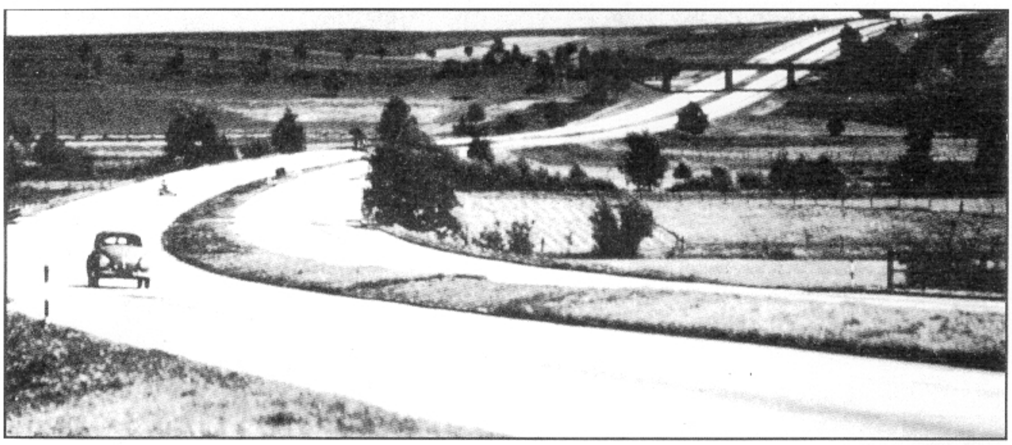
10. Horizontal alignment should be co-ordinated with vertical alignment to avoid the appearance of inconsistent distortion.

Figure B-3.2a through B-3.2l are illustrations of design form which show good and bad examples of the concepts.

Figure B-3.2a Illustration of Design Form

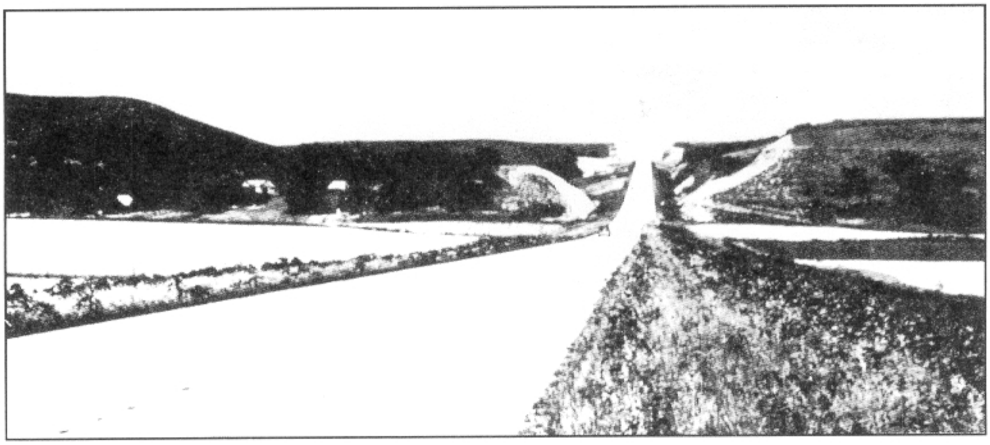
To more fully appreciate the value of good design, it is instructive to observe a few examples of discontinuous alignment where how it looks was considered unimportant. A continuous curve, beginning at the bottom of the picture and ending where the right lane disappears, would have been a much superior design.

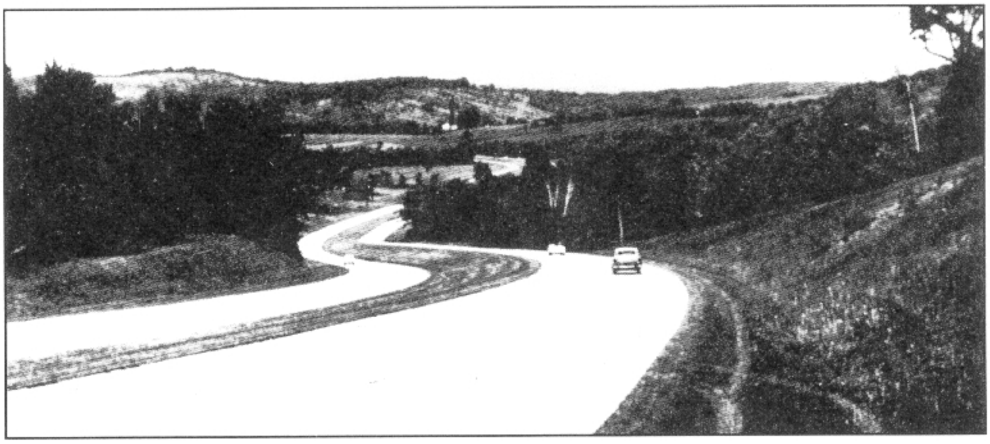




These contrasting photographs illustrate vividly the difference between long tangent-short curve design versus continuous curvilinear alignment. The top view gives one the impression the designer laid out each segment of highway on a separate sheet of plan paper without regard to the continuity of the entire roadway. The other highway (bottom) flows with the natural contours of the terrain with a minimum of sudden changes in alignment or grade.

Figure B-3.2b Illustration of Design Form

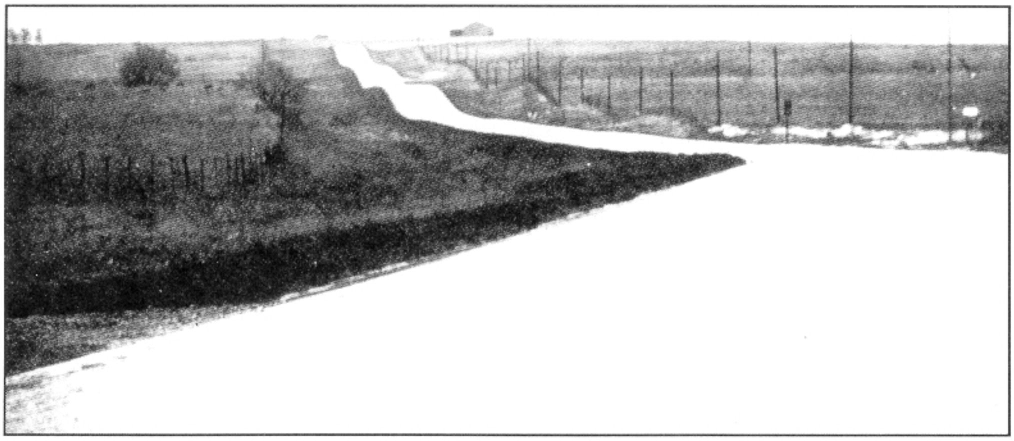




Alignment should be as directional as possible, but should be consistent with topography. A flowing line that generally conforms to the natural contours is aesthetically preferable to one with long tangents that slash through the terrain. The construction scars can be kept to a minimum and natural slopes and plant growth can be preserved.

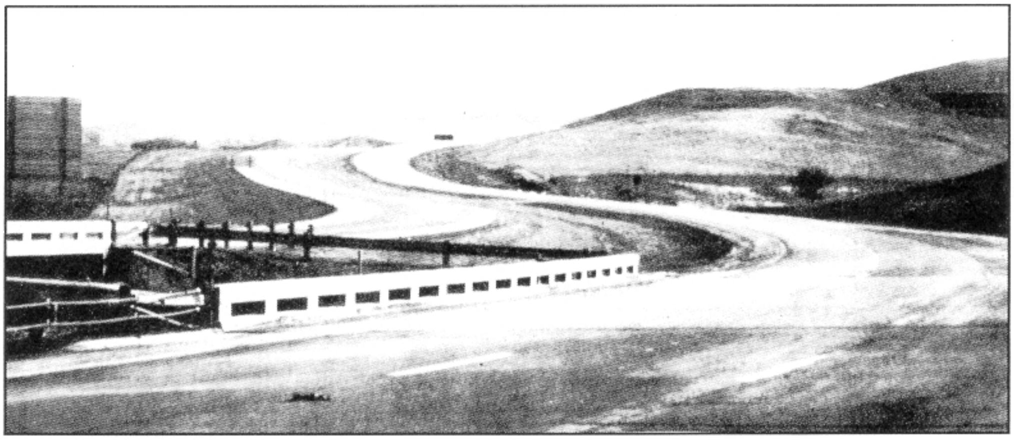
Because of straight alignment, one can often see a long distance ahead. When this happens, it is almost impossible to avoid a roller coaster appearance. Also, any median width changes are difficult to conceal. Observe the width change just above the grade separation structure.

Figure B-3.2c Illustration of Design Form



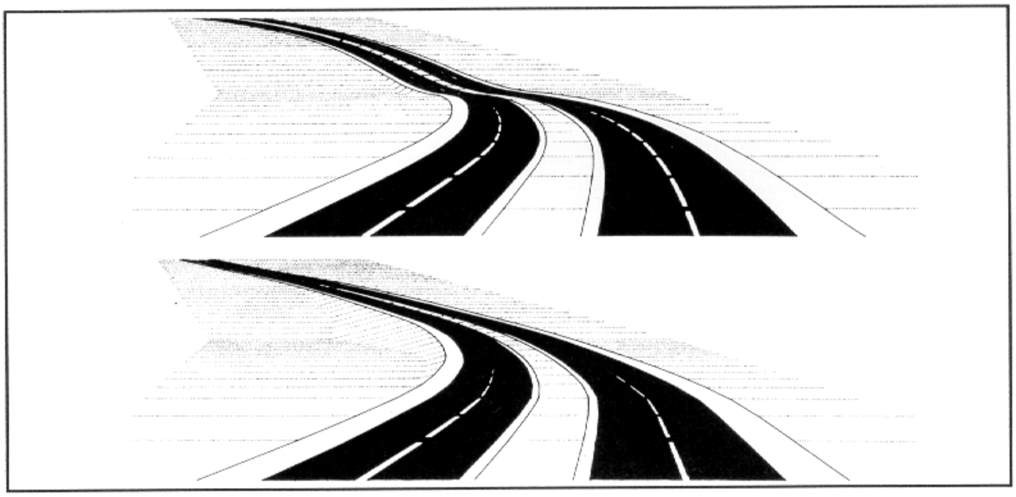
The roller-coaster or the hidden-dip type profile should be avoided. In general, such profiles occur on relatively straight, horizontal alignment where the roadway profile closely follows a rolling natural ground line. They are unpleasant aesthetically and hazardous.

The vertical alignment, which attempts to match the rather minor humps and hollows, is not in scale with the more liberal horizontal alignment.

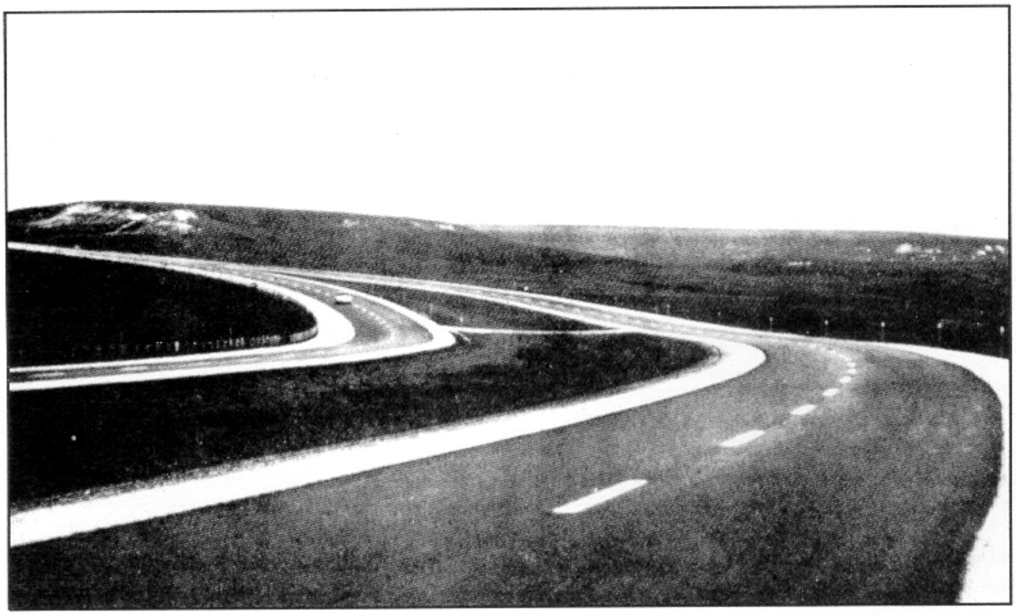


This example of curvilinear alignment enables the driver to scan the surrounding landscape without turning his head for a better view.

Figure B-3.2d Illustration of Design Form

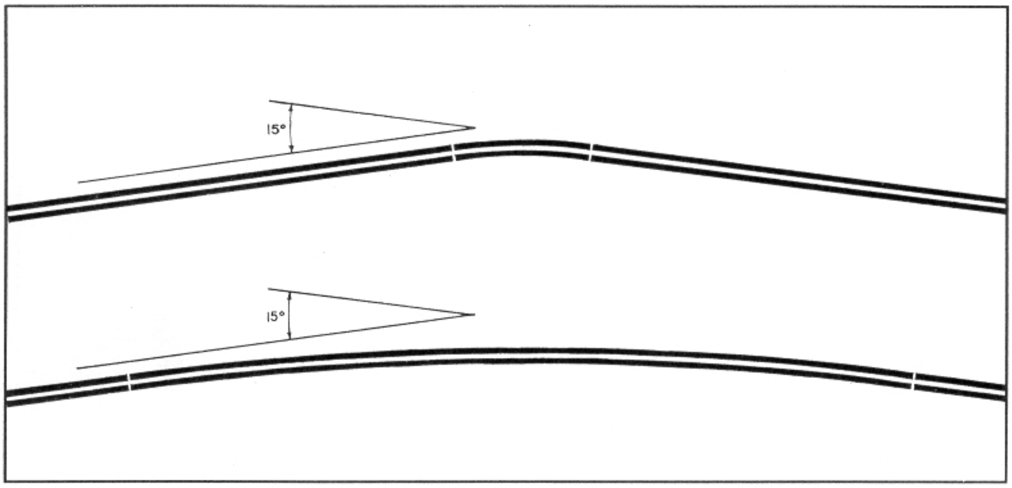


This drawing illustrates the effect of superimposing a short vertical curve on a relatively long horizontal curve. To eleminate the appearance of a settlement of the roadway, it is necessary to increase the length of vertical curve to that nearly of the horizontal curve.

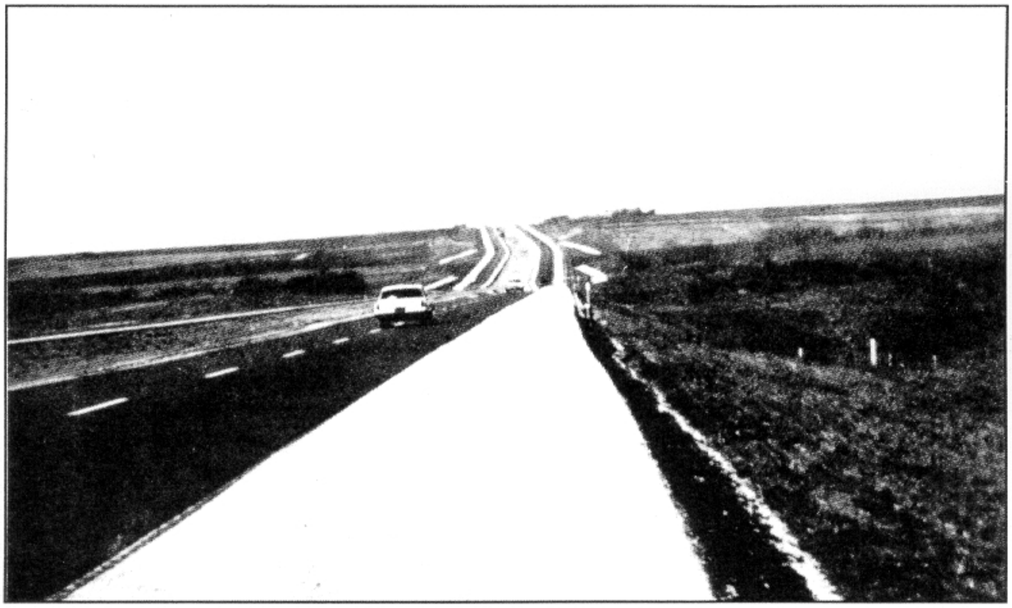


The sagging effect is clearly evident in this picture.

Figure B-3.2e Illustration of Design Form

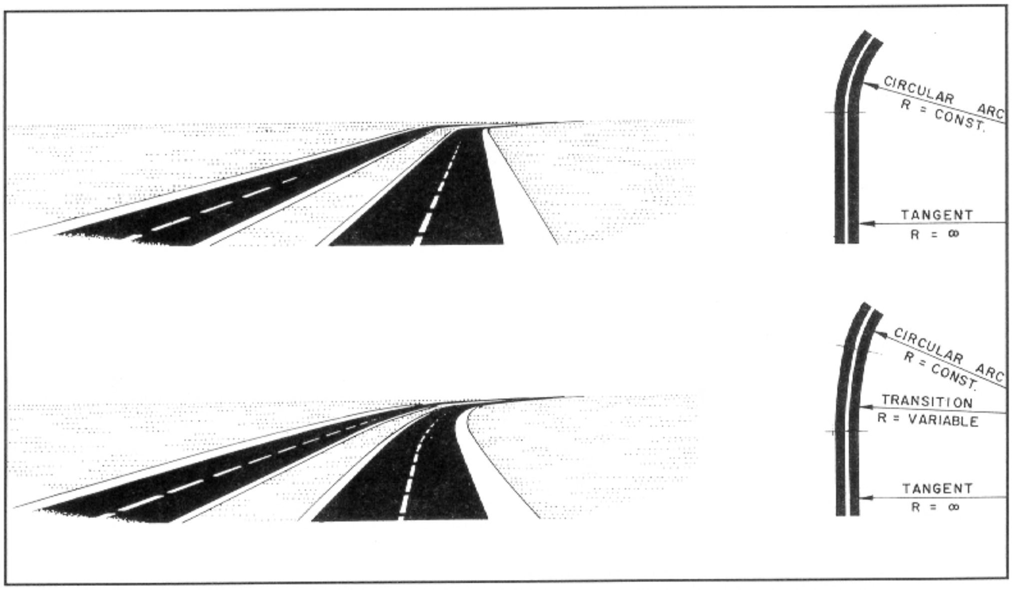


For small deflection angles, curves should be sufficiently long to avoid the appearance of a kink.

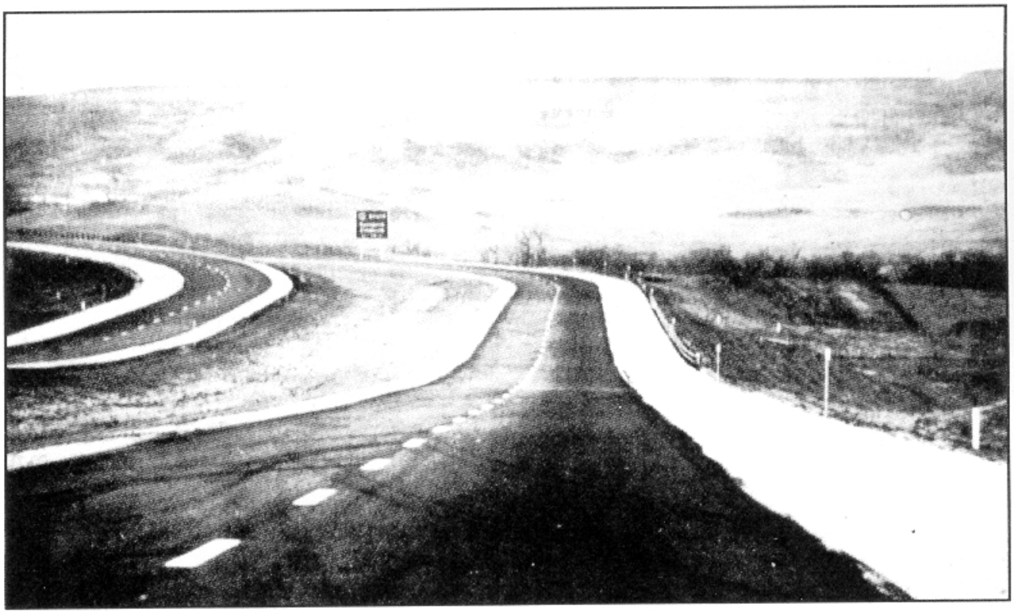


This view gives the feeling that the designer changed his mind rather suddenly and did not plan very far ahead. To avoid this, the length of curve should be proportional to the maximum distance from which one views the curve.

Figure B-3.2f Illustration of Design Form

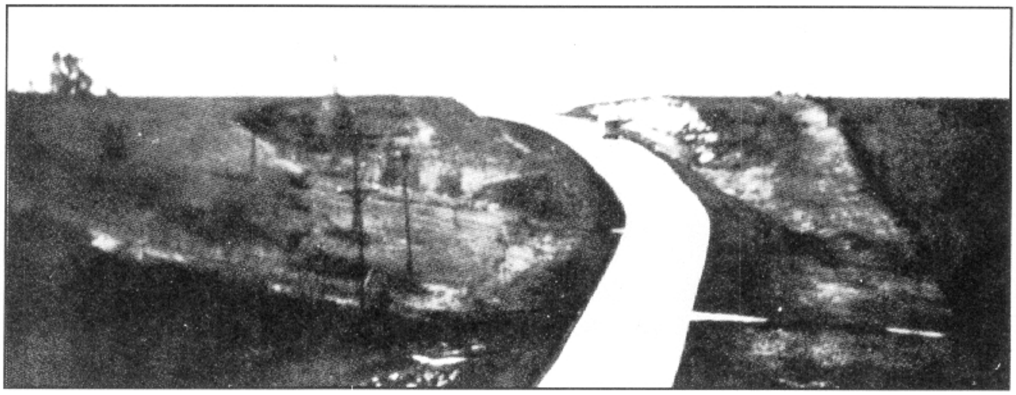


One effect of perspective viewing is that distant objects seem nearer than they really are. The circular curve consequently appears to diverge from the tangent rather rapidly and the curve no longer seems continuous. This gives the impression the designer was unable to make the curve meet the tangent properly. To remedy this situation, the use of long spirals is suggested and is illustrated in sketch B, above.

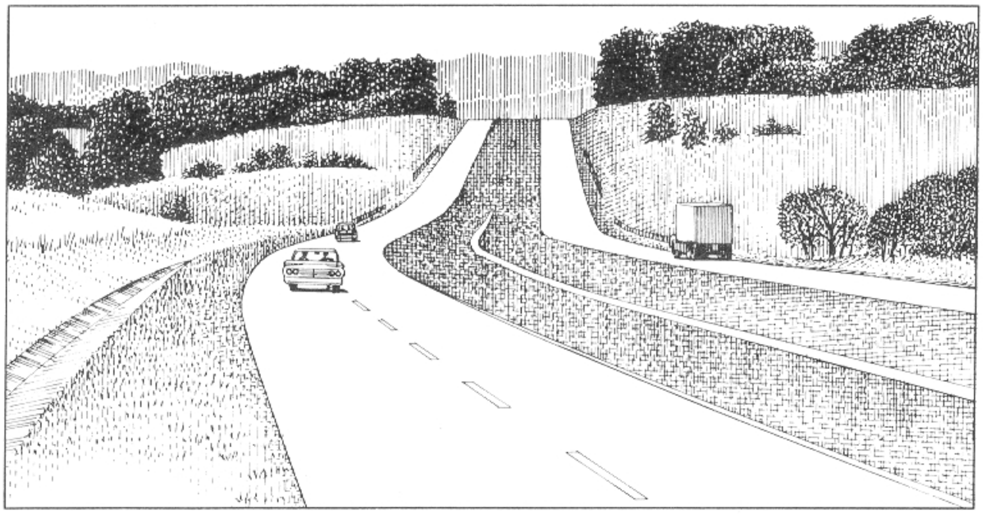


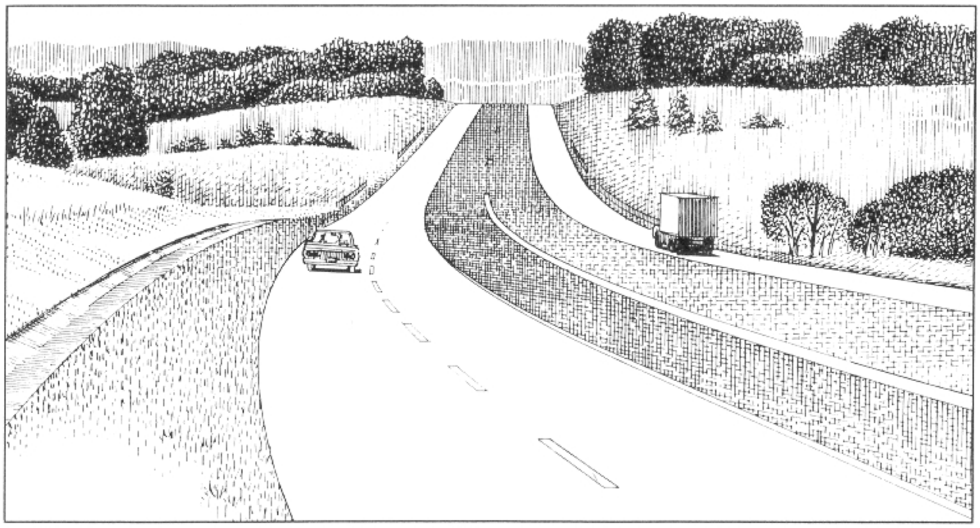
The horizontal curve does not appear to be tangent to the straight alignment. In fact, it visually jerks away from the tangent alignment. The left-hand roadway does, however, give the driver a good clue that the road continues to the left and does not merely fade away.

Figure B-3.2g Illustration of Design Form



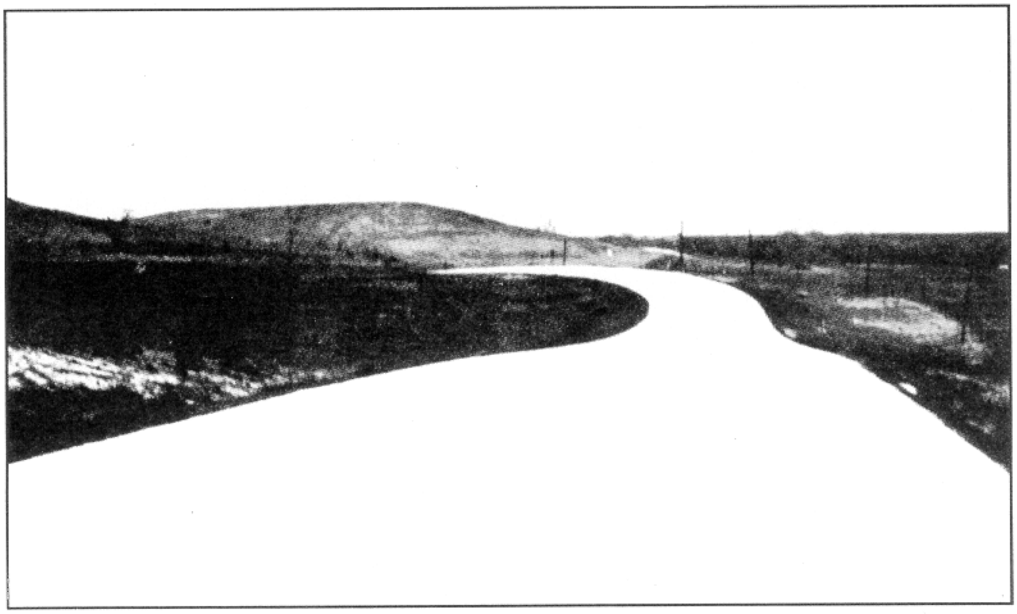
A long spiral beginning at the first entrance at the bottom of the hill and ending near the position of the truck would have improved the appearance of this curve.



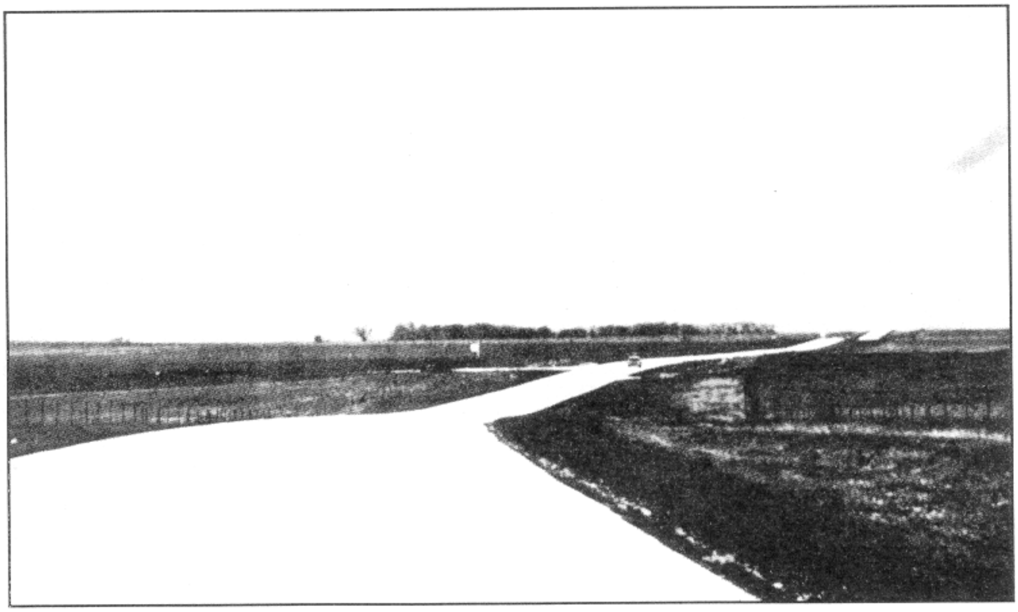


Short vertical curvature at the end of a long horizontal curve will usually produce a warped appearance. This situation can be improved by using a longer vertical curve than otherwise would be needed.

Figure B-3.2h Illustration of Design Form

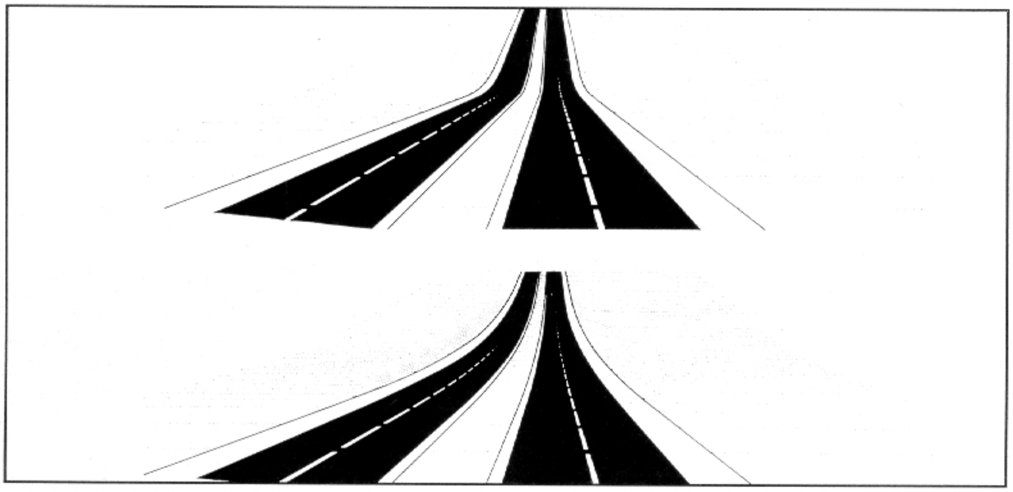


A short vertical curve at the beginning of a horizontal curve. Again, this is not a well-balanced design.

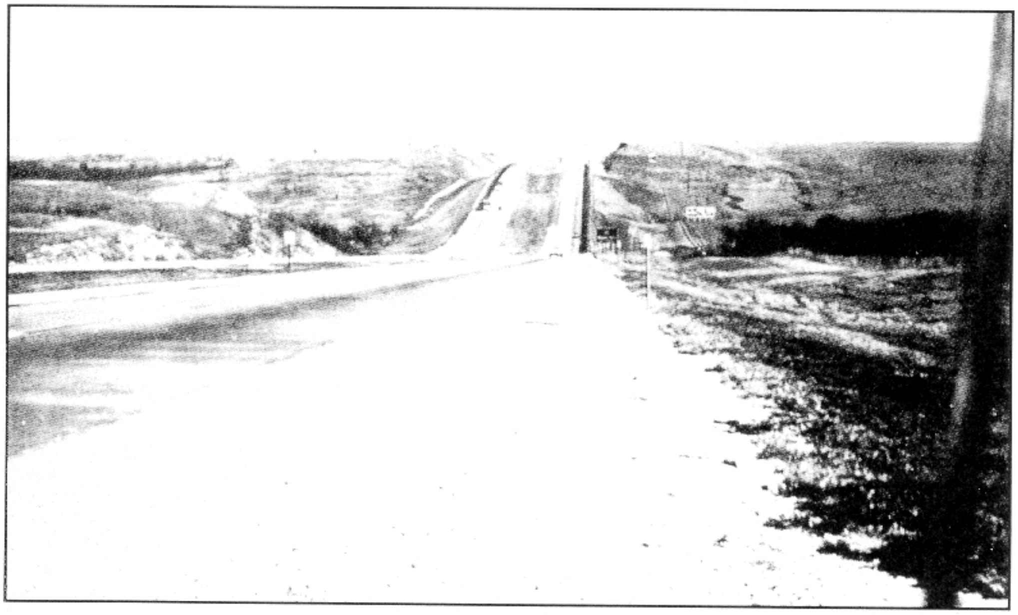


Almost in trouble, visually.

Figure B-3.2i Illustration of Design Form

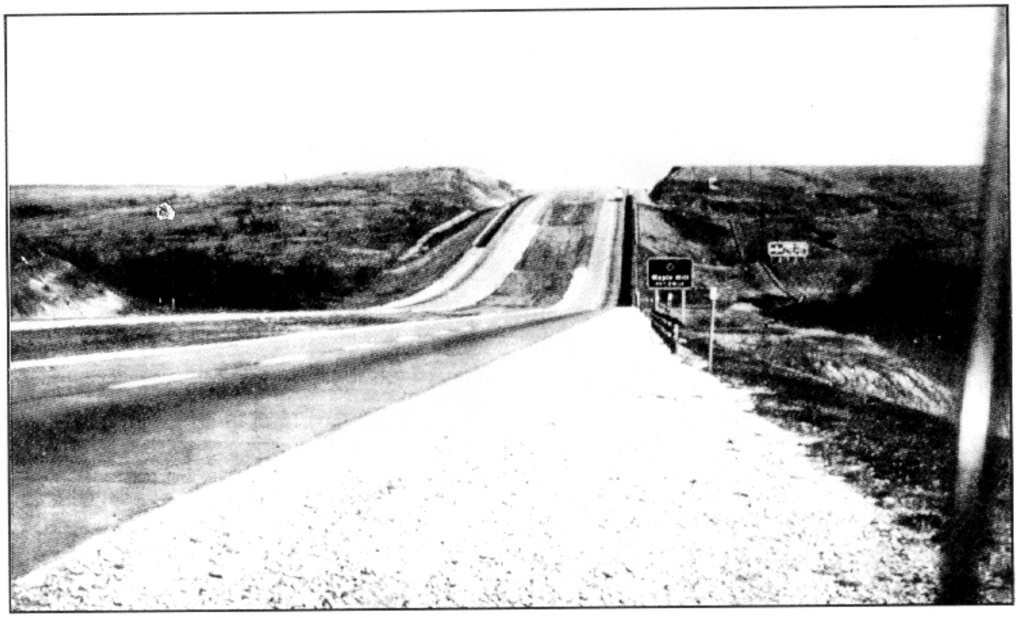


When the relatively short vertical curve in the upper picture is viewed from some distance, the transition from downgrade to upgrade appears rather abrupt. The alternatives to this design are longer curves and/or curvilinear alignment to shorten the long look ahead.

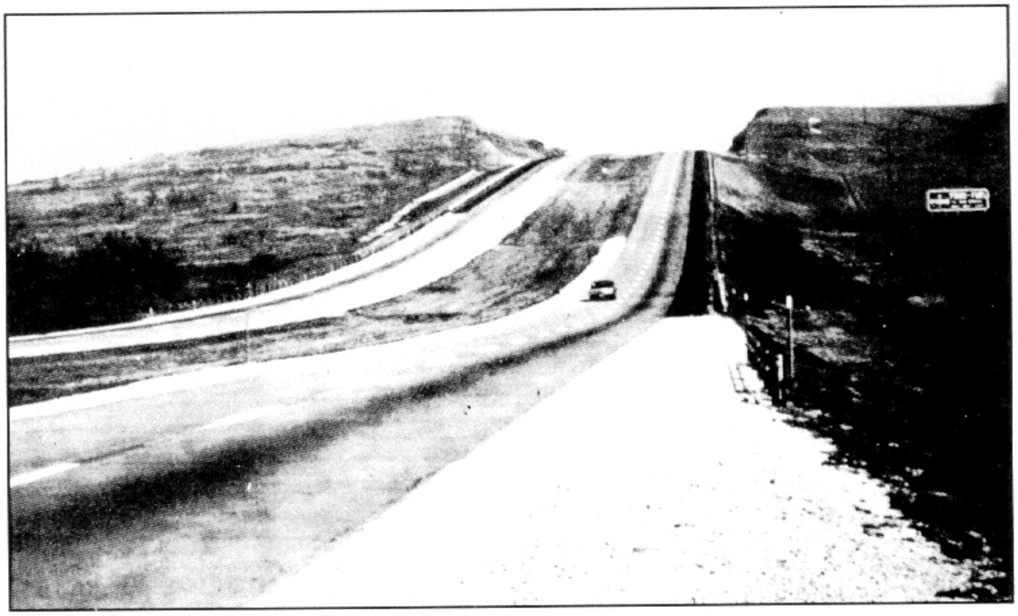


The curve at the bottom of the hill is too short when viewed from this distance.

Figure B-3.2j Illustration of Design Form



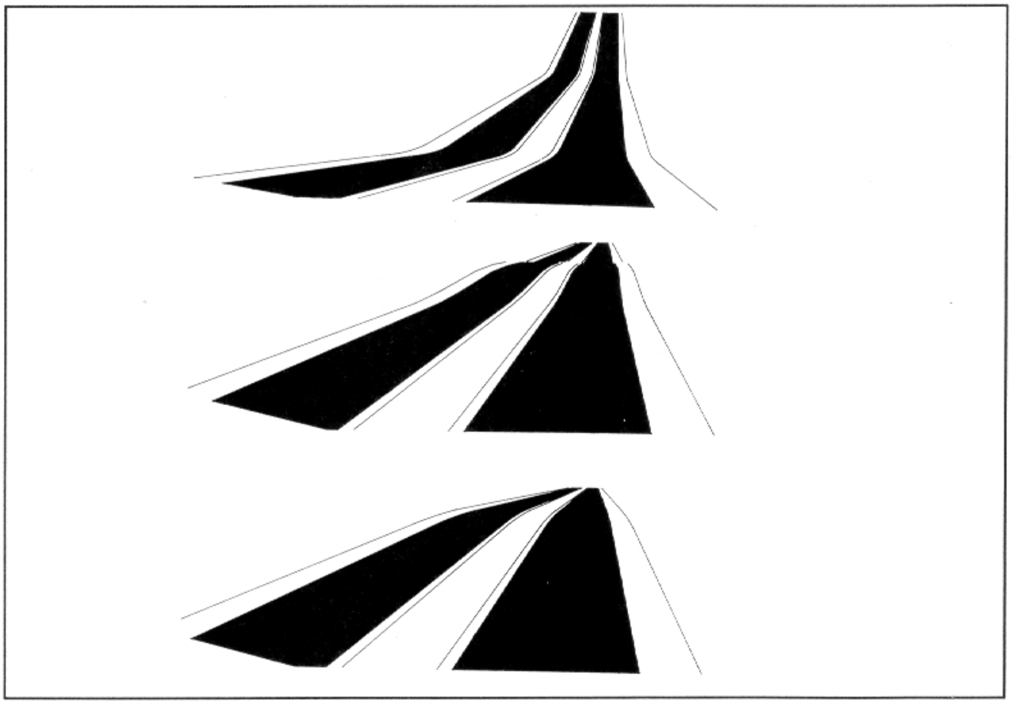
From an intermediate point, the curve is only a little too short.



From this position, the length of curve is about right.

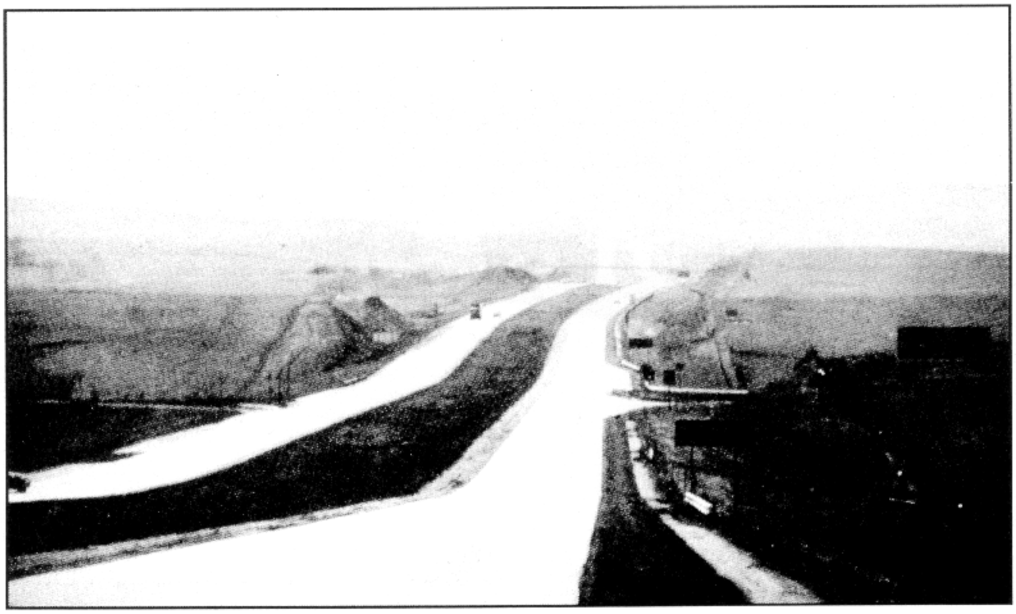
Figure B-3.2k Illustration of Design Form

Because the vertical curve is too short, the left edge of the pavement forms a V at the bottom of the hill.



The broken-back arrangement of curves (flat-back, or short tangent between two curves in the same direction) should be avoided.

Figure B-3.2I Illustration of Design Form



The broken-back vertical alignment with small grade changes. This type of design destroys the flowing continuity of a high-speed highway. Note the jerk in horizontal alignment.

B.3.3 Maximum Safe Side Friction Factors

The friction co-efficient, at which side skidding is imminent, depends upon a number of factors including: vehicle speed, type and condition of roadway surface, and type and condition of tires.

On any curve, it can be expected that some drivers will travel in excess of the design speed. In making lane changes or passing manoeuvres, a path of smaller radius than the control line is possible. Recognizing this, a safety factor has been incorporated into side friction factors. From available data and experience, the American Association of State Highway and Transportation Officials has established maximum safe side friction factor values for use in highway curvature design. These values are primarily based on an empirical relationship between side friction factor and design speed. The maximum safe values for each design speed are shown in the following table.

Table B.3.3
Maximum Safe Side Friction Factors
(for rural and high speed urban design)

Design Speed (km/h)	Safe Side Friction Factors
40	0.17
50	0.16
60	0.15
70	0.15
80	0.14
90	0.13
100	0.12
110	0.10
120	0.09
130	0.08

B.3.4 Maximum Superelevation

The maximum rates of superelevation usable for highway design are controlled by several factors:

1. Climatic conditions
   - Frequency and amount of snow and icing
2. Terrain conditions
   - Flat versus mountainous
3. Type of area
   - Rural or urban
4. Frequency of very slow moving vehicles that would be subject to uncertain conditions.

In Canada, provincial highway authorities typically chose either 0.06m/m or 0.08m/m as the maximum superelevation rate for rural highways. For the following reasons, there has been a recent trend towards adoption of 0.06 m/m as the maximum rate.

1. Adoption of the 0.06 m/m maximum table results in better horizontal alignments in cases where the minimum radii are used. Use of the minimum radii shown on the 0.08 m/m maximum table can result in sharp curves not consistent with driver expectations in a rural environment. Use of isolated sharp curves in a generally smooth high speed rural alignment is discouraged because it breaks one of the cardinal rules of highway design, that is, no surprises.

2. Use of the 0.06 m/m maximum table is expected to improve operational characteristics for vehicles traveling at lower speeds during adverse weather conditions, or for other reasons, while not adversely affecting higher speed vehicles. This is especially important for highways located where winter conditions prevail several months of the year.

A maximum superelevation rate of 0.06 m/m is recommended for all rural roads.

In an urban environment, superelevation is generally not applied on local streets, and is used only occasionally on collector streets. Topographic considerations may suggest the use of superelevation on collector streets, and to a lesser extent on local streets, to provide a better elevation match between street facilities and adjacent developments. Maximum superelevation rates in these cases are in the range of 0.02m/m to 0.04m/m. For further information on urban design refer to the most recent TAC publication on this subject.

B.3.5 Minimum Radius

The minimum allowable radius is a limiting value for a given design speed determined from the maximum rate of superelevation and the maximum side friction factor. For that design speed, use of sharper curvature would call for superelevation beyond the limit considered practical or for operation with tire friction beyond the safe limit or some combination of both.

The formula for calculating the radius is:

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

Where R_{min} is the minimum radius of the circular curve (m)
V is the vehicle speed (km/h)
e_{max} is the maximum roadway superelevation (m/m)
f is the maximum side friction factor

Minimum radii for corresponding design speeds are shown in Tables B.3.6a and B.3.6b.

B.3.6 Rates of Superelevation for Design

B.3.6.1 Speed to be Used for Superelevation

For design purposes, the speed to be used for selecting the superelevation rate is based on the expected 85th percentile running speed on the completed facility. In Alberta, this is generally 10 km/h higher than the posted speed. Therefore in general where the posted speed on undivided highways is 100 km/h, the speed to be used for superelevation is 110 km/h. Similarly on divided highways where the posted speed is generally 110 km/h, the speed to be used for superelevation is 120 km/h. Notwithstanding the above, horizontal alignments on divided highways in Alberta are generally set out based on a design speed of 130 km/h to allow for a possible increase in speeds (posted and running) that may occur in the future.

Where the design speed is less than 110 km/h, it is generally appropriate to use the full design speed for setting the superelevation rate as the 85th percentile running speed may match or exceed the design speed.

B.3.6.2 Superelevation Rates

The recommended superelevation rates for various radii for each design speed are given in Tables B.3.6a and B.3.6b. These tables are based on the form of superelevation distribution and side friction described as Method 5 in the 1990 AASHTO publication entitled A Policy on Geometric Design of Highways and Streets. The values shown in the tables are also consistent with those currently recommended by TAC.

Design Superelevation Tables

Design superelevation rates can be read directly from Tables B.3.6a and B.3.6b. When superelevation is used, the minimum rate should not be less than the rate of crossfall of the normal crown rate per travel lane, that is, normally 0.02m/m for paved roads and 0.03m/m for gravel roads.

Table B.3.6a (e_{max} = 0.06 m/m), which is normally used on all rural highway curves, includes an inset table entitled Values for Superelevation on Horizontal Curves Containing Major Intersections. The table provides values for superelevation related to design speed and circular curve radii for curved alignments containing major intersections. An intersection is considered major if intersection treatment, that is flaring or channelization, is provided. In addition to providing superelevation rates, the inset table also indicates those curves on which intersections may be permitted. It is noted that intersections on curve are undesirable for safety reasons and should be avoided where possible.

However, intersections on curves may be permitted where the combination of design speed and radius (as shown on this special inset table), or the radius is greater than 4000m. Also, for design speeds from 40 km/h to 90 km/h, which are not covered by the inset table, intersections are generally permitted only if e is less than or equal to 0.038m/m.

DESIGN SPEED (km/h)

RADIUS (m)	40 e	40 A Min.	40 A Des.	50 e	50 A Min.	50 A Des.	60 e	60 A Min.	60 A Des.
10 000	NC			NC			NC		
9 000	NC			NC			NC		
8 000	NC			NC			NC		
7 000	NC			NC			NC		
6 000	NC			NC			NC		
5 000	NC			NC			NC		
4 500	NC			NC			NC		
4 000	NC			NC			NC		
3 500	NC			NC			NC		
3 000	NC			NC			NC		
2 500	NC			NC			NC		
2 200	NC			NC			NC		
2 000	NC			NC			NC		
1 800	NC			NC			RC	245	245
1 700	NC			NC			RC	240	240
1 600	NC			NC			RC	230	230
1 500	NC			NC			RC	225	225
1 400	NC			NC			RC	225	225
1 300	NC			RC	200	200	RC	225	225
1 250	NC			RC	195	195	RC	200	200
1 200	NC			RC	190	190	RC	200	200
1 150	NC			RC	185	185	RC	200	200
1 100	NC			RC	180	180	0.021	200	200
1 000	NC			RC	175	175	0.021	175	175
1 050	NC			RC	170	170	0.021	175	175
950	NC			RC	170	170	0.022	175	175
900	NC			RC	150	150	0.023	175	175
850	NC			RC	150	150	0.024	175	175
800	RC	140	140	RC	150	150	0.025	160	160
750	RC	135	135	0.021	150	150	0.026	160	160
700	RC	130	130	0.021	140	140	0.027	150	150
650	RC	125	125	0.022	140	140	0.029	150	150
600	RC	120	120	0.024	125	125	0.030	140	140
575	RC	120	120	0.025	125	125	0.031	140	140
550	RC	120	120	0.025	125	125	0.032	125	125
525	RC	100	100	0.026	120	120	0.033	125	125
500	RC	100	100	0.027	120	120	0.034	125	125
475	0.021	100	100	0.028	120	120	0.035	125	125
450	0.021	100	100	0.029	120	120	0.036	125	125
420	0.022	90	90	0.030	100	100	0.037	115	120
400	0.023	90	90	0.031	100	100	0.038	115	120
380	0.024	90	90	0.032	100	100	0.039	115	120
350	0.025	90	90	0.034	100	100	0.041	110	115
340	0.026	90	90	0.035	100	100	0.042	110	115
320	0.027	80	80	0.036	90	100	0.043	100	110
300	0.028	80	80	0.037	90	100	0.044	100	110
280	0.029	80	80	0.038	90	100	0.046	100	110
250	0.031	75	80	0.040	85	90	0.048	90	100
240	0.032	75	80	0.041	85	90	0.049	90	100
230	0.033	70	80	0.042	80	90	0.049	90	100
220	0.034	70	80	0.043	80	90	0.050	90	100
210	0.035	70	80	0.044	80	90	0.051	90	100
200	0.036	70	75	0.045	75	90	0.052	85	100
190	0.037	70	75	0.046	75	90	0.053	85	100
180	0.038	60	75	0.047	70	90	0.054	85	95
170	0.039	60	75	0.048	70	90	0.055	85	95
160	0.040	60	75	0.049	70	85	0.056	85	90
150	0.041	60	75	0.051	70	85	0.057	85	90
140	0.043	60	70	0.052	65	80	0.059	85	90
130	0.044	60	70	0.053	65	80	0.060	85	90
125	0.045	60	65	0.054	65	75	minimum R=130		
120	0.046	60	65	0.055	65	75			
115	0.047	60	65	0.056	65	75			
110	0.047	60	65	0.057	65	75			
105	0.048	50	65	0.057	65	70			
100	0.049	50	65	0.058	65	70			
95	0.050	50	65	0.059	65	70			
90	0.051	50	60	0.060	65	70			
85	0.052	50	60	minimum R=90					
80	0.054	50	60						
75	0.055	50	60						
70	0.056	50	60						
65	0.057	50	60						
60	0.059	50	60						
55	0.060	50	60						
	minimum R=55								

RADIUS (m)	70 e	70 A Min.	70 A Des.	80 e	80 A Min.	80 A Des.	90 e	90 A Min.	90 A Des.
10 000	NC			NC			NC		
9 000	NC			NC			NC		
8 000	NC			NC			NC		
7 000	NC			NC			NC		
6 000	NC			NC			NC		
5 000	NC			NC			NC		
4 500	NC			NC			NC		
4 000	NC			NC			RC	450	450
3 500	NC			NC			RC	420	420
3 000	NC			RC	375	375	RC	390	400
2 500	RC	315	315	RC	325	325	0.021	360	380
2 200	RC	295	295	RC	325	325	0.022	330	375
2 000	RC	285	285	RC	300	300	0.023	300	350
1 800	RC	270	270	0.021	300	300	0.025	300	350
1 700	RC	260	260	0.022	275	275	0.026	300	350
1 600	RC	255	255	0.023	275	275	0.027	270	275
1 500	RC	250	250	0.024	250	250	0.028	270	275
1 400	0.021	240	240	0.025	250	250	0.030	270	275
1 300	0.021	230	230	0.026	250	250	0.031	270	275
1 250	0.022	225	225	0.027	225	225	0.032	240	240
1 200	0.023	220	220	0.028	225	225	0.033	240	240
1 150	0.024	215	215	0.029	225	225	0.034	240	240
1 100	0.025	210	210	0.030	225	225	0.035	240	240
1 000	0.026	205	205	0.031	200	200	0.036	225	225
1 050	0.027	200	200	0.032	200	200	0.037	225	225
950	0.028	200	200	0.033	200	200	0.038	225	225
900	0.029	180	180	0.034	200	200	0.039	200	200
850	0.030	180	180	0.035	200	200	0.041	200	200
800	0.031	175	175	0.036	175	175	0.042	200	200
750	0.032	175	175	0.037	175	175	0.043	200	200
700	0.034	175	175	0.039	175	175	0.045	185	195
650	0.035	175	175	0.041	175	175	0.046	185	195
600	0.037	150	150	0.042	175	175	0.048	175	185
575	0.038	150	150	0.043	175	175	0.049	175	185
550	0.039	150	150	0.044	175	175	0.050	175	185
525	0.040	140	150	0.045	150	160	0.051	160	175
500	0.041	140	150	0.046	150	160	0.052	160	175
475	0.042	140	150	0.047	150	160	0.053	160	175
450	0.043	140	150	0.048	150	160	0.054	160	175
420	0.044	125	135	0.050	135	150	0.056	160	165
400	0.045	125	135	0.051	135	150	0.057	160	165
380	0.046	125	135	0.052	135	150	0.058	160	165
350	0.048	120	125	0.054	125	140	0.059	160	160
340	0.049	120	125	0.054	125	140	0.060	160	160
320	0.050	120	125	0.056	125	135	minimum R=340		
300	0.051	120	125	0.057	125	135			
280	0.053	120	125	0.058	125	135			
250	0.055	110	120	0.060	125	125			
240	0.056	110	120	minimum R=250					
230	0.056	110	110						
220	0.057	110	110						
210	0.058	110	110						
200	0.059	110	110						
190	0.060	110	110						
180	minimum R=190								

RADIUS (m)	100 e	100 A Min.	100 A Des.	110 e	110 A Min.	110 A Des.	120 e	120 A Min.	120 A Des.	130 e	130 A Min.	130 A Des.	RADIUS (m)
10 000	NC			NC			NC			NC			10 000
9 000	NC			NC			NC			NC			9 000
8 000	NC			NC			NC			RC	760	760	8 000
7 000	NC			NC			RC	685	685	RC	710	710	7 000
6 000	NC			RC	600	600	RC	635	635	RC	660	660	6 000
5 000	RC	530	530	RC	555	555	RC	580	580	RC	600	600	5 000
4 500	RC	490	490	RC	525	525	RC	550	550	RC	570	570	4 500
4 000	RC	475	475	RC	495	495	RC	515	515	0.022	540	540	4 000
3 500	RC	440	440	RC	460	460	0.022	485	485	0.024	500	500	3 500
3 000	RC	410	410	RC	430	430	0.024	450	450	0.028	465	465	3 000
2 500	0.022	380	380	0.023	390	390	0.029	410	410	0.032	425	425	2 500
2 200	0.025	350	350	0.027	370	370	0.032	385	385	0.036	400	400	2 200
2 000	0.026	335	335	0.029	350	350	0.034	365	365	0.041	380	380	2 000
1 800	0.029	310	310	0.033	330	330	0.038	350	350	0.045	350	360	1 800
1 700	0.030	310	310	0.034	325	325	0.039	340	340	0.046	350	360	1 700
1 600	0.031	290	290	0.036	310	310	0.041	325	325	0.048	340	350	1 600
1 500	0.033	290	290	0.037	305	305	0.042	315	315	0.049	330	335	1 500
1 400	0.035	280	280	0.039	290	290	0.044	305	305	0.051	330	335	1 400
1 300	0.036	260	260	0.041	290	290	0.046	300	300	0.052	315	335	1 300
1 250	0.037	260	260	0.042	280	280	0.048	285	290	0.053	300	330	1 250
1 200	0.038	260	260	0.043	270	270	0.049	285	290	0.055	295	320	1 200
1 150	0.039	250	250	0.044	270	270	0.050	275	285	0.056	295	320	1 150
1 100	0.040	250	250	0.045	260	260	0.052	270	285	0.057	295	320	1 100
1 000	0.041	235	235	0.047	260	260	0.053	260	280	0.058	280	300	1 050
1 050	0.042	235	235	0.048	245	255	0.054	260	280	0.059	280	300	1 000
950	0.043	235	235	0.049	245	255	0.055	260	280	0.060	280	300	950
900	0.044	225	225	0.051	235	250	0.057	250	270	minimum R=950			
850	0.046	225	225	0.053	235	250	0.058	250	270				
800	0.047	210	215	0.054	220	240	0.059	250	260				
750	0.048	210	215	0.056	220	240	0.060	250	250				
700	0.050	200	210	0.058	220	235	minimum R=750						
650	0.052	200	210	0.059	220	235							
600	0.054	190	200	0.060	220	220							
575	0.055	190	200	minimum R=600									
550	0.056	190	200										
525	0.057	190	190										
500	0.059	190	190										
475	0.059	190	190										
450	0.060	190	190										
420	minimum R=440												

TABLE B.3.6a

Recommended for new construction projects on Alberta's rural roads.

NOTES:

e is superelevation (m/m).

A is spiral parameter in meters.

NC is normal cross-section.

RC is remove adverse crown and superelevate at normal crown rate.

Spiral length, L=A²÷Radius.

For 3 and 4 lane pavements, use desirable "A" values.

For 6 lane pavements, use desirable "A" values x 1.15.

A divided roadway having a median width less than 3m may be treated as a single roadway.

e max. = 0.06

INSET TABLE
VALUES FOR SUPERELEVATION ON HORIZONTAL CURVES
CONTAINING MAJOR INTERSECTIONS

RADIUS (m)	100 e	110 e	120 e	130 e	RADIUS (m)
4 000	RC	RC	RC	RC	4 000
3 500	RC	RC	RC	RC	3 500
3 000	RC	RC	RC	RC	3 000
2 500	RC	RC	RC	0.022	2 500
2 200	RC	RC	0.022	0.025	2 200
2 000	RC	RC	0.024		
1 800	RC	0.023	0.029		
1 700	RC	0.024			
1 600	0.021	0.026			
1 500	0.021	0.029			
1 400	0.024	0.032			
1 300	0.026				
1 250	0.028				
1 200	0.030				
1 150	0.032				

NOTES:

This table provides values for superelevation related to design speeds and circular curve radii for curves containing major intersections. An intersection is considered major if intersection treatment, ie., flaring or channelisation, is provided.

Intersections on curve are undesirable for safety reasons and should be avoided where possible. However intersections on curves are permitted where the combination of design speed and radius is shown on this special table or if the radius is greater than 4000m. For design speeds from 40 km/h to 90 km/h intersections are only permitted if e is less than or equal to 0.038.

VALUES FOR SUPERELEVATION AND SPIRAL PARAMETERS RELATED TO DESIGN SPEEDS AND CIRCULAR CURVE RADII.
e max = 0.06 m/m

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DESIGN SPEED (km/h)

RADIUS (m)	40			50			60			70			80			90			100			110			120			130			RADIUS (m)
	e	A Min.	A Des.	e	A Min.	A Des.	e	A Min.	A Des.	e	A Min.	A Des.	e	A Min.	A Des.	e	A Min.	A Des.	e	A Min.	A Des.	e	A Min.	A Des.	e	A Min.	A Des.	e	A Min.	A Des.	
10 000	NC			NC			NC			NC			NC			NC			NC			NC			NC			NC			10 000
9 000	NC			NC			NC			NC			NC			NC			NC			NC			NC			NC			9 000
8 000	NC			NC			NC			NC			NC			NC			NC			NC			NC			RC	760	760	8 000
7 000	NC			NC			NC			NC			NC			NC			NC			NC			RC	685	685	RC	710	710	7 000
6 000	NC			NC			NC			NC			NC			NC			NC			RC	600	600	RC	635	635	RC	660	660	6 000
5 000	NC			NC			NC			NC			NC			NC			RC	530	530	RC	555	555	RC	580	580	0.021	600	600	5 000
4 500	NC			NC			NC			NC			NC			NC			RC	490	490	RC	525	525	RC	550	550	0.023	570	570	4 500
4 000	NC			NC			NC			NC			NC			RC	450	450	RC	480	480	RC	495	495	0.021	515	515	0.026	540	540	4 000
3 500	NC			NC			NC			NC			NC			RC	420	420	0.021	440	440	0.022	460	460	0.024	485	485	0.030	500	500	3 500
3 000	NC			NC			NC			NC			RC	375	375	RC	390	400	0.023	410	410	0.025	430	430	0.028	450	450	0.033	465	465	3 000
2 500	NC			NC			NC			RC	315	315	RC	325	325	0.021	360	380	0.027	380	380	0.030	390	390	0.034	410	410	0.040	425	425	2 500
2 200	NC			NC			NC			RC	295	295	0.021	325	325	0.023	330	375	0.028	350	350	0.032	370	370	0.037	385	385	0.044	400	400	2 200
2 000	NC			NC			NC			RC	285	285	0.021	300	300	0.026	300	350	0.030	335	335	0.035	350	350	0.040	365	365	0.047	380	380	2 000
1 800	NC			NC			RC	245	245	RC	270	270	0.023	300	300	0.028	300	350	0.032	310	310	0.038	330	330	0.044	350	350	0.051	360	375	1 800
1 700	NC			NC			RC	240	240	RC	260	260	0.024	275	275	0.029	300	350	0.034	310	310	0.039	325	325	0.046	340	340	0.053	350	375	1 700
1 600	NC			NC			RC	230	230	0.021	255	255	0.025	275	275	0.031	270	275	0.036	290	290	0.040	310	310	0.048	325	325	0.055	340	365	1 600
1 500	NC			NC			RC	225	225	0.021	250	250	0.027	250	250	0.032	270	275	0.037	290	290	0.042	305	305	0.050	315	330	0.058	330	365	1 500
1 400	NC			NC			RC	225	225	0.022	240	240	0.029	250	250	0.034	270	275	0.039	280	280	0.044	290	290	0.053	305	330	0.060	330	365	1 400
1 300	NC			RC	200	200	RC	225	225	0.024	230	230	0.030	250	250	0.036	270	275	0.041	260	260	0.047	290	290	0.055	300	330	0.063	315	365	1 300
1 250	NC			RC	195	195	RC	200	200	0.025	225	225	0.031	225	225	0.037	240	240	0.043	260	260	0.048	280	285	0.057	285	320	0.065	300	350	1 250
1 200	NC			RC	190	190	RC	200	200	0.026	220	220	0.032	225	225	0.038	240	240	0.044	260	260	0.050	270	285	0.059	285	320	0.067	295	350	1 200
1 150	NC			RC	185	185	RC	200	200	0.026	215	215	0.033	225	225	0.039	240	240	0.045	250	250	0.051	270	285	0.061	285	320	0.068	295	350	1 150
1 100	NC			RC	180	180	0.021	200	200	0.027	210	210	0.034	225	225	0.040	240	240	0.046	250	250	0.053	260	285	0.062	285	320	0.070	295	350	1 100
1 050	NC			RC	170	170	0.022	175	175	0.028	205	205	0.035	200	200	0.041	225	225	0.048	240	240	0.055	260	280	0.064	260	310	0.072	280	340	1 050
1 000	NC			RC	170	170	0.023	175	175	0.029	200	200	0.036	200	200	0.043	225	225	0.049	240	240	0.057	250	280	0.066	260	310	0.074	280	340	1 000
950	NC			RC	170	170	0.024	175	175	0.030	200	200	0.038	200	200	0.044	225	225	0.050	225	240	0.059	250	280	0.068	260	310	0.076	280	340	950
900	NC			RC	150	150	0.025	175	175	0.032	180	180	0.039	200	200	0.046	200	220	0.052	225	240	0.062	235	275	0.071	250	300	0.078	280	330	900
850	NC			RC	150	150	0.026	175	175	0.033	180	180	0.040	200	200	0.048	200	220	0.053	225	240	0.064	235	275	0.073	250	300	0.079	280	330	850
800	RC	140	140	RC	150	150	0.027	160	160	0.035	175	175	0.042	175	195	0.049	200	220	0.055	210	235	0.066	220	270	0.075	250	295	minimum R=830			
750	RC	135	135	0.021	150	150	0.028	160	160	0.036	175	175	0.044	175	195	0.051	200	220	0.058	210	235	0.068	220	270	0.077	250	295				
700	RC	130	130	0.023	140	140	0.030	150	150	0.038	165	165	0.046	175	190	0.053	185	200	0.061	200	230	0.072	220	260	0.079	250	285				
650	RC	125	125	0.024	140	140	0.032	150	150	0.040	165	165	0.048	175	190	0.055	185	200	0.064	200	230	0.074	220	260	minimum R=670						
600	RC	120	120	0.026	125	125	0.034	140	140	0.042	150	160	0.050	165	185	0.058	175	200	0.067	190	225	0.077	220	250							
575	RC	120	120	0.027	125	125	0.035	140	140	0.043	150	160	0.052	165	185	0.059	175	200	0.069	190	225	0.078	220	250							
550	0.021	120	120	0.028	125	125	0.036	140	140	0.045	150	160	0.053	165	185	0.061	175	200	0.070	190	225	0.079	220	250							
525	0.021	100	100	0.029	120	120	0.038	125	135	0.046	140	150	0.054	150	175	0.062	160	200	0.071	190	215	minimum R=530									
500	0.021	100	100	0.030	120	120	0.039	125	135	0.048	140	150	0.056	150	175	0.064	160	200	0.073	190	215										
475	0.022	100	100	0.031	120	120	0.040	125	135	0.050	140	150	0.058	150	175	0.066	160	200	0.074	190	215										
450	0.023	100	100	0.032	120	120	0.041	125	135	0.051	140	150	0.059	150	175	0.067	160	200	0.075	190	215										
420	0.024	90	90	0.034	100	110	0.043	115	125	0.053	125	150	0.061	135	165	0.069	160	185	0.077	190	200										
400	0.025	90	90	0.035	100	110	0.045	115	125	0.054	125	150	0.063	135	165	0.071	160	185	0.079	190	200										
380	0.026	90	90	0.036	100	110	0.046	115	125	0.056	125	150	0.064	135	165	0.072	160	185	minimum R=390												
350	0.028	90	90	0.038	100	105	0.049	110	125	0.058	120	150	0.067	125	160	0.075	160	175													
340	0.029	90	90	0.039	100	105	0.050	110	125	0.059	120	150	0.068	125	160	0.076	160	175													
320	0.030	80	90	0.040	90	100	0.051	100	120	0.061	120	140	0.070	125	150	0.078	160	175													
300	0.031	80	90	0.042	90	100	0.053	100	120	0.063	120	140	0.072	125	150	0.080	160	175													
280	0.032	80	90	0.044	90	100	0.055	100	120	0.065	120	140	0.074	125	150	minimum R=300															
250	0.035	75	85	0.047	85	100	0.059	100	120	0.069	110	135	0.078	125	150																
240	0.036	75	85	0.048	85	100	0.060	100	120	0.070	110	135	0.079	125	150																
	0.037	70	80	0.049	80	100	0.061	95	110	0.071	110	125	0.080	125	150																
220	0.039	70	80	0.051	80	100	0.062	95	110	0.073	110	125	minimum R=230																		
210	0.040	70	80	0.052	80	100	0.063	95	110	0.074	110	125																			
200	0.041	70	80	0.054	80	100	0.065	90	110	0.075	110	125																			
190	0.042	70	80	0.056	80	100	0.066	90	110	0.076	110	125																			
180	0.044	65	80	0.057	75	95	0.068	90	105	0.078	110	120																			
170	0.046	65	80	0.058	75	95	0.070	90	105	0.080	110	120																			
160	0.047	65	80	0.060	75	90	0.072	85	100	minimum R=170																					
150	0.049	65	80	0.062	75	90	0.074	85	100																						
140	0.051	65	75	0.064	70	90	0.076	85	100																						
130	0.053	65	75	0.066	70	90	0.078	85	100																						
125	0.054	60	75	0.068	70	85	0.079	85	95																						
120	0.055	60	75	0.069	70	85	0.080	85	95																						
115	0.056	60	75	0.070	70	85	minimum R=120																								
110	0.058	60	75	0.071	70	85																									
105	0.059	55	70	0.072	65	80																									
100	0.061	55	70	0.074	65	80																									
95	0.063	55	70	0.075	65	80																									
90	0.064	55	70	0.077	65	80																									
85	0.065	55	70	0.078	65	80																									
80	0.067	55	65	0.080	65	75																									
75	0.069	55	65	minimum R=80																											
70	0.071	50	60																												
65	0.074	50	60																												
60	0.077	50	60																												
55	0.078	50	60																												
50	0.080	50	60																												
	minimum R=50																														

TABLE B.3.6b

This table is not recommended for new construction projects on Alberta's rural roads. Table B.4.6a (e max=0.06) should be used.

e max. = 0.08

NOTES:

e is superelevation (m/m).

A is spiral parameter in meters.

NC is normal cross-section.

RC is remove adverse crown and superelevate at normal crown rate.

Spiral length, $L = A^2 \div \text{Radius}$.

For 3 and 4 lane pavements, use desirable "A" values.

For 6 lane pavements, use desirable "A" values x 1.15.

A divided roadway having a median width less than 3m may be treated as a single roadway.

VALUES FOR SUPERELEVATION AND SPIRAL PARAMETERS RELATED TO DESIGN SPEEDS AND CIRCULAR CURVE RADII. e max = 0.08 m/m

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The following paragraphs describe in more detail the various methods of distribution of e and f over a range of curves that are available. Although Method 5 is currently used by AI, this does not imply that other methods are not acceptable or that there is no flexibility. The form of distribution of e and f is currently under review by AI.

Distribution of e and f Over a Range of Curves

For a given design speed, there are five methods for counteracting centrifugal force on curves by use of e and f, or both. These methods follow, and their resulting relation is illustrated in Figure B-3.6.

1. Superelevation and side friction are directly proportional to the degree of curve (D), that is, a straight-line relation exists between D = O and D = D_{max}.

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

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

4. Method 4 is the same as Method 3, except that it is based on average running speed instead of design speed.

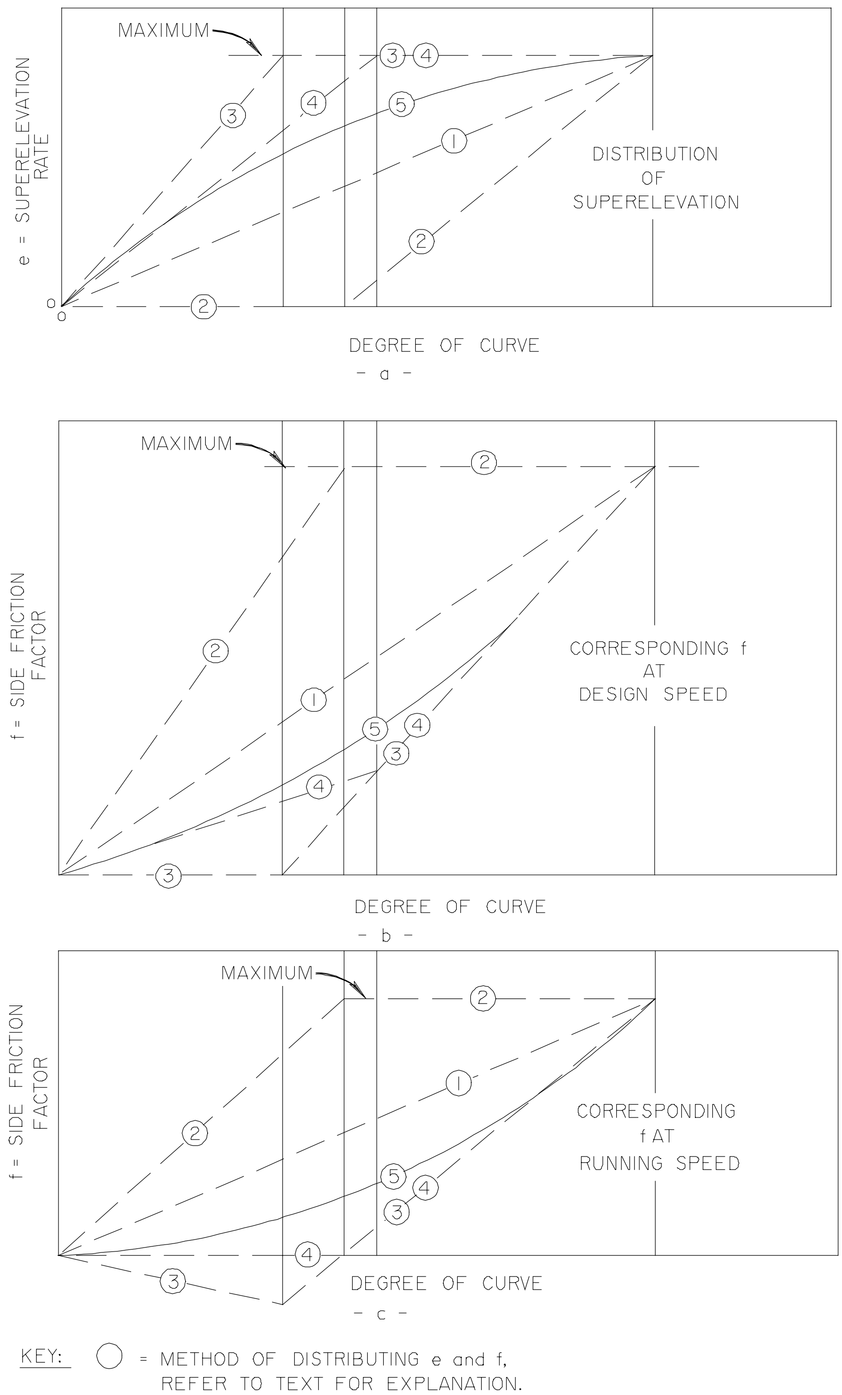
5. Superelevation and side friction are in a curvilinear relation with degrees of curve, with values between those of methods 1 and 3.

Table B.3.6a shows the comparative relations of superelevation versus degree of curve for these methods. Table B.3.6b shows the corresponding value of side friction for a vehicle travelling at design speed. Figure B-3.6 shows the value of side friction for a vehicle travelling at the corresponding average running speed.

To favor the overdriving characteristics that occur on flat to intermediate curves, it is desirable that the superelevation approximate that obtained by Method 4. Overdriving on such curves is not dangerous because superelevation counteracts nearly all centrifugal force at average running speed, and considerable side friction is available for greater speed. On the other hand, it also is desirable to favour Method 1, which avoids a substantial part of the range of curves with maximum superelevation. Using Method 5, a curved line (curve 5, as shown within the triangular working range between curves 1 and 4 in Figure B-3.6) represents a superelevation and side friction distribution reasonably satisfying both aspects. Curve 5, of parabolic form, represents a practical distribution over the range of curvature.

There is some flexibility when applying superelevation in urban areas due to the willingness of drivers to accept higher friction values than on open highways and due to other controls that frequently apply. For current practice for application of superelevation to urban roadways, refer to the most recent TAC publication dealing with this subject.

Figure B-3.6 Methods of Distributing Superelevation and Side Friction (Source: AASHTO 1994)



B.3.7 Development of Superelevation

To superelevate two-lane highways the pavement should generally be rotated about its centreline. In cases where centreline rotation would cause drainage problems or adversely affect the profile of guard rails or retaining walls, either inside or outside edge of pavement may be used as the point of rotation.

For multi-lane divided highways with depressed median widths greater than 13m, superelevation may be attained by rotating travel lane centrelines. Where there is a possibility of future widening of a four-lane divided highway to a six-lane divided highway by adding lanes to the inside of each roadway, superelevation should be developed by rotation about the inside edge of the future pavement. Figures B-3.7a, B-3.7b and B-3.7c illustrate the desirable methods of developing superelevation for both simple and spiral curves.

It should be noted that simple curves are rarely used in new construction. Spiral curves are required on all new roadways except where superelevation is not required (see Table B.3.6a) or on the flatter curves on local roads (see Table H.3.3.1a). When appropriate, simple curves may be used on existing paved roads to avoid the need to make minor realignments. The use of simple curves to tie in to existing paved alignments is considered a design exception.

When spiral transition curves are used, the adverse crown should be completely removed at the beginning of the spiral. For two-lane undivided and four-lane divided highways, transition from normal crown to where the adverse crown is removed is accomplished by means of a 30m tangent runout. For six-lane or eight-lane divided highways, the tangent runout length is to be determined using a slope on the outside pavement edge in relation to the centreline of one m in 400m, as suggested in TAC. From the beginning of the spiral curve to the beginning of the circular curve, the slope of the pavement edge is governed by the spiral parameter requirements and pavement width. Vertical curves should be used whenever a change in grade occurs at the shoulder line, at the beginning of the spiral curve, and at the beginning of the circular curve.

To superelevate simple circular curves, two-thirds of the full superelevation should be in place at the beginning of the circular curve, with the remaining third developed on the circular curve. Transition, from normal crown to where the adverse crown is removed, is accomplished by means of a 30m tangent runout. The length, Lr, over which the superelevation is applied, is determined by reference to Table B.3.7.

Where a transition occurs near the end of a bridge, the horizontal alignment is usually adjusted, if possible, to keep the superelevation transition off the bridge and maintain the normal cross-fall or constant superelevation throughout the structure.

On curves where the adverse crown is removed, but where the superelevation required does not exceed the crown slope, the superelevation should be adjusted to equal the crown slope.

Table B.3.7 - Length Required for Superelevation Runoff on Simple Curves

e (m/m)	Length of Runoff, Lr (m) for Design Speed (km/h)								
	50	60	70	80	90	100	110	120	130
0.02	30	30	30	30	30	30	30	30	30
0.03	30	30	30	30	40	40	40	40	50
0.04	30	40	40	40	50	50	50	60	60
0.05	40	50	50	50	60	60	70	70	70
0.06	50	60	60	70	70	80	80	80	90
0.07	60	60	70	70	80	90	90	90	100
0.08	70	70	80	80	90	100	110	110	120

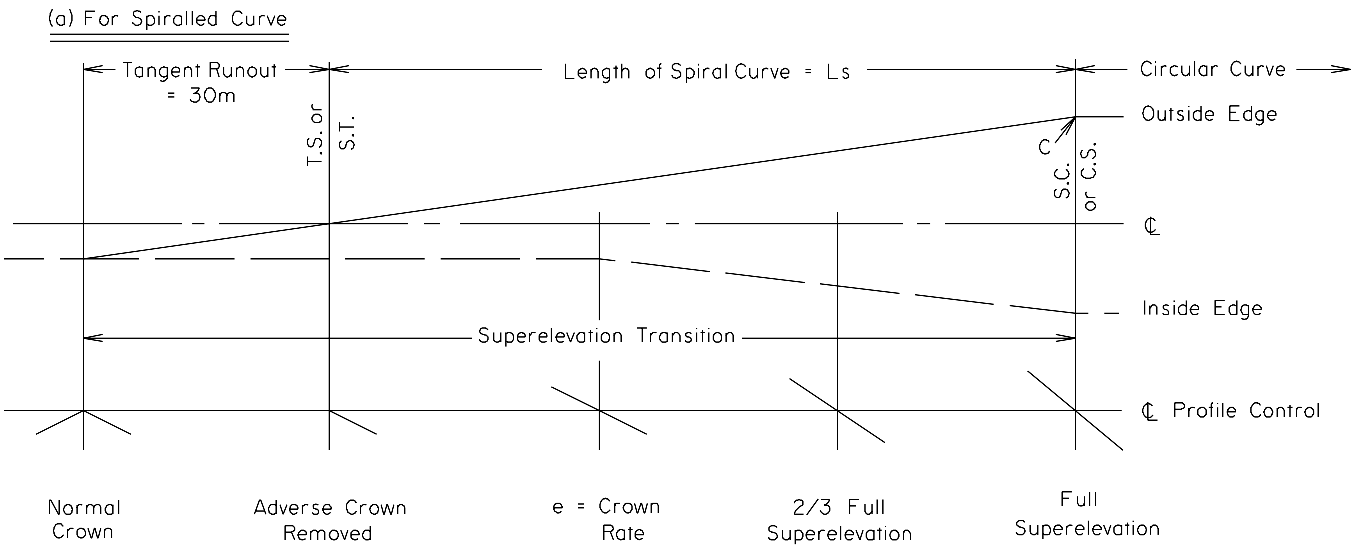
The above runoff lengths, Lr, are required for two-lane and four-lane undivided pavements. For six-lane undivided pavements, use 1.3 times the tabular values.

FIGURE B-3.7a SUPERELEVATION TRANSITION (CASE I)

METHOD OF ATTAINING SUPERELEVATION

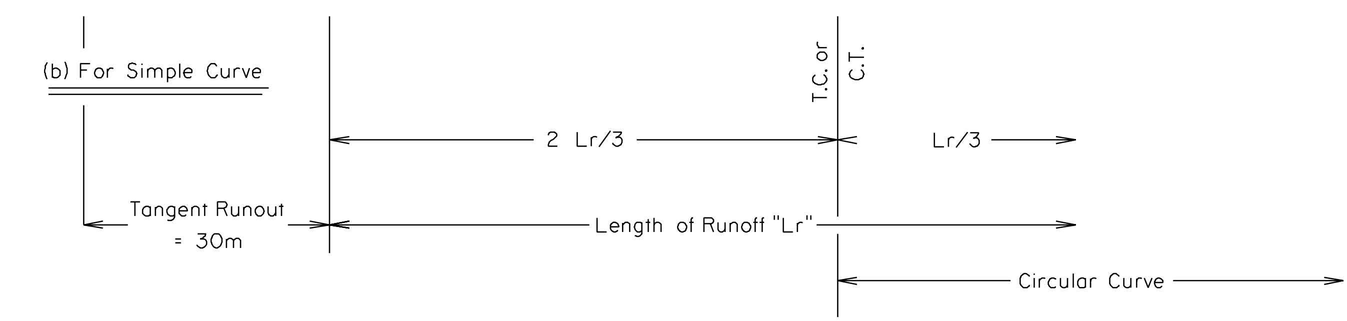
REVOLVED ABOUT CENTRE LINE

(- normally used on rural highways)



L_s = Length of Spiral Curve
A = Spiral Parameter (See Table B.4.6a or B.4.6b)
R = Radius of Circular Curve

$$L_s = \frac{A^2}{R}$$



L_r = Length of Runoff. (See Table B.4.7)

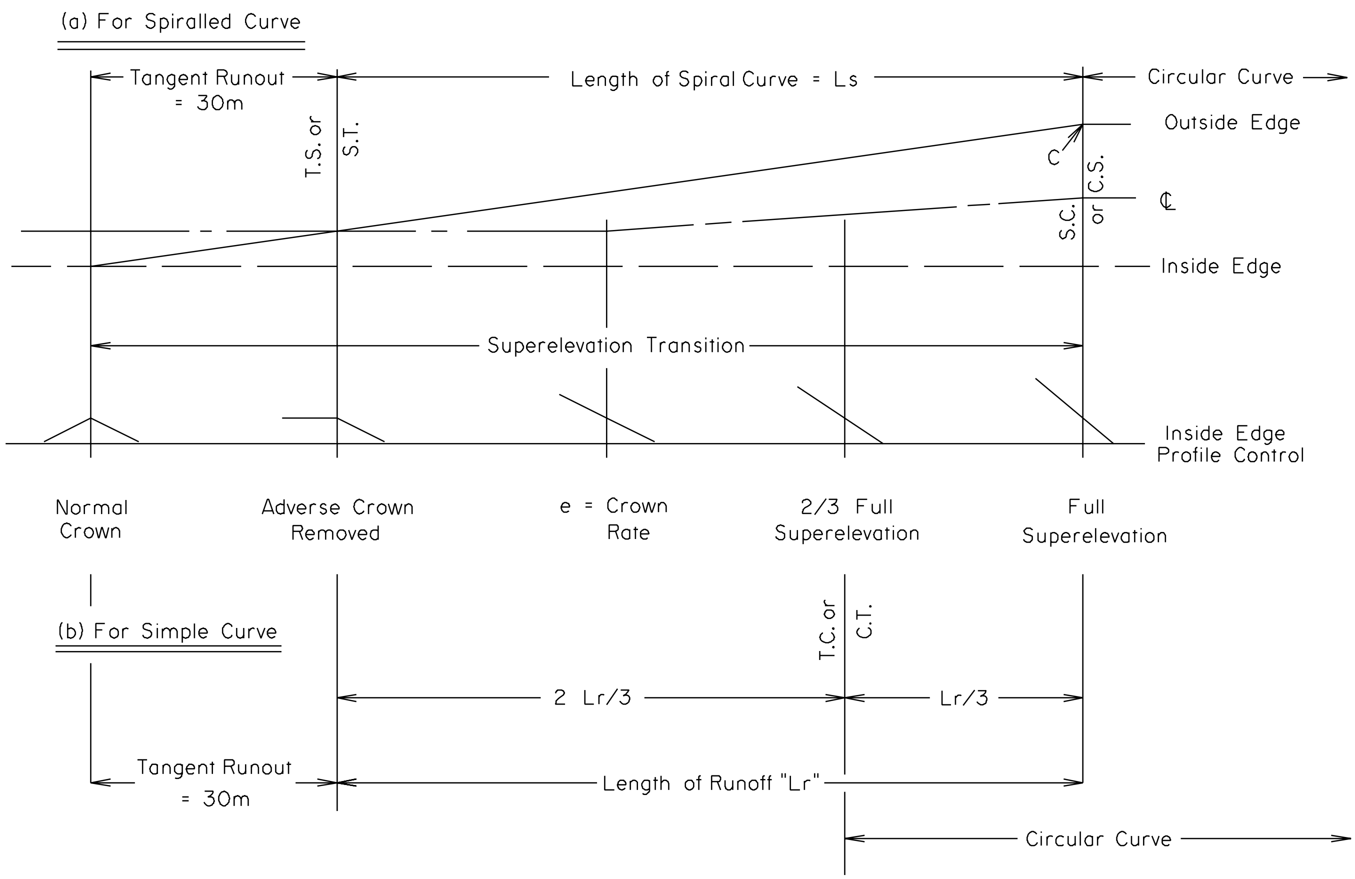
This method of attaining superelevation is to be used on 2-lane undivided highways and divided highways with narrow raised medians. In some cases it may be advantageous to use this method for curves at interchanges or intersections.
For multilane divided highways with depressed medians, superelevation is attained by revolving about centreline of travel lane in each direction.

A 30m tangent runout is applicable for 2-lane undivided highways or 4-lane divided highways. This tangent runout length is based on a 3.7m travel lane. For 6 or 8-lane divided highways, the tangent runout is to be determined, using a slope on the outside pavement edge in relation to the centreline of 1m in 400m.

NOTE: Use short vertical curve at points marked "C".

FIGURE B-3.7b SUPERELEVATION TRANSITION (CASE II)

METHOD OF ATTAINING SUPERELEVATION
REVOLVED ABOUT INSIDE EDGE

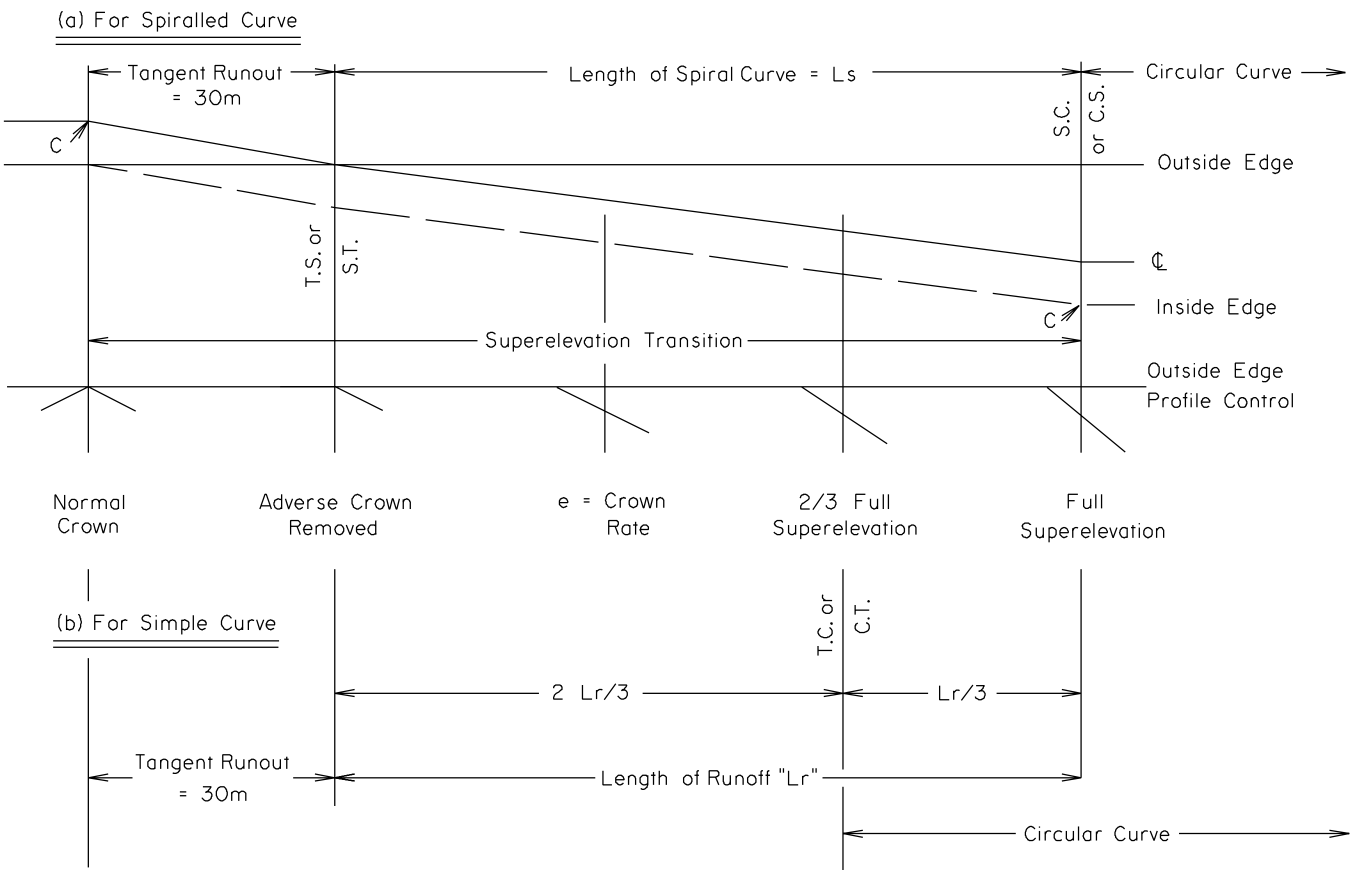


This method of attaining superelevation can be used on 2-lane highways, if required to match physical features on roadside or to facilitate drainage, and on divided highways where the median shoulder is on the inside of the curve. (see section B.4.7)

NOTE: Use short vertical curve at points marked "C".

FIGURE B-3.7c SUPERELEVATION TRANSITION (CASE III)

METHOD OF ATTAINING SUPERELEVATION
REVOLVED ABOUT OUTSIDE EDGE



This method of attaining superelevation can be used on divided highways where the median shoulder is on the outside of the curve. In some cases it may be advantageous to use this method for curves at interchanges or intersections.

NOTE: Use short vertical curve at points marked "C".

B.3.8 Spiral Curves

Spiral curves provide a gradual change in curvature from a straight to a circular path. The advantages of spiral curves are as follows:

1. Spiral curves provide a natural path for a motorist to follow, allowing centrifugal force to increase and decrease gradually as the vehicle enters and leaves the circular portion of the curve. This minimizes encroachment upon adjoining traffic lanes, promotes speed uniformity and increases safety.

2. The spiral curve length provides a desirable arrangement for superelevation runoff. A change from normal to a fully superelevated cross-slope is applied along the spiral curve length.

3. Where the pavement section is to be widened around a circular curve, the spiral facilitates the transition in width. Spirals simplify the design procedure and provide flexibility so that widening of sharp curves can be applied, in part, on the outside of the pavement without a reverse-edge alignment.

4. Highway appearance is improved by the application of spirals. Breaks that appear at the beginning and end of circular curves, which can be further distorted by superelevation runoff, should be avoided.

B.3.8.1 Form and Properties

Spiral curves are defined by three values: R (radius), L (length) and A (the spiral parameter). The square of the spiral parameter is the rate of change of length with respect to curvature; that is, the reciprocal of radius. This is expressed mathematically as follows:

R varies with L
RL is constant
$RL = A^2$ where A is a constant
$L = A^2 / R$

In the above expression, each term (L, A and R) is expressed in units of length. All spiral curves are the same shape and vary only in size. The spiral parameter is a measure of the spiral flatness — the larger the parameter, the flatter the spiral.

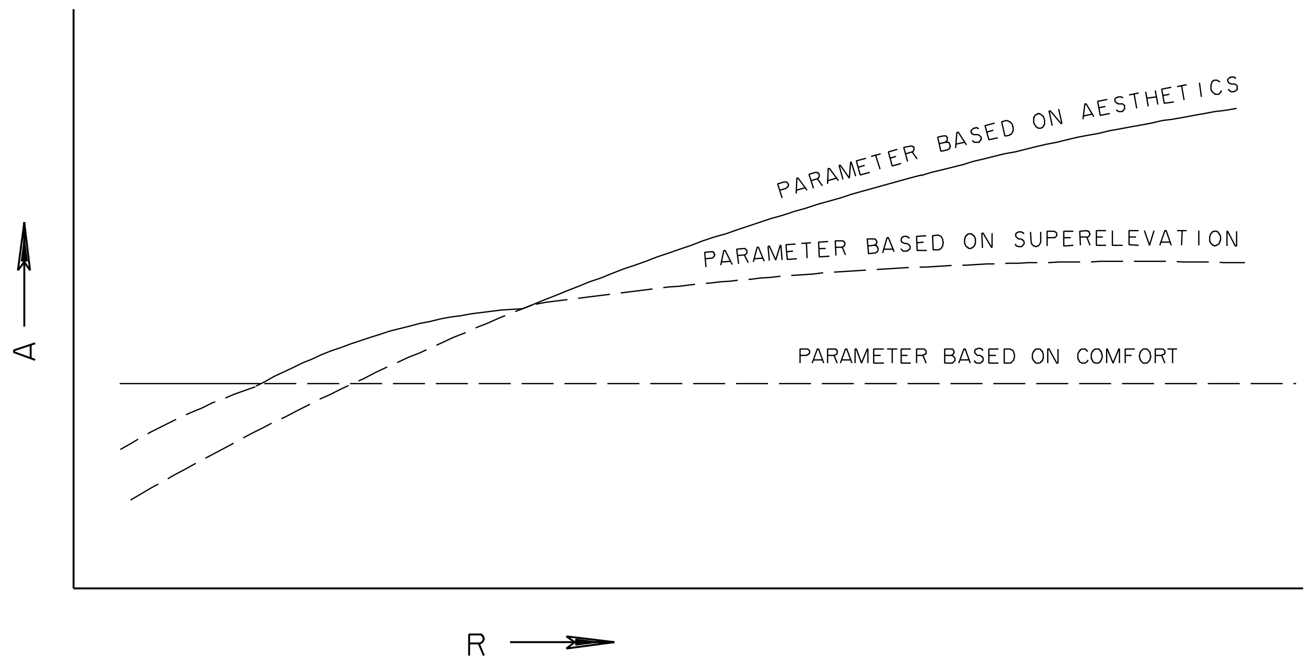
B.3.8.2 Basis of Design

As illustrated in Figure B-3.8.2, spiral design is based on three considerations: comfort, superelevation runoff and aesthetics. For any given design speed and radius, the highest value of spiral parameter, as determined by this criterion, is adopted for design.

FIGURE B-3.8.2 MINIMUM SPIRAL PARAMETER CONSIDERATIONS

MINIMUM SPIRAL PARAMETER

MINIMUM SPIRAL PARAMETER GOVERNED BY COMFORT, SUPERELEVATION AND AESTHETIC CRITERIA IN RELATION TO CURVE RADII FOR A GIVEN DESIGN SPEED



—— MINIMUM SPIRAL PARAMETER BASED ON THE THREE CRITERIA.

B.3.8.2.1 Comfort

A vehicle travelling along a spiral curve from tangent to the end radius at a constant speed experiences a centripetal force which varies at a constant rate along the transition length. For a given speed and end radius, the change rate of the centripetal force is a function of the spiral length — the shorter the spiral, the more rapid the rate of change. If it is very short, passengers will experience discomfort. The rate of change of centripetal force is proportional to the rate of change of radial acceleration and this is a measure of the severity of the discomfort. Tolerable radial acceleration varies between drivers. As a basis for design, the maximum value used to provide the minimum acceptable comfort and safety suitable for passengers is 0.6 metres per second cubed. The minimum spiral parameter, based on comfort, can be calculated for each design speed using the following expression.

Note: The minimum spiral parameter based on comfort considerations is independent of the radius, as illustrated in Figure B-3.8.2 by the comfort line paralleling the X-axis (representing the radius).

$$A = 0.189\ V^{1.5}$$

Where A is the spiral parameter (m)
V is the design speed (km / h)

B.3.8.2.2 Superelevation Runoff

As a vehicle traverses a spiral curve, the decreasing radius requires a corresponding increase in superelevation. In order to ensure a gradual change in the resulting radial acceleration, the superelevation is applied over the length of the spiral by raising or lowering the edge of pavement relative to some fixed profile control line. The slope of the outer edge of pavement, if permitted to become excessive relative to the profile control line, creates an aesthetically unpleasant kink in the vertical alignment of the pavement edge. The upper limiting values for relative slope between the outer edge of pavement and centreline (that is, profile control) for two-lane pavements at various design speeds are shown in Table B.3.8.2.2.

Table B.3.8.2.2 - Maximum Relative Slope Between Outer Edge of Pavement and Centreline for Two-Lane Roadway

Design Speed (km/h)	Relative Slope (%)
40	0.70
50	0.65
60	0.60
70	0.56
80	0.51
90	0.47
100	0.44
110	0.41
120	0.38
130	0.36

Utilizing the maximum permissible values for relative slope from Table B.3.8.2.2, the minimum length of spiral, L, can be found from the following expression:

$$L = \frac{100\,we}{2s}$$

Where L is the length of spiral (m)
w is the width of pavement (m)
e is the superelevation being developed (m / m)
s is the relative slope (%)

For a given design 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:

$$A^2 = RL$$

Where A is the spiral parameter
R is the radius (m)
L is the length of spiral (m)

B.3.8.2.3 Aesthetics

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

$$A^2 = 0.56RV$$

Where A is the spiral parameter
R is the radius (m)
V is the design speed (km/h)

The minimum spiral requirement for design is the highest of the three values required for comfort, runoff, and aesthetics. For smaller radii the comfort criterion controls, for the next larger set of radii the relative slope criterion controls, and for the larger radii the aesthetic criterion controls.

B.3.8.3 Design Values for Spiral Parameters

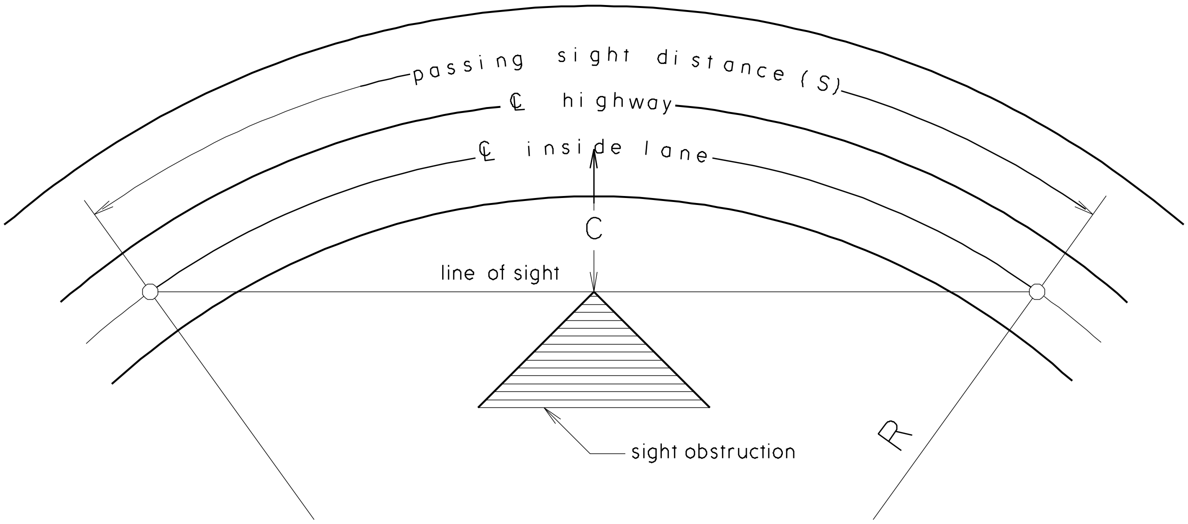
Spiral A parameter values for design are shown in Tables B.3.6a and B.3.6b for maximum superelevation rates of 0.06 m/m and 0.08 m/m, respectively. For each design speed and radius, minimum and desirable A parameters are given. On two-lane pavements desirable values should be used whenever possible. For three-lane and four-lane pavements desirable values are to be used, and on six-lane pavements desirable values multiplied by 1.15 are to be used. Spiral A parameter values are shown rounded to the nearest whole metre.

B.3.9 Passing Sight Distance and Stopping Sight Distance on Horizontal Curves

The minimum lateral clearances required on curves for each design speed to provide safe passing sight distance and stopping sight distance for a wide range of radii, are given in Figures B-3.9a and B-3.9b. The co-ordinates were calculated using the values shown in Table B.2.4 and in Table B.2.3, respectively.

Horizontal stopping sight distance should be provided along the entire road length. This may only involve the changing of a backslope, the removal of trees and hedges, or setback of a bridge handrail. Exceptions should be made only in rare cases, where the cost of such provision would be excessive, and then only with approval of a design exception. Horizontal sight distance for passing should be provided as frequently as possible.

FIGURE B-3.9a LATERAL CLEARANCE ON HORIZONTAL CURVES FOR PASSING SIGHT DISTANCES



$$C = R\left[1 - \cos\left(\frac{90S}{\pi R}\right)\right]$$

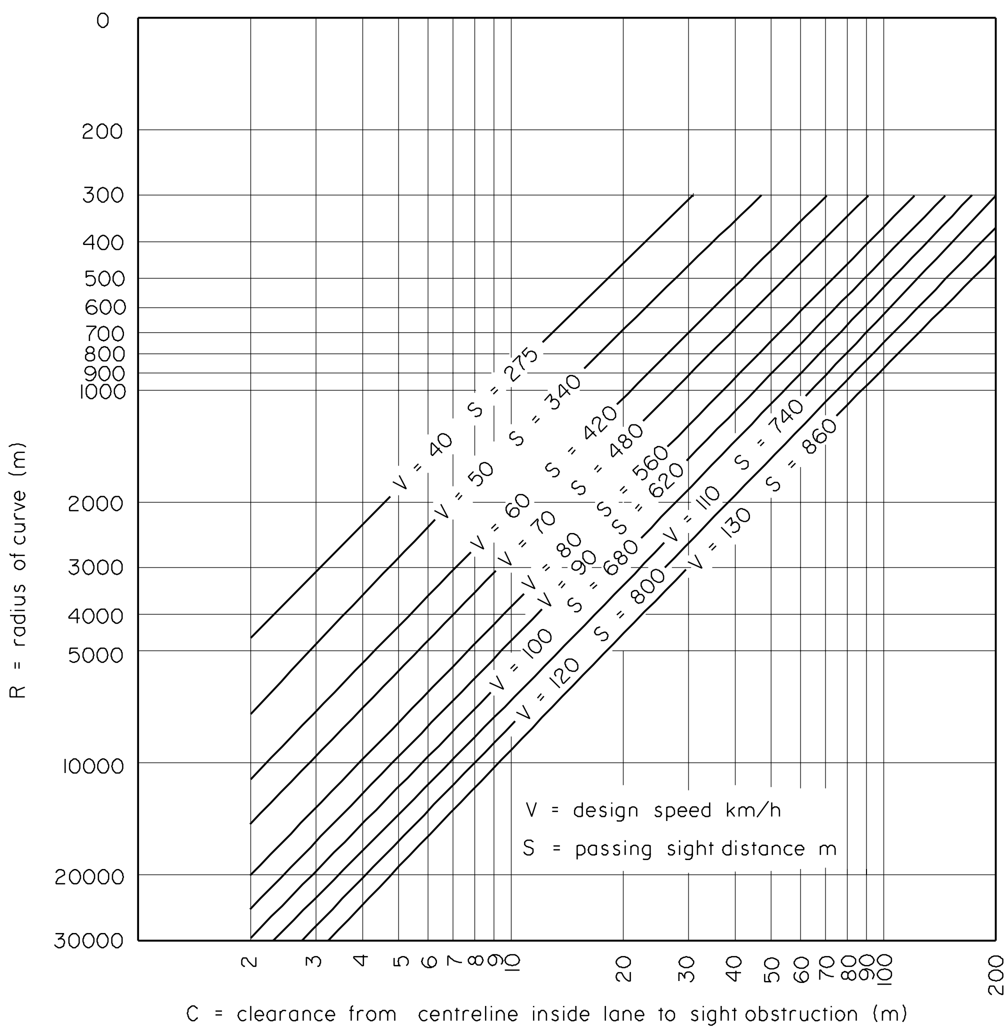


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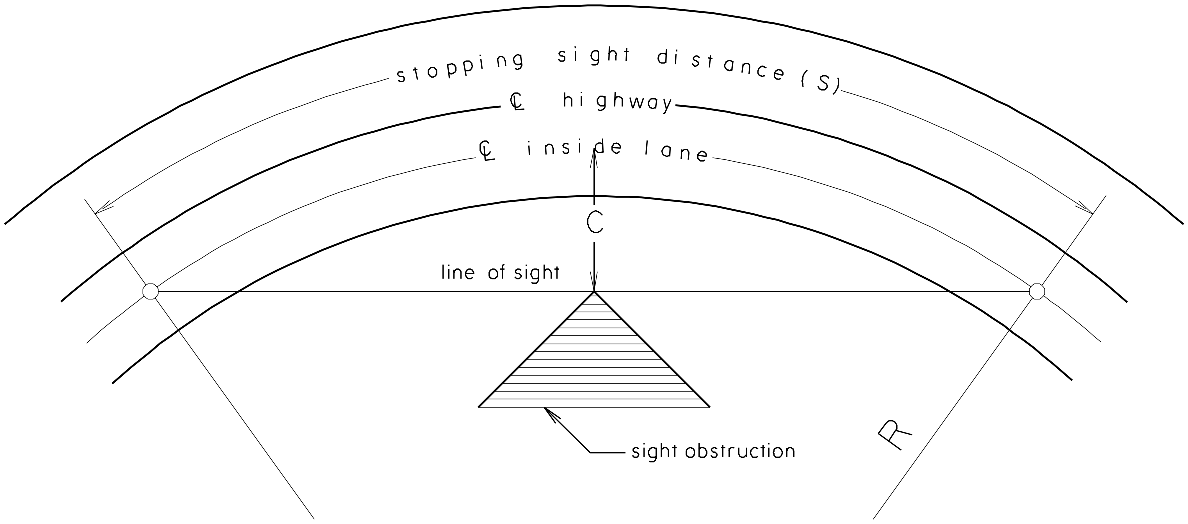
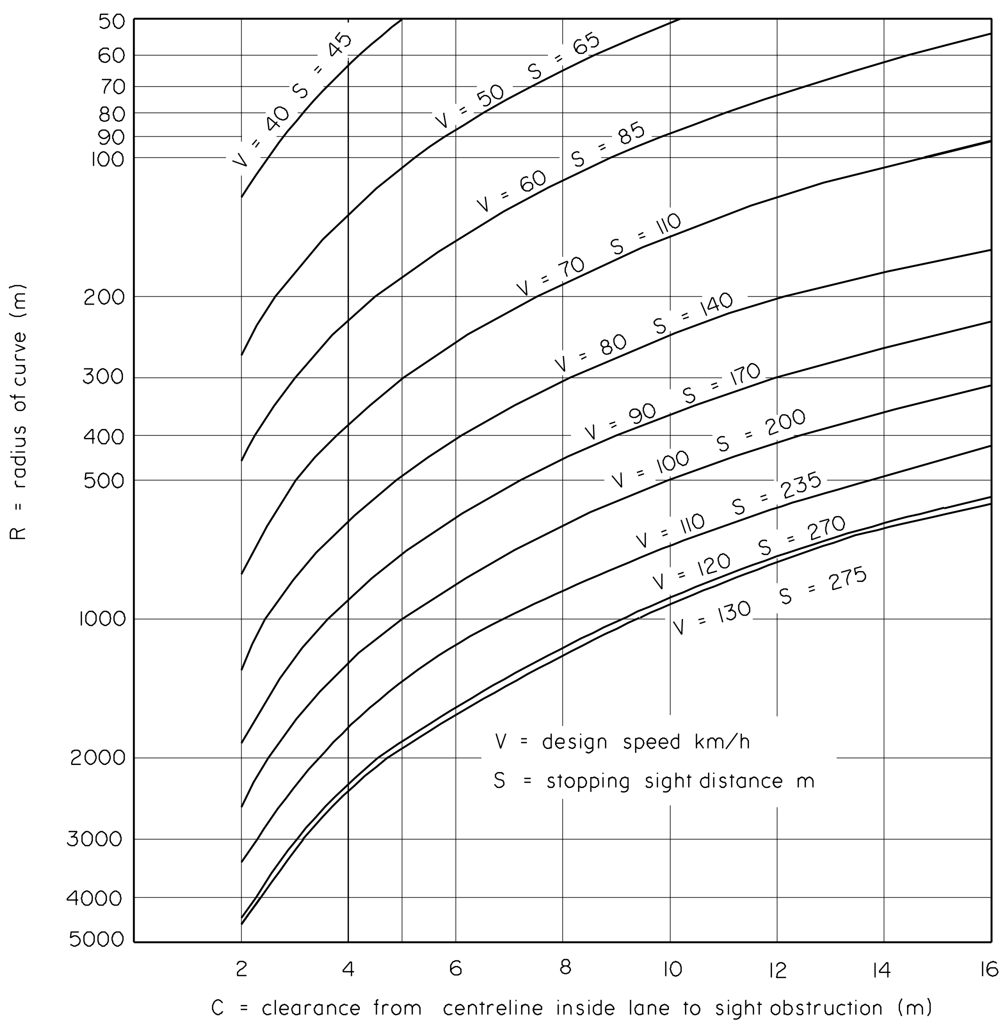
FIGURE B-3.9b LATERAL CLEARANCE ON HORIZONTAL CURVES FOR STOPPING SIGHT DISTANCE



$$C = R\left[1 - \cos\left(\frac{90S}{\pi R}\right)\right]$$



B.4 VERTICAL ALIGNMENT

B.4.1 General Controls for Vertical Alignment

The following general controls should be considered in design of vertical alignment, in addition to the specific controls related to sight distance, vehicle performance, drainage, etc., that are detailed later in this chapter.

1. A smooth gradeline with gradual changes, consistent with the type of highway and terrain, should be preferred over a line with numerous breaks and short lengths of grades. Detailed design values are the maximum grade and the critical length of grade, but the manner in which they are applied and fitted to the terrain on a continuous line determines the suitability and appearance of the finished product.

2. The roller-coaster or the hidden-dip type of profile should be avoided. Such profiles generally occur on relatively straight horizontal alignment where the roadway profile closely follows a rolling natural ground line. Examples of these undesirable profiles are evident on many older highways. They are unpleasant aesthetically and more difficult to drive. Hidden dips contribute to passing manoeuvre problems for drivers. The passing driver is deceived by the view of the road or street beyond the dip that appears to be free of opposing vehicles. Even with shallow dips, this type of profile is disconcerting because the driver does not know if there are oncoming vehicles hidden beyond the rise. This type of profile is avoided by use of horizontal curves or by more gradual grades.

3. Undulating gradelines, involving substantial lengths of grade, should be appraised for their effect on traffic operation. Such profiles permit heavy trucks to operate at higher overall speeds than is possible when an upgrade is not preceded by a downgrade. However, they may encourage excessive speeds of trucks with attendant conflicts with other traffic.

4. A broken-back gradeline (two vertical curves in the same direction separated by short section of tangent grade) generally should be avoided, particularly in sags where the full view of both vertical curves is not pleasing. This effect is very noticeable on divided roadways with open median sections.

5. On long grades it may be preferable to place the steepest grades at the bottom and lighten the grades near the top of the ascent. Another option is to break the sustained grade by short intervals of lighter grade instead of a uniform sustained grade that might be only slightly below the allowable maximum. This is particularly applicable to low-design-speed highways.

6. Where at-grade intersections occur on roadway sections with moderate to steep grades, it is desirable to reduce the gradient through the intersection to less than three percent. Such a profile change is beneficial for all vehicles making turns and serves to reduce the potential hazards.

7. Sag vertical curves should be avoided in cuts unless adequate drainage can be provided.

8. Generally, to ensure a smooth gradeline, a minimum Vertical Point of Intersection (VPI) spacing of 300m is used. A minimum length of vertical curve of 120m is used also to ensure that the parabolic shape is achieved using a survey control with 20m stations. Asymmetric parabolic curves may be used in special cases to suit terrain. This is achieved by using a different length for the second half of the curve compared to the first half; that is, the K value will change.

B.4.2 Maximum Gradient

Table A.7 provides the desirable maximum gradient that should be used for each design designation. The desirable maximum gradient is three percent on divided highways and RAU-213.4 roadways, five percent on undivided highways with pavement width from nine metres to 11.8m, and six percent on undivided two-lane eight metre roadways. Higher maximums are permitted on local roads due to the lower speeds and lower volumes. The use of these desirable maximum gradients will contribute towards very uniform vehicle speeds on the higher standard roadways with greater speed variation on the lower design designation roadways. With all other factors being equal, this would result in higher levels of service on the higher designation facilities.

The desirable maximum gradients shown in Table A.7 provide maximum gradients that should not be exceeded whenever practical. However, the maximum gradient is site specific. On alignments where construction costs increase substantially depending on the maximum gradient, an economic analysis should be undertaken to determine the suitable maximum gradient for that roadway section. This is why an absolute maximum gradient is not suggested in Table A.7. The economic analysis should include road user costs for collisions and vehicle running costs over the design life of the highway as well as construction costs, highway maintenance costs, and any other costs that are impacted by the choice of maximum gradient. It should be noted that the choice of maximum gradient may have a bearing on whether or not a climbing lane or truck runaway lane is required and this should be included in the economic analysis. For economic analysis purposes, the department uses a guide entitled Benefit-Cost Analysis which was prepared by K.E. Howery, P. Eng., and Applications Management Consulting Ltd. This guide presents the analysis methods and unit costs to be used for collisions and other road user costs to ensure uniformity for various evaluations. This guide can be obtained from Planning Services Branch.

An additional consideration that will not be apparent specifically from the economic analysis is level of service. Level of service can be downgraded considerably by the choice of a maximum gradient exceeding three percent on a long grade with significant truck traffic, especially where a climbing lane is not warranted on a two-lane roadway.

B.4.2.1 Vehicle Operating Characteristics on Grades

Passenger cars The practices of passenger car operators with respect to grades vary greatly. There is a general acceptance that nearly all passenger cars can readily negotiate grades as steep as four to five percent without appreciable loss in speed below that normally maintained on level highways, except for cars with high weight/horsepower ratios, including some compact and subcompact cars.

Studies show that operation on a three percent upgrade, compared with that on the level, has only a slight effect on passenger car speeds under uncongested conditions. On steeper grades, the speeds decrease progressively with an increase in the ascending grade. On downgrades, passenger car speeds generally are slightly higher than on level sections but local conditions govern.

Trucks The effect of grades on truck speeds is much more pronounced than on passenger car speeds. Average speed of trucks on level sections of highway approximates the average speed of passenger cars. Trucks display up to about a five percent increase in speed on downgrades and about a seven percent or more decrease in speed on upgrades as compared to operation on the level. On upgrades the maximum speed that can be maintained by a truck depends primarily on the length and steepness of the grade and the mass/power ratio, which is the gross vehicle weight in grams divided by the engine horsepower in watts. The effect of rate and length of grade on the speed of typical heavy trucks is shown in Table B.5.3.1a through Table B.5.3.1e in the climbing lane section of this chapter. Performance characteristics for various different mass:power ratios are shown. It has been found that trucks with a mass:power ratio of about 180g/W have acceptable operating characteristics from the standpoint of the highway user. Such a mass/power ratio assures a minimum crawl speed of about 43 km/h on a long three percent upgrade. There is evidence that the automotive industry finds a mass/power ratio of this magnitude acceptable as a minimum goal in the design of commercial vehicles. There is also evidence that the carrier operators are voluntarily recognizing this ratio as a minimum performance control in the loads placed on trucks of different power. The overall result is that mass/power ratios of trucks on highways have improved in recent years; that is, truck performance has improved.

A mass/power ratio of 180g/W should be used in general for the design truck for the purpose of simulation of vehicle performance on grade for Alberta highways. Field surveys of truck ratings have shown the 180g/W generally provides for more than 85 percent of the trucks in the traffic stream. Additional performance charts are provided for special situations; for example, where the design truck is a log haul truck or where at least 20 percent of the heavy vehicles consist of seven or eight axle trucks, as may be the case on divided highways, 200g/W should be used. For light trucks or recreation vehicles 60g/W is appropriate; that is, it should only be used where at least 85 percent of the heavy vehicles are recreational vehicles or light trucks (two axles). Similarly, the 120g/W and 150g/W are appropriate where the design vehicle is a three axle or five axle truck, respectively.

B.4.3 Minimum Gradient

B.4.3.1 Rural Highways

Level grades (0 percent) on uncurbed rural highways are considered perfectly acceptable provided that the roadway surface is adequately crowned to drain the surface laterally. A two percent crown is standard for paved roadways and three percent for gravel surfaces. Although a level grade is acceptable on the roadway surface, some positive drainage is required in the roadside ditch to ensure that ponding does not occur. A different gradient in the ditch compared to the road surface can be built using a design ditch. This is achieved by adjusting the ditch depth. This practice is also known as false grading. Table B.4.3 provides the desirable and absolute minimum gradients for narrow and wide roadside ditches. Flatter gradients are permitted on the wider ditches based on the assumption that the ditch bottom will slope away from the roadway. Since this cross-slope helps keep water away from the subgrade, a smaller value for longitudinal gradients may be used. For wide ditches the desirable minimum longitudinal grade is 0.2 percent and absolute minimum is 0.05 percent. Where the longitudinal ditch gradient is quite flat, designers should consider using a five percent cross-slope on the ditch bottom (sloping away from the subgrade) to provide a drier embankment.

B.4.3.2 Curbed Roadways

On curbed pavements where drainage is adjacent to the travel lanes, longitudinal gradients must be set to eliminate excessive water accumulation on the pavement. A minimum grade for the usual case is 0.5 percent, but an absolute minimum of 0.35 percent may be used where the pavement surface is accurately crowned and supported on firm subgrade.

Table B.4.3 gives the standard minimum longitudinal gradients for roadways and ditches on all classes of roads.

Table B.4.3 Minimum Gradient

Roadway Type	GRADIENT	
	Desirable Minimum %	Absolute Minimum %
Rural Highways	Longitudinal Gradient for Ditch	
Wide Ditches (greater than 3m)	0.2	0.05
Narrow Ditches (less than 3m)	0.5	0.2
Urban Roadways (curbed)	Longitudinal Gradient for Roadway Surface	
Roadway Surface	0.5	0.35

B.4.4 Vertical Curves

B.4.4.1 K Parameter

The K parameter is a coefficient for defining the rate of gradient change. For example, a K value of 90 means a horizontal distance of 90m is required for every one percent gradient change.

For metric curve calculation the length of a vertical curve is based on the K value, using the formula:

$$L = KA$$

Where	L	is the horizontal length of the vertical curve (m)
	A	is the algebraic difference in gradient between the two intersecting gradelines (%)
	K	is a coefficient, as described above

Table B.4.4 provides a summary of all the minimum vertical curve K parameters based on stopping, non-striping and passing sight distance criteria. 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 needs to see to make a decision).

B.4.4.2 Crest Vertical Curves

Minimum crest vertical curves that satisfy stopping, non-striping and passing sight distance requirements are given in Table B.4.4. These minimum curvatures are expressed in terms of the K parameter for each design speed.

Minimum crest curve K values which satisfy stopping sight distance requirements were derived based on an eye height of 1.05m, object height of 0.38m and using the unrounded minimum stopping sight distances (see Table B.2.3). These values are given for minimum curve lengths for each design speed, but in practice a higher value should be used whenever possible.

The designer should use crest vertical curves that provide passing sight distance on two-lane highways wherever economically practicable. This will result in improved traffic flow, increased capacity and probably some reduction in the number of collisions. Minimum crest vertical curves that satisfy passing sight distance requirements were derived based on an eye height of 1.05m, height of opposing vehicle of 1.30m and using minimum passing sight distances as shown in Table B.2.4.

Passing sight distance is desirable on crest vertical curves; however, where it is not economically feasible to provide passing sight distance, the provision of non-striping sight distance is of some benefit. Non-striping sight distance is still adequate to allow drivers to complete safe passes; however, drivers must be prepared to abort a pass if an oncoming vehicle comes into view at the critical moment, that is, when actual passing is about to occur. Section B.2 includes further information about the differences between passing sight distance and non-striping sight distance.

Using a K value greater than the minimum stopping sight K but less than the minimum non-striping K will increase the length of the barrier line; that is, it will increase the length of the no passing zone on the crest vertical curve.

Figures B-4.4.2a, b and c show the models and formulae used to determine the crest vertical curvature K parameters.

Table B.4.4 Minimum Vertical Curve Criteria

Design Speed (km/h)	Assumed Running Speed (km/h)	MINIMUM K VALUES OF VERTICAL CURVES				
		Vertical Crest Curves			Vertical Sag Curves (SSD)	
		SSD	NSSD*	PSD	Headlight Control	Comfort Control
40	40	5	-	80	7	5
50	50	10	-	125	12	7
60	60	15	-	190	20	10
70	70	25	-	245	25	13
80	80	35	-	335	35	17
90	90	55	-	410	40	21
100	100	75	250	495	50	26
110	108	100	250	585	60	30
120	115	130	250	685	70	35
130	115	140	-	790	70	35
height of eye (m)		1.05	1.15	1.05	1.05	-
height of object (m)		0.38	1.15	1.3	0	-

SSD = Stopping Sight Distance
NSSD = Non-Striping Sight Distance
PSD = Passing Sight Distance

* Although a “K” value of 250 should be used on new construction as a desirable parameter where passing is to be permitted, a lower “K” value may also allow passing. Using the current AI practice for marking barrier lines, based on a sight distance of 425 m and eye height = object height = 1.15 m, a “K” value of 197 m may allow passing. Also where the crest curve is less than 425 m long, passing may be permitted even on sharper crests (this can be established from the profile plan or through measurements taken in the field).

FIGURE B-4.4.2a MINIMUM STOPPING SIGHT DISTANCE ON CREST VERTICAL CURVES

STOPPING SIGHT DISTANCE ON CREST VERTICAL CURVES

(i) For use in design of two-lane highways as an absolute minimum only.
(ii) For use in design of all divided highways and interchanges.

L = Minimum length of vertical curve in metres
A = Algebraic difference in grades, percent
SSD = Minimum stopping sight distance in metres
K = Rate of Vertical Curvature, Length in Metres Per Percent change of A.

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

When SSD < L

$$K = \frac{SSD^2}{200\,(\sqrt{h_1} + \sqrt{h_2})^2} = \frac{SSD^2}{538.67}$$

When SSD > L

$$K = \frac{2\ SSD}{A} - \frac{200\,(\sqrt{h_1} + \sqrt{h_2})^2}{A^2} = \frac{2\ SSD}{A} - \frac{538.67}{A^2}$$

Design Speed (km/h)	Assumed Running Speed (km/h)	Minimum Stopping Sight Distance (m)	Minimum K Values Vertical Crest Curves
40	40	45	5
50	50	65	10
60	60	85	15
70	70	110	25
80	80	140	35
90	90	170	55
100	100	200	75
110	108	235	100
120	115	270	130
130	115	275	140

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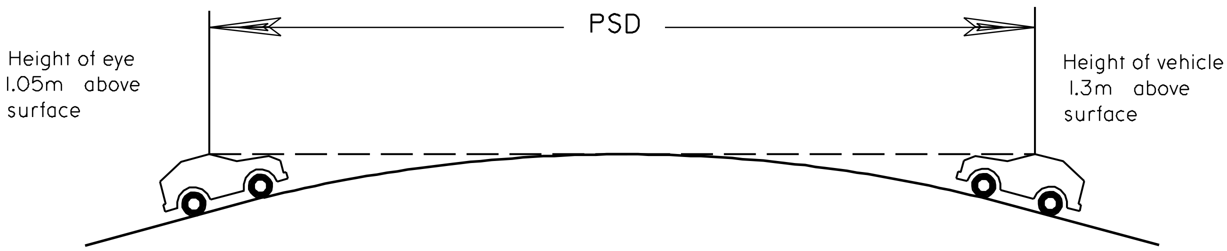
FIGURE B-4.4.2b PASSING SIGHT DISTANCE

PASSING SIGHT DISTANCE
ON
CREST VERTICAL CURVES

Desirable for design of two-lane highways carrying two-way traffic



L = Minimum length of vertical curves in metres
A = Algebraic difference in grades, percent
PSD = Passing sight distance in metres
(Use PSD designation on profiles.)

When PSD < L

$$K = \frac{PSD^2}{200\,(\sqrt{h_1} + \sqrt{h_2})^2} = \frac{PSD^2}{937.3}$$

When PSD > L

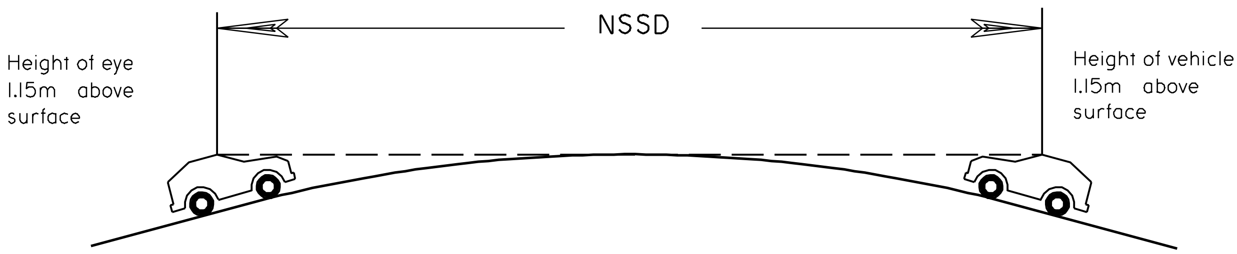
$$K = \frac{2\ PSD}{A} - \frac{200\,(\sqrt{h_1} + \sqrt{h_2})^2}{A^2} = \frac{2\ PSD}{A} - \frac{937.3}{A^2}$$

Design Speed (km/h)	Minimum PSD for Crest or Sag Curves (m)	Minimum K Value for PSD on Crest Curves
40	275	85
50	340	125
60	420	190
70	480	245
80	560	335
90	620	410
100	680	495
110	740	585
120	800	685
130	860	790

FIGURE B-4.4.2c MINIMUM NON-STRIPPING SIGHT DISTANCE ON CREST VERTICAL CURVES

NON-STRIPING SIGHT DISTANCE ON CREST VERTICAL CURVES

For use in design of two-lane, two-way highways



L = Minimum length of vertical curve in metres
A = Algebraic difference in grades, percent
NSSD = Non-striping sight distance in metres

When NSSD < L

$$K = \frac{NSSD^2}{200\,(\sqrt{h_1}+\sqrt{h_2})^2} = \frac{NSSD^2}{920}$$

When NSSD > L

$$K = \frac{2\ NSSD}{A} - \frac{200\,(\sqrt{h_1}+\sqrt{h_2})^2}{A^2} = \frac{2\ NSSD}{A} - \frac{920}{A^2}$$

Design Speed (km/h)	Posted Speed (km/h)	Minimum Non-Striping Sight Distance (m)	Minimum K Value for Non-striping
100	100	480	250
110	100	480	250
120	100	480	250

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B.4.4.3 Sag Vertical Curves

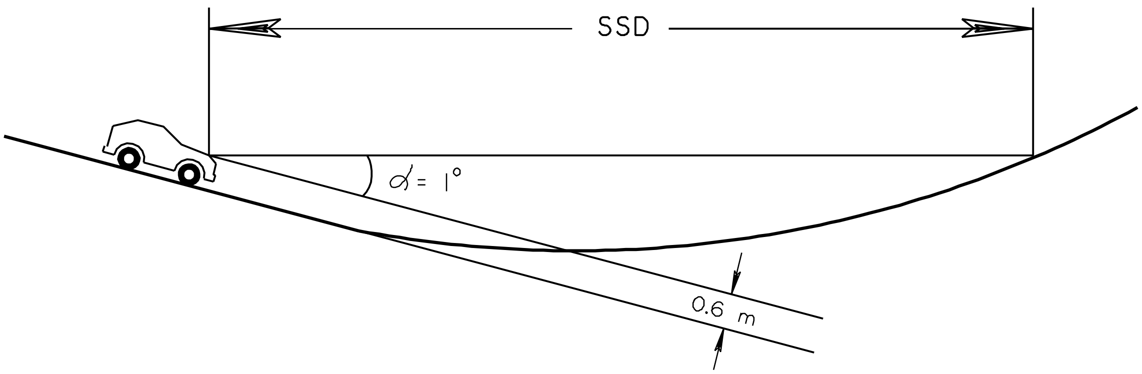
The minimum vertical curvature on sag curves, shown in Figure B-4.4.3, is based on providing stopping sight distance within the headlight beam. This is described as headlight control and is only appropriate for roadways that are not illuminated. Headlight control is based on the following assumptions: headlight beams slope upward at an angle of one degree from the plane of the vehicle, height of headlights is 0.6m above the driving surface, and the object (to be stopped for) comes into view when the vehicle is the minimum stopping sight distance away.

On illuminated sag curves, it is acceptable to use comfort control rather than headlight control to select K values for sag vertical curves. These values are normally exceeded where feasible, in consideration of possible power failures and other malfunctions to the streetlighting systems. The comfort K value is based on the radial acceleration experienced by occupants of vehicles travelling at the assumed running speed at the bottom of the sag vertical curve. The maximum acceptable radial acceleration adopted by Alberta Infrastructure is $0.3 m/s^2$, based on TAC. The comfort K values shown in Figure B-4.4.3 and Table A.7 are based on this allowable radial acceleration.

FIGURE B-4.4.3 MINIMUM STOPPING SIGHT DISTANCE ON SAG VERTICAL CURVES

STOPPING SIGHT DISTANCE ON SAG CURVES

For use in design of all highways and interchanges.



h_3 = 0.6m, headlight height above driving surface in metres
L = Minimum length of vertical curve in metres
A = Algebraic difference in grades, percent
SSD = Minimum stopping sight distance in metres
α = upward angle of headlight beam
K = Rate of vertical curvature, length in metres per percent change of "A"

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

Formulas for minimum K with HEADLIGHT control

When SSD < L

$$K = \frac{SSD^2}{200\ (h_3 + SSD \tan 1^\circ)}$$

When SSD > L

$$K = \frac{2\ SSD}{A} - \frac{200\ (h_3 + SSD \tan 1^\circ)}{A^2}$$

Formula for minimum K for COMFORT control

$$K = \frac{V^2}{390}$$

When V is the assumed running speed in km/h.

Design Speed (km/h)	Assumed Running Speed (km/h)	Minimum Stopping Sight Distance (m)	Minimum K Values Vertical Sag Curves	
			Headlight Control	Comfort Control*
40	40	45	7	5
50	50	65	12	7
60	60	85	20	10
70	70	110	25	13
80	80	140	35	17
90	90	170	40	21
100	100	200	50	26
110	108	235	55	30
120	115	270	60	35
130	115	275	65	35

* To be used on illuminated roads only.

* Note: Designers should use vertical curve K values higher than the minimums shown whenever economically practical.

B.5 CLIMBING AND PASSING LANES

B.5.1 Introduction

Auxiliary lanes are additional lanes that can be provided at selected locations along highways to facilitate turning, deceleration, acceleration, passing or low velocity climbing (climbing lanes). Auxiliary lanes for turning, deceleration, or acceleration are normally provided at intersection treatments and therefore are included in Chapter D. Climbing lanes and passing lanes are generally required due to the characteristics of the vertical and horizontal alignment, together with other factors, and therefore are covered in this chapter.

B.5.2 Geometric Features of Climbing and Passing Lanes

The following geometric criteria should be met in provision of climbing or passing lanes.

B.5.2.1 Lane Width

The width of the auxiliary lane should be the same as the through lane, that is, 3.7m for design designation of RAU-211.8 and higher or 3.5m for design designations of RAU-210 and lower.

B.5.2.2 Shoulder Width

The shoulder adjacent to the auxiliary lane should be equal to the lesser of 1.5m or the standard shoulder width on that design designation of highway.

B.5.2.3 Superelevation

Superelevation on the climbing lane portion of the roadway surface should generally be the same as on the adjacent through lane. However, where operating speeds of heavy vehicles can be expected to be much lower than design speed, the designer may use judgment in selecting a lower superelevation rate. Superelevation on the passing lane portion of the roadway surface should be the same as the adjacent through lane.

B.5.2.4 Tapers

The taper at the beginning and end of climbing/passing lane should be 60:1. The 60:1 taper on the diverge should promote use of the right hand lane by all vehicles except those intending to overtake slower vehicles.

B.5.2.5 Proximity to Intersections

Locations that include or are in close proximity to intersections should be avoided because of possible operational difficulties. Where these situations cannot be avoided, a site specific analysis should be undertaken to determine the intersection treatment required. The treatment may require construction of an additional lane or relocation of the intersection.

B.5.2.6 Start and End Points and Length

The full width of a climbing lane should begin when the design truck has experienced a 15 km/h speed reduction. It should not be terminated until the design truck has regained the speed that it had at the beginning of the climbing lane.

A climbing lane could be started earlier or ended later if this would result in a noticeable improvement in traffic operations; for example, on roadways where the passing demand is high (due to high volume and/or high percentage of heavy vehicles) and the length of grade is short. Where it has been decided that a climbing lane should be lengthened, it is generally preferable to add to the beginning of the climbing lane. Beginning a climbing lane earlier (that is, before heavier vehicles have decreased their speed by 15 km/h) will allow following vehicles to pass without having to decelerate to 80 km/h. This results in a more efficient climbing lane when the passing demand is high and a generally higher level of service for the roadway. It is preferable that the length of climbing lane be minimized to less than two to three km to provide greater cost-effectiveness. Very long climbing lanes, especially on lower volume roads, tend to be under-utilized.

The desirable length of a passing lane is between 1.5 km and 2.0 km. This range is long enough to be adequate for dispersing queues while still being short enough to be cost effective.

With long continuous grades, it is occasionally impractical to continue a climbing lane for the complete length required for the design truck to regain the entry speed. In this case, it is necessary to terminate the extra lane prematurely. It is important to ensure that there is good sight distance using decision sight distance criteria at the end point. It is also good practice to provide an extra wide shoulder (3.5m) for some length after the termination point. This length of wide shoulder should be sufficient to allow a vehicle travelling in the upgrade direction to come to a safe stop in an emergency situation, assuming the vehicle is at a reduced speed on the upgrade as shown by the design vehicle performance charts. The designer may use the appropriate stopping sight distance as a guide. The wide shoulder will serve as an escape lane and should reduce the occurrence of collisions at the merge area. The merge area can be very problematic for recreational vehicles and trucks, especially if the lane is ended prematurely. Under these circumstances, the absence of an escape lane can reduce the utilization and effectiveness of a climbing lane.

Very long passing or climbing lanes are especially undesirable on high volume two-lane highways because of the restricted passing for the opposing traffic stream. Current pavement marking guidelines in Alberta suggest that a double solid barrier line (prohibiting passing in the single lane direction) be painted at all passing/climbing lane locations on undivided highways where the AADT exceeds 4000. Where the AADT is less than 4000, passing is permitted in the single lane direction provided that passing sight distance is available. This is illustrated on Figure B-5.2.7.

B.5.2.7 Sight Distance at Start and End Points

Decision sight distance should be available for drivers of passenger vehicles to see the pavement surface in the first half of the taper at the termination of a climbing lane or passing lane. A similar sight distance is desirable but not essential at the beginning of climbing or passing lanes. When measuring the decision sight distance, a height of eye 1.05m (corresponding to a passenger vehicle) and a height of object of 0 (corresponding to the roadway surface) should be used. The range of decision sight distances suggested for the termination of an auxiliary lane is shown on Table B.2.6.

For the purpose of measuring decision sight distance, the object can be assumed to be 120m past the beginning of taper at the termination of the climbing lane. The reasons for selecting this location are as follows:

1. A driver seeing the pavement surface at this point will know that there is a taper. (That is, the driver will already have seen the two arrows on the pavement, the end of the auxiliary lane line and the narrower pavement.)

2. The decision sight distance requirement includes four seconds for a manoeuvre (lane change) which could occur on the first half of the taper (a vehicle travelling at 110 km/h will travel approximately 120 m in four seconds).

For example, for a design speed of 110 km/h, the driver of a passenger vehicle should be able to see the pavement surface over the first 120m of taper from a point 210m - 310m before the taper begins. This should enhance the safety of merging operations. Figure B-5.2.7 illustrates the general layout of a climbing/passing lane including typical signing and pavement markings and the decision sight distance requirement.

FIGURE B-5.2 CLIMBING/PASSING LANES FOR VARIOUS PAVEMENT WIDTHS

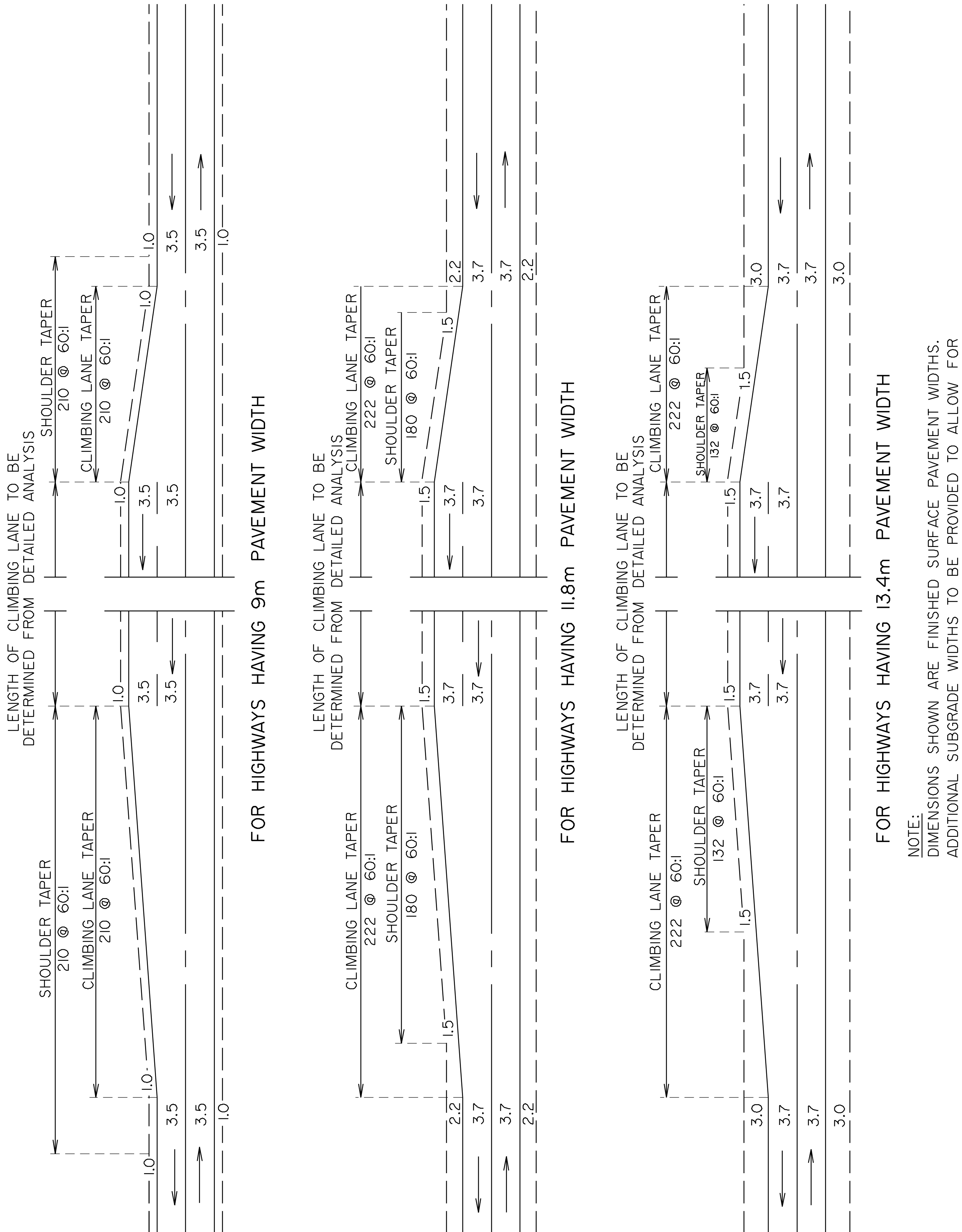
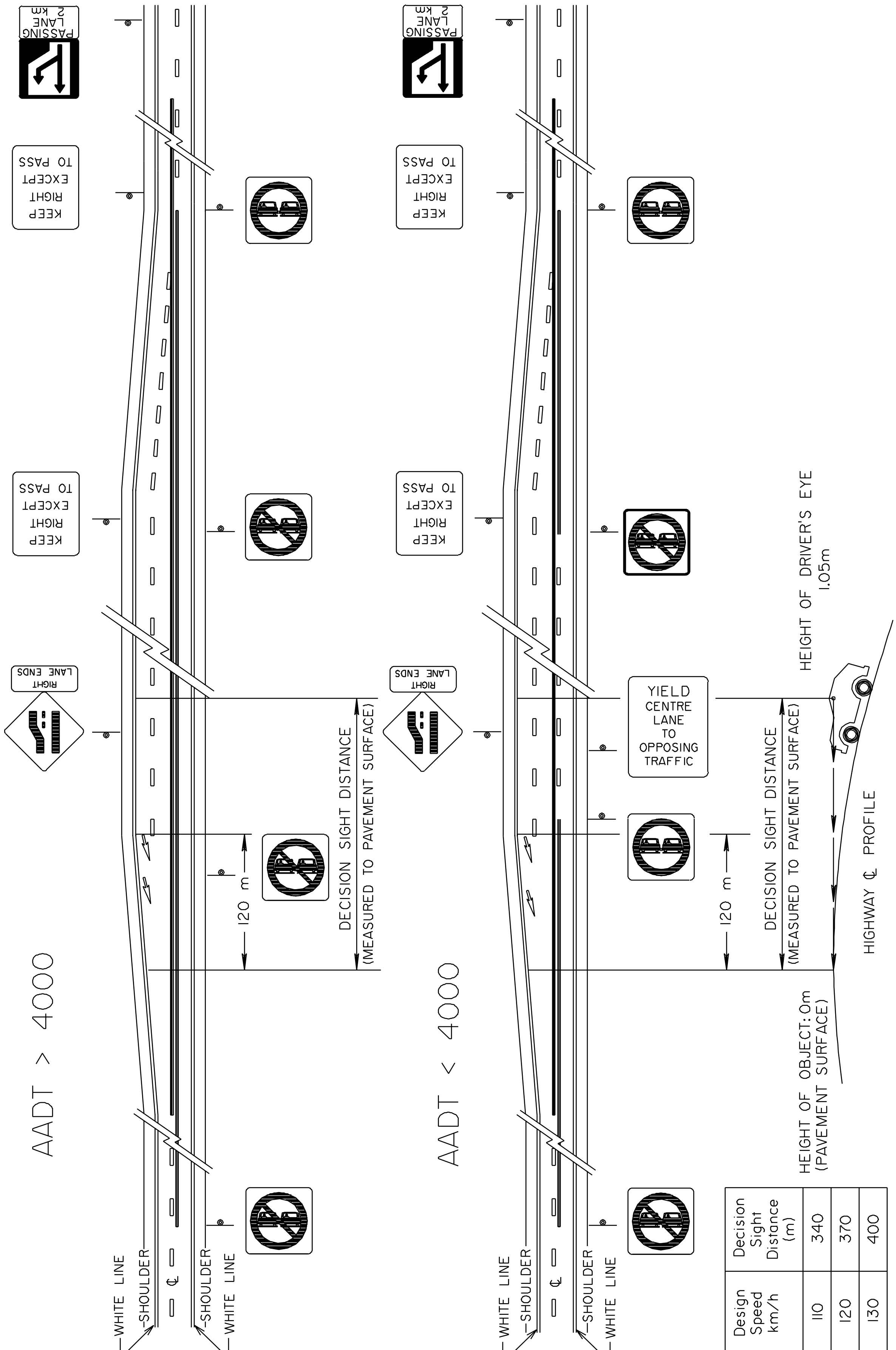


FIGURE B-5.2.7 TYPICAL SIGNING FOR PASSING AND CLIMBING LANES AND DECISION SIGHT DISTANCE REQUIREMENT AT MERGE TAPER



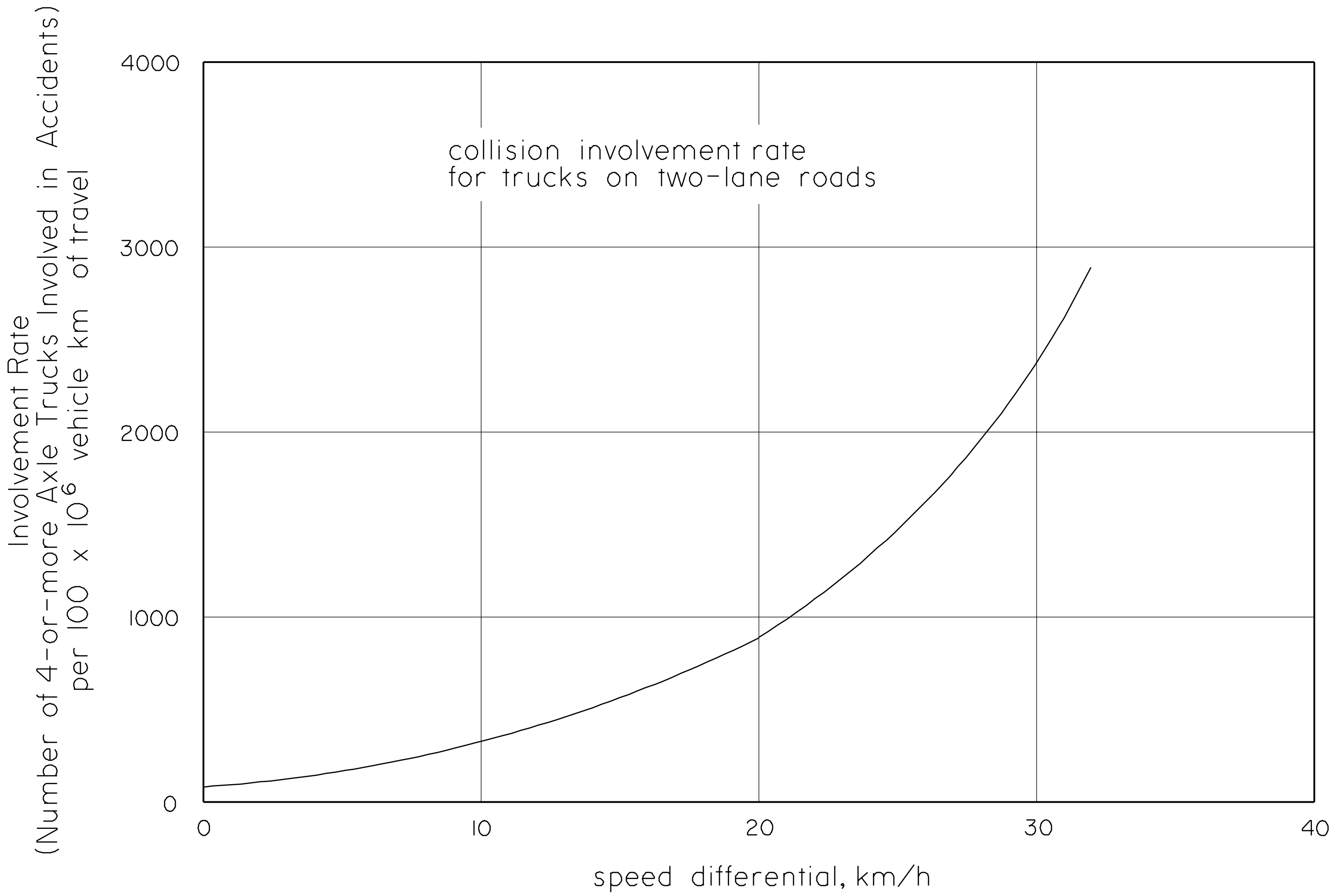
Design Speed km/h	Decision Sight Distance (m)
110	340
120	370
130	400

B.5.3 Climbing Lanes

Level of service and safety of operation on two-lane highways are impacted by the extent and frequency of passing sections. They are also adversely affected by heavily loaded vehicles operating on grades of sufficient length to result in speeds that could impede following vehicles. Because of the high number of accidents occurring on grades involving heavy vehicles, climbing lanes are commonly included in new construction of busier highways and additional lanes on existing highways are frequently built as safety improvement projects. The justification for these safety improvements is demonstrated by a plot of accident involvement rate for trucks on two-lane roads versus speed reduction. See Figure B-5.3.

It is desirable to provide a climbing lane as an extra lane on the upgrade side of a two-lane highway where the grade, traffic volume and heavy vehicle component combine to degrade traffic operations from those on the approach to the grade. Where climbing lanes have been provided, there has been a high degree of compliance in their use by truck drivers. On highways with low volumes, only the occasional car is delayed. Climbing lanes, although desirable, may not be justified economically even where the critical length of grade is exceeded. A warrant system is used to identify those cases where a climbing lane is called for based on safety and overall cost-effectiveness.

FIGURE B-5.3 COLLISION INVOLVEMENT RATE OF TRUCKS FOR WHICH RUNNING SPEEDS ARE REDUCED BELOW AVERAGE RUNNING SPEEDS OF ALL TRAFFIC



Source: AASHTO, A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS, 1994 (PAGE 237)

B.5.3.1 Climbing Lane Warrant for Two-Lane Undivided Highways

Climbing lanes should normally be provided only when both the speed reduction warrant and traffic volume warrant are met on a particular upgrade segment of roadway.

Speed Reduction Warrant

A 15 km/h speed reduction is experienced by the design truck.

For the purpose of calculating the speed reduction of trucks on gradient the following assumptions are used:

- The truck entry speed is 95 km/h
- The mass power ratio for the design truck is 180 g/W.

The truck entry speed is based on mean speed recorded for trucks on two-lane highways in Alberta. The mass/power ratio is based on a survey of the Alberta trucking industry together with a spot survey taken at provincial vehicle inspection stations. The 180 g/W rating, which corresponds approximately to 300lbs/hp used by many U.S. Transportation Departments, is based on the 85th percentile mass/power ratio, that is, 85 percent of the heavy vehicles in the upgrade traffic stream should be able to perform as well or better than the design truck.

Exceptions to the standard design truck mass/power ratio should only be made where records of the actual mass/power ratio of the trucks in the traffic stream indicate that a different value would more closely represent the 85th percentile heavy vehicle. An example of this may be a predominantly recreational route where more than 85 percent of the heavy vehicles are recreational, in which case a lower mass/power ratio (probably 120 g/W) would be appropriate.

Table B.5.3.1a may be used as a quick reference to determine if the speed reduction warrant is met on a particular grade. The truck performance curves should be used together with other considerations to determine the exact start and end point of the climbing lane.

Table B.5.3.1a - Critical Length of Grade in Metres for a Speed Reduction of 15 km/h

Design Truck Mass/Power Rating	Grade in Percentage						
Metric Imperial	2	3	4	5	6	7	8
60 g/W (100lb/hp)	N/A	N/A	740	410	240	190	180
120 g/W (200 lb/hp	N/A	N/A	440	280	240	200	160
150 g/W (250 lb/hp)	730	360	280	220	170	140	-
180 g/W (300 lb/hp)*	**550**	**340**	**260**	**210**	**160**	**120**	**-**
200 g/W (325 lb/hp)	520	320	260	210	160	120	

Note: * 180 g/W is normally used for 2 lane highways.

1. Length of specified grade at which the designated design truck speed is reduced by 15 km/h from its entry speed (entry speed assumed to be 95 km/h)
2. Conversion factor: 1 g/W = 1.645 lb/hp
3. Values shown above have been rounded.

Volume Warrant: Two-Lane Undivided Highways

The volume warrant for climbing lanes on undivided roadways is shown in Tables B.5.3.1b through B.5.3.1e. This volume warrant is based on two conditions:

1. The heavy traffic (T) must exceed 150 veh/day

2. The level of service on the grade must drop below LOS A in the design hour on the two-lane roadway, that is, if the level of service on the grade in the design hour is LOS A, a climbing lane is not required.

Note: T is defined as the total number of tractor trailer-combinations and single unit trucks plus half of the recreational vehicles plus half of the buses. Buses and recreational vehicles generally perform better than trucks on grades.

If the traffic volume required for the warrant is projected to occur in the first half of the design life, the climbing lane shall be considered warranted, that is, it is not necessary to justify a climbing lane based on existing traffic volume.

On particular projects, it may be possible to show that the construction of climbing lanes is cost effective even though the traffic volume does not meet the general warrant. In this case, the construction of climbing lanes would be considered good design.

The daily volume to be used for design purposes is generally the AADT, unless the ASDT or AWDT is more than 15 percent greater than AADT, in which case the higher number should be used.

The reasons for recommending this volume warrant are as follows:

1. It is necessary to choose a minimum volume for which climbing lanes would be built. Use of the level of service criteria alone could result in some very low volume roads warranting climbing lanes, even though they are not cost effective, based on collision reduction or road user savings. A volume of 150 heavy vehicles per day was chosen because this represents one heavily loaded vehicle travelling in the upgrade direction for every 10 minutes in the design hour, that is, 150 heavy veh/day = 22 heavy vehicles/hour = 11 loaded heavy vehicles/hour = 6 loaded heavy vehicles/direction/hour. This is based on the assumption that the design hour volume equals 15 percent of the AADT. The presence of one loaded heavy vehicle travelling in the upgrade direction every 10 minutes in the design hour (30th highest hour of the design year) does not represent a serious congestion problem nor would it normally be a serious safety problem.

2. A warrant which is based on volume only without consideration of length of grade, steepness and traffic composition would be too simplistic. Alberta's warrant considers all those variables by using the level of service on the upgrade and the minimum number of heavy vehicles.

3. A review of the geometry and traffic conditions at 91 existing climbing lanes on Alberta's primary and secondary highway system shows that neither the volume nor the level of service criteria recommended in this warrant are too high. In fact, there are some climbing lanes in Alberta that have lower volume and higher level of service than required by the warrant.

Although 150 heavy vehicles/day is suggested as a general warrant, it is noted that inclusion of climbing lanes in low volume situations should be considered if shown to be cost-effective. Construction of climbing lanes may be less costly on new construction projects or on projects where the existing or proposed shoulder is wide. The benefits of providing climbing lanes may be greater if:

1. There is a high percentage of loaded trucks in the upgrade traffic stream

2. If the geometry of the highway, prior to the grade, is very restrictive for passing, thus resulting in a high demand for passing.

To be considered cost-effective, the benefits (considering road user costs, time savings and reduction in collision costs) should be sufficient to give a four percent internal rate of return on the extra investment required for construction of the climbing lane. The four percent internal rate of return should result before the end of the design life of the improvement. This is usually 20 years but may be less if future twinning is scheduled. The department's Benefit Cost Analysis guidelines should be followed for the economic analysis.

The following is an example of the use of the climbing lane warrant for two-lane highways.

Example of Use of Climbing Lane Warrant for Two-Lane Highways

Listed below is the geometric and traffic information for a particular segment of two-lane roadway where construction of a climbing lane is being considered.

Design Designation:	RAU-211.8-110
Length of Grade:	500m
Average Gradient:	3%
Percentage of Passing Zones on Upgrade Segment:	50%, i.e., on a 2 lane roadway 50% of the centreline would be painted as a barrier zone.
Design Truck:	180 g/W
Design/Existing AADT:	2000/1333 Based on 20 year design life and 2.5% annual growth not compounded.
Traffic Composition:	TRTL: 8% SU: 3% RV: 6% BUS: 2% PV: 81%
Design Hour Factor (K)	0.15, i.e., Design Hour Volume = Design AADT x 0.15 = 300

Step 1: Check Speed Reduction Warrant

According to Table B.5.3.1a, a 15 km/h speed reduction would have occurred after 340 m at three percent using a 180 g/w design truck. Therefore, speed reduction warrant is definitely met on a 500 m long three percent grade. The speed distance charts should be used to locate the points at which the 15 km/h speed reduction occurs on both the deceleration and acceleration portions of the vertical alignment.

Step 2: Check Traffic Volume Warrant

Based on the traffic composition, a value for T is calculated as shown below:

$$T = TRTL + SU + 1/2\,(RV + BUS)$$
$$= 8 + 3 + 1/2\,(6 + 2)$$
$$= 15\%$$

Because the percentage of passing zones on the segment in question is 50 percent, Table B.5.3.1d is used as the volume warrant. The assumptions used are all consistent with this design designation and the traffic characteristics shown.

The minimum volume required to warrant a climbing lane for a three percent grade of 500 m length with T = 15% is 1,871 AADT.

The existing AADT is 1333.

The design AADT is 2000, that is after 20 years with 2.5 percent annual growth not compounded.

The projected AADT for the 16th year is 1,866; that is, 1333 x [1 + 16 (0.025)].

The projected AADT for the 17th year is 1,899; that is, 1333 x [1 + 17 (0.025)].

The volume warrant for this grade is only achieved in the 17th year and therefore a climbing lane is not warranted at this time.

Note: If the volume warrant was met on or before the 10th year, a climbing lane would be warranted.

TABLE B-5.3.1b VOLUME WARRANTS FOR TRUCK CLIMBING LANES ON TWO-LANE HIGHWAYS - PASSING OPPORTUNITY = 100%

(on the two-lane highway)

Grade %	Length km	AADT									
		T=5%	T=10%	T=13%	T=15%	T=17%	T=20%	T=25%	T=30%	T=40%	T=50%
3	0.5	3,658	3,190	2,935	2,807	2,722	2,578	2,320	2,147	1,847	1,587
	1.0	3,374	2,833	2,610	2,441	2,315	2,147	1,933	1,727	1,447	1,273
	1.5	3,000	2,406	2,160	2,000	1,873	1,713	1,527	1,360	1,127	967
	2.0	3,000	1,660	1,433	1,300	1,220	1,067	927	820	633	567
	3.0	3,000	1,500	1,154	1,000	882	750	600	500	375	300
4	0.5	3,196	2,728	2,494	2,378	2,260	2,107	1,893	1,733	1,460	1,273
	1.0	3,000	2,265	2,040	1,887	1,773	1,640	1,453	1,260	1,040	893
	1.5	3,000	1,713	1,500	1,373	1,267	1,153	987	880	707	607
	2.0	3,000	1,500	1,154	1,000	882	750	600	500	375	300
5	0.5	3,000	2,084	1,880	1,780	1,687	1,553	1,360	1,247	1,013	893
	1.0	3,000	1,593	1,413	1,287	1,213	1,087	933	847	667	580
	1.5	3,000	1,500	1,154	1,000	882	750	600	500	387	333
	2.0	3,000	1,500	1,154	1,000	882	750	600	500	375	300
6	0.5	3,000	1,500	1,154	1,000	882	760	680	607	500	433
	1.0	3,000	1,500	1,154	1,000	882	750	600	500	375	300
7	0.5	3,000	1,500	1,154	1,000	882	750	600	500	375	300

Warrant: AADT must exceed the numbers shown above to satisfy the traffic volume warrant for climbing lanes.

T = % heavy vehicles = % [T.R.T.L. + S.U. + ½ (R.V. + BUS)]

T.R.T.L. = Tractor Trailers
S.U. = Single Unit Trucks
R.V. = Recreational Vehicle

Assumptions: Design speed 110 km/h. Passing opportunities 100% (based on pavement markings) peak hour factor 0.92, directional distribution 60/40, lane width 3.7m, shoulder width at least 1.8m, K=0.15 (Design Hour Volume = K x AADT). This table is based on the principle that level of service A is acceptable on grade before climbing lane is warranted and that there should be at least 150 heavy vehicles on the grade each day before climbing lane is warranted on 2 lane roadways. With volumes exceeding the numbers shown on this chart, the heavy vehicle traffic (T) will exceed 150 per day and the level of service on the upgrade will be less than A in the design hour using the assumptions shown. Exceptions to this warrant may be made where cost-effectiveness is demonstrated. A designer should consider the traffic volume and composition over the design life of a facility when deciding whether or not a climbing lane is required. If the warrant is met at or before the 10th year on a project with a design life of 20 years, construction of a climbing lane is suggested.

TABLE B-5.3.1c VOLUME WARRANTS FOR TRUCK CLIMBING LANES ON TWO-LANE HIGHWAYS - PASSING OPPORTUNITY = 70%
(on the two-lane highway)

Grade %	Length km	AADT									
		T=5%	T=10%	T=13%	T=15%	T=17%	T=20%	T=25%	T=30%	T=40%	T=50%
3	0.5	3,000	2,488	2,280	2,187	2,120	2,007	1,807	1,673	1,440	1,233
	1.0	3,000	2,200	2,027	1,900	1,800	1,667	1,507	1,340	1,127	993
	1.5	3,000	1,873	1,680	1,553	1,460	1,333	1,187	1,060	873	753
	2.0	3,000	1,500	1,154	1,007	947	833	720	640	493	440
	3.0	3,000	1,500	1,154	1,000	882	750	600	500	375	300
4	0.5	3,000	2,127	1,947	1,853	1,760	1,640	1,473	1,353	1.133	993
	1.0	3,000	1,767	1,593	1,473	1,387	1,280	1,133	980	813	700
	1.5	3,000	1,500	1,173	1,073	987	900	767	687	553	473
	2.0	3,000	1,500	1,154	1,000	882	750	600	500	375	300
5	0.5	3,000	1,540	1,387	1,313	1,247	1,147	1,007	920	747	660
	1.0	3,000	1,500	1,154	1,000	893	800	687	627	493	427
	1.5	3,000	1,500	1,154	1,000	882	750	600	500	375	300
6	0.5	3,000	1,500	1,154	1,000	882	750	600	500	375	320
	1.0	3,000	1,500	1,154	1,000	882	750	600	500	375	300
7	0.5	3,000	1,500	1,154	1,000	882	750	600	500	375	300

Warrant: AADT must exceed the numbers shown above to satisfy the traffic volume warrant for climbing lanes.
T = % heavy vehicles = % [T.R.T.L. + S.U. + ½ (R.V. + BUS)]

T.R.T.L. = Tractor Trailers
S.U. = Single Unit Trucks
R.V. = Recreational Vehicle

Assumptions: Design speed 110 km/h. Passing opportunities 70% (based on pavement markings), peak factor 0.92, directional distribution 60/40, lane width 3.7m, shoulder width at least 1.8m. K=0.15 (Design Hour Volume = K x AADT). This table is based on the principle that level of service A is acceptable on grade before climbing lane is warranted on 2 lane roadways. With volumes exceeding the numbers shown on this chart, the heavy vehicle traffic (T) will exceed 150 per day and the level of service on the upgrade will be less than A in the design hour using the assumptions shown. Exceptions to this warrant may be made where cost-effectiveness is demonstrated. A designer should consider the traffic volume and composition over the design life of a facility when deciding whether or not a climbing lane is required. If the warrant is met at or before the 10th year on a project with a design life of 20 years, construction of a climbing lane is suggested.

TABLE B-5.3.1d VOLUME WARRANTS FOR TRUCK CLIMBING LANES ON TWO-LANE HIGHWAYS - PASSING OPPORTUNITY = 50% (on the two-lane highway)

Grade %	Length km	AADT									
		T=5%	T=10%	T=13%	T=15%	T=17%	T=20%	T=25%	T=30%	T=40%	T=50%
3	0.5	3,000	2,127	1,956	1,871	1,815	1,719	1,547	1,433	1,233	1,060
	1.0	3,000	1,888	1,740	1,628	1,547	1,433	1,935	1,153	967	853
	1.5	3,000	1,604	1,440	1,333	1,253	1,140	1,020	907	747	647
	2.0	3,000	1,500	1,154	1,000	882	750	620	547	420	373
	3.0	3,000	1,500	1,154	1,000	882	750	600	500	375	300
4	0.5	3,000	1,855	1.696	1,617	1,540	1,433	1,287	1,180	993	867
	1.0	3,000	1,541	1,387	2,187	1,207	1,113	987	853	707	607
	1.5	3,000	1,500	1,154	1,000	882	787	673	593	480	413
	2.0	3,000	1,500	1,154	1,000	882	750	600	500	375	300
5	0.5	3,000	1,500	1,160	1,100	1,040	960	840	773	627	553
	1.0	3,000	1,500	1,154	1,000	882	750	600	527	413	360
	1.5	3,000	1,500	1,154	1,000	882	750	600	500	375	300
6	0.5	3,000	1,500	1,154	1,000	882	750	600	500	375	300
7	0.5	3,000	1,500	1,154	1,000	882	750	600	500	375	300

Warrant: AADT must exceed the numbers shown above to satisfy the traffic volume warrant for climbing lanes.

T = % heavy vehicles = % [T.R.T.L. + S.U. + ½(R.V. + BUS)]

T.R.T.L. = Tractor Trailers
S.U. = Single Unit Trucks
R.V. = Recreational Vehicle

Assumptions: Design speed 110 km/h. Passing opportunities 50% (based on pavement markings), peak hour 0.92, directional distribution 60/40, lane width 3.7m, shoulder width at least 1.8m, K=0.15 (Design Hour Volume = K x AADT). This table is based on the principle that level of service A is acceptable on grade before climbing lane is warranted and that there should be at least 150 heavy vehicles on the grade each day before climbing lane is warranted on 2 lane roadways. With volumes exceeding the numbers shown on this chart, the heavy vehicle traffic (T) will exceed 150 per day and the level of service on the upgrade will be less than A in the design hour using the assumptions shown. Exceptions to this warrant may be made where cost-effectiveness is demonstrated. A designer should consider the traffic volume and composition over the design life of a facility when deciding whether or not a climbing lane is required. If the warrant is met at or before the 10th year on a project with a design life of 20 years, construction of a climbing lane is suggested.

TABLE B-5.3.1e VOLUME WARRANTS FOR TRUCK CLIMBING LANES ON TWO-LANE HIGHWAYS - PASSING OPPORTUNITY = 30% (on the two-lane highway)

Grade %	Length km	AADT									
		T=5%	T=10%	T=13%	T=15%	T=17%	T=20%	T=25%	T=30%	T=40%	T=50%
3	0.5	3,000	1,831	1,685	1,612	1,563	1,480	1,333	1,233	1,060	913
	1.0	3,000	1,626	1,498	1,402	1,327	1,233	1,113	993	833	733
	1.5	3,000	1,500	1,241	1,147	1,080	980	873	780	647	553
	2.0	3,000	1,500	1,154	1,000	882	750	600	500	360	327
	3.0	3,000	1,500	1,154	1,000	882	750	600	500	375	300
4	0.5	3,000	1,582	1,447	1,379	1,313	1,220	1,093	1,007	847	740
	1.0	3,000	1,500	1,180	1,093	1,027	953	840	733	607	520
	1.5	3,000	1,500	1,154	1,000	882	750	600	507	413	353
	2.0	3,000	1,500	1,154	1,000	882	750	600	500	375	300
5	0.5	3,000	1,500	1,154	1,100	882	813	713	653	533	467
	1.0	3,000	1,500	1,154	1,000	882	750	600	500	375	300
6	0.5	3,000	1,500	1,154	1,000	882	750	600	500	375	300
7	0.5	3,000	1,500	1,154	1,000	882	750	600	500	375	300

Warrant: AADT must exceed the numbers shown above to satisfy the traffic volume warrant for climbing lanes.

T = % heavy vehicles = % [T.R.T.L. + S.U. + ½ (R.V. + BUS)]

T.R.T.L. = Tractor Trailers
S.U. = Single Unit Trucks
R.V. = Recreational Vehicle

Assumptions: Design speed 110 km/h. Passing opportunities 30% (based on pavement markings) peak hour 0.92, directiional distribution 60/40, lane width 3.7m, shoulder width at least 1.8m, K=0.15 (Design Hour Volume = K x AADT). This table is based on the principle that level of service A is acceptable on grade before climbing lane is warranted and that there should be at least 150 heavy vehicles on the grade each day before climbing lane is warranted on 2 lane roadways. With volumes exceeding the numbers shown on this chart, the heavy vehicle traffic (T) will exceed 150 per day and the level of service on the upgrade will be less than A in the design hour using the assumptions shown. Exceptions to this warrant may be made where cost-effectiveness is demonstrated. A designer should consider the traffic volume and composition over the design life of a facility when deciding whether or not a climbing lane is required. If the warrant is met at or before the 10th year on a project with a design life of 20 years, construction of a climbing lane is suggested.

B.5.3.2 Climbing Lane Warrant for Four-Lane Divided Highways

The addition of climbing lanes to four-lane divided highways need not be considered if the AADT is less than 12,000, regardless of grades or percentages of trucks, because of the generally high level of service provided by a four-lane divided facility with this traffic volume. If the AADT exceeds 12,000 and the design truck experiences a speed reduction exceeding 15 km/h, the level of service on the upgrade segment in the design hour should be compared to the level of service on the approach segment. If there is a reduction of at least one complete level of service when going from the approach segment to the upgrade, a climbing lane is warranted.

There has been little application of climbing lanes to divided highways in Alberta to date, due to the generally high level of service that exists on provincial divided facilities.

B.5.3.3 Determining Length and Location of Climbing Lanes

Once the need for a climbing lane has been established by satisfying the speed reduction and traffic volume warrants, the exact start and end points and length are determined using the truck performance curves (Figures B-5.3.3b through B-5.3.3k).

The following example illustrates the use of the truck performance curves.

Example of use of truck performance curves

The vertical alignment and truck performance curves are shown on Figure B-5.3.3a. The design truck is assumed to have a mass/power ratio of 180 g/W, as this is the standard truck. The dashed lines superimposed on the performance curves of Figure B-5.3.3a show the plot of the design truck speed throughout the alignment section as follows.

1. Entry speed = 95 km/h (assumed) at PI #1 (point of intersection)
2. Truck decelerates to 52 km/h at PI #2 due to 800m upgrade at four percent
3. Truck decelerates to crawl speed (26 km/h) due to 600m upgrade at six percent

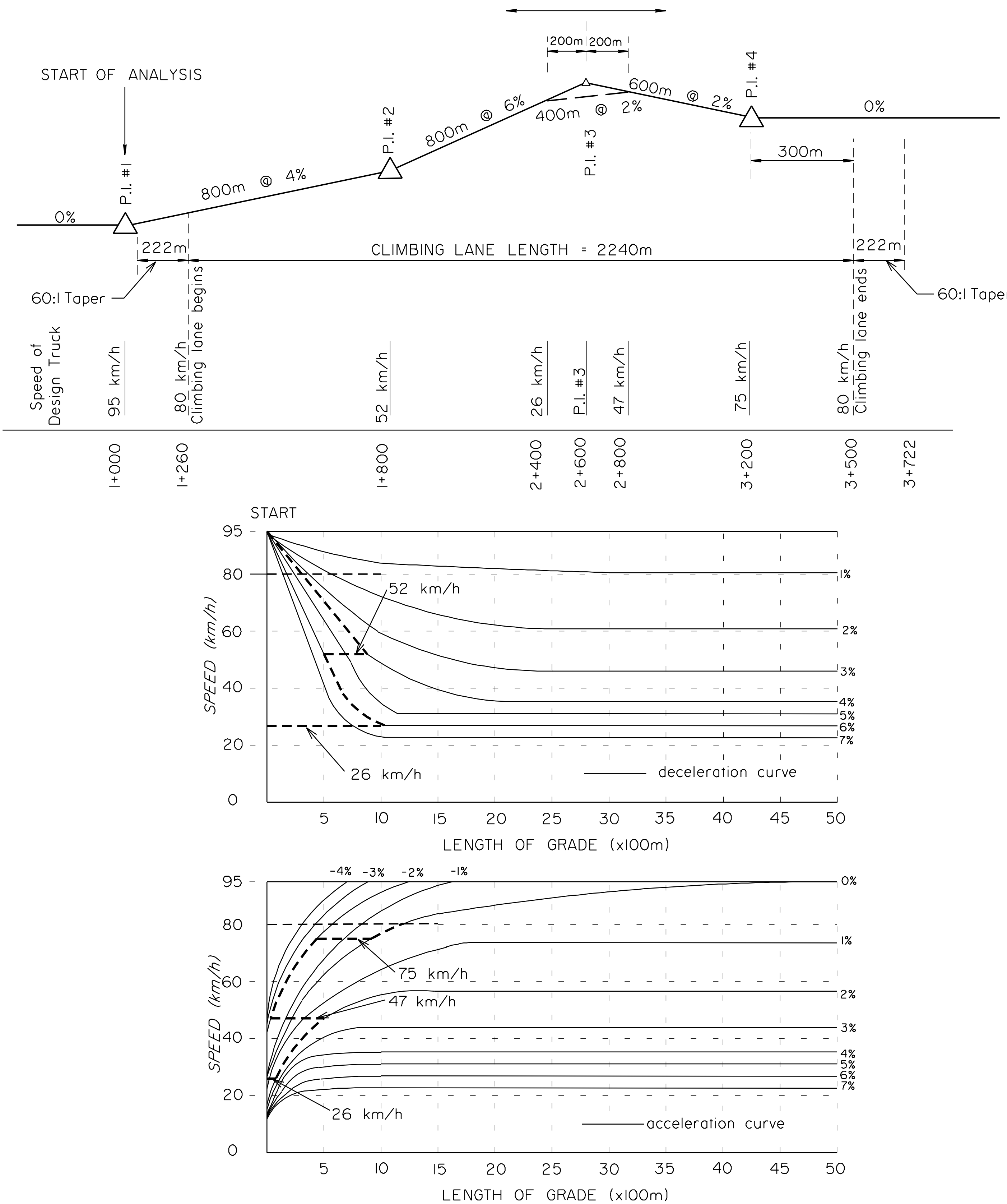
The design truck now experiences a grade change whose algebraic difference exceeds four percent; that is, +6% - (-2%) = +8%.

When the algebraic difference exceeds four percent, the vertical curve connecting the grades is approximated through the average grades connecting the quarter points on the semi-tangents of the vertical curve. These quarter points act as new PI's for the purpose of estimating the design vehicle speed. In this example, the length of the vertical curve is 800m. Therefore the quarter points occur at 200m on either side of the real PI and the grade connecting the quarter points has been estimated at two percent. This approximated grade, 400m in length, reduces the length of the preceding and following grades by 200m each. (The dashed line now enters the acceleration portion of the chart, as the design truck accelerates on the two percent upgrade).

4. Truck accelerates from crawl speed (26 km/h) to 47 km/h on the 400m, two percent upgrade
5. Truck accelerates from 47 km/h to 75 km/h at PI #4 on the 400m, two percent downgrade
6. Truck accelerates from 75 km/h to 80 km/h (the merge speed) on a 300m, zero percent grade.

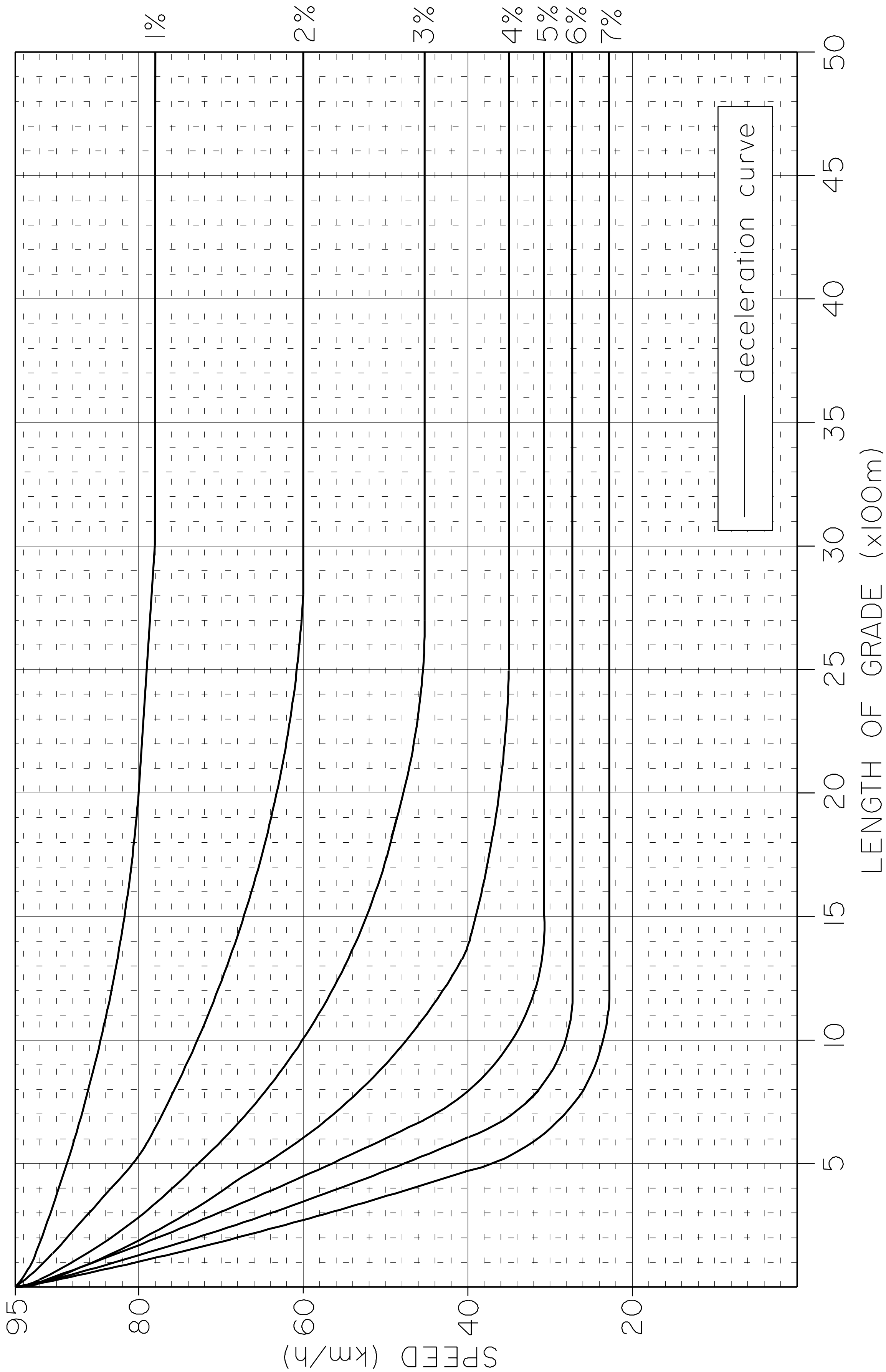
As per the plot shown on Figure B.5.3.3a, the climbing lane should begin when the design truck speed reaches 80 km/h (this occurs at 1+260). The 60:1 taper should be introduced before this point. The end point of the climbing lane can be placed anywhere after the merge speed has been achieved, that is, after 3+500, provided that the decision sight distance is available. The merge taper is placed after the end of climbing lane.

FIGURE B-5.3.3a CLIMBING LANE DESIGN EXAMPLE



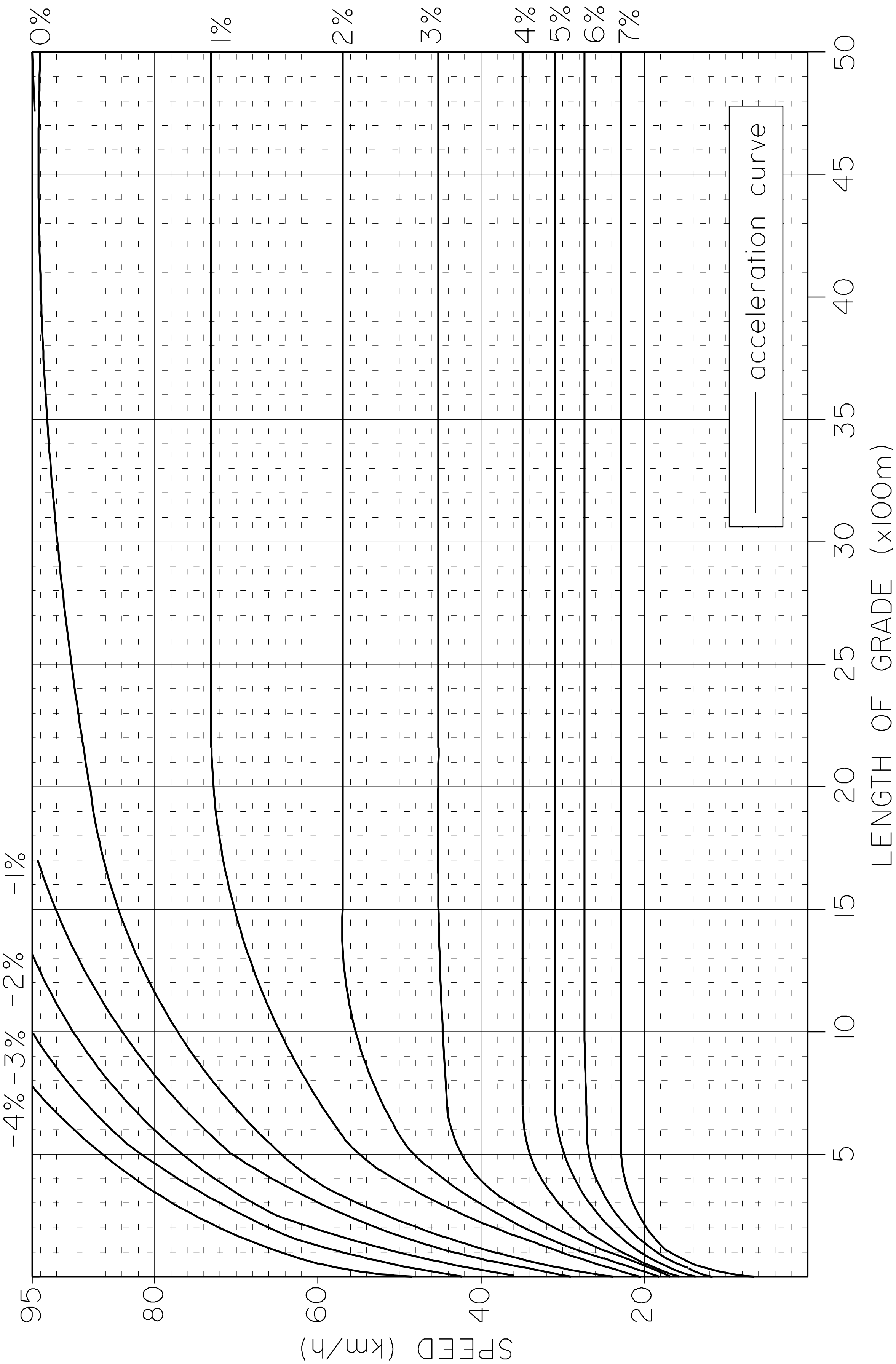
Performance Curves for Heavy Trucks (180g/W)
Adopted from Highway Capacity Manual 1985

FIGURE B-5.3.3b PERFORMANCE CURVES FOR HEAVY TRUCKS 180 g/w
DECELERATION CURVE



Adapted from Highway Capacity Manual, Figure I.3-4, published by
Transportation Research Board, National Research Council, Washington D.C. (1985)

FIGURE B-5.3.3c PERFORMANCE CURVES FOR HEAVY TRUCKS 180 g/w
ACCELERATION CURVE



Adapted from Highway Capacity Manual, Figure I.3-4, published by
Transportation Research Board, National Research Council, Washington D.C. (1985)

FIGURE B-5.3.3d PERFORMANCE CURVES FOR HEAVY TRUCKS 200 g/w
DECELERATION CURVE

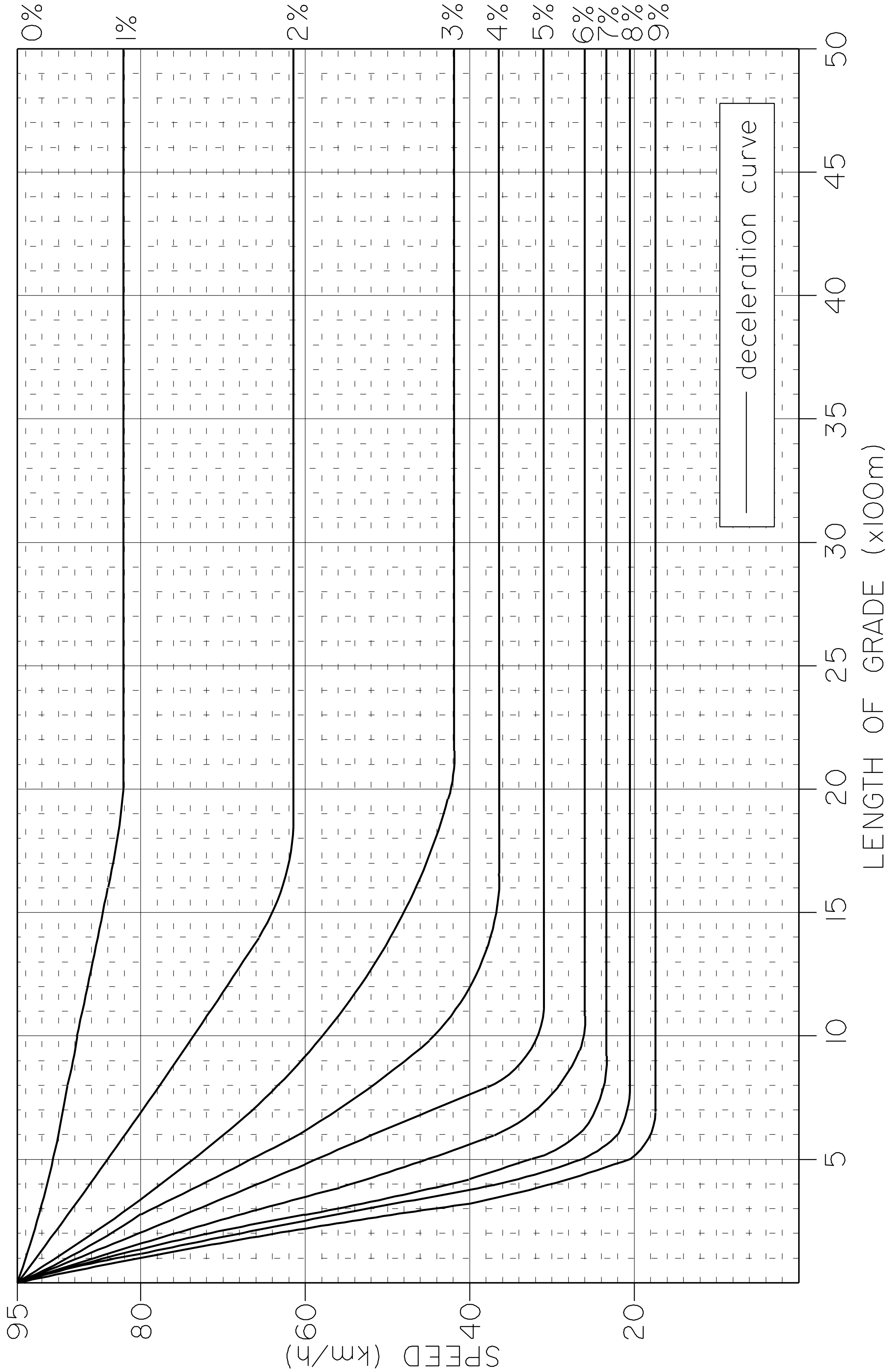
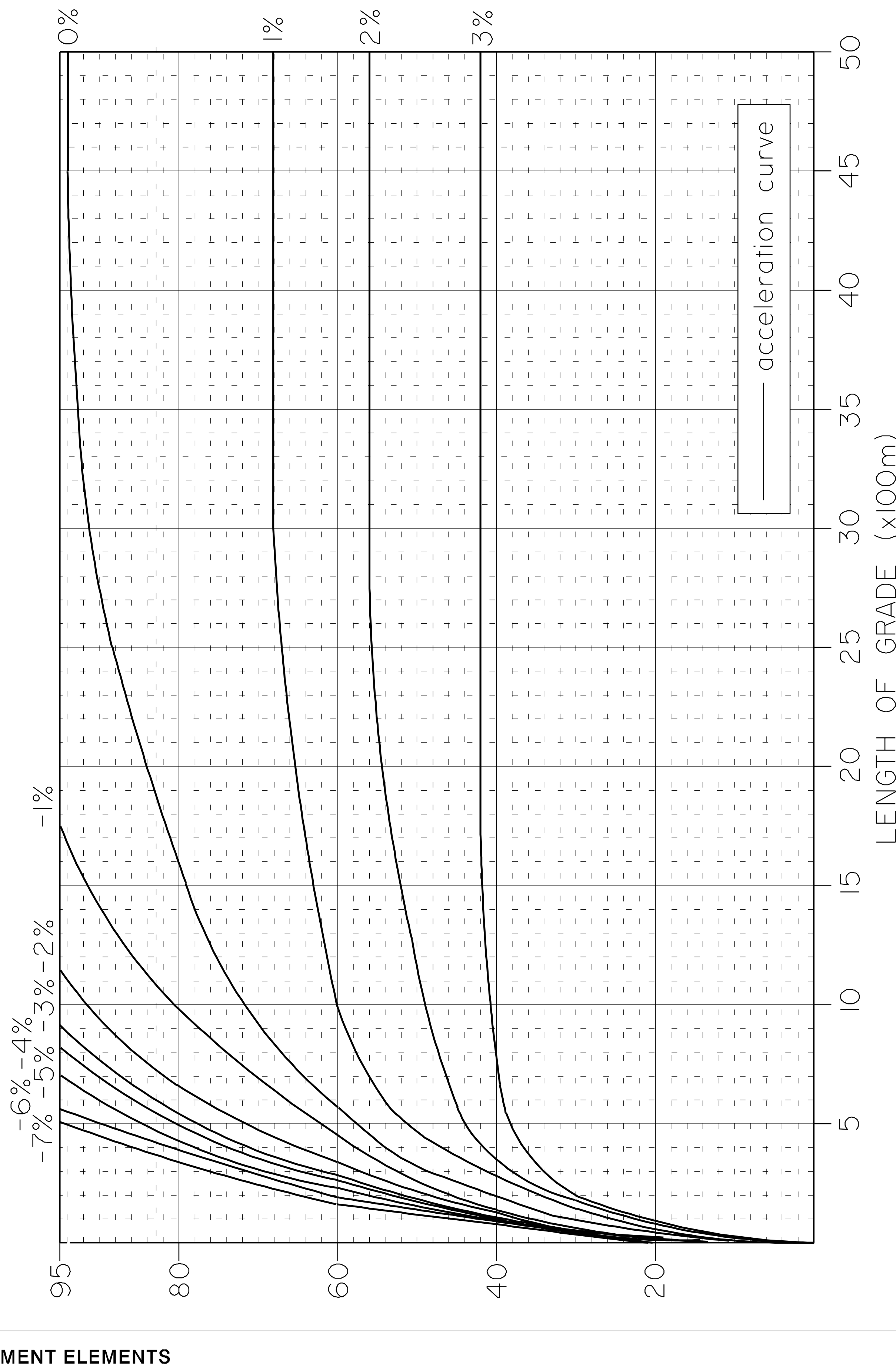


FIGURE B-5.3.3e PERFORMANCE CURVES FOR HEAVY TRUCKS 200 g/w
ACCELERATION CURVE



GRAPHICS FILE: TIGSFS01\REB\MANUAL\CHAPTERS\CHAP-B\DEBB533E.MAN

FIGURE B-5.3.3f PERFORMANCE CURVES FOR HEAVY TRUCKS 150 g/w
DECELERATION CURVE

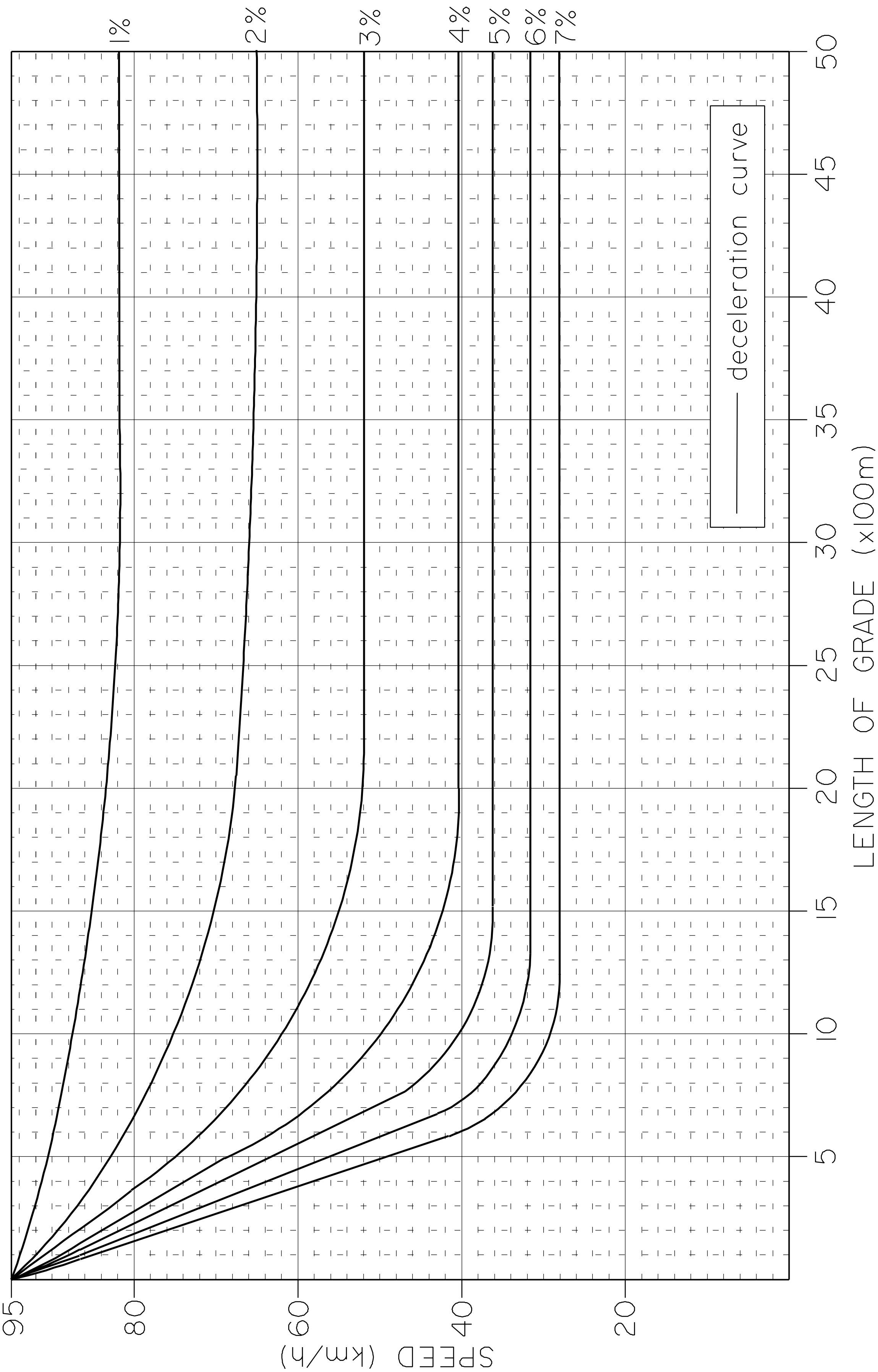
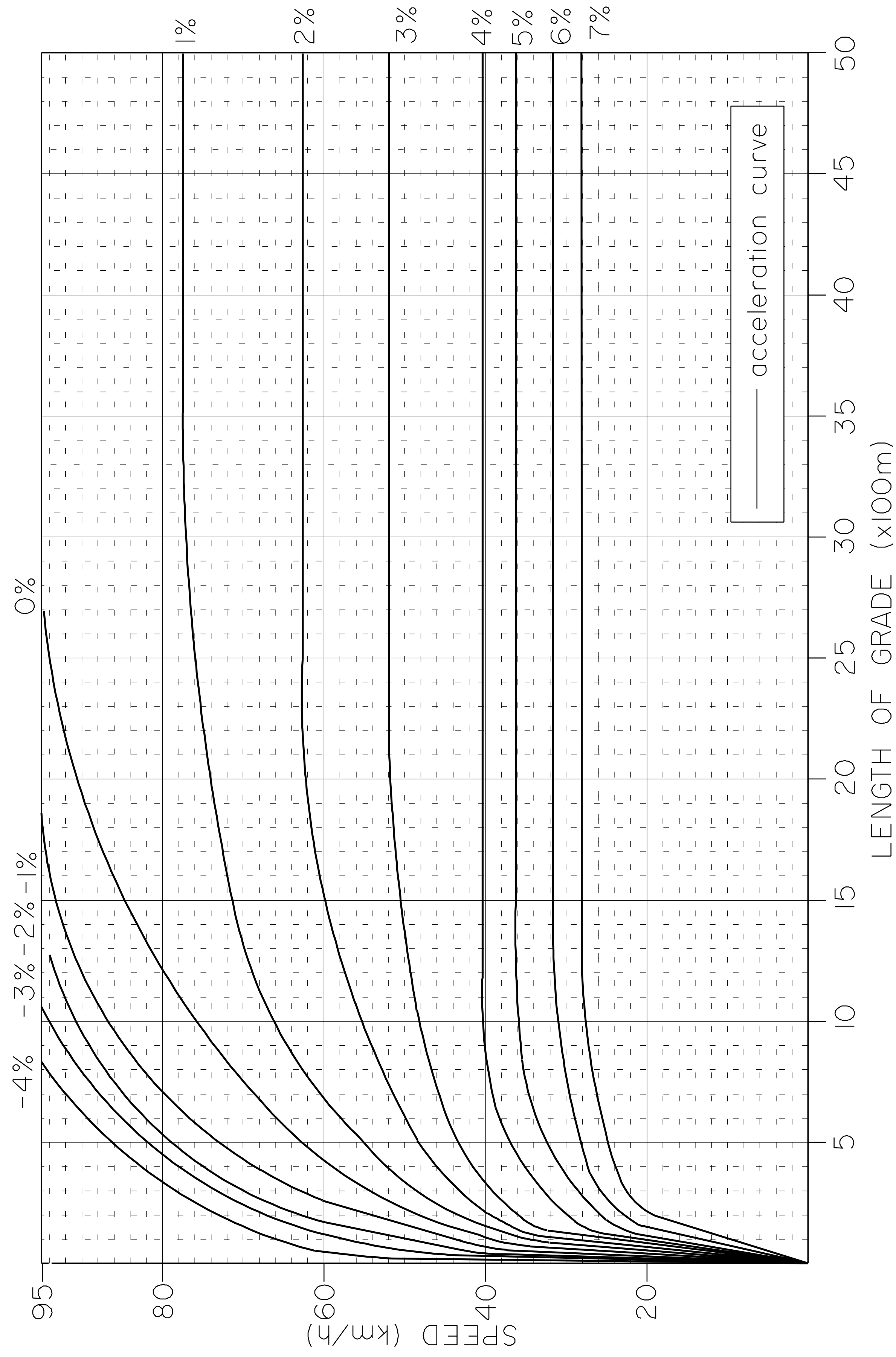


FIGURE B-5.3.3g PERFORMANCE CURVES FOR HEAVY TRUCKS 150 g/w
ACCELERATION CURVE



GRAPHICS FILE: TIGSFS01\REB\MANUAL\CHAPTERS\CHAP-B\DEBB533G.MAN

FIGURE B-5.3.3h PERFORMANCE CURVES FOR HEAVY TRUCKS 120 g/w
DECELERATION CURVE

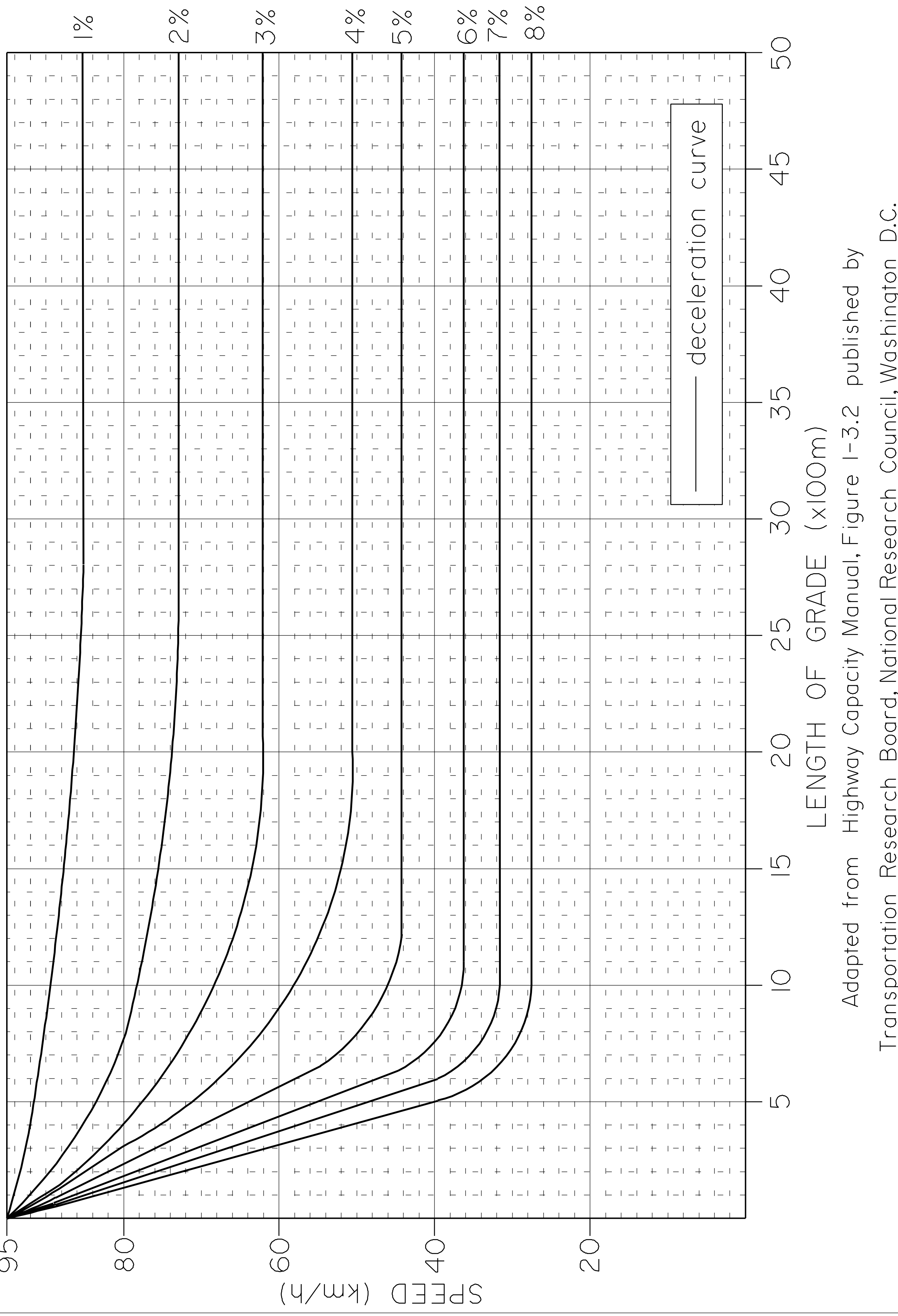
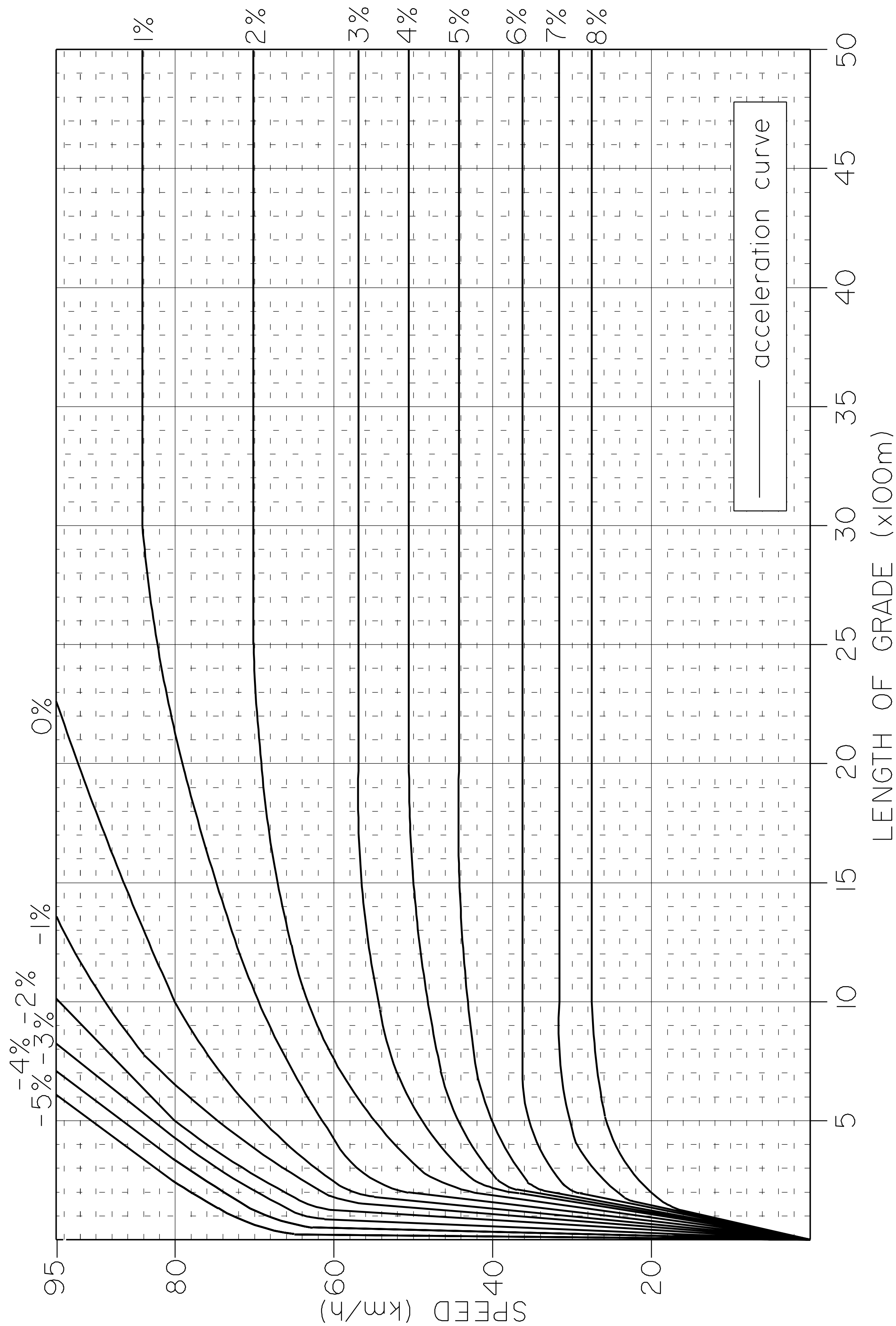


FIGURE B-5.3.3i PERFORMANCE CURVES FOR HEAVY TRUCKS 120 g/w
ACCELERATION CURVE



Adapted from Highway Capacity Manual, Figure I-3.2, published by
Transportation Research Board, National Research Council, Washington D.C. (1985)

FIGURE B-5.3.3j PERFORMANCE CURVES FOR HEAVY TRUCKS 60 g/w
DECELERATION CURVE

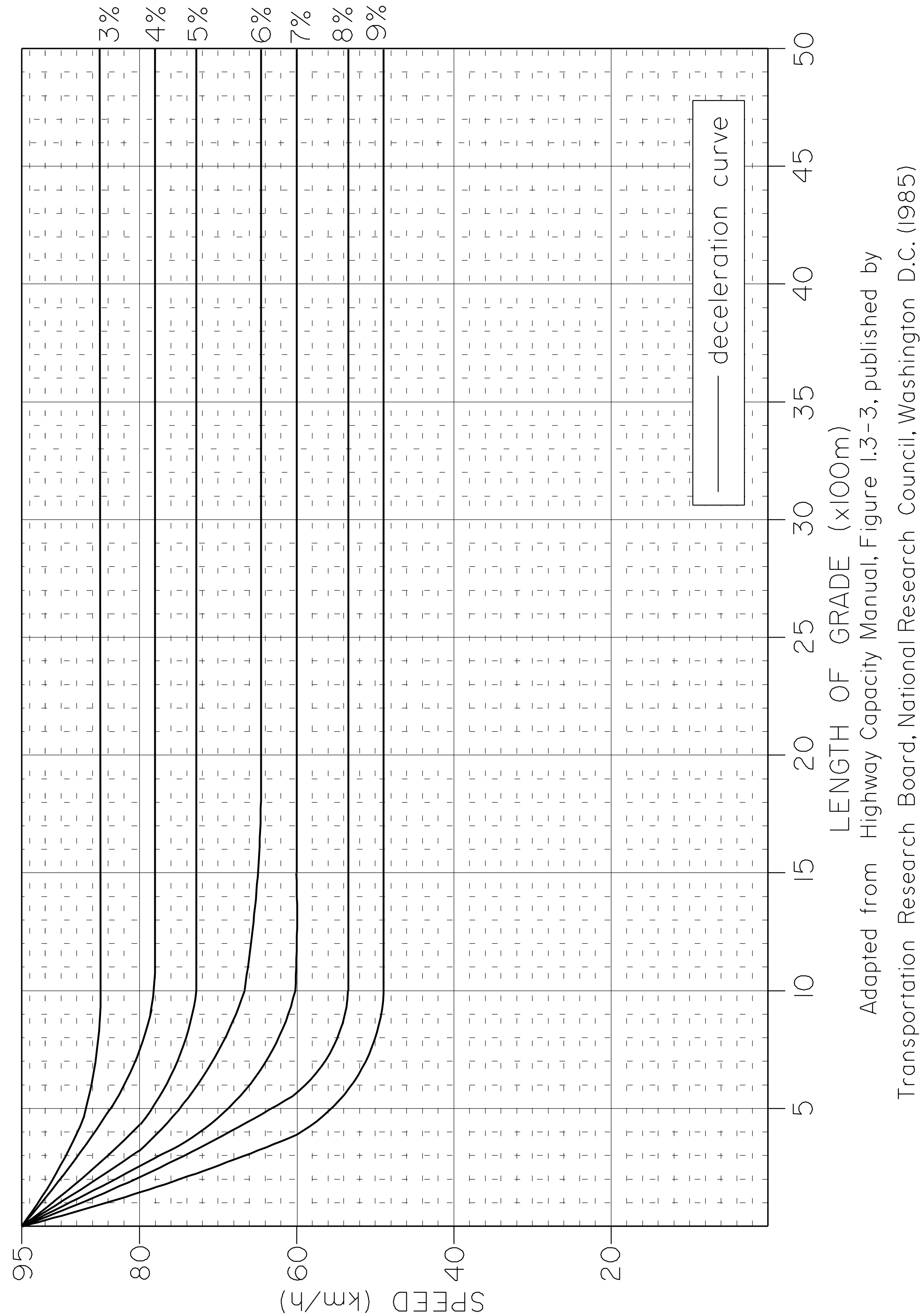
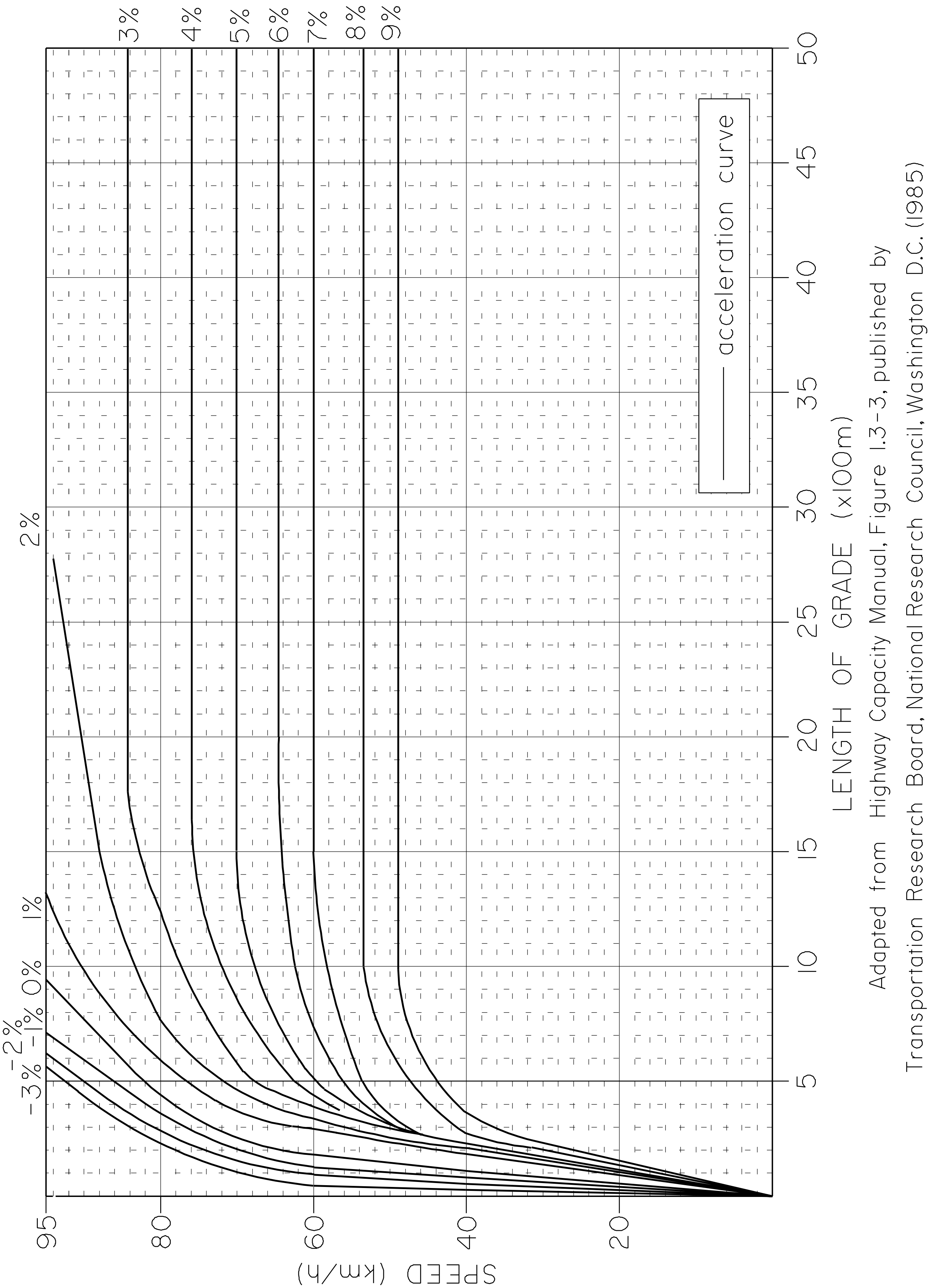


FIGURE B-5.3.3k PERFORMANCE CURVES FOR HEAVY TRUCKS 60 g/w

ACCELERATION CURVE



B.5.4 Passing Lanes

Passing lanes are additional parallel auxiliary lanes provided, on two-lane undivided highways, for the exclusive purpose of improving passing opportunities. Passing lanes should be considered as a cost-effective geometric improvement on two-lane roads where: the length and location of passing zones on the existing highway are less than desirable, and the traffic volume is high enough that the level of service is noticeably low. Passing lanes should also be considered on new construction or major realignment projects to achieve the desired level-of-service. Passing lanes may also be a cost-effective solution where:

1. Volumes on a two-lane highway are increasing and will soon warrant twinning

2. Where the provision of passing lanes may postpone the construction of a divided facility, for example for five to 10 years.

B.5.4.1 Passing Lane Warrant

To establish the need for passing lanes on an existing two-lane undivided rural highway, the net passing opportunity (NPO) concept is to be used.

The following is a brief description of the net passing opportunity concept and how it should be applied:

Net passing opportunity is a function of both passing opportunities provided by highway geometry and the number of gaps in the opposing traffic stream. The probability of time gaps greater than 30 seconds available for overtaking, known as P(GAO), can be estimated using the following formula:

$$P(GAO) = e^{(-0.0023381\ Vopp)}$$

Where Vopp = opposing traffic volume in vehicles per hour

Values for P(GAO) for various values of Vopp have been tabulated in Table B.5.4.1a. The net passing opportunity for one direction on a particular segment of highway in the hour of interest is equal to the product of the P(GAO) for that hour and the percentage of passing zones (% PZ) available according to pavement markings.

$$NPO = P(GAO) \times (\%\ PZ)$$

NPO = Net Passing Opportunity

P(GAO) = Percentage of the hour with gaps (greater than 30 seconds) available for passing

% PZ = percentage of the road segment where passing is allowed by pavement markings.

For example NPO = 0.792 x 0.730 = 0.578 = 57.8%; that is, the net passing opportunity is 57.8% on a roadway with 73% passing zones if the probability of gaps is 79.2% (this corresponds to an opposing volume of 100 veh / h).

To establish a warrant for provision of passing lanes, each direction of travel should be examined separately. This is because the NPO in one direction may be very low while it is satisfactory in the other direction, depending on traffic and geometric conditions.

Table B.5.4.1a
Percent of the Hour With Gaps Available for Overtaking as a Function of Volume.

Opposing Volume, Vopp (veh/h)	Probability of Time Gaps Available for Overtaking P(GAO)
50	0.890
100	0.792
150	0.704
200	0.627
250	0.557
300	0.496
400	0.393
500	0.311
600	0.246
Note: $P(GAO) = e^{(-0.0023381\ Vopp)}$	

For passing lane warrant purposes the Design Hour Volume (DHV) for the roadway segment of interest is to be used. The DHV is calculated as follows: DHV = K (Design AADT). Where the K value is unknown, 0.15 may be assumed. The DHV will normally correspond to the 30th highest hourly volume in the design year (approximately).

To obtain values for NPO based on opposing volume and percent no passing zones, Table B.5.4.1b may be used. It should be noted that the AADT values shown in Table B.5.4.1b have been calculated using AADT = DHV/0.15. AADT values, assuming various directional splits (50:50, 55:45, and 60:40) have been provided at the top of Table B.5.4.1b. If the traffic conditions in the design hour on the highway being studied differ from above, make the appropriate adjustments.

Alberta's passing lane warrant is shown in Table B.5.4.1b and can be summarized as follows:

1. If NPO ≥ 40%, the percentage of passing zones is satisfactory.

2. If 40% > NPO ≥ 30% , the percentage of passing zones is marginal.

3. If NPO < 30%, the passing improvement is warranted.

In Alberta, there is considerable variation in the percentage of passing zones available on various projects. The network-wide average availability is 73 percent on the paved roadways that have been videologged. The roadways with higher traffic volumes tend to have better geometry and therefore more passing zones.

For warranting purposes, designers should consider the net passing opportunity for traffic derived from the higher volume directional split in the design hour because this is the direction with the highest passing demand.

The directional split in the design hour has a significant impact on the net passing opportunity for a given AADT. This is why three lines are provided on Table B.5.4.1b to show the various AADT values that correspond to the three directional splits. A designer should use project specific traffic information where it is available, otherwise a 50:50 split should be assumed.

Passing demand is proportional to the square of the one way stream flow according to Wardrop's formula; that is:

$$\mathbf{P} = \frac{0.56\,\sigma\,\mathbf{Q}^2}{\mathbf{V}^2}$$

Where P = overtaking demand (overtakings/km/h)
Q = stream flow (veh/h)
V = mean unimpeded speed (km/h)
σ = standard deviation of the unimpeded speed distribution (km/h)

Therefore, with a 60:40 split, the passing demand in the high volume direction is 2.25 times higher than the passing demand in the low volume direction.

The construction of passing lanes is only one solution to the problem of a less than desirable level of service. Other possible solutions are geometric improvements, such as horizontal and/or vertical realignments (to provide more passing zones) or twinning, which would provide unlimited percentage of passing zones for both traffic streams. On existing paved roadways, the provision of additional passing opportunities through construction of passing lanes may be more cost-effective than realignment and general grade widening. However, in many cases geometric improvements may be desirable to reduce road-user costs or necessary for safety reasons.

On higher volume rural roadways, especially above 5000 AADT with K = 0.15 (that is, DHV > 300), it will be difficult to achieve a high level of service in the design hour without twinning. Therefore, passing lanes would have limited application. Passing lanes may, however, be cost effective on high volume undivided roadways if they can be used to temporarily alleviate traffic problems and hence delay the need to twin for a period, for example five to 10 years. If passing lanes can be installed on existing RAU-213.4 roadways by altering the pavement markings and without doing any grade widening, this can be a very cost effective improvement. However, the shoulders will be less than the desirable width. Therefore, this type of improvement should be considered as an interim measure only (less than five years).

Care must be taken when designing passing lanes to ensure that the overall two-directional Net Passing Opportunity is significantly improved by the addition of the passing lanes at the proposed locations. Otherwise, the cost effectiveness will be questionable.

TABLE B.5.4.Ib PASSING LANE WARRANT

Net passing opportunities for traffic in the high volume direction as a function of percent no passing zones and opposing volumes.

		NET PASSING OPPORTUNITIES (%)																
Opposing Volume (Vopp) Veh/h (one way)		50	100	150	175	200	225	250	275	300	325	350	375	400	425	450	500	600
AADT (2-way) assuming directional split 50:50		667	1,333	2,000	2,333	2,667	3,000	3,333	3,667	4,000	4,333	4,667	5,000	5,333	5,667	6,000	6,667	8,000
AADT (2-way) assuming directional split (55:45)		740	1,481	2,222	2,593	2,963	3,333	3,704	4,074	4,444	4,815	5,185	5,556	5,926	6,296	6,667	7,404	8,888
AADT (2-way) assuming directional split 60:40		830	1,670	2,500	2,920	3,330	3,750	4,170	4,580	5,000	5,420	5,830	6,250	6,660	7,080	7,500	8,340	10,000
Percent no passing zone according to current pavement marking standards, i.e., sight distance must exceed 425m using an eye height of 1.15m and an object height of 1.15m for passing to be permitted.	0	89	79	70	66	63	59	56	53	50	47	44	42	39	37	35	31	25
	10	80	71	63	60	56	53	50	47	45	42	40	37	35	33	31	28	22
	20	71	63	56	53	50	47	45	42	40	37	35	33	31	30	28	25	20
	30	62	55	49	46	44	41	39	37	35	33	31	29	28	26	24	22	17
	40	53	47	42	40	38	35	34	32	30	28	26	25	24	22	21	19	15
	50	46	40	35	33	31	30	28	26	25	23	22	21	20	19	17	16	12
	60	36	32	28	27	25	24	22	21	20	19	18	17	16	15	14	12	10
	80	18	16	14	13	13	12	11	11	10	9	9	8	8	7	7	6	5
	100	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Possible Improvements to achieve greater Net Passing Opportunities		Geometric Improvements (50 – 325)																
		Passing Lanes (150 – 500)																
		Twinning (250 – 600)																

Passing Opportunity is Satisfactory

Passing Opportunity

Is Marginal

Passing Improvement Warranted

Assumptions: $AADT = \frac{DHV}{0.15}$

NOTE: This table depicts Net Passing Opportunities (N.P.O.) for traffic in high volume direction in the design hour based on geometric and traffic characteristics. N.P.O. values above the solid line are considered satisfactory. When the N.P.O. is below the solid line, consideration should be given to improving the passing opportunity through geometric improvements, addition of passing lanes or twinning. When the volumes are relatively low (i.e., <3000) geometric improvements may be appropriate, where the volumes are at an intermediate level (i.e., 3000 to 5000) passing lanes may be the most effective method of providing the desirable N.P.O. and where the volume exceeds 5000 on a rural route (i.e., with K = 0.15) twinning should be considered. A detailed analysis may be required to estimate the timing of future twinning and determine if it is cost effective to construct passing lanes, advance the twinning or delay improvements.

B.5.4.2 Considerations for Location and Spacing of Passing Lanes

1. Passing lane locations should be chosen with a view to minimizing unnecessary costs. Locations requiring large culverts, bridge widenings, major cuts or major fills should be avoided where possible unless the construction of passing lanes is beneficial in terms of balancing earthwork quantities.

2. Locations that include or are in close proximity to intersections should be avoided because of possible operational difficulties. Where these situations cannot be avoided, the intersection should be relocated or a site specific analysis undertaken to determine the intersection treatment required.

3. It is important to ensure that the two-way percent passing zones will be improved as a result of passing lane construction. It must be remembered that if the existing AADT exceeds 4000, the pavement will be marked with a double barrier line through the passing lane section. Therefore, no passing will be allowed for the opposing traffic flow. Consequently, it is imperative to locate passing lanes in sections of limited passing zones to ensure overall passing opportunities are improved, especially where the existing AADT exceeds 4000.

4. The addition of passing lanes should not result in an imbalance in percentage of passing zones between the two directions of travel on an undivided roadway. To achieve a balance, it may be necessary to add passing zones for each direction of travel alternately.

   Where passing lanes are being added to both sides of a two-lane highway, it is generally preferable to place a passing lane after a zone of restricted passing rather than before. This is referred to as the tail-to-tail configuration. The after location allows the platoons, which have been built up over the zone of restricted passing, to be dissipated on the passing lane. Also, driver frustration in the zone of restricted passing may be alleviated by signs advising of the upcoming passing lane.

5. Passing lane locations should be selected based on review of geometric and traffic conditions in both the upstream and downstream directions. Passing lanes in close proximity to four-lane sections, or downstream from climbing lanes, are not particularly effective in improving the overall level of service.

6. Generally passing lanes should not exceed about 25 percent of the highway section length for each direction of travel. For example, on a 40 km section of highway, up to five passing lanes each two km in length, could be constructed in each direction. This provides an average spacing of six km between the beginning of successive passing lanes in the same direction.

B.6 TYPICAL HIGHWAY TRANSITIONS

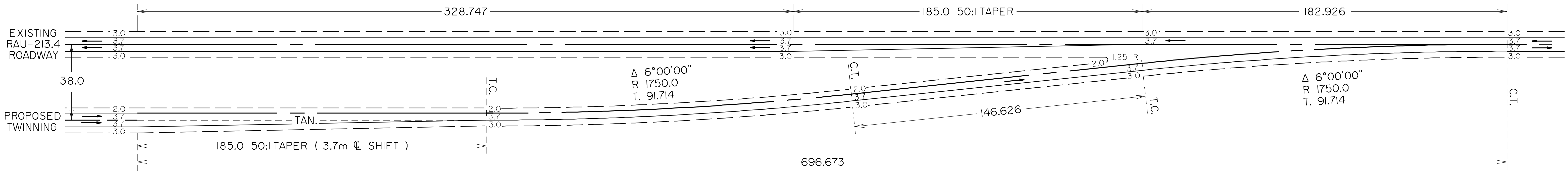
B.6.1 Introduction

Typical highway transitions from four-lane divided to two-lane undivided and the reverse are shown in Figures B-6.1a and B-6.1b. Figure B-6.1a shows transitions based on a 38m centreline offset while Figure B-6.1b shows transitions based on a 30m centreline offset.

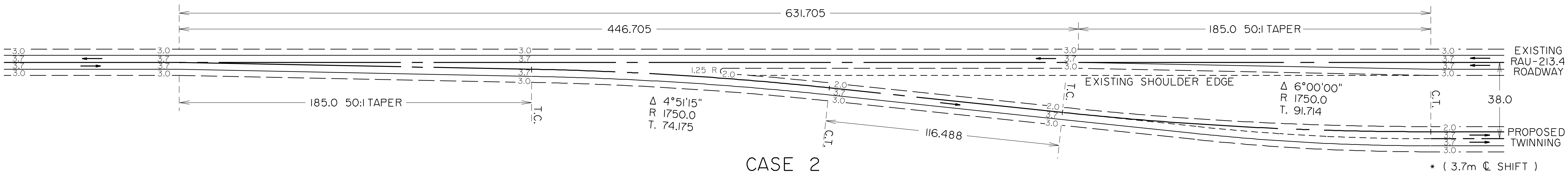
B.6.2 Construction Practices at Typical Highway Transitions

On projects where twinning of an existing two-lane facility is expected to continue over the next construction season or within the next five years, a parallel alignment section of the new two-lane, one-way roadway, should be built. This extension (grading and base course) beyond the point at which transition back to a two-lane undivided highway occurs is called a stub. The stub of the new roadway should continue until there is a physical separation between the temporary transition section and the new two-lane, one-way roadway. The stub should proceed with the -0.02 m/m cross-slope, extending across the temporary transition section, until the gore has occurred between the two roadways. Beyond the gore, the transition roadway section may be rotated, attaining the proper cross-slope (-0.02 m/m), at the point where transition back to a two-lane undivided highway is complete. Superelevation on the curved portion of the transition is usually kept to two percent to match the future crown. However, it is built low to allow for one additional lift of ACP.

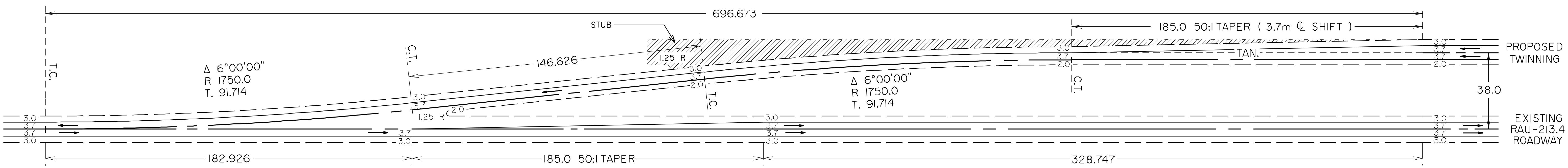
The benefit of this construction procedure is that traffic on the transition lane is not disrupted when construction resumes on the next twinned portion. There is also a cost savings in not having to reconstruct the transition portion of the highway.



CASE 1



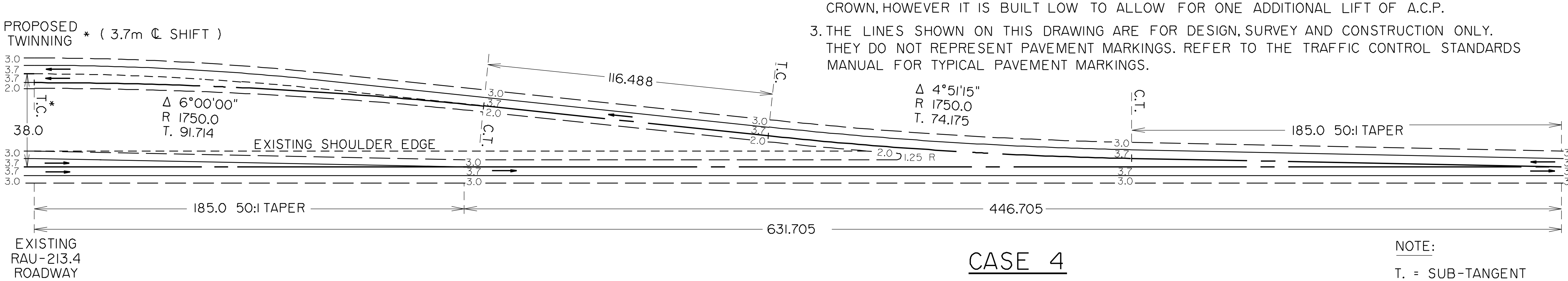
CASE 2



CASE 3

NOTES

1. TRANSITIONS ARE OFTEN BUILT WITH A 'STUB ' FOR CONTINUATION OF FUTURE TWINNING ESPECIALLY IN CASES WHERE THE NEXT STAGE IS TO BE BUILT IN LESS THAN 5 YEARS. A TYPICAL STUB IS SHOWN IN THE CASE 3 SKETCH.
2. SUPERELEVATION ON TRANSITION IS USUALLY KEPT AT 2% TO MATCH THE FUTURE CROWN, HOWEVER IT IS BUILT LOW TO ALLOW FOR ONE ADDITIONAL LIFT OF A.C.P.
3. THE LINES SHOWN ON THIS DRAWING ARE FOR DESIGN, SURVEY AND CONSTRUCTION ONLY. THEY DO NOT REPRESENT PAVEMENT MARKINGS. REFER TO THE TRAFFIC CONTROL STANDARDS MANUAL FOR TYPICAL PAVEMENT MARKINGS.



CASE 4

NOTE:

T. = SUB-TANGENT

No.		BY	DATE
2			
1			

Alberta INFRASTRUCTURE

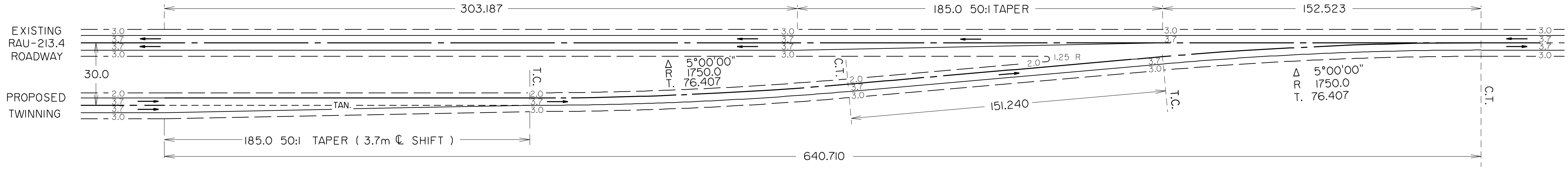
FIGURE B-6.1.a

Date: APRIL 1995

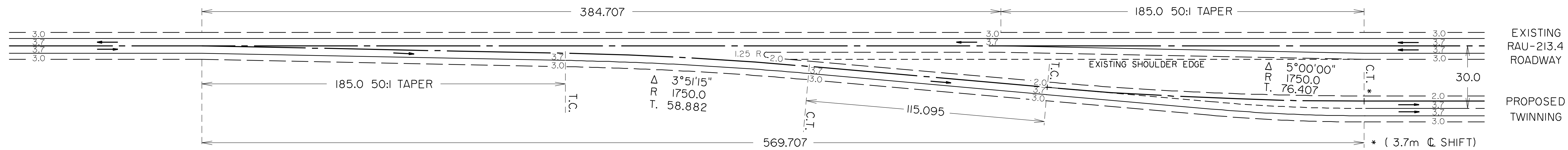
TYPICAL HIGHWAY TRANSITIONS (TWO-LANE UNDIVIDED, FOUR-LANE DIVIDED) 38m ℄ TO ℄ SPACING

Prepared By: R.T. | Checked By: B.K | Scale: N.T.S.

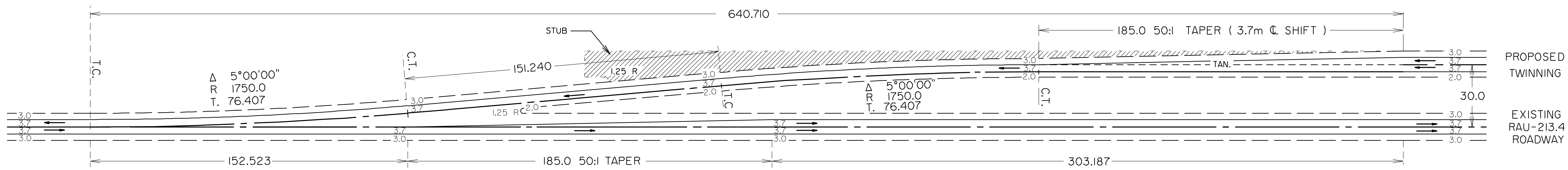
GRAPHICS FILE: tigfs01\reb\manual\chapters\chap-b\debb61a.man



CASE 1



CASE 2



CASE 3

NOTES:

1. TRANSITIONS ARE OFTEN BUILT WITH A 'STUB ' FOR CONTINUATION OF FUTURE TWINNING ESPECIALLY IN CASES WHERE THE NEXT STAGE IS TO BE BUILT IN LESS THAN 5 YEARS. A TYPICAL STUB IS SHOWN IN THE CASE 3 SKETCH.
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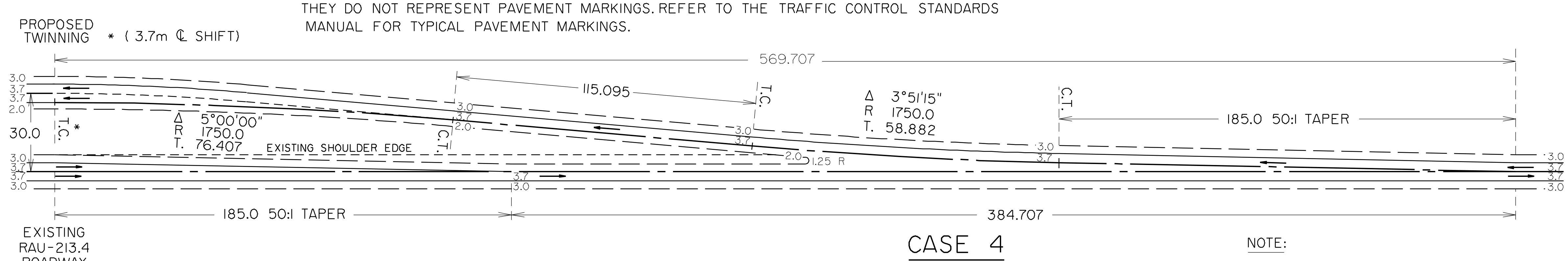
3. THE LINES SHOWN ON THIS DRAWING ARE FOR DESIGN, SURVEY AND CONSTRUCTION ONLY. THEY DO NOT REPRESENT PAVEMENT MARKINGS. REFER TO THE TRAFFIC CONTROL STANDARDS MANUAL FOR TYPICAL PAVEMENT MARKINGS.



CASE 4

NOTE:

T. = SUB-TANGENT

No.		BY	DATE
2			
1			

Alberta INFRASTRUCTURE

FIGURE B-6.1b

Date: APRIL 1995

TYPICAL HIGHWAY TRANSITIONS (TWO-LANE UNDIVIDED, FOUR-LANE DIVIDED) 30m ℄ TO ℄ SPACING

Prepared By: R.T.	Checked By: B.K	Scale: N.T.S.	PAGE B-87

B.7 TEMPORARY HIGHWAY DETOURS

B.7.1 Introduction

Temporary highway detours that are to be open for public use in Alberta must be designed and built under the supervision of a professional engineer as per the Engineering, Geological and Geophysical Professionals Act.

The following guidelines should be used where temporary detours are required to accommodate traffic on existing highways due to bridge construction or other activities. Designers may exceed the standards shown here for efficiency or safety but lower standards for the geometric elements are generally not appropriate. The guidelines provide advice on the following topics.

- Guidelines for surfacing of the detour;
- Appropriate design speed for the detour;
- Minimum geometric parameters that should be used for each design speed on a detour;
- Horizontal alignment guidelines.

A typical cross-section and two typical detour plans are also provided.

B.7.2 Guidelines for Surfacing of Detour

Detour surfacing requirements generally depend on the road type, the duration of the detour, the time of year and the expected daily traffic volume during the time the detour will be in place.

In winter conditions gravel surfacing is generally adequate structurally due to the frozen ground conditions. Asphalt stabilized base-course (ASBC) is not a good option for detours built in winter because the material is not readily available. In spring and summer conditions ASBC (alone) does not provide sufficient structural strength for heavy trucks and therefore a gravel surface with dust abatement treatment is more appropriate. A paved surface on a detour is generally only warranted where the daily traffic volumes exceed 4000, the detour will be in place for a long duration, or no alternate routes are available. A "long duration" is typically defined as more than 3 weeks on a divided highway or more than 4 months on an undivided highway.

In some cases the department may stipulate the type of surface required on a detour however, where this is not the case, Table B.7.2 is suggested as a guide. The surfacing options are asphalt concrete pavement, or road gravel with dust abatement treatment.

Table B.7.2 Guidelines for Surfacing of Detours

Road Type	Short Duration < 4 Months		Long Duration > 4 Months	
	AADT	Detour Surfacing	AADT	Detour Surfacing
Divided Primary Highway	All	Gravel/Pavement	All	Pavement
Undivided Primary Highway	> 4000	Gravel	> 4000	Pavement/Gravel
	< 4000	Gravel	< 4000	Gravel
Secondary Highway or Local Road	All	Gravel	All	Gravel

B.7.2.1 Design Speed of Detour

Design speed of the detour is selected based on the traffic volume and the expected duration of the detour.

Generally a 60 km/h minimum design speed is appropriate for all detours. Detours that are designed for 60 km/h may be posted for 50 km/h. A 40 km/h posted speed may be used if necessary to reduce vehicle speeds in the construction zone. Higher design speeds may be appropriate in some cases, for example where the traffic will use the detour for a long duration or where the detour itself is long. A 50 km/h design speed may be appropriate for lower volumes (AADT < 500) or short duration detours.

Illumination, using at least two overhead streetlights, one at each detour entrance to the highway, is generally provided on all detours where any hazard is involved.

Where a temporary structure is required on the detour to accommodate the flow of water, the contractor's engineer should select the size based on an engineering assessment and professional judgement unless a size has been specified by the Department. Detour crossing structures should generally be designed to handle a 1 in 5 year flood if the structure will be in placed during the stream's typical flood season.

B.7.2.2 Horizontal Alignment Guidelines

At the conclusion to the permanent highway the detour should begin with 40 m long 20:1 tapers. This provides a visual cue to the driver. A horizontal curve can begin at the end of taper without any spiral transition. A curve in the opposite direction may be joined directly to the end of this curve, without spiral transitions or tangents between curves.

The maximum deflection angles and minimum radii are shown in Table B.7.2.2a. The minimum radii have been selected based on a maximum superelevation of 0.05 m/m and maximum side-friction factors for the limit of comfortable driving as recommended for low-speed urban design. These friction factors are chosen because detours do not have to provide the same level of comfort that is appropriate on the open highway, however, they must be safe. The purpose of using a 0.05 m/m superelevation rate is to safely accommodate the full range of speeds, including very low speeds coupled with adverse weather/road conditions. When road conditions are good and speeds are higher the surface will still provide an adequate margin of safety against sideslipping. Table B.7.2.2b "Superelevation for Detour" has been developed to show recommended "e" values for each radius for design speeds from 40 km/h to 90 km/h. For higher design speeds the 0.06 m/m superelevation table in Section B.3.6 may be used. The distribution of e and f through the range of curves shown in Table B.7.2.2b is based on Method 5 as described in AASHTO's 1994 publication "A Policy on Geometric Design of Highways and Streets". Method 5 represents a practical distribution over the range of curvature.

Figures B-7.2.2b and B-7.2.2c show typical plan views of bridge detours using three-curve and four curve layouts respectively. The four-curve layout is appropriate where a temporary bridge structure (overhead truss type) requiring a tangent approach is utilized.

Table B.7.2.2a Geometric Parameters of Detours

Design Speed of Detour (km/h)	50	60	70	80	90
Max. Deflection (degrees) for first curve joining the highway	30	25	20	15	10
Max. Grade (%) ○ - Primary Highways or Secondary Highways > 200 AADT	8	8	8	8	7
Max. Grade (%) ○ - Secondary Highways or Local Roads < 200 AADT	10	10	9	8	7
Stopping Sight Distance (m) *	45	65	85	110	140
Min. Crest K. * (based on stopping sight distance)	5	10	15	25	35
Min. Sag K * (based on comfort control)	7	10	13	17	21
Max. Superelevation (m / m)	Use 0.05 m / m as a maximum on the detour				
Superelevation	Use Table B.7.2.2b Superelevation for Detours				
Min. Radius (m) **	75	120	185	250	350
Min. Width (m)	9 m if AADT < 2000, 10 m if AADT > 2000 See notes on Figure B-7.2.2a				
Sideslope on unprotected fills +	3:1	3:1	3:1	3:1	3:1

○ Grades higher than the suggested maximum may be permitted as directed by the Engineer.

* Based on 10 km / h less than the design speed.

** Based on a maximum superelevation of 0.05 m / m and maximum side friction factors for "limit of comfort", as per low-speed urban design.

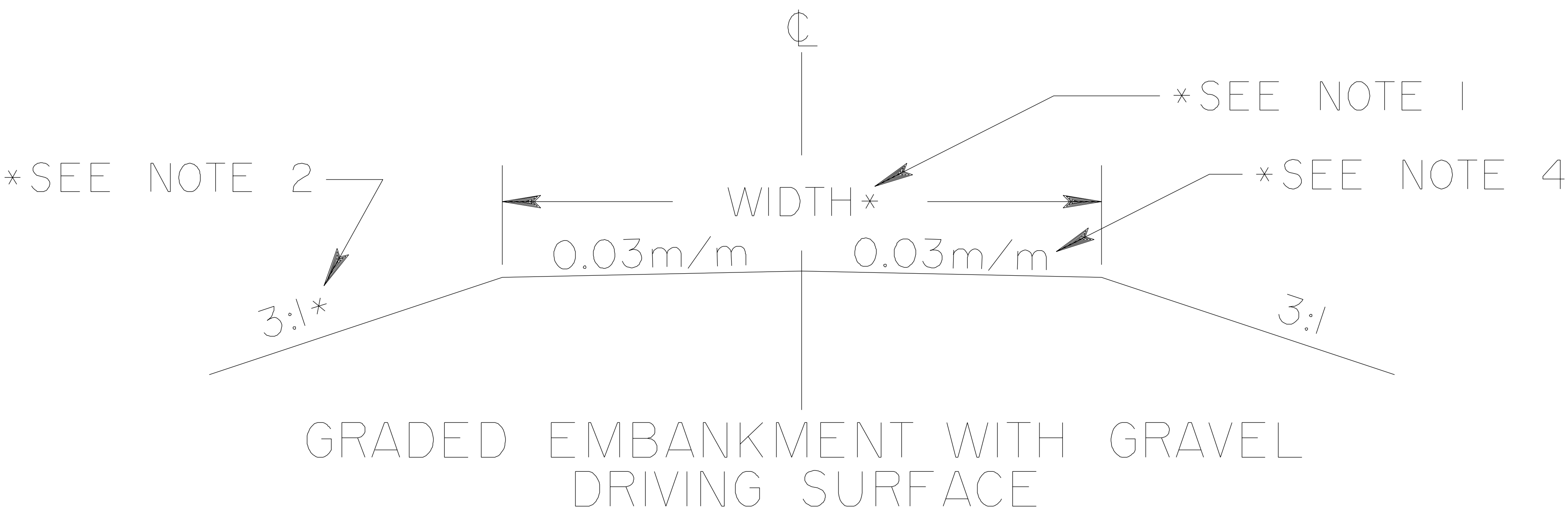
\+ Where traffic is protected from the slope by traffic barrier, a steeper slope may be used behind the barrier based on stability of soil.

Note: Adequate warning and decision sight distance (based on the design speeds of the highway) should be provided on the approaches to the detour. The guidelines for construction zone signs and other traffic control devices should also be followed.

Table B.7.2.2b Superelevation for Detours

Radius (m)	Design Speed (km/h)					
	40	50	60	70	80	90
4500	NC	NC	NC	NC	NC	NC
4000	NC	NC	NC	NC	NC	RC
3000	NC	NC	NC	NC	RC	RC
2000	NC	NC	NC	RC	RC	RC
1500	NC	NC	RC	RC	0.024	0.028
1000	NC	RC	0.021	0.026	0.031	0.036
750	RC	0.021	0.026	0.032	0.036	0.041
500	RC	0.027	0.034	0.041	0.044	0.048
350	0.025	0.034	0.041	0.048	0.048	0.050
250	0.031	0.040	0.046	0.049	0.050	-
220	0.034	0.043	0.048	0.050	-	-
185	0.037	0.046	0.049	0.50	-	-
150	0.041	0.048	0.050	-	-	-
120	0.045	0.049	0.50	-	-	-
100	0.047	0.050	-	-	-	-
75	0.049	0.050	-	-	-	-
45	0.050	-	-	-	-	-

Notes:
1. The maximum superelevation rate for temporary detours is 0.05 m / m.
2. Simple circular curves are adequate for temporary detours, i.e., spiral transitions are not required.



NOTES

1. WIDTH = 9m FOR AADT LESS THAN 2000.
   WIDTH = 10m FOR AADT GREATER THAN 2000 OR FOR LOG HAUL ROUTES.
   AN ADDITIONAL 0.5m WIDTH IS TO BE PROVIDED ADJACENT TO TRAFFIC BARRIER ON EACH SIDE.
2. SIDESLOPE SHALL BE 3:1 OR FLATTER UNLESS PROTECTED BY TRAFFIC BARRIER, (GUARDRAIL OR CONCRETE F BARRIER). IF TRAFFIC IS PROTECTED FROM SLOPE, THE SLOPE MAY BE BUILT AS STEEP AS POSSIBLE BASED ON THE STABILITY OF THE SOIL (POSSIBLY 1.5:1 OR 2:1).
3. TRAFFIC SHALL BE PROTECTED FROM ANY NON-TRANSVERSABLE HAZARDS ADJACENT TO THE DETOUR. THIS INCLUDES CULVERT OPENINGS IN EXCESS OF 1.4m DIAMETER WITHIN 5m OF THE DRIVING SURFACE. MORE SEVERE HAZARDS MAY NEED TO BE MITIGATED EVEN AT GREATER OFFSETS BASED ON ENGINEERING JUDGEMENT.
4. IF THE DETOUR ROADTOP IS TO BE PAVED, A 0.02m/m NORMAL CROWN CROSSFALL MAY BE USED.

No.	REVISIONS	BY	DATE
2			
1			

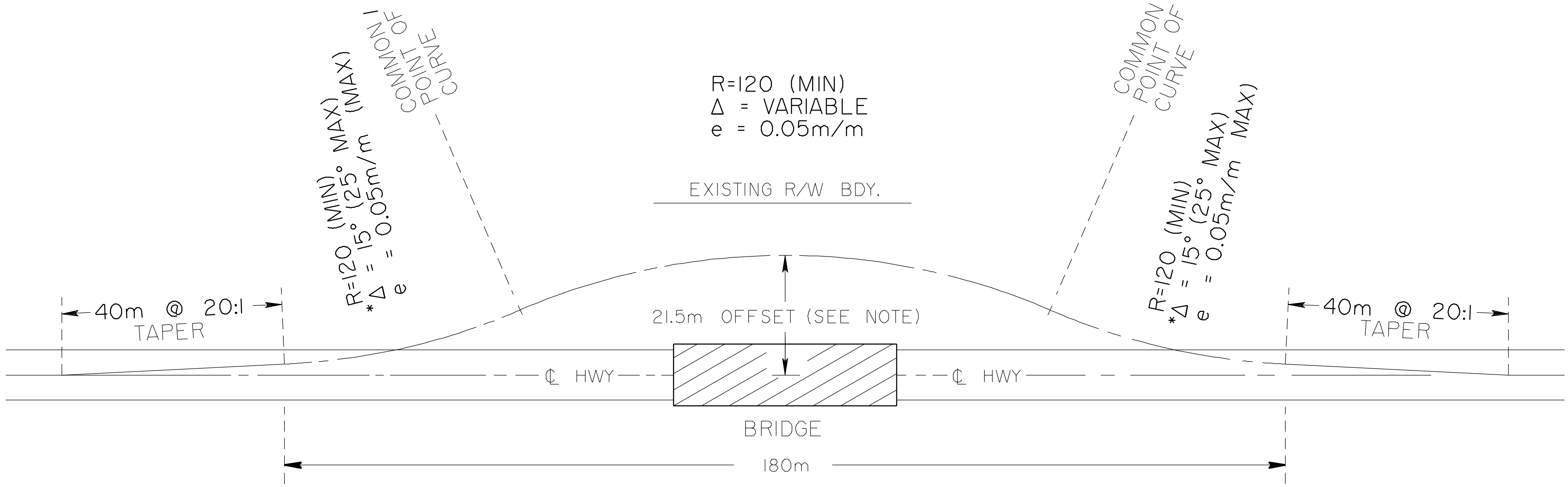
Alberta INFRASTRUCTURE

FIGURE B.7.2.2a

Date: MARCH 28, 1996

TYPICAL CROSS SECTION FOR DETOUR

Prepared By: R.M.	Checked By: B.K.	Scale: N.T.S.	PAGE B-93



NOTES

1. THE DETOUR OFFSET IS VARIABLE AND MAY BE ADJUSTED TO SUIT THE SITE BY USING DIFFERENT ANGLES, RADII AND/OR ADDING TANGENTS BETWEEN THE CURVES. THIS DRAWING SHOWS A 21.5m OFFSET.

*2. THE MAXIMUM DEFLECTION ANGLES SHOWN IN THE TABLE BELOW REFER TO THE FIRST CURVE THAT TRAFFIC TRAVELS ON WHILE LEAVING THE HIGHWAY.

CURVE PARAMETERS		
DESIGN SPEED (km/h)	MINIMUM RADIUS (m)	MAXIMUM DEFLECTION Δ_I (DEGREES)
50	80	30
60	120	25
70	170	20
80	230	15
90	300	10

No.	REVISIONS	BY	DATE
2			
1			

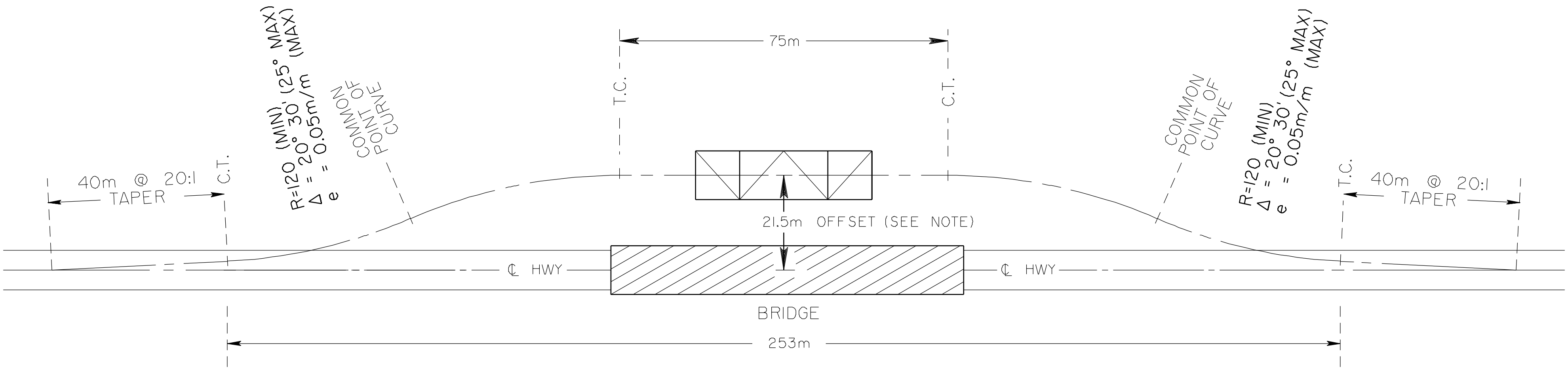
Alberta INFRASTRUCTURE

FIGURE B.7.2.2b

Date: MARCH 28, 1996

TYPICAL PLAN VIEW
BRIDGE DETOUR (3 CURVES)
60km/h DESIGN SPEED

Prepared By: R.M. | Checked By: B.K. | Scale: N.T.S.



NOTES

1. THE TANGENT LENGTH REQUIRED ON THE APPROACHES TO THE TEMPORARY STRUCTURE WILL DEPEND ON THE CLEAR WIDTH OF THE STRUCTURE, THE DESIGN VEHICLE AND THE NUMBER OF LANES OF TRAFFIC TO BE ACCOMMODATED. THE DESIGNER MAY USE VEHICLE TURNING TEMPLATES TO DETERMINE THE TANGENT LENGTH NEEDED.
2. THE DETOUR OFFSET IS VARIABLE. AND MAY BE ADJUSTED TO SUIT THE SITE BY USING DIFFERENT ANGLES, RADII AND/OR ADDING TANGENTS BETWEEN CURVES. THIS DRAWING SHOWS A 21.5m OFFSET.
3. THE MAXIMUM DEFLECTION ANGLES SHOWN IN THE TABLE BELOW REFER TO THE FIRST CURVE THAT TRAFFIC TRAVELS ON WHILE LEAVING THE HIGHWAY.

CURVE PARAMETERS		
DESIGN SPEED (km/h)	MINIMUM RADIUS (m)	MAXIMUM DEFLECTION Δ° (DEGREES)
50	80	30
60	120	25
70	170	20
80	230	15
90	300	10

No.	REVISIONS	BY	DATE
2			
1			

Alberta INFRASTRUCTURE

FIGURE B.7.2.2c

Date: MARCH 28, 1996

TYPICAL PLAN VIEW
BRIDGE DETOUR (4 CURVES)
60km/h DESIGN SPEED

Prepared By: R.M.	Checked By: B.K.	Scale: N.T.S.	PAGE B-95

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