CHAPTER B
ALIGNMENT ELEMENTS

TABLE OF CONTENTS

B.1 INTRODUCTION **1**
B.2 SIGHT DISTANCE **1**
B.2.1 General Considerations 1
B.2.2 Criteria for Determining Sight Distance 2
B.2.2.1 Height of Driver's Eye 2
B.2.2.2 Height of Object 2
B.2.3 Stopping Sight Distance 2
B.2.4 Passing Sight Distance 6
B.2.5 No Passing Zone Sight Distance 9
B.2.5.1 No Passing Zones on Undivided Highways 11
B.2.6 Decision Sight Distance 12
B.3 HORIZONTAL ALIGNMENT **14**
B.3.1 Introduction 14
B.3.2 General Controls 14
B.3.3 Maximum Safe Side Friction Factors 26
B.3.4 Minimum Radius 26
B.3.5 Rates of Superelevation for Design 27
B.3.5.1 Speed to be used for Superelevation 27
B.3.5.2 Maximum Superelevation 27
B.3.5.3 Minimum Superelevation 29
B.3.5.4 Distribution of e and f 35
B.3.6 Development of Superelevation 37
B.3.7 Spiral Curves 42
B.3.7.1 Form and Properties 42
B.3.7.2 Basis of Design 43
B.3.7.3 Design Values for Spiral Parameters 45
B.3.8 Providing Sight Distance on Horizontal Curves 45
B.3.9 Urban Fringe Transition 46
B.4 VERTICAL ALIGNMENT **47**
B.4.1 General Controls for Vertical Alignment 47
B.4.2 Maximum Gradient 50
B.4.2.1 Vehicle Operating Characteristics on Grades 51
B.4.3 Minimum Gradient 51
B.4.3.1 Rural Highways 51
B.4.3.2 Urban Curbed Roadways 52
B.4.3.3 Minimum Transition Grades and Drainage Considerations 52
B.4.4 Vertical Curves 53
B.4.4.1 K Value 53
B.4.4.2 Crest Vertical Curves 54
B.4.4.3 Sag Vertical Curves 58
B.4.4.4 Sight Distance at Underpasses 60
B.5 CLIMBING AND PASSING LANES **60**
B.5.1 Introduction 60
B.5.2 Geometric Features of Climbing and Passing Lanes 60
B.5.2.1 Lane Width 60
B.5.2.2 Shoulder Width 60
B.5.2.3 Superelevation 60

B.5.2.4 Tapers 60
B.5.2.5 Proximity to Intersections 62
B.5.2.6 Start and End Points and Length 62
B.5.2.7 Sight Distance at Start and End Points 62
B.5.3 Climbing Lanes 65
B.5.3.1 Climbing Lane Warrant for Two-Lane Undivided Highways 66
B.5.3.2 Climbing Lane Warrant for Multi-lane Highways 71
B.5.3.3 Determining Length and Location of Climbing Lanes 71
B.5.4 Passing Lanes 79
B.5.4.1 Passing Lane Warrant 79
B.5.4.2 Location, Spacing, and Length of Passing Lanes 82
B.6 TYPICAL HIGHWAY TRANSITIONS 83
B.6.1 Introduction 83
B.6.2 Construction Practices at Typical Highway Transitions 83
B.7 TEMPORARY HIGHWAY DETOURS 85
B.7.1 Introduction 85
B.7.2 Accommodation of Pedestrians and Cyclists 85
B.7.3 Guidelines for Surfacing of Detours 85
B.7.4 Design Speed of Detours 86
B.7.5 Horizontal Alignment Guidelines for Detours 86
REFERENCES 92

CHAPTER B
ALIGNMENT ELEMENTS

LIST OF FIGURES

Figure B-2-4a Elements of Passing Sight Distance for Two-Lane Undivided Highways 8
Figure B-3-2a Illustration of Designs 17
Figure B-3-2b Illustration of Designs 18
Figure B-3-2c Illustration of Designs 19
Figure B-3-2d Illustration of Designs 20
Figure B-3-2e Illustration of Designs 21
Figure B-3-2f Illustration of Designs 22
Figure B-3-2g Illustration of Designs 23
Figure B-3-2h Illustration of Designs 24
Figure B-3-2i Illustration of Designs 25
Figure B-3-5-4a Distribution of Superelevation and Side Friction 36
Figure B-3-6a Superelevation Transition (Case I) 39
Figure B-3-6b Superelevation Transition (Case II) 40
Figure B-3-6c Superelevation Transition (Case III) 41
Figure B-3-7-2a Minimum Spiral Parameter Considerations 43
Figure B-3-8a Lateral Clearance on Horizontal Curves for Sight Distance 46
Figure B-4-1a Illustration of Design: Roller-Coaster / Hidden Dip Profile 47
Figure B-4-1b Illustration of Design: Brocken-back Curve 48
Figure B-4-4-2a Stopping Sight Distance on Crest Vertical Curves 55
Figure B-4-4-2b Passing Sight Distance on Crest Vertical Curves 56
Figure B-4-4-2c No Passing Zone Sight Distance on Crest Vertical Curves 57
Figure B-4-4-3a Stopping Sight Distance on Sag Vertical Curves 59
Figure B-5-2-4a Climbing/Passing Lanes for Various Pavement Widths 61
Figure B-5-2-7a Passing & Climbing Lanes Decision Sight Distance at Merge Taper 64
Figure B-5-3a Collision Involvement Rate for Trucks on Two-lane Roads 65
Figure B-5-3-3a Climbing Lane Design Example 72
Figure B-5-3-3b Performance Curves for Heavy Trucks – 180 g/W Deceleration 73
Figure B-5-3-3c Performance Curves for Heavy Trucks – 180 g/W Acceleration 74
Figure B-5-3-3d Performance Curves for Heavy Trucks – 150 g/W Deceleration 75
Figure B-5-3-3e Performance Curves for Heavy Trucks – 150 g/W Acceleration 76
Figure B-5-3-3f Performance Curves for Heavy Trucks – 120 g/W Deceleration 77
Figure B-5-3-3g Performance Curves for Heavy Trucks – 120 g/W Acceleration 78
Figure B-6-1a Typical Highway Transitions (Two-lane Undivided to Four-lane Divided) 84
Figure B-7-5a Typical Cross-Section for Detour 89
Figure B-7-5b Typical Plan View for Bridge Detour (3 Curves) 90
Figure B-7-5c Typical Plan View for Bridge Detour (4 Curves) 91

CHAPTER B
ALIGNMENT ELEMENTS

LIST OF TABLES

Table B-2-3a	Minimum Stopping Sight Distance on Level Roadways	5
Table B-2-3b	Stopping Sight Distance on Grades	6
Table B-2-4a	Minimum Passing Sight Distance on Two-Lane Undivided Highways	9
Table B-2-5a	Minimum No Passing Zone Sight Distance on Undivided Highways	11
Table B-2-6a	Minimum Decision Sight Distance	13
Table B-3-3a	Maximum Safe Side Friction Factors	26
Table B-3-5-3a	Values for Superelevation and Spiral Parameters (e_{max} = 0.06 m/m)	30
Table B-3-5-3b	Values for Superelevation and Spiral Parameters (e_{max} = 0.08 m/m)	31
Table B-3-5-3c	Values for Superelevation and Spiral Parameters (e_{max} = 0.04 m/m)	32
Table B-3-5-3d	Superelevation Rate for Urban Design (e_{max} = 0.04 m/m)	33
Table B-3-5-3e	Superelevation Rate for Urban Design (e_{max} = 0.06 m/m)	34
Table B-3-6a	Length Required for Superelevation Runoff on Simple Curves	38
Table B-3-7-2-2a	Maximum Relative Slope between Outer Edge of Pavement and Centreline	44
Table B-4-1a	Minimum Design Vertical Clearances below Structures	49
Table B-4-2a	Desirable Maximum Gradient (%)	50
Table B-4-3-2a	Minimum Ditch or Gutter Gradient	52
Table B-4-4-2a	Minimum Vertical Curve Criteria for K Value	54
Table B-5-3-1a	Critical Length of Grade (Metres) for a 15 km/h Speed Reduction	67
Table B-5-4-1a	Probability of Time Gaps Available for Overtaking	80
Table B-5-4-1b	Passing Lane Warrant	81
Table B-7-3a	Guidelines for Surfacing of Detours	86
Table B-7-5a	Geometric Parameters of Detours	87
Table B-7-5b	Superelevation for Detours	88

CHAPTER B ALIGNMENT ELEMENTS

B.1 INTRODUCTION

Roads are traditionally designed in three views: plan, profile and cross-section. The highway designer will often design each view independently. 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.

Every effort should be made to provide greater than the minimum value for various design parameters. Generally, the use of minimum values should be avoided; reserving the minimum for particular circumstances at critical locations. When topography is rugged or rolling, considerable construction cost savings may be achieved when minimum design values are considered at selected locations. When reductions in design values are considered, the horizontal alignment standard is generally more critical than the vertical alignment standard. Consequently, it is important to attach a high priority to the horizontal alignment parameters, while more flexibility can be exercised with vertical alignments. Notwithstanding the above, vertical alignment standards above the minimum should be used near intersections, due to the importance of intersection sight distance.

The examination of alternative alignments is encouraged to maximize the benefits considering construction costs, road user costs, reduced collisions, and safety.

Certain combinations of horizontal and vertical curves can result in an apparent disjointed appearance in the alignment or grade although the horizontal and vertical curves comply with the design standards outlined in Tables A-10-1a and A-10-1b.

Although this guide does not identify all concepts of good design, Figures B-3-2a through B-3-2i and Figures B-4-1a through B-4-1-b illustrate roadways with horizontal and vertical design components that are visually displeasing. Suggestions for improvement are also identified.

Constraints due to bridges can have a significant impact on road geometry. These constraints can be much more restrictive than typical roadway geometric design criteria. Identification of potential bridge constraints and accounting for them during preliminary geometric layout of the road is often the most cost-effective method of optimizing the overall project. Refer to the latest version of the Alberta Transportation “Bridge Conceptual Design Guidelines” for specific bridge-related geometric requirements.

B.2 SIGHT DISTANCE

B.2.1 General Considerations

The ability to see ahead is of utmost importance for 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 drivers can avoid striking unexpected objects on the roadway surface. Two-lane, undivided highways should also have sufficient sight distance to enable drivers to safely occupy the opposing traffic lane during passing manoeuvres. Two-lane, undivided, 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 the highway function. Generally, for two-lane, undivided, rural, arterial highways, it is desirable to provide passing sight distance over at least 75% of the length.

Four sight distance concepts are presented:

1. The criteria for determining various sight distances for use in design;
2. Stopping sight distance (the distance required for stopping; applicable on all highways);
3. Passing sight distance (the distance required for passing manoeuvres; applicable only on two-lane, undivided highways); and
4. Decision sight distance (the distance needed for decisions at complex locations).

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 Determining Sight Distance

The height of the driver's eye and the height of an object are two elements used to determine the available stopping, passing and decision sight distances.

B.2.2.1 Height of Driver's Eye

For passenger car sight distance calculations, the driver's eye height is considered to be 1.08 m 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. The driver eye height exceeds 1.08 m in more than 90% of all passenger cars [1].

An eye height of 1.80 m is adopted for the single unit vehicle (SU) and the bus design vehicle (B).

An eye height of 2.30 m is adopted for all large trucks and tractor-trailer combinations, based on the height of typical highway tractors.

B.2.2.2 Height of Object

For stopping sight distance, an object height of 0.6 m is used. The selection of the 0.6 m object height is based on research indicating that objects with a height less than 0.6 m are seldom involved in crashes [1]. An object height of 0.6 m is representative of the height of automobile headlights and tail lights. Using an object height less than 0.6 m would result in longer crest vertical curves without a documented decrease in severity or frequencies of collisions [1].

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

The object height used for 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

Stopping 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 the perception-reaction time of the driver, the initial vehicle speed and the rate of deceleration.

The sight distance that is available to a driver (i.e., the distance along the roadway that a driver can see to an object) depends on the driver's eye height, the object height and the geometry of the roadway (vertical and horizontal alignment).

The minimum available sight distance on a roadway must be long enough to allow a vehicle, travelling at an assumed speed (which is the same as the design speed), to stop before reaching a stationary object in its path. Although greater length is desirable, the available sight distance at every point along the highway must be at least the distance required for a vehicle to stop (the stopping sight distance). When the required stopping sight distance is not met, a design exception request may be required. Refer to the Alberta Transportation, "Design Exceptions Guideline" [2] for further details.

The following criteria have been adopted by Alberta Transportation to determine minimum stopping sight distance (SSD) requirements:

- a fixed perception-reaction time of 2.5 seconds;
- deceleration rate of 3.4 m/s^2;
- eye height of 1.08 m; and
- object height of 0.60 m.

Approximately 90% of all drivers decelerate at rates greater than 3.4 m/s^2 [3]. Such deceleration is within a driver's capability to stay within their lane and maintain steering control during the braking manoeuvre on wet surfaces. Therefore, 3.4 m/s^2 is considered a comfortable deceleration for most drivers and is recommended as the deceleration threshold for determining stopping sight distance.

Most vehicle braking systems and the tire-pavement friction levels of most roadways are capable of providing a deceleration rate of at least 3.4 m/s^2. In addition, the friction available on most wet pavement surfaces and the capabilities of most vehicle braking systems can provide braking friction that exceeds this deceleration rate [3].

These criteria and the resulting minimum SSD values are tabulated in Table B-2-3a. An available sight distance exceeding the values in Table B-2-3a should be provided wherever practical. Providing a longer stopping sight distance increases the margin for error for all drivers and, in particular, for those who operate at or above the design speed during wet pavement conditions.

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

Stopping sight distance is the sum of the distance travelled during the perception-reaction time (d_{pr}) and the braking distance time (d_b).

$$\mathrm{SSD} = \mathrm{d_{pr}} + \mathrm{d_b} = 0.2778\,\mathrm{V}\,\mathrm{t} + \frac{0.0386\,\mathrm{V}^2}{a}$$

where:

SSD = stopping sight distance (m)
V = design speed (km/h)
t = perception-reaction time, 2.5 s
a = deceleration rate, 3.4 m/s^2

Perception-reaction distance is the distance a vehicle travels the instant the driver sights an object and decides to stop before the brakes are applied. This distance is determined using the following equation:

$$d_{pr} = \frac{V\,t}{\left(\frac{3600}{1000}\right)} = \frac{\mathbf{V\,t}}{\mathbf{3.6}} = 0.2778\,V\,t$$

where:

d_{pr} = distance travelled during perception-reaction time (m)
V = design speed (km/h)
t = perception-reaction time, 2.5 s

Braking distance is the distance it takes to stop a vehicle once the brakes have been applied. On a level roadway, this distance is determined using the following equation:

$$d_b = 0.0386\,\frac{V^2}{a}$$

where:

d_b = braking distance (m)
V = design speed (km/h)
a = deceleration rate, 3.4 m/s^2

Braking distance is derived from the general equations:

$$U^2 = V^2 - 2\,g\,f\,d_b \quad \text{and} \quad a = g\,f$$

where:

U = final speed (km/h)
V = initial speed = design speed (km/h)
g = acceleration rate due to gravity, 9.81 m/s^2
f = coefficient of friction
d_b = braking distance (m)
a = deceleration rate, 3.4 m/s^2

In this case, U = zero, therefore:

$$0 = V^2 - 2\,g\,f\,d_b$$

$$d_b = \frac{V^2}{2\,g\,f} = \frac{V^2}{\left(\frac{3600}{1000}\right)^2 2\,a} = \frac{\mathbf{V^2}}{\mathbf{25.92}\,\boldsymbol{a}} = \frac{0.0386\,V^2}{a}$$

where:

V = design speed, (km/h)
a = deceleration rate, 3.4 m/s^2

Several situations 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 eye height is of little value on long downgrades. Truck speeds may approach or exceed those of passenger cars. Although the average truck operator may 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 provide a stopping sight distance that exceeds the values in Table B-2-3a or Table B-2-3b.

Another situation where truck operators may have difficulty seeing up the road is near underpasses due to railway grade separations and interchanges. Overpasses affect the ability to stop due to preferential bridge icing, and impact available sight distances at intersections located near the structure, due to bridge rails. 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 that satisfy the criteria for stopping sight distance are given in Section B.4.4.

Table B-2-3a Minimum Stopping Sight Distance on Level Roadways

Design Speed (km/h)	Perception-Reaction Distance (m)	Braking Distance (m)	Stopping Sight Distance (SSD)	
			Calculated (m)	Design (m)
40	27.8	18.2	45.93	50
50	34.7	28.4	63.09	65
60	41.7	40.8	82.52	85
70	48.6	55.6	104.21	105
80	55.6	72.6	128.18	130
90	62.5	91.9	154.41	160
100	69.4	113.5	182.92	185
110	76.4	137.3	213.69	220
120	83.3	163.4	246.73	250
130	90.3	191.8	282.04	285

Braking distance will increase on downgrades and decrease on upgrades. As indicated earlier,

$$\mathrm{d_b} = \frac{\mathrm{V}^2}{25.92\,a}$$

Since $a = \mathrm{g}\,f$,

$$\mathrm{d_b} = \frac{\mathrm{V}^2}{25.92\,\mathrm{g}\,f} = \frac{\mathrm{V}^2}{25.92\,(9.81)\,f} = \frac{V^2}{254.28\,f}$$

When the roadway is on a grade, the equation for braking distance is modified by adding the grade of the roadway to the coefficient of friction, as follows [3]:

$$\mathrm{d_b} = \frac{\mathrm{V}^2}{254.28 * [\,f + \mathrm{G}\,]} = \frac{\mathrm{V}^2}{254.28 * \left[\left(\frac{a}{9.81}\right) + \mathrm{G}\right]}$$

where:

- $\mathrm{d_b}$ = braking distance (m)
- V = design speed (km/h)
- g = acceleration rate due to gravity, 9.81 m/s^2
- f = coefficient of friction = a / g = a / 9.81
- a = deceleration rate, 3.4 m/s^2
- G = grade of roadway (m/m) (G is positive if the vehicle travels uphill and negative if downhill)

Many drivers, particularly those in automobiles, do not compensate completely (by acceleration or deceleration) for the changes in speed caused by grade. In many cases, the sight distance available on downgrades is greater than on upgrades, which can help to provide the necessary corrections for grade. The following Table B-2-3b summarizes the stopping sight distances on grades for a variety of design speeds.

Table B-2-3b Stopping Sight Distance on Grades

Design Speed (km/h)	Stopping Sight Distance (m)					
	Downgrades (%)			Upgrades (%)		
	3	6	9	3	6	9
40	50	50	53	45	44	43
50	66	70	74	61	59	58
60	87	92	97	80	77	75
70	110	116	124	100	97	93
80	136	144	154	123	118	114
90	164	174	187	148	141	136
100	194	207	223	174	167	160
110	227	243	262	203	194	186
120	263	281	304	234	223	214
130	302	323	350	267	254	243

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 manoeuvres without interfering with the smooth flow of traffic in either direction.

Many passes are accomplished without the driver being able to see ahead for the entire passing sight distance. Because many cautious drivers would not attempt to pass under such conditions, design on this basis would comprise highway function.

Passing sight distance (PSD) for use in design is determined based on 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, passing sight distance is determined for a single vehicle passing a single vehicle. It does not account for a car passing a longer vehicle such as an LCV, which can restrict sight lines to the opposing lane and may be causing spray during inclement weather. Together, these factors result in much longer passing times and distances [4]. Longer sight distances occur naturally along the roadway 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, undivided highways. The assumed control for driver behavior should be that practiced by a high percentage of drivers, rather than the average driver.

Alberta Transportation follows the same passing sight distance model indicated in the 2004 American Association of State Highway Transportation Officials (AASHTO), "A Policy on Geometric Design of Highways and Streets" [5]. The assumptions made in the 2004 AASHTO model are:

1. The overtaken vehicle travels at a uniform speed;

2. The passing vehicle trails the overtaken vehicle and has reduced its speed as it enters the passing section;
3. When the passing section is reached, the driver requires a short time period 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 15 km/h higher than that of the overtaken vehicle; and
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 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 undivided highways is determined as the sum of the following four distances, as indicated in Figure B-2-4a.

- 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; and
- d_4 Distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane.

The minimum passing sight distances have been developed based on extensive field observations of driver behavior during passing manoeuvres. The average speed differential of 15 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-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. Largely 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 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. 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.

The distance travelled during the initial manoeuvre is:

$$d_1 = \frac{t_1}{3.6}\left(V - m + \frac{a\,t_1}{2}\right)$$

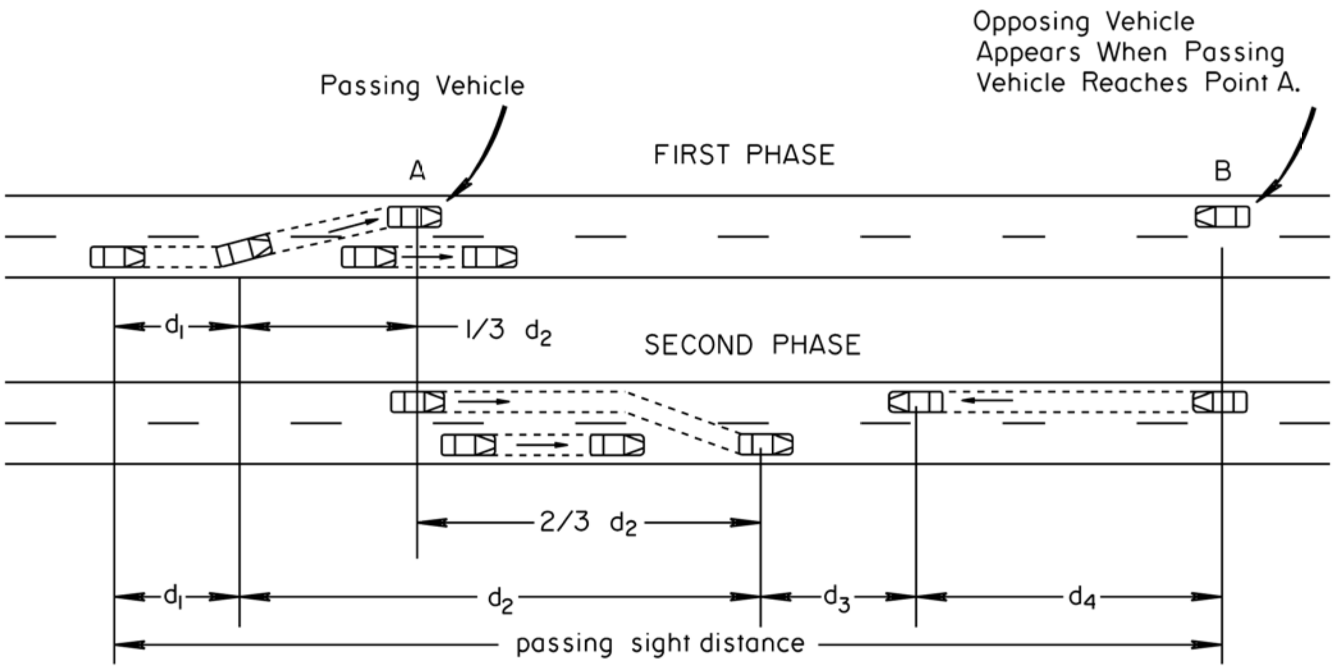
where:

d_1 = the distance travelled during the initial manoeuvre
t_1 = time of the initial manoeuvre (s)
V = average speed of the passing vehicle (km/h)
m = difference in speed between the passing and passed vehicles (km/h)
a = average acceleration (km/h/s)

The time, average speed and average acceleration varies depending on the design speed.

Figure B-2-4a Elements of Passing Sight Distance for Two-Lane Undivided Highways

(Source: Exhibit 3-4, AASHTO 2004 [5])



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, depending on the design speed. These studies involved extensive field observations of driver behavior during passing manoeuvres. The distance travelled in the left lane by the passing vehicle is:

$$d_2 = \frac{V\,t_2}{3.6}$$

where:

d_2 = the distance travelled while the passing vehicle occupies the left lane (m)
V = average speed of the passing vehicle (km/h)
t_2 = time the passing vehicle occupies the left lane (s)

Clearance length (d_3)

The clearance length between the opposing and passing vehicles at the end of the manoeuvre found in the passing study varied from 33 m to 92 m. 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:

$$d_4 = \frac{2\,d_2}{3}$$

The minimum passing sight distances for two-lane undivided highways are indicated in Table B-2-4a.

Table B-2-4a Minimum Passing Sight Distance on Two-Lane Undivided Highways

Design Speed (km/h)	Minimum Passing Sight Distance (m)
40	275
50	345
60	420
70	485
80	560
90	620
100	680
110	740
120	800
130	860

These minimum passing sight distances for design are different than the distances used as warrants for placing pavement markings for no-passing zones. Those values, described in Section B.2.5, are substantially less than design passing sight distances and are derived for traffic operation needs that are based on different assumptions than those for highway design.

Passing sight distance is based on an eye height of 1.08 m and an object height (of the opposing vehicle) of 1.30 metres. The minimum vertical crest curves that satisfy the criteria for passing sight distance are given in Section B.4.4.

B.2.5 No Passing Zone Sight Distance

The no-passing zone sight distance (NPZSD) was formerly known as the non-striping sight distance (NSSD).

The no passing zone sight distance is the limiting value used to determine when no-passing pavement markings are required. The pavement marking is a solid yellow directional dividing line (also known as a barrier line). The no passing zones shall be established where the available sight distance is equal to, or less than, the required no passing zone sight distance. Although passing sight distance is a desirable condition on two-lane undivided highways, the no passing zone sight distance is generally adequate for safe passing manoeuvres.

The no passing zone 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 completed where full passing sight distance is not available, depending on the timing of oncoming vehicles.

Refer to Figure B-2-4a depicting the phases and elements for passing sight distance. In the case of passing sight distance, if an oncoming vehicle comes into view at the critical moment (i.e., at the end of the first phase), there is deemed to be sufficient time to complete the pass safely. In the case of no passing zone sight distance, if an oncoming vehicle appears at the critical moment, there is only sufficient time available to abort the pass.

For design in Alberta, the chosen values for no passing zone sight distance are based on the “Manual of Uniform Traffic Control Devices for Canada” (MUTCDC) [6]. Both Alberta Transportation and MUTCDC use an eye height and object height of 1.15 metres. Alberta Transportation uses the design speed of the roadway to determine the no passing zone sight distance.

Effort should be made to achieve at least the minimum passing sight distance indicated in Table B-2-4a. If the minimum passing sight distance cannot be attained, effort should be made to provide the minimum no passing zone sight distance indicated in Table B-2-5a. This maximizes passing opportunities and consequently improves the level of service. This is especially important on higher volume highways in rolling terrain.

Roadways with available sight distances that are immediately above the minimum values for no passing zone sight distance may cause a false feeling of safety for passing because of the absence of barrier lines. Sight distances only slightly greater than the no passing zone values should be increased as much as economically possible.

Frequent barrier lines (due to not meeting the minimum value for no passing zone sight distance) are likely to appear unreasonable to a driver and may be ignored. If it is not feasible to provide the no passing zone 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. Designers may, however, consider increasing the length of barrier lines, where appropriate, to take into account a short separation between successive no-passing zones, steep upgrades, hazardous conditions, complex combinations of geometric features, etc. These measures should be done with careful consideration of their implications on the level of service.

In general, designers should strive to achieve at least 75 percent of the length of a highway as free of barrier lines. Higher percentages are desirable on higher volume roads. Barrier lines are used at intersections and some climbing/passing lanes even where there is no sight-line restriction. Where passing opportunities are limited, designers should assess the passing demand to determine if the construction of climbing or passing lanes is appropriate. This is especially true on new construction projects.

Alberta Transportation has adopted a no passing zone sight distance of 475 m for design speeds of 100 km/h through 130 km/h. Although generally reserved for freeways and divided arterials, the 120 km/h and 130 km/h design speeds are included because many divided roadways are built in stages, with the first stage being an undivided roadway with a design speed of 120 km/h or 130 km/h. Although the design speed is higher, the posted speed is 100 km/h because the roadway is two-way undivided. Because these roadways may operate for many years as an undivided facility, a no passing zone sight distance of 475 m has been adopted for these design speeds.

The no passing zone sight distances for two-lane, undivided highways are indicated in Table B-2-5a. The minimum vertical crest curves that satisfy the criteria for no passing zone sight distance are given in Section B.4.4.

Table B-2-5a Minimum No Passing Zone Sight Distance on Undivided Highways

(Source: Adapted from Table C2-2, 1998 MUTCDC [6])

Design Speed (km/h)	No Passing Zone Sight Distance (m)
50	-
60	-
70	240
80	275
90	330
100	475
110	475
120	475
130	475

B.2.5.1 No Passing Zones on Undivided Highways

The current practice of Alberta Transportation is to establish no passing zones on two-lane or three-lane undivided highways where passing must be prohibited because of restricted sight distance or other hazardous conditions on horizontal and vertical curves.

On a two-lane undivided highway, a solid yellow line indicates a no passing zone for the direction in which passing is prohibited. The directional dividing line is a double, solid, yellow line where a no-passing zone applies in both directions. A simultaneous solid yellow line and broken yellow line, indicates that passing is prohibited from the lane bounded by the solid line.

On a three-lane highway where there are two lanes in one direction and one lane in the other direction, the practice of Alberta Transportation is to use a double, solid, yellow line as the directional dividing line if the existing AADT exceeds 4000. If the existing AADT is less than 4000, a simultaneous solid yellow line and broken yellow line may be used to allow passing in the direction of the single lane, if sufficient sight distance is available.

Illustrations of these pavement markings on a two-lane undivided highway (sometimes referred to as barrier lines) are in the Alberta Transportation, “Highway Pavement Marking Guide” [7].

During the Design Stage

Designers are responsible for identifying required pavement markings and messages and determining where these markings are required (e.g., railway crossings, school zones, R.C.M.P. aircraft patrol zones, stop lines, etc.). Designers shall strive to achieve highly cost-effective designs while providing the standards that are appropriate for the highway according to these guidelines. Pavement marking requirements for all no passing zones are shown in the design drawings or design reports with the locations of the termini. The locations of the termini of the no passing zones will be derived from the designed horizontal and vertical alignments using the appropriate design speed and sight distance.

During the Construction Stage

Designers are required to determine the beginning and end points of the solid lines for no passing zones using the method outlined in the Alberta Transportation, “Highway Pavement Marking Guide” [7], so that the contractor can paint the lines accordingly. After painting has been completed for all types of construction projects, designers must confirm that the proper paint and message markings have been placed at the correct locations and meet the specified dimensions.

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. 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 appropriate, standard traffic control devices, stopping sight distance might not provide sufficient visible distance for drivers to appreciate advance warnings and perform the necessary manoeuvres. There are many locations where it would be better to provide a 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 information or the threat potential of a hazard;
- select appropriate action; and
- 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 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 (e.g., from roadway elements, traffic, traffic control devices and advertising signs);
- construction zones; and
- locations with a high likelihood of unfamiliar drivers, such as airports.

The decision sight distances in Table B-2-6a are appropriate at critical locations and serve as criteria for evaluating the suitability of the available sight distance at these locations. Because of the additional safety and manoeuverability these lengths provide, 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 that are likely to be encountered.

Ranges of values for decision sight distance, that are applicable to most situations, have 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 calculated decision sight distances are given in Table B-2-6a.

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

For determining the available decision sight distance, an eye height of 1.08 m is used together with an appropriate object height, depending on 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 eye height for the various design vehicles, minimum crest vertical curves that satisfy the requirements for decision sight distance are not given.

Table B-2-6a shows a range of minimum values for decision sight distance. The required decision sight distance increases with the design speed, the complexity of the action taken by the driver, and with the complexity of the surroundings. When using these sight distances, planners and designers should consider eye and object heights appropriate for specific applications. Refer to Section B.2.1, General Considerations and Section B.2.2, Criteria for Measuring Sight Distance and Chapter D.5, Design Vehicle, for further information.

Table B-2-6a Minimum Decision Sight Distance

(Source: Adapted from Table 3-3, AASHTO 2018 [3])

Design Speed (km/h)	Minimum Decision Sight Distance for Avoidance Maneuver (m) (Rounded)									
	Manoeuvre A		Manoeuvre B		Manoeuvre C		Manoeuvre D		Manoeuvre E	
	t	DSD	t	DSD	t	DSD	t	DSD	t	DSD
50	3.0	70	9.1	155	10.2	145	12.1	170	14.0	195
60	3.0	95	9.1	195	10.2	170	12.1	205	14.0	235
70	3.0	115	9.1	235	10.2	200	12.1	240	14.0	275
80	3.0	140	9.1	275	10.2	230	12.1	270	14.0	315
90	3.0	170	9.1	320	11.2	280	12.9	325	14.5	365
100	3.0	200	9.1	370	10.7	300	12.5	350	14.0	390
110	3.0	230	9.1	420	10.7	330	12.5	385	14.0	430
120	3.0	265	9.1	470	10.7	360	12.5	420	14.0	470
130	3.0	300	9.1	525	10.7	390	12.5	455	14.0	510

Notes:

- Avoidance manoeuvre A: Stop on rural roadway, t = 3.0s
- Avoidance manoeuvre B: Stop on urban roadway, t = 9.1s
- Avoidance manoeuvre C: Speed/path/direction change on rural roadway; t varies between 10.2 and 11.2 s, and is specific for each design speed.
- Avoidance manoeuvre D: Speed/path /direction change on suburban roadway; t varies between 12.1 and 12.9 s, and is specific for each design speed.
- Avoidance manoeuvre E: Speed /path/direction change on urban roadway; t varies between 14.0 and 14.5 s, and is specific for each design speed.

For avoidance manoeuvres A and B, the required decision sight distance is the sum of the distance travelled during the perception-reaction (pre-manoeuvre) time and the braking distance to stop a vehicle once the brakes have been applied:

$$\mathrm{DSD} = 0.2778\,\mathrm{V}\,\mathrm{t} + \frac{0.0386\,\mathrm{V}^2}{a}$$

where:

DSD = decision sight distance (m)
V = design speed (km/h)
t = perception-reaction (pre-manoeuvre) time (s)
a = deceleration rate, 3.4 m/s^2

For avoidance manoeuvres C, D, and E, the required decision sight distance is the distance travelled during the perception-reaction (pre-manoeuvre) and manoeuvre times:

$$\mathrm{DSD} = \frac{\mathrm{V}\,\mathrm{t}}{\left(\frac{3600}{1000}\right)} = \frac{\mathrm{V}\,\mathrm{t}}{3.6} = 0.2778\,\mathrm{V}\,\mathrm{t}$$

where:

DSD = decision sight distance (m)
V = design speed (km/h)
t = total of perception-reaction (pre-manoeuvre) time and manoeuvre time (s)

B.3 HORIZONTAL ALIGNMENT

B.3.1 Introduction

The horizontal alignment of a road is usually a series of tangents and curves. When a spiral curve is not used, the tangents are connected with a simple curve. A curvilinear alignment is a horizontal alignment in which curves are connected by spiral transitions, generally without connecting tangents.

Deficiencies in horizontal alignment are typically more critical than those in vertical alignment. Horizontal alignment deficiencies may necessitate a lower posted speed at site-specific locations. Lowering the posted speed due to vertical alignment deficiencies may not be required unless they occur at hazardous locations such as near intersections.

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 roadways. Specific controls for horizontal alignment are discussed in other sections of this chapter.

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 a winding alignment, composed of short curves, should be avoided since it tends to cause erratic operation, which may lead to collisions. Along with the importance of the aesthetic qualities of curvilinear alignment, long tangents must be provided on two-lane undivided highways to meet passing sight distance requirements.

2. The use of the minimum curve radius should be avoided, if possible, to establish an alignment based on the selected design speed. Generally use a larger radius; reserving the minimum radius for critical locations.
3. Alignment based on a consistent design speed should be provided. Small-radius 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 use of a small-radius curve, the critical curve should be preceded by curves with successively smaller radii. In this way, erratic operation and collisions can be minimized because the driver may 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 (a sudden change in direction). Curves should generally be long enough to provide an aesthetically pleasing alignment (refer to Figure B-3-2d). A deflection angle of 30 minutes requires a curve; smaller deflections do not. For smaller deflection curves (between 30 minutes and 1 degree), that occur on rural roads in open country, a minimum curve length of 350 m should be used to maintain a pleasing appearance. For determining curve length where spiral curves are applied, 50% 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 superelevation (i.e., normal crown curves) are very desirable for small deflections.
5. Small-radius 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 may appear unpleasant. Some drivers may 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 of the connecting tangent (in metres), is less than four times the design speed (in kilometres per hour). Broken back curves are not permitted on bridge structures. Inflection points on structures introduce safety concerns related to drainage / icing while adding complexity to the structural design.
7. Long spirals should be used whenever possible rather than compounding simple curves. If it is necessary to compound simple curves without a spiral between them, the ratio of the longer radius to the shorter radius should not exceed 1.5. A ratio of 1:1.25 is more desirable on high-speed roads where the speeds are at or near the maximum for the curvature (Section 3.2.6.1 of the 2017 Transportation Association of Canada (TAC), “Geometric Design Guide for Canadian Roads” [8]).
8. Abrupt alignment reversal must be avoided. When reverse curves (see Figure B-3-2c) are too close, it is difficult to superelevate them properly, which may result 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. Insufficient tangent length may result in a flat road surface (no cross-slope) during the superelevation transition from one curve to the next.
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. The horizontal alignment should be co-ordinated with the vertical alignment to avoid locations where flat spots on the road surface occur when superelevation transitions on a crest or sag curve, or where the beginning of a horizontal curve is obscured by a preceding crest curve.
11. An appropriate design speed should be maintained to support driver expectation. Speeds should match the terrain and alignment of the roadway, and should be consistent to the best extent possible.

12. The horizontal alignment may be affected by climate conditions. Constraints may be present due to conditions such as blowing snow in open terrain or known avalanche areas.
13. Traffic volumes and vehicle mix (current, projected, and seasonal variations) should be taken into account.
14. The presence of major utilities can pose design constraints as they may be difficult and costly to relocate, and the horizontal alignment may have to be designed around them. This is especially common in high-density urban areas.
15. Bridge rails can affect the available horizontal sight distance along a roadway. Solutions include a tangent alignment of the bridge, increased curve radius and a wider shoulder on the inside of the curve. Bridge rails can also reduce the available intersection sight distance from a ramp intersection at a grade separation, or other intersections near the bridge. Solutions include an increased K value for vertical curves, a wider shoulder and a longer distance between the end of the bridge and the intersection.
16. Bridge decks, including abutments, are typically susceptible to preferential icing (the bridge is icy when the approach road is not). As such, braking and steering adjustments on the bridge deck should be avoided. Solutions include reducing the longitudinal gradient, reducing the superelevation (provide a tangent alignment or large radius curve), providing a constant road cross section along the bridge (no spirals, avoid tapers), avoiding intersections on, or in close proximity to, the bridge (to avoid braking or acceleration on the structure) and considering a culvert structure. Anti-icing measures may be required if a suitable solution cannot be developed. Refer to the latest version of the Alberta Transportation, “Bridge Conceptual Design Guidelines” for further information.

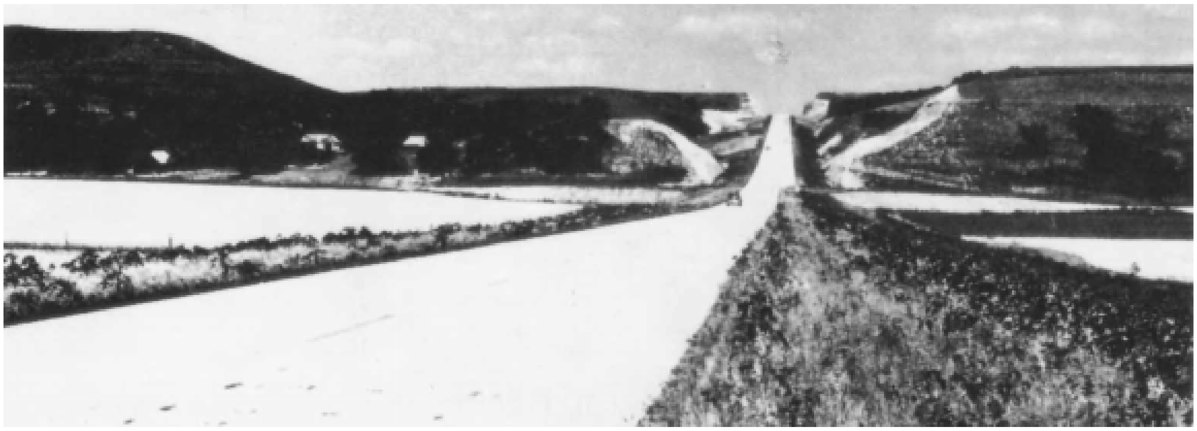
Figures B-3-2a through B-3-2i are illustrations that show examples of horizontal alignment concepts.

Figure B-3-2a Illustration of Design

The figure above is an example of a discontinuous alignment. Horizontal curves with larger radii and less deflection would have been a more pleasing design.

These contrasting photographs illustrate the difference between a design with long tangents and short curves versus a continuous curvilinear alignment. The top view indicates separate segments of highway with little continuity along 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



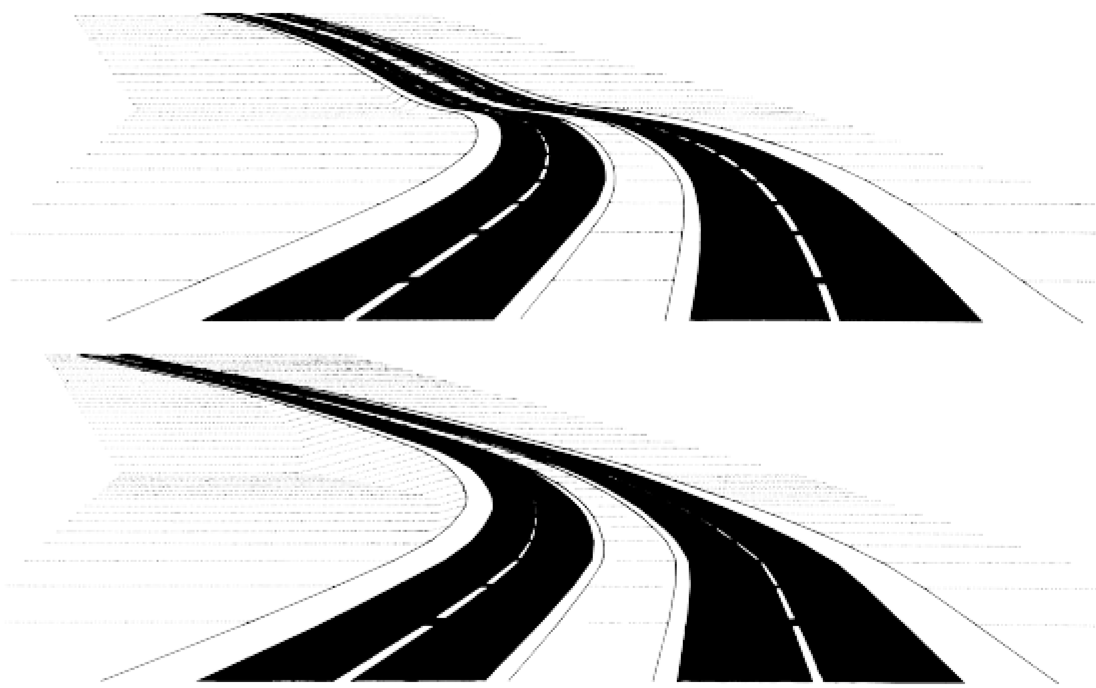
Alignments should be as directional as possible, but should be consistent with the topography. A flowing line that generally conforms to the natural contours is aesthetically preferable to one with long tangents that slash through the terrain. The construction scars should be kept to a minimum and natural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

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

Figure B-3-2c Illustration of Design

Image from: "Driving" (https://driving.ca) [9]

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

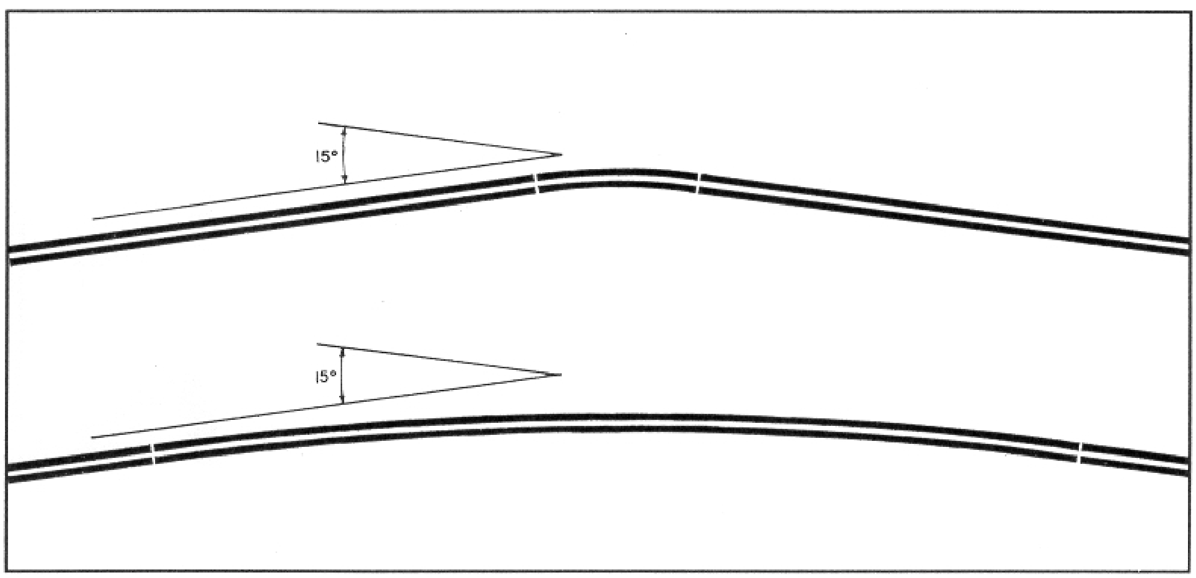


This drawing illustrates the effect of superimposing a short vertical curve on a relatively long horizontal curve. The appearance of a settlement of the roadway can be eliminated by increasing the length of the vertical curve.

Figure B-3-2d Illustration of Design

Image from: "OZROADS: The Australian Roads Website" (https://www.ozroads.com.au) [10]

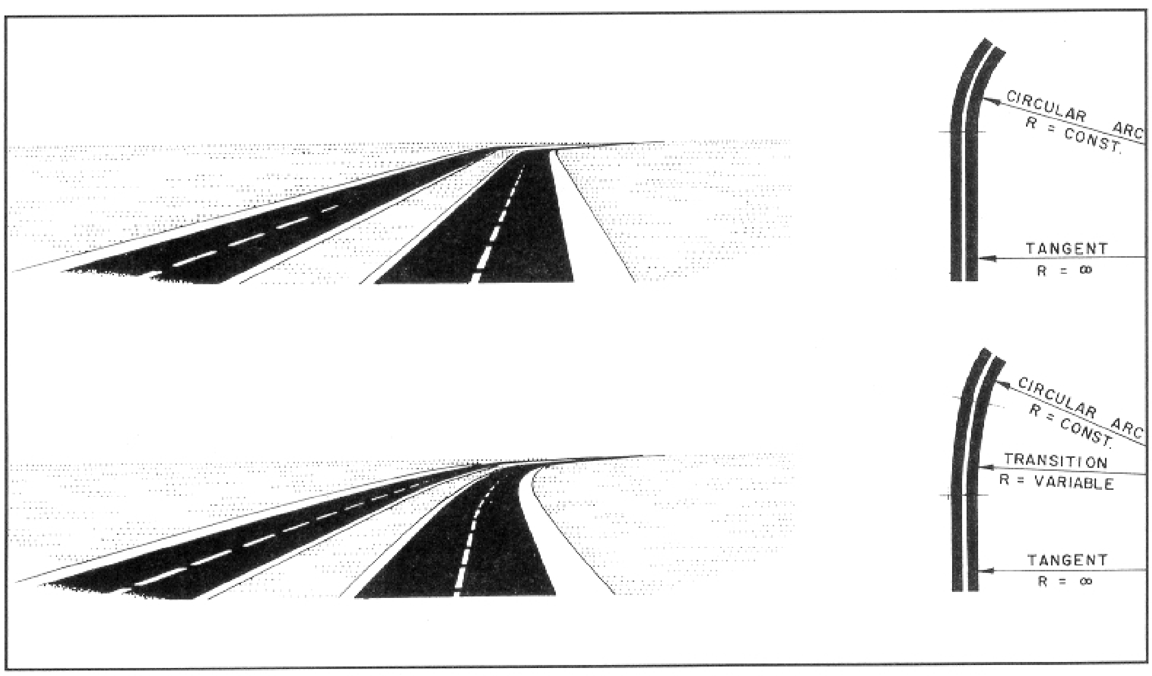
The sagging effect is evident in this picture.



For small deflection angles, curves should be sufficiently long (by increasing the radius) to avoid the appearance of a kink.

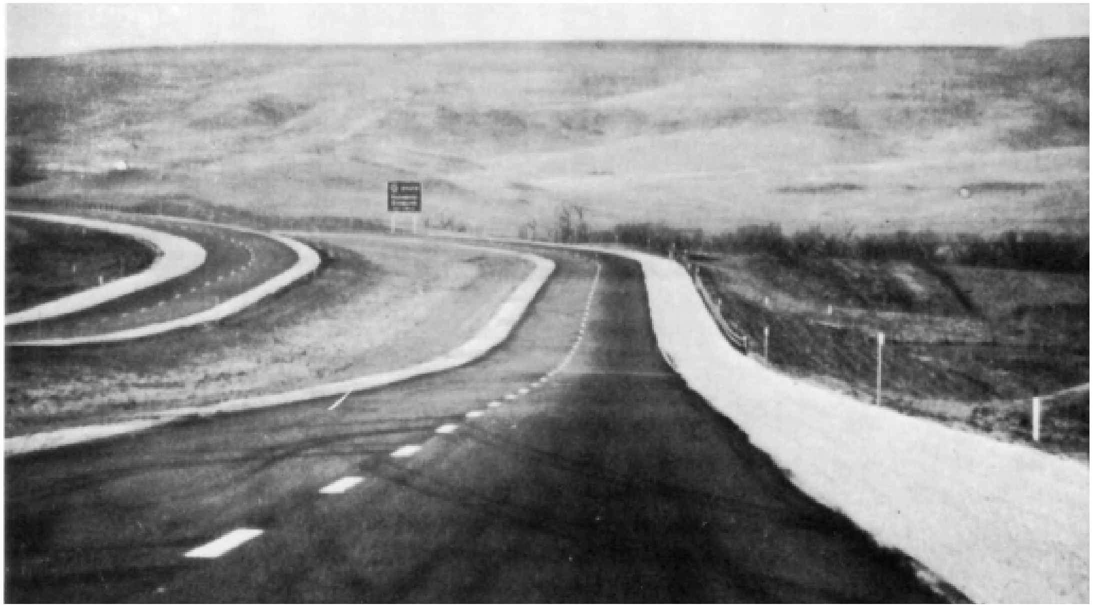
Figure B-3-2e Illustration of Design

A larger curve radius (and resultant larger curve length) would remove the appearance of the kink from this alignment.

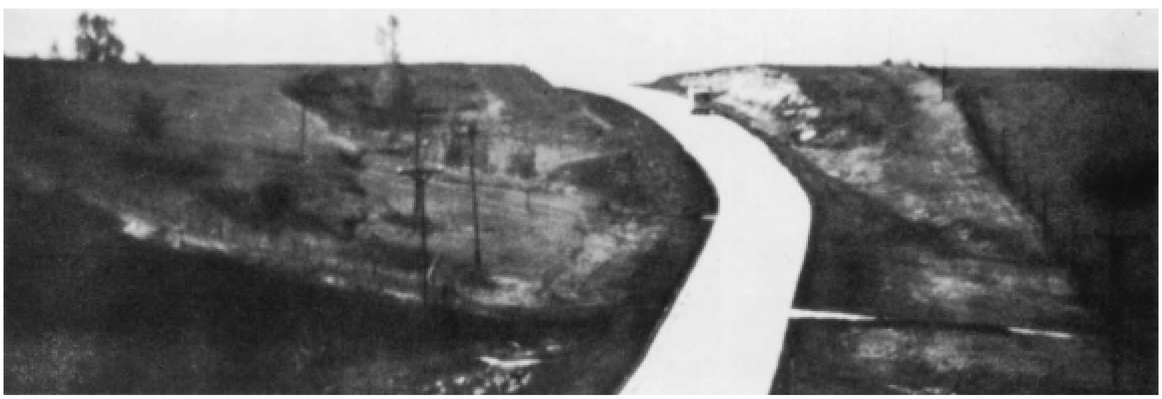


One effect of perspective viewing is that distant objects seem nearer than they really are. The simple curve (at the top) appears to diverge from the tangent rather rapidly and the curve no longer seems continuous. To remedy this situation, the use of long spirals is recommended (creating a spiral curve).

Figure B-3-2f Illustration of Design

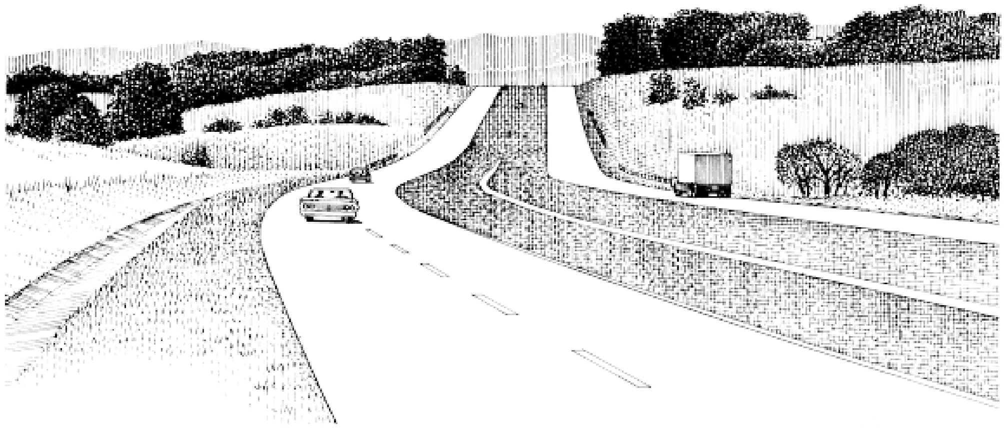


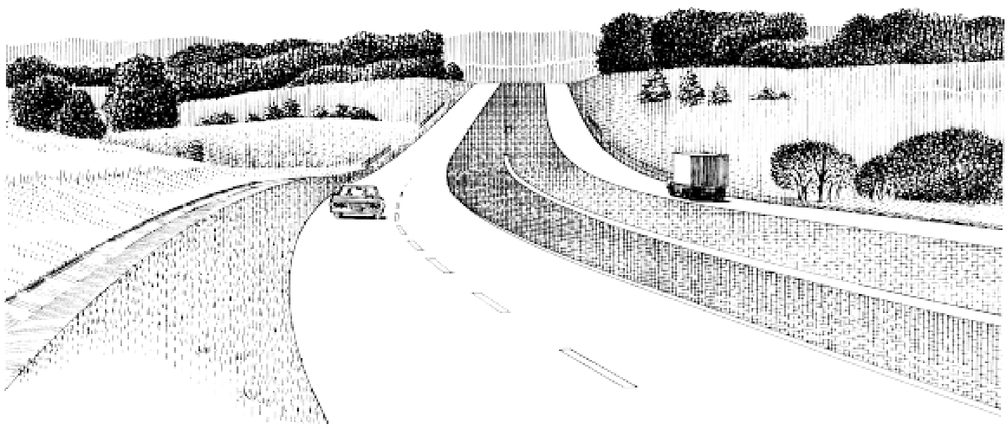
The horizontal curve does not appear to be blend in with the straight alignment. It visually jerks away from the tangent alignment. The left-hand roadway gives the driver a good clue that the road continues to the left and does not merely fade away.



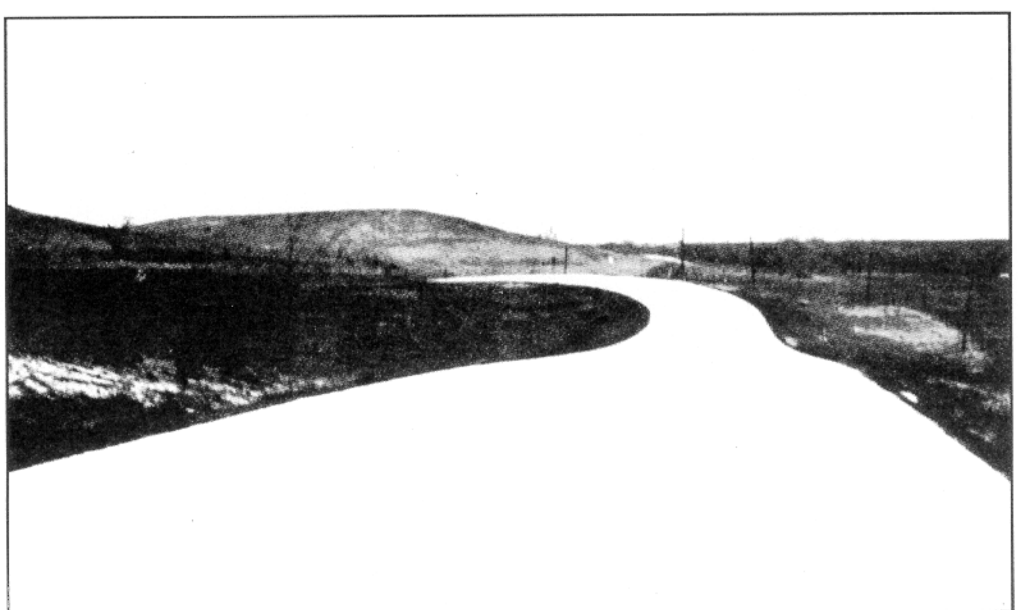
A long spiral beginning at the first entrance at the bottom of the hill and ending near the position of the truck would have improved the appearance of this curve.

Figure B-3-2g Illustration of Design



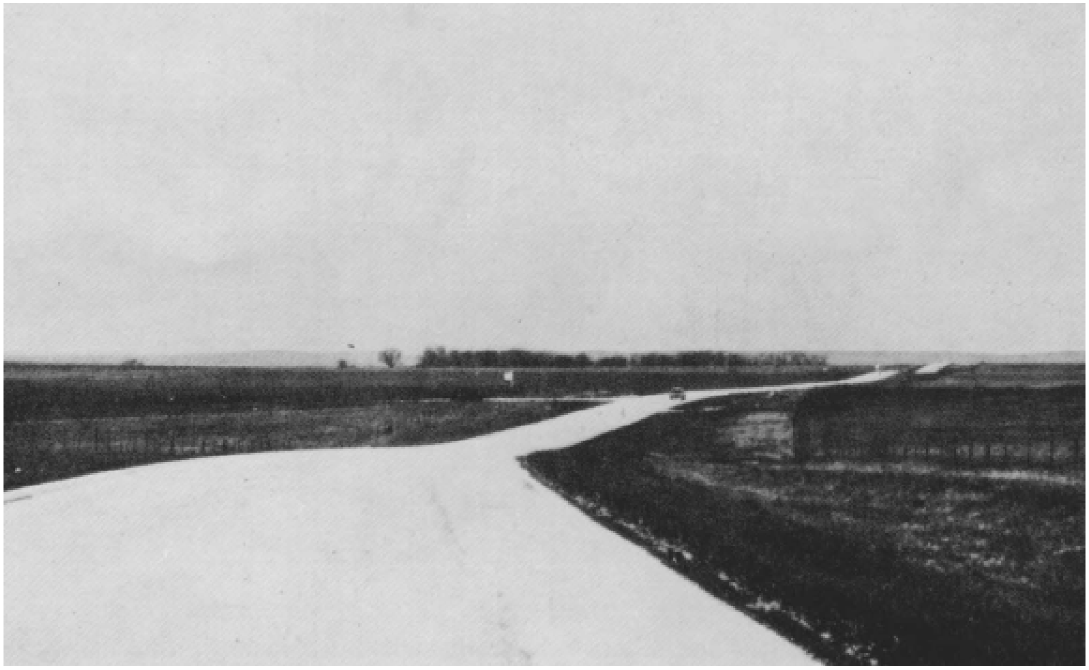


Short vertical curvature at the end of a long horizontal curve will usually produce a warped appearance. This situation can be improved by using a longer vertical curve.

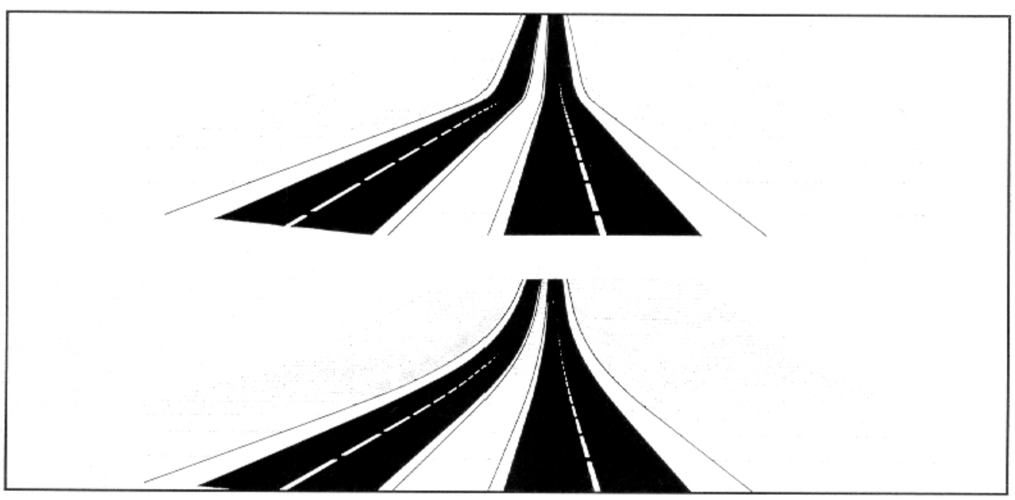


A short vertical curve at the beginning of a horizontal curve is not a well-balanced design.

Figure B-3-2h Illustration of Design



The sag curve near the end of the horizontal curve makes the horizontal curve appear too sharp. A flatter (longer) vertical curve would have been desirable.



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 alternative is a longer vertical curve.

Figure B-3-2i Illustration of Design

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

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

When designing a curve, how the driver will navigate through it and what challenges they will face should be considered. Curves are visually demanding because the current position of the vehicle is in a different place from the future position. Drivers can underestimate curvature, especially on tight curves, leading to speeds that are inappropriate for the curve radius. Considering such factors, the presence of curves can lead to a higher incidence of run-off-road collisions; some of which can be fatal. Approximately 95% of the reasons for a run-off-road collision are due to behaviour on the driver's part (e.g., over-compensation, high speeds, distractions, etc.) [11].

B.3.3 Maximum Safe Side Friction Factors

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

On any curve, some drivers will travel in excess of the design speed. When 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.

TAC has established maximum values for the safe side friction factor used in highway curve design. The maximum safe values for each design speed are shown in Table B-3-3a.

Table B-3-3a Maximum Safe Side Friction Factors

(Source: Table 3.2.1 and Table 3.2.2 of TAC 2017 [8])

Design Speed (km/h)	Maximum Safe Side Friction Factor	
	Rural Roads and High Speed Urban Roads	Low Speed Urban Roads
30		0.31
40	0.17	0.25
50	0.16	0.21
60	0.15	0.18
70	0.15	
80	0.14	
90	0.13	
100	0.12	
110	0.10	
120	0.09	
130	0.08	

B.3.4 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 a particular design speed, use of a smaller radius curve would call for superelevation beyond the limit considered practical, for operation with tire friction beyond the safe limit, or some combination of both. On steep downgrades, the minimum curve radius should be increased by 10% for each 1% increase in grade over 3%. Designers may refer to Section 3.2.2.4 of TAC [8] for more information.

The radius is determined using the following equation:

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

where:

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

Minimum radii for corresponding design speeds are shown in Tables B-3-5-3a through B-3-5-3e.

B.3.5 Rates of Superelevation for Design

B.3.5.1 Speed to be used for Superelevation

The speed to be used for selecting the superelevation rate is the design speed of the roadway.

B.3.5.2 Maximum Superelevation

Several factors influence the maximum rate of superelevation that is used for highway design:

1. Climate conditions
   - Frequency and amount of snow and icing
2. Terrain conditions
   - Flat versus mountainous
3. Type of area
   - Rural or urban
   - Adjacent land use
4. Frequency of very slow moving vehicles that would be subject to uncertain conditions
5. Maintenance
6. Constructability
7. Presence of a bridge structure (maximum desirable 4% resultant gradient)

B.3.5.2.1 Rural Superelevation Rates

In Canada, provincial highway authorities typically chose either 0.06 m/m or 0.08 m/m as the maximum superelevation rate for rural highways, although some may specify 0.04 m/m. 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 0.08 m/m maximum table can result in smaller-radius curves not consistent with driver expectations in a rural environment. Use of isolated smaller-radius curves in a generally smooth, high-speed, rural alignment is discouraged.
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 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.

Alberta Transportation uses a maximum superelevation rate of 0.06 m/m for all rural roads. Use of the 0.08 m/m superelevation rate may be permitted at a constrained area where a curve radius at or above the minimum, based on a superelevation rate of 0.06 m/m, cannot be achieved.

The recommended superelevation rates for various radii for each design speed are given in Tables B-3-5-3a and B-3-5-3b. These tables are based on the form of superelevation distribution and side friction described as Method 5 in the 2018 AASHTO, “A Policy on Geometric Design of Highways and Streets” [3]. The methods are described later in this section. The values shown in the tables are also consistent with those currently recommended by TAC [8].

Table B-3-5-3a (e = 0.06 m/m), which is normally used on all rural highway curves, includes an inset table, “Values for Superelevation on Horizontal Curves Containing Major Intersections”. The table provides values for superelevation related to design speed and curve radii for curved alignments containing major intersections. An intersection is considered major if intersection treatment (flaring or channelization) is provided. In addition to providing superelevation rates, the inset table also indicates those curves on which intersections may be permitted. Intersections on curve are undesirable for safety reasons and should be avoided where possible. Intersections on curves may be permitted where the combination of design speed and radius is shown on the special inset table, or the radius is greater than 4000 m. 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 0.038 m/m.

B.3.5.2.2 Urban Superelevation Rates

In urban areas, the maximum superelevation values used on Alberta Transportation roadways can range from 0.02 m/m to 0.06 m/m. The following values indicate the generally allowable maximum superelevation rates:

1. Local Roads – generally normal crown
2. Collector Roads – used occasionally with maximum rates of 0.02 m/m (reverse crown) or 0.04 m/m.
3. Arterials (undivided) – 0.04 m/m to 0.06 m/m
4. Arterials (divided) – 0.06 m/m
5. Expressways and Freeways – 0.06 m/m
6. Interchange Ramps – 0.06 m/m

Considerations when selecting the appropriate superelevation rate in an urban environment include:

- 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 adjacent streets and development accesses.
- The 0.04 m/m superelevation table may be used for an urban roadway system, and is appropriate where surface icing and interrupted flow is expected. Refer to Table B-3-5-3c (e_{max} = 0.04 m/m), which is based on TAC Table 3.2.5 [8], for more information.
- Superelevation rates in excess of 0.04 m/m are not recommended where curved alignments pass through existing or future intersections.
- A normal crown on a low speed, urban, local road may result in slower speeds by drivers in a residential area.
- On low speed (30 km/h to 60 km/h) urban roads, drivers have developed a higher threshold of discomfort through conditioning, and are willing to accept more side friction than in rural or higher speed (≥ 70 km/h) urban conditions. Using higher side friction values reduces the superelevation requirement. Designers may refer to Table 3.2.2 and Figure 3.2.4 of TAC [8] for more information regarding low speed urban roads.
- For urban freeways and expressways (including interchange ramps), the superelevation rates are designed the same as for rural roadways using Table B-3-5-3a (e_{max} = 0.06 m/m).
- For high-speed urban arterials where intersections are widely spaced and access is restricted, the designer should use Table B-3-5-3a (e_{max} = 0.06 m/m). Table B-3-5-3c (e_{max} = 0.04 m/m) may be used in localized urban areas where there is an intersection on a curve or higher superelevation rates are not attainable due to surrounding constraints. This table is derived using

the same methods as Table B-3-5-3a and the superelevation rates are based on low side friction values.

- For low speed (30 km/h to 60 km/h) urban arterials, the roadway should first be assessed to determine if superelevation Table B-3-5-3a (e_{max} = 0.06 m/m) or Table B-3-5-3c (e_{max} = 0.04 m/m) can be applied. If constraints do not allow use of either of these tables, Table B-3-5-3d (Urban Design, e_{max} = 0.04 m/m) or Table B-3-5-3e (Urban Design, e_{max} = 0.06 m/m), both from TAC [8] may be used. These tables show normal crown, reverse crown (2% superelevation), or higher superelevation depending on radius and design speed. They also indicate the minimum radii for the maximum superelevation rate.
- Designers should use Table B-3-5-3a, (e_{max} = 0.06 m/m) or Table B-3-5-3c (e_{max} = 0.04 m/m) in normal conditions and Table B-3-5-3d (Urban Design, e_{max} = 0.04 m/m) or Table B-3-5-3e (Urban Design, e_{max} = 0.06 m/m) in constrained conditions. Table B-3-5-3b (e_{max} = 0.08 m/m) is only to be used where the 0.08 m/m maximum superelevation is justified for some reason (e.g., a curve constraint or retrofitting to suit for an existing curve).
- The superelevation rates given in Tables B-3-5-3d and B-3-5-3e are derived using higher side friction values than those given in Tables B-3-5-3a and B-3-5-3b.

For further information on the development of superelevation rates in an urban setting, refer to the 2018 AASHTO, “A Policy on Geometric Design of Highways and Streets” [3] or the 2017 TAC, “Geometric Design Guide for Canadian Roads” [8].

B.3.5.3 Minimum Superelevation

When superelevation is used, the minimum rate is not less than the rate of crossfall of the normal crown rate (normally 0.02 m/m for paved roads and 0.03 m/m for gravel roads).

Table B-3-5-3a Values for Superelevation and Spiral Parameters (e_{max} = 0.06 m/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.	70 e	70 A Min.	70 A Des.
10 000	NC			NC			NC			NC		
9 000	NC			NC			NC			NC		
8 000	NC			NC			NC			NC		
7 000	NC			NC			NC			NC		
6 000	NC			NC			NC			NC		
5 000	NC			NC			NC			NC		
4 500	NC			NC			NC			NC		
4 000	NC			NC			NC			NC		
3 500	NC			NC			NC			NC		
3 000	NC			NC			NC			NC		
2 500	NC			NC			NC			RC	315	315
2 200	NC			NC			NC			RC	295	295
2 000	NC			NC			NC			RC	275	275
1 800	NC			NC			RC	245	245	RC	270	270
1 700	NC			NC			RC	240	240	RC	260	260
1 600	NC			NC			RC	230	230	RC	255	255
1 500	NC			NC			RC	225	225	RC	250	250
1 400	NC			NC			RC	225	225	0.021	240	240
1 300	NC			NC			RC	225	225	0.021	230	230
1 250	NC			NC			RC	200	200	0.022	225	225
1 200	NC			NC			RC	200	200	0.023	225	225
1 150	NC			NC			RC	200	200	0.024	215	215
1 100	NC			NC			0.021	200	200	0.025	210	210
1 050	NC			NC			0.021	175	175	0.026	205	205
1 000	NC			RC	170	170	0.021	175	175	0.027	200	200
950	NC			RC	170	170	0.022	175	175	0.028	200	200
900	NC			RC	150	150	0.023	175	175	0.029	200	200
850	NC			RC	150	150	0.024	175	175	0.030	180	180
800	NC			RC	150	150	0.025	160	160	0.031	175	175
750	NC			0.021	150	150	0.026	160	160	0.032	175	175
700	NC			0.021	140	140	0.027	150	150	0.034	175	175
650	RC	120	120	0.022	140	140	0.029	150	150	0.035	175	175
600	RC	120	120	0.024	125	125	0.030	140	140	0.037	175	175
575	RC	120	120	0.025	125	125	0.031	140	140	0.038	150	150
550	RC	120	120	0.025	125	125	0.032	125	125	0.039	150	150
525	RC	100	100	0.026	120	120	0.033	125	125	0.040	150	150
500	RC	100	100	0.027	120	120	0.034	125	125	0.041	150	150
475	0.021	100	100	0.028	120	120	0.035	125	125	0.042	140	150
450	0.021	100	100	0.029	120	120	0.036	125	125	0.043	140	150
420	0.022	90	90	0.030	100	100	0.037	115	120	0.044	140	150
400	0.023	90	90	0.031	100	100	0.038	115	120	0.045	140	150
380	0.024	90	90	0.032	100	100	0.039	115	120	0.046	125	135
350	0.025	90	90	0.034	100	100	0.041	110	115	0.048	125	135
340	0.026	90	90	0.035	100	100	0.042	110	115	0.049	120	125
320	0.027	80	80	0.036	90	100	0.043	100	110	0.050	120	125
300	0.028	80	80	0.037	90	100	0.044	100	110	0.051	120	125
280	0.029	80	80	0.038	90	100	0.046	100	110	0.053	120	125
250	0.031	75	80	0.040	85	90	0.048	90	100	0.055	120	125
240	0.032	75	80	0.041	85	90	0.049	90	100	0.056	110	120
230	0.033	70	80	0.042	80	90	0.049	90	100	0.056	110	120
220	0.034	70	80	0.043	80	90	0.050	90	100	0.057	110	120
210	0.035	70	80	0.044	80	90	0.051	90	100	0.058	110	110
200	0.036	70	75	0.045	75	90	0.052	85	100	0.059	110	110
190	0.037	70	75	0.046	75	90	0.053	85	100	0.060	110	110
180	0.038	60	75	0.047	70	90	0.054	85	90			
170	0.039	60	75	0.048	70	90	0.055	85	90			
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	75	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	70	0.054	65	75						
120	0.046	60	70	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									
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			minimum R=90			minimum R=130			minimum R=190		

RADIUS (m)	80 e	80 A Min.	80 A Des.	90 e	90 A Min.	90 A Des.	100 e	100 A Min.	100 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			RC	490	490
4 000	NC			NC			RC	475	475
3 500	NC			RC	420	420	RC	440	440
3 000	RC	375	375	RC	390	390	RC	410	410
2 500	RC	325	325	0.021	360	360	0.022	380	380
2 200	RC	325	325	0.022	330	330	0.025	350	350
2 000	RC	300	300	0.023	300	300	0.026	335	335
1 800	0.021	300	300	0.025	300	300	0.029	310	310
1 700	0.022	275	275	0.026	300	300	0.030	310	310
1 600	0.023	275	275	0.028	270	270	0.031	290	290
1 500	0.024	250	250	0.029	270	270	0.032	290	290
1 400	0.025	250	250	0.030	270	270	0.034	280	280
1 300	0.026	250	250	0.031	270	270	0.036	260	260
1 250	0.027	225	225	0.032	240	240	0.037	260	260
1 200	0.028	225	225	0.033	240	240	0.038	260	260
1 150	0.029	225	225	0.034	240	240	0.040	250	250
1 100	0.030	225	225	0.035	240	240	0.041	250	250
1 050	0.031	200	200	0.036	225	225	0.042	235	235
1 000	0.032	200	200	0.037	225	225	0.043	235	235
950	0.033	200	200	0.038	225	225	0.044	235	235
900	0.034	200	200	0.039	200	200	0.045	225	225
850	0.035	200	200	0.041	200	200	0.047	225	225
800	0.036	175	175	0.042	200	200	0.048	210	215
750	0.037	175	175	0.043	200	200	0.050	210	215
700	0.039	175	175	0.045	185	195	0.051	200	210
650	0.041	175	175	0.046	185	195	0.052	200	210
600	0.042	175	175	0.048	175	185	0.054	190	200
575	0.043	175	175	0.049	175	185	0.055	190	200
550	0.044	175	175	0.050	175	185	0.056	190	200
525	0.045	150	160	0.051	160	175	0.057	190	190
500	0.046	150	160	0.052	160	175	0.059	190	190
475	0.047	150	160	0.053	160	175	0.059	190	190
450	0.048	150	160	0.054	160	175	0.060	190	190
420	0.050	135	150	0.056	160	165			
400	0.051	135	150	0.057	160	165			
380	0.052	135	150	0.058	160	165			
350	0.054	125	140	0.059	160	160			
340	0.054	125	140	0.060	160	160			
320	0.056	125	135						
300	0.057	125	135						
280	0.058	125	135						
250	0.060	125	125						
	minimum R=250			minimum R=340			minimum R=440		

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

TABLE B-3-5-3a

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 Reverse Crown (superelevate at normal crown rate = 0.02 m/m)

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

For 2 lane pavements, use min. "A" values.

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

For 6 lane pavements: above dashed line use desirable "A" values, below the dashed line use desirable "A" value x 1.15.

A divided roadway having a median width less than 3 m 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.

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 4000 m. 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.

Table B-3-5-3b Values for Superelevation and Spiral Parameters (e_{max} = 0.08 m/m)

DESIGN SPEED (km/h)

RADIUS (m)	40			50			60			70			80		
	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		
9 000	NC			NC			NC			NC			NC		
8 000	NC			NC			NC			NC			NC		
7 000	NC			NC			NC			NC			NC		
6 000	NC			NC			NC			NC			NC		
5 000	NC			NC			NC			NC			NC		
4 500	NC			NC			NC			NC			NC		
4 000	NC			NC			NC			NC			NC		
3 500	NC			NC			NC			NC			NC		
3 000	NC			NC			NC			NC			RC	375	375
2 500	NC			NC			NC			RC	315	315	RC	325	325
2 200	NC			NC			NC			RC	295	295	0.021	325	325
2 000	NC			NC			NC			RC	270	270	0.021	300	300
1 800	NC			NC			RC	245	245	RC	270	270	0.023	300	300
1 700	NC			NC			RC	240	240	RC	260	260	0.024	275	275
1 600	NC			NC			RC	230	230	0.021	255	255	0.025	275	275
1 500	NC			NC			RC	225	225	0.021	255	255	0.027	250	250
1 400	NC			NC			RC	225	225	0.022	240	240	0.029	250	250
1 300	NC			NC			RC	225	225	0.024	230	230	0.030	250	250
1 250	NC			NC			RC	200	200	0.025	225	225	0.031	225	225
1 200	NC			NC			RC	200	200	0.026	220	220	0.032	225	225
1 150	NC			RC	185	185	RC	200	200	0.026	215	215	0.033	225	225
1 100	NC			RC	180	180	0.021	200	200	0.027	210	210	0.034	225	225
1 050	NC			RC	170	170	0.022	175	175	0.028	205	205	0.035	200	200
1 000	NC			RC	170	170	0.023	175	175	0.029	200	200	0.036	200	200
950	NC			RC	170	170	0.024	175	175	0.030	200	200	0.038	200	200
900	NC			RC	150	150	0.025	175	175	0.032	180	180	0.039	200	200
850	NC			RC	150	150	0.026	175	175	0.033	180	180	0.040	200	200
800	NC			0.020	150	150	0.027	160	160	0.035	175	175	0.042	175	195
750	NC			0.022	150	150	0.028	160	160	0.036	175	175	0.044	175	195
700	NC			0.023	140	140	0.030	150	150	0.038	165	165	0.046	175	190
650	RC	125	125	0.024	140	140	0.032	150	150	0.040	165	165	0.048	175	190
600	RC	120	120	0.026	125	125	0.034	140	140	0.042	150	160	0.050	165	185
575	RC	120	120	0.027	125	125	0.035	140	140	0.043	150	160	0.052	165	185
550	0.021	120	120	0.028	125	125	0.036	140	140	0.045	150	160	0.053	165	185
525	0.021	100	100	0.029	120	120	0.038	125	135	0.046	140	150	0.054	150	175
500	0.021	100	100	0.030	120	120	0.039	125	135	0.048	140	150	0.056	150	175
475	0.022	100	100	0.031	120	120	0.040	125	135	0.050	140	150	0.058	150	175
450	0.023	100	100	0.032	120	120	0.041	125	135	0.051	140	150	0.059	150	175
420	0.024	90	90	0.034	100	110	0.043	115	125	0.053	125	150	0.061	135	165
400	0.025	90	90	0.035	100	110	0.045	115	125	0.054	125	150	0.063	135	165
380	0.026	90	90	0.036	100	110	0.046	115	125	0.056	125	150	0.064	135	165
350	0.028	90	90	0.038	100	105	0.049	110	125	0.058	120	150	0.067	125	160
340	0.029	90	90	0.039	100	105	0.050	110	125	0.059	120	150	0.068	125	160
320	0.030	80	90	0.040	90	100	0.051	100	120	0.061	120	140	0.070	125	150
300	0.031	80	90	0.042	90	100	0.053	100	120	0.063	120	140	0.072	125	150
280	0.032	80	90	0.044	90	100	0.055	100	120	0.065	120	140	0.074	125	150
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
230	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	0.073	50	60												
60	0.075	50	60												
55	0.078	50	60												
50	0.080	50	60												
	minimum R=50														

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

TABLE B-3-5-3b

This table is not recommended for new construction projects on Alberta's rural roads. Table B-3-5-3a (e max=0.06) should be used for new construction projects.

e max. = 0.08

NOTES:

e is superelevation (m/m).

A is spiral parameter in meters.

NC is normal cross-section.

RC is Reverse Crown (superelevate at normal crown rate = 0.02 m/m)

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

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

For 6 lane pavements: above dashed line use desirable "A" values, below the dashed line use desirable "A" value x 1.15.

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

Table B-3-5-3c Values for Superelevation and Spiral Parameters (e_{max} = 0.04 m/m)

(Source: Table 3.2.5 of TAC 2017 [8])

design speed (km/h)	40		50		60		70		80		90		100	
radius (m)	e	A	e	A	e	A	e	A	e	A	e	A	e	A
7000	NC		NC		NC		NC		NC		NC		NC	
5000	NC		NC		NC		NC		NC		NC		NC	
4000	NC		NC		NC		NC		NC		NC		NC	
3000	NC		NC		NC		NC		NC		NC		RC	410
2000	NC		NC		NC		NC		RC	300	RC	315	RC	335
1500	NC		NC		NC		RC	240	RC	260	RC	275	0.022	290
1200	NC		NC		NC		RC	215	RC	230	0.021	245	0.026	260
1000	NC		NC		RC	180	RC	200	0.020	210	0.025	225	0.030	235
900	NC		NC		RC	175	RC	190	0.021	200	0.027	210	0.032	225
800	NC		NC		RC	165	RC	180	0.023	190	0.029	200	0.034	210
700	NC		RC	140	RC	150	0.020	165	0.026	175	0.032	185	0.036	200
600	NC		RC	130	RC	140	0.023	155	0.029	165	0.035	175	0.039	190
500	NC		RC	120	0.021	130	0.026	140	0.033	150	0.038	160	0.040	190
400	RC	95	RC	105	0.025	115	0.031	125	0.037	135	0.040	160		
350	RC	90	0.020	100	0.027	110	0.033	120	0.039	135				
300	RC	80	0.023	90	0.031	100	0.036	110	0.040	135				
250	RC	75	0.026	85	0.034	90	0.038	110						
220	RC	70	0.029	80	0.036	90	0.039	110						
200	0.021	65	0.031	75	0.038	90	0.040	110						
180	0.023	65	0.033	70	0.039	90								
160	0.025	60	0.035	70	0.040	90								
140	0.028	55	0.037	70										
120	0.031	50	0.039	70										
100	0.034	50	0.040	70										
90	0.036	50												
80	0.038	50												
70	0.040	50												
60	0.040	50												
	0.040	50	0.040	70	0.040	90	0.040	110	0.040	135	0.040	160	0.040	190
	minimum	R=60	minimum	R=100	minimum	R=150	minimum	R=200	minimum	R=280	minimum	R=380	minimum	R=490

NOTES:

e is superelevation (m/m).

A is spiral parameter in meters.

NC is normal cross-section.

RC is Reverse Crown (superelevate at normal crown rate = 0.02 m/m)

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

For 2,3 and 4 lane pavements, use indicated "A" values.

For 6 lane pavements: above dashed line use indicated "A" values, below the dashed line use indicated "A" value x 1.15.

Spiral parameters are minimum and higher-values should be used where possible.

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

e max. = 0.04

Table B-3-5-3d Superelevation Rate for Urban Design (e_{max} = 0.04 m/m)

(Source: Table 3.2.8 of TAC 2017 [8])

Radius (m)	Design Speed (km/h) 30	40	50	60	70	80	90	100
7000	NC	NC	NC	NC	NC	NC	NC	NC
5000								
4000								NC
3000						NC	NC	RC
2000					NC	RC	RC	
1500				NC	RC			
1200				RC				
1000								
900			NC					
800			RC					
700		NC						RC
600		RC					RC	0.027
500	NC						0.022	0.039
400	RC					RC	0.036	Min R=490
350						0.026	Min R=380	
300					RC	0.035		
250					0.027	Min R=280		
200				RC	0.040			
180				0.021	Min R=200			
160				0.027				
140				0.035				
120			RC	Min R=130				
100			0.026					
90			0.032					
80			0.039					
70		RC	Min R=80					
60		0.022						
50		0.032						
40		Min R=45						
30	RC							
20	0.040							
	Min R=20							
min. radius for normal crown	420	660	950	1290	1680	2130	2620	3180
min. radius for reverse crown	30	65	115	185	290	400	530	690

e_{max} = 0.04 m/m
NC = normal crown (-0.02 m/m)
RC = reverse crown (+0.02 m/m)

Table B-3-5-3e Superelevation Rate for Urban Design (e_{max} = 0.06 m/m)

(Source: Table 3.2.9 of TAC 2017 [8])

Radius (m)	Design Speed (km/h)							
	30	40	50	60	70	80	90	100
7000	NC	NC	NC	NC	NC	NC	NC	NC
5000								
4000								NC
3000						NC	NC	RC
2000					NC	RC	RC	
1500				NC	RC			
1200				RC				
1000								
900			NC					
800			RC					RC
700		NC						0.025
600		RC					RC	0.035
500	NC					RC	0.030	0.048
400	RC					0.026	0.045	Min R=440
350					RC	0.035	0.056	
300					0.025	0.045	Min R=340	
250				RC	0.036	Min R=250		
200				0.024	0.053			
180				0.030	Min R=190			
160				0.037				
140			RC	0.046				
120			0.026	Min R=120				
100			0.036					
90			0.043					
80		RC	0.052					
70		0.024	Min R=75					
60		0.032						
50		0.044						
40	RC	Min R=40						
30	0.030							
20	0.056							
	Min R=20							
min. radius for normal crown	420	660	950	1290	1680	2130	2620	3180
min. radius for reverse crown	40	80	135	220	330	450	600	770

e_{max} = 0.06 m/m
NC = normal crown (-0.02 m/m)
RC = reverse crown (+0.02 m/m)

B.3.5.4 Distribution of e and f

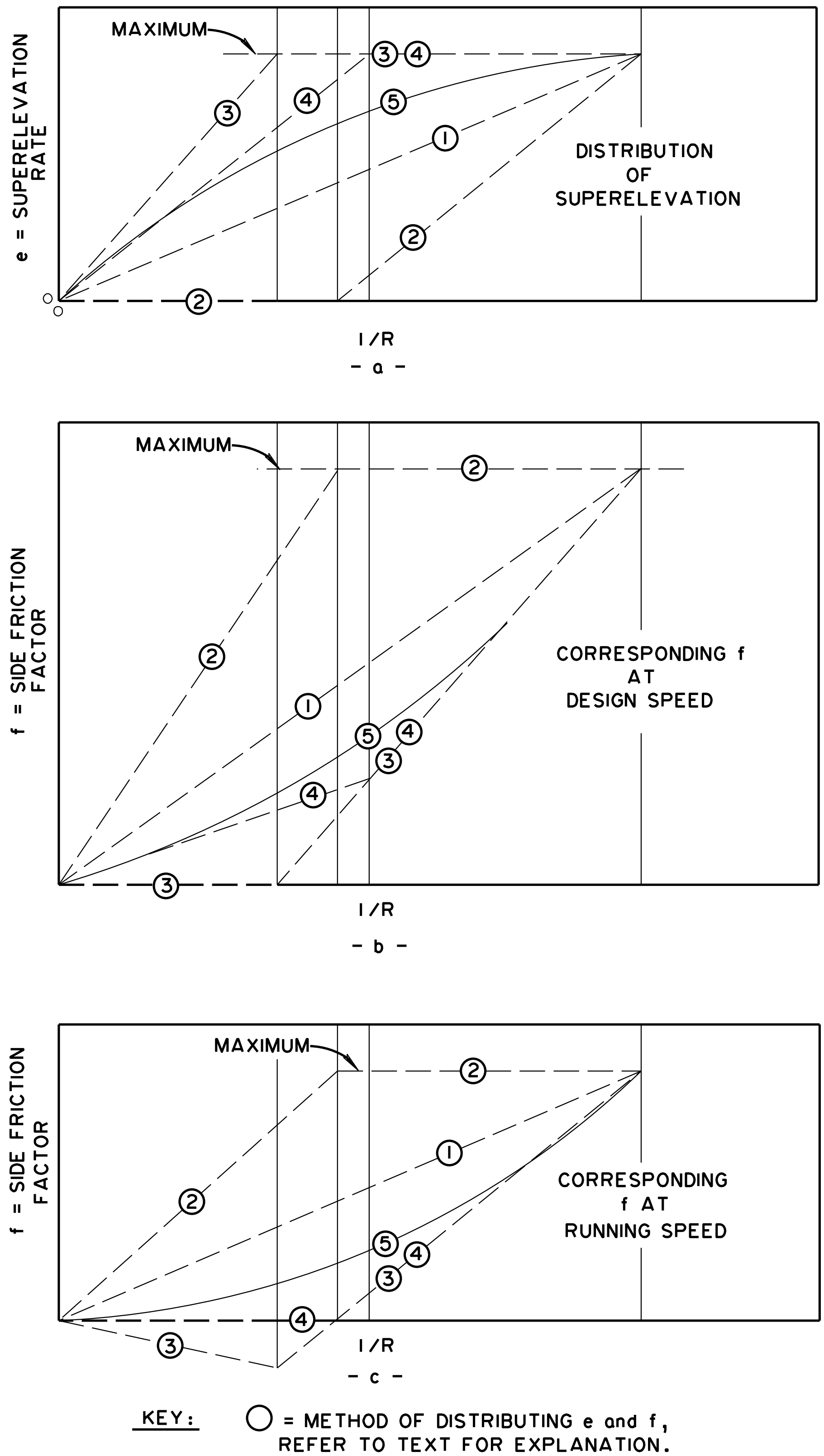
For a given design speed, there are five methods for sustaining lateral acceleration on curves by use of superelevation (e), or side friction (f), or both (2018 AASHTO, "A Policy on Geometric Design of Highways and Streets" [3]). The methods are summarised below, and the relationships are illustrated in Figure B-3-5-4a.

- Method 1 –Superelevation and side friction are directly proportional to the inverse of the radius (i.e., a straight-line relation exists between 1/R = O and 1/R = $1/R_{min}$).
- Method 2 – Side friction is such that a vehicle travelling at the design speed has all lateral acceleration sustained by side friction on curves up to those designed for f_{max}. For sharper curves, f remains equal to f_{max} and superelevation is then used to sustain lateral acceleration until e reaches e_{max}. In this method, first f and then e are increased in inverse proportion to the radius of curvature.
- Method 3 – Superelevation is such that a vehicle travelling at the design speed has all lateral acceleration sustained by superelevation on curves up to those designed for e_{max}. For sharper curves, e remains equal to e_{max} and side friction is then used to sustain lateral acceleration until f reaches f_{max}. In this method, first e and then f are increased in inverse proportion to the radius of curvature.
- Method 4 – Is the same as Method 3, except that it is based on average running speed instead of design speed.
- Method 5 – Superelevation and side friction are in a curvilinear relation with the inverse of the radius of the curve, with values between those of Method 1 and Method 3.

For rural and high-speed urban roadways, the method used for distributing e and f is referred to as "Method 5" as described above. For low speed urban streets, "Method 2" is used to distribute superelevation due to various constraints.

Figure B-3-5-4a Distribution of Superelevation and Side Friction

(Source: Figure 3-5 of AASHTO 2018 [3])



B.3.6 Development of Superelevation

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

For multi-lane divided highways with sufficiently wide depressed medians, superelevation may be developed by rotating the pavement about the crown of each carriageway, without adversely affecting the median sideslope or bottom width. Where adding lanes to the outside of each roadway is expected (future widening from four-lane divided to six-lane divided), superelevation should also be developed by rotation about the crown. Where there is a possibility of future widening by adding lanes to the inside of each roadway, superelevation should be developed by rotation about the inside edge of the future pavement.

On a divided highway with a narrower median, superelevation should be developed by rotation about the inside (median) edge of each carriageway. That way, the elevation of the inside (median) edge of pavement of both carriageways does not change and there is no adverse effect on the geometry of the depressed median.

Figures B-3-6a, B-3-6b, and B-3-6c illustrate the desirable methods of developing superelevation for both simple and spiral curves.

Simple curves are rarely used in new construction. Spiral curves are required on all new roadways except where normal crown is maintained on larger radius curves (i.e., superelevation is not required). See Table B-3-5-3a or Table H-3-3-1a (for local roads). When appropriate, simple curves may be used on existing paved roads to avoid the need to make minor realignments. The use of a simple curve to tie into an existing paved alignment is considered a design exception. Refer to the Alberta Transportation, "Design Exceptions Guideline" [2] for further information.

When spiral transition curves are used, the adverse crown is completely removed by the beginning of the spiral. For two-lane undivided and four-lane divided highways, the transition from normal crown to where the adverse crown is removed is accomplished by means of a 30 m tangent runout. For six-lane or eight-lane divided highways, the tangent runout length is determined using a slope from the lane edge to the centreline, of one metre in 400 m, as suggested in TAC [8].

For example,

$$\text{Tangent Runout} = 400 \times 0.02 \times 3.7 \times 1 = 29.6 \text{ m (rounded to 30 m)}$$

where:

0.02 = normal crown rate (m/m)
3.7 = lane width (m)
1 = number of lanes

From the beginning of the spiral to the beginning of the circular portion of the curve, the slope of the pavement edge is governed by the spiral parameter requirements and pavement width.

To superelevate simple curves, two-thirds of the full superelevation is in place at the beginning of the curve, with the remaining third developed on the curve. The transition from normal crown to where the adverse crown is removed is accomplished by means of a 30 m tangent runout. The Length of Runoff (Lr), over which the superelevation is applied, is indicated in Table B-3-6a.

Table B-3-6a 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

Note: The above Lengths of Runoff (Lr) are required for two-lane undivided and four-lane undivided pavements. For six-lane undivided pavements, use 1.3 times the tabular values.

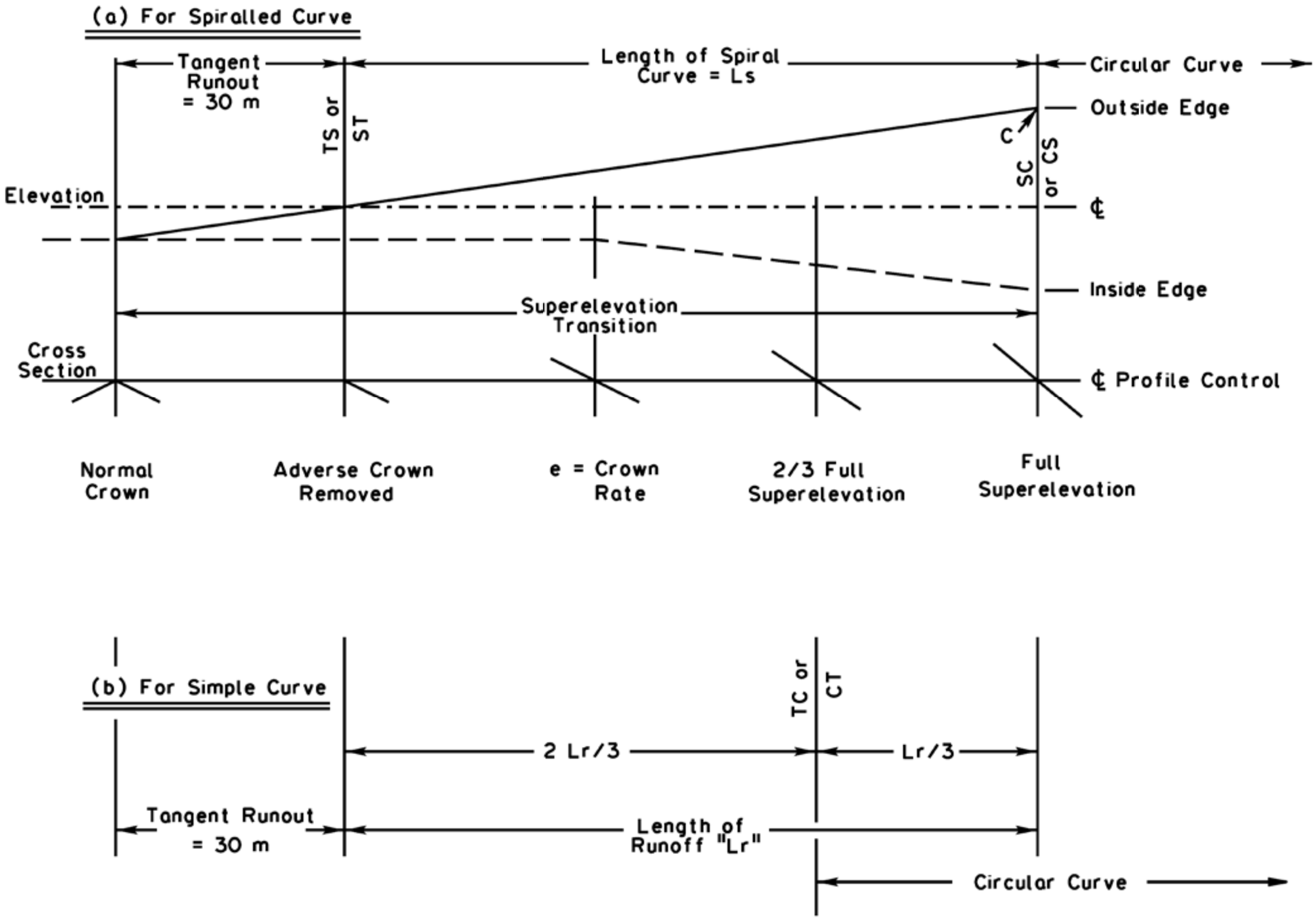
Where a transition occurs near the end of a bridge, the horizontal alignment is usually adjusted, if possible, to keep the superelevation transition off the bridge and maintain the normal crossfall (or constant superelevation) throughout the structure.

Figure B-3-6a Superelevation Transition (Case I)

METHOD OF ATTAINING SUPERELEVATION

REVOLVED ABOUT CENTRE LINE

(- normally used on rural highways)



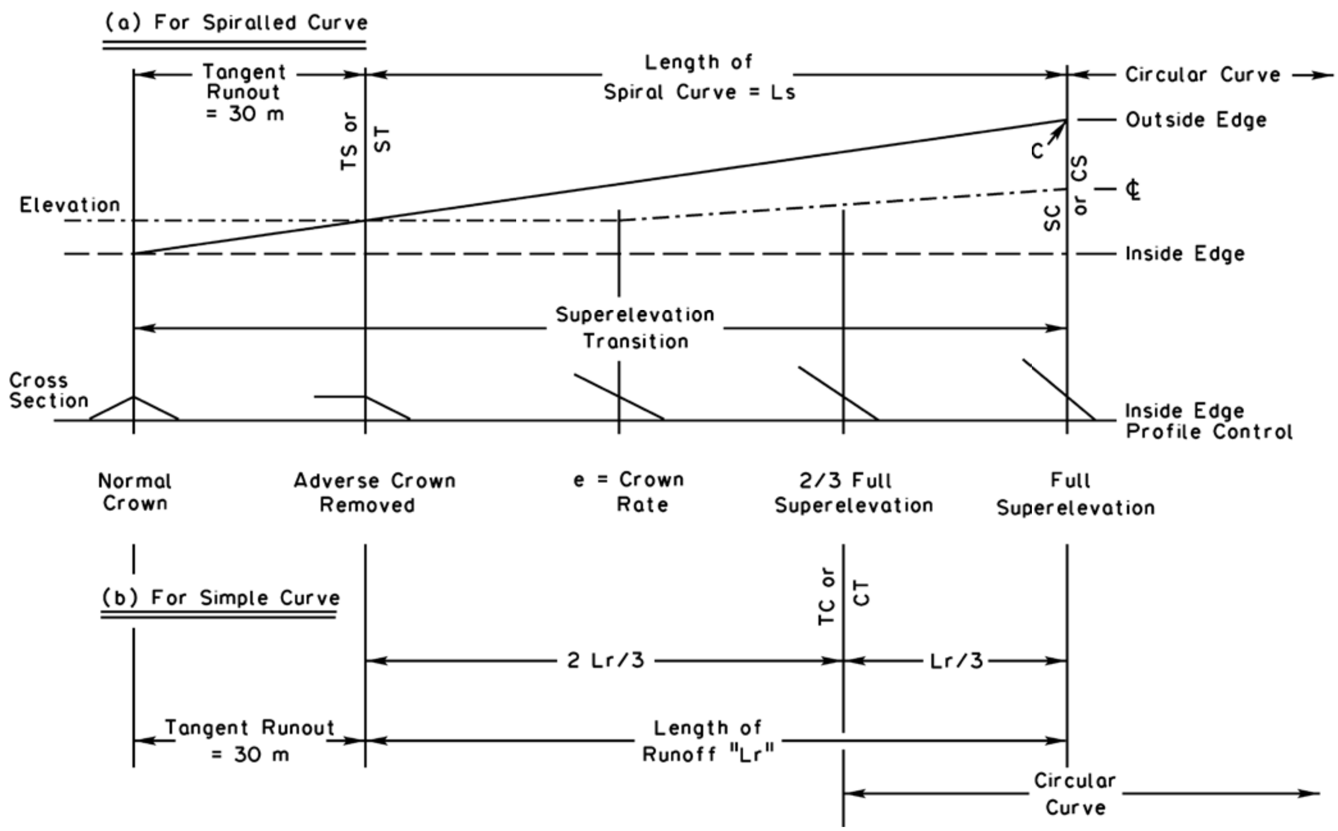
This method of attaining superelevation is to be used on 2-lane undivided highways and divided highways with narrow raised medians. In some cases it may be advantageous to use this method for curves at interchanges or intersections.
For multilane divided highways with depressed medians, superelevation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

A 30 m tangent runout is applicable for 2-lane undivided highways or 4-lane divided highways. This tangent runout length is based on a 3.7 m 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 400 m.

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

Figure B-3-6b Superelevation Transition (Case II)

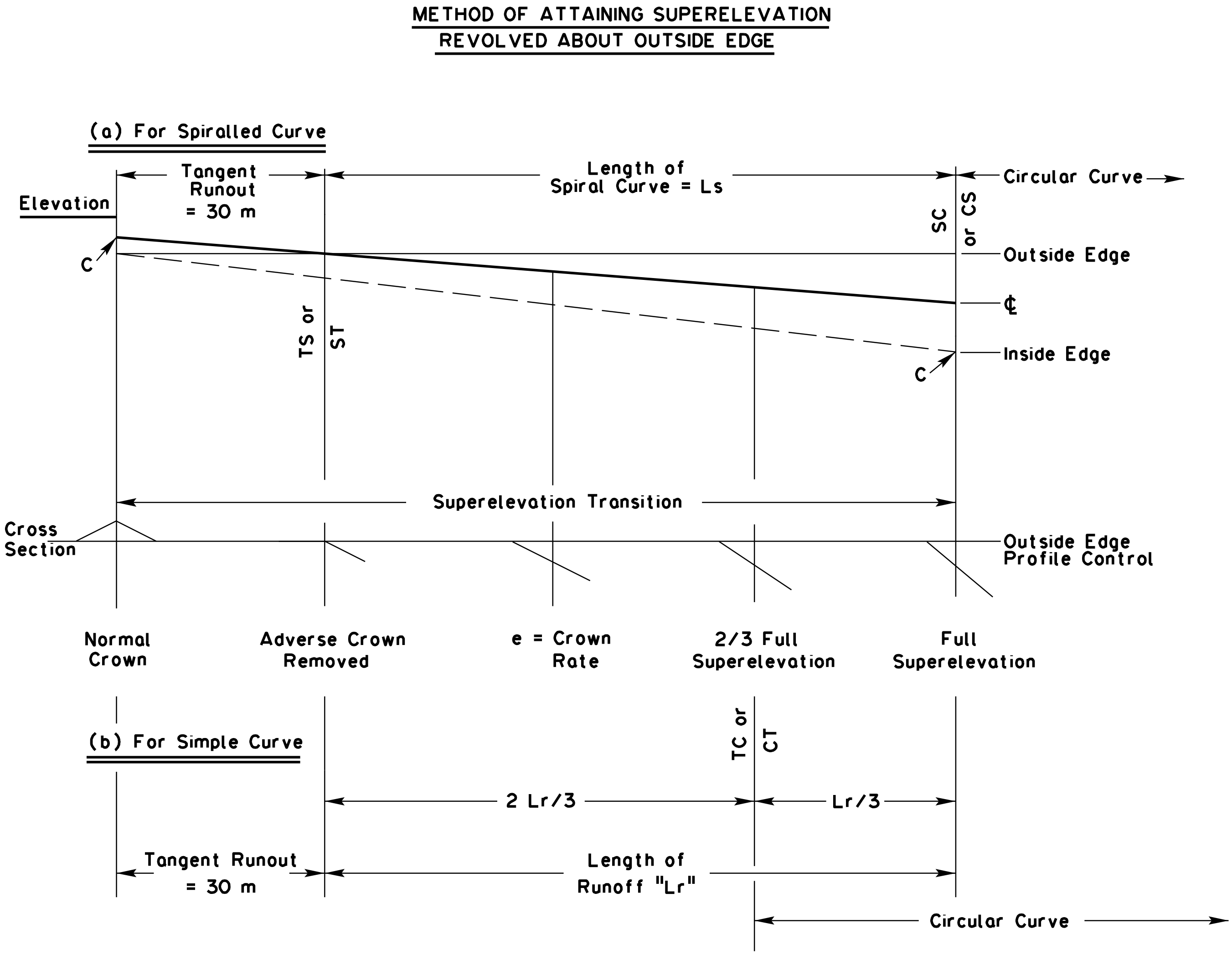
METHOD OF ATTAINING SUPERELEVATION
REVOLVED ABOUT INSIDE EDGE



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.

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

Figure B-3-6c Superelevation Transition (Case III)



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.3.7 Spiral Curves

Spiral curves provide a gradual change in curvature from a straight path (on tangent) to a circular path (with a given radius). The advantages of spiral curves are:

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 portion of the curve that has a constant radius. This minimizes encroachment upon adjoining traffic lanes, promotes speed uniformity and increases safety.
2. The spiral curve provides a desirable arrangement for superelevation development. A change from normal crown to a fully superelevated cross-slope is applied along the length of the spiral.
3. Where the pavement section is to be widened around a curve, the spiral facilitates the transition in width. Spirals provide flexibility so that widening of smaller-radius curves can occur on the outside of the pavement, without a rapid reversal of the alignment of the outer edge of pavement.
4. Alignment changes that occur at the beginning and end of simple curves may be distorted by superelevation development. The roadway appearance is improved by the use of spirals.

Spiral curves are discouraged on bridge structures due to complications related to structural design (variable cross-slope), and operations (drainage, driver behavior due to icy conditions).

B.3.7.1 Form and Properties

Spiral curves are defined by three variables: R (radius), L (length) and A (spiral parameter). The square of the spiral parameter is the rate of change of radius with respect to the length. The radius varies with the reciprocal of length (i.e., the radius decreases as the distance from the beginning of the spiral increases). This is expressed mathematically as follows:

$$\mathrm{L} = \frac{\mathrm{A}^2}{\mathrm{R}} \qquad \Rightarrow \qquad \mathrm{R\,L} = \mathrm{A}^2$$

where:

L = length of the spiral (m)
A = spiral parameter (m) (a constant)
R = radius at the end of the spiral (m)

All spirals are the same shape and vary only in size. The spiral parameter is a measure of the spiral “ flatness” - the larger the spiral parameter, the “flatter” the spiral (i.e., for a given radius, the larger the spiral parameter, the longer the spiral length).

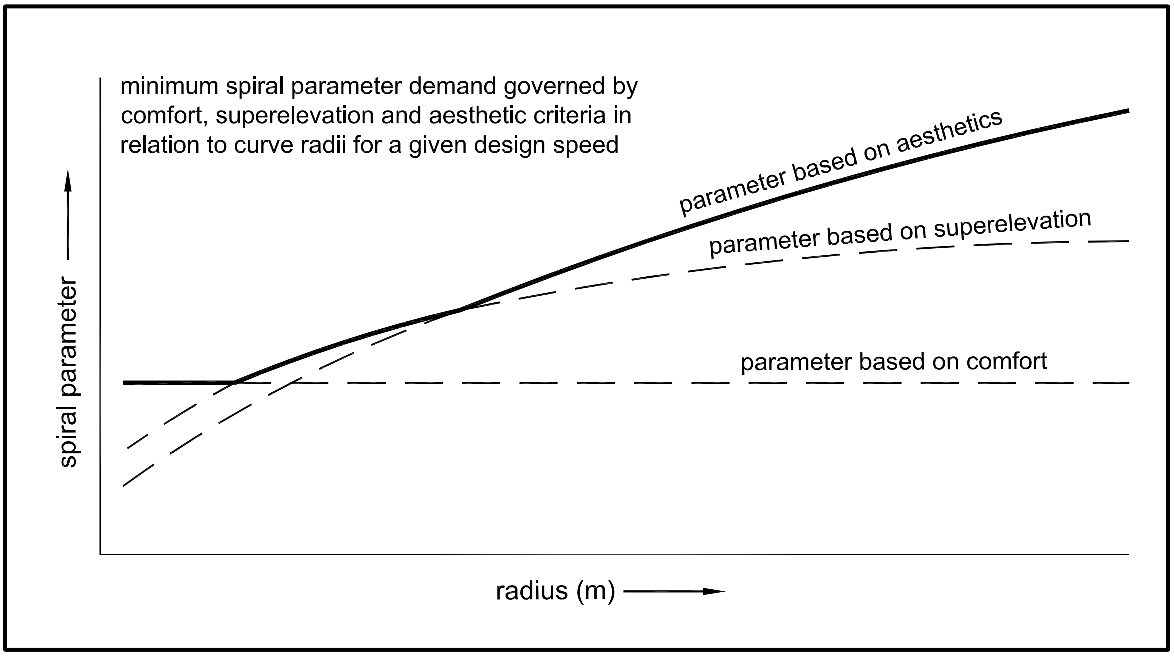
As indicated in Tables B-3-5-3a and B-3-5-3b, as the radius and design speed increases, the minimum and desirable spiral parameters increase.

B.3.7.2 Basis of Design

As illustrated in Figure B-3-7-2a, spiral design is based on three considerations: comfort, superelevation 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-3-7-2a Minimum Spiral Parameter Considerations

(Source: Figure 3.2.6 of TAC 2017 [8])



B.3.7.2.1 Comfort

A vehicle travelling along a spiral from tangent to the end radius at a constant speed, experiences a centripetal force that varies along the transition length. For a given speed and end radius, the rate of change of the centripetal force is a function of the spiral length – the shorter the spiral, the more rapid the rate of change. If the spiral is very short, passengers may 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 for the rate of change of radial acceleration, used to provide the minimum acceptable comfort and safety suitable for passengers, is 0.6 m/s^3. For each design speed, the minimum spiral parameter, based on comfort, is determined using the following equation:

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

where:

A = spiral parameter (m)
V = design speed (km/h)

Note: The minimum spiral parameter, based on comfort considerations, is independent of the radius, as illustrated in Figure B-3-7-2a. The comfort line parallels the X-axis (representing the radius).

B.3.7.2.2 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 developed over the length of the spiral by raising or lowering the edge of pavement relative to some fixed profile control line (generally the centreline). If the slope of the outer edge of pavement, in relation to the profile control line, is permitted to become excessive, it 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 the centreline, for two-lane pavements, at various design speeds, are shown in Table B-3-7-2-2a.

Table B-3-7-2-2a Maximum Relative Slope between Outer Edge of Pavement and Centreline

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-3-7-2-2a, the minimum length of spiral is determined using the following equation:

$$L = \frac{100\,w\,e}{2\,s}$$

where:

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

For a given design speed and radius, the superelevation and relative slope are known, and the minimum length can be calculated. From minimum length and radius, the minimum spiral parameter is determined using the following equation:

$$A^2 = R\,L$$

where:

A = spiral parameter
R = radius (m)
L = length of spiral (m)

B.3.7.2.3 Aesthetics

Short spirals are visually unpleasant. The driving time along the length of the spiral is generally accepted to be at least two seconds. For a given speed, the minimum spiral length is determined using the following equation:

$$\mathrm{L} = \frac{\mathrm{V\,t}}{\left(\frac{3600}{1000}\right)} = \frac{\mathrm{V\,t}}{3.6} = 0.556\,\mathrm{V}$$

where:

L = the minimum spiral length (m)
V = design speed (km/h)
t = minimum travel time along the spiral, 2 s

The minimum spiral parameter is determined using the following equation:

$$\mathrm{A}^2 = \mathrm{R\,L} = 0.556\,\mathrm{R\,V}$$

where:

A = spiral parameter
R = radius (m)
V = design speed (km/h)

B.3.7.3 Design Values for Spiral Parameters

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

Spiral parameter values for design are shown in Tables B-3-5-3a and B-3-5-3b 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. On six-lane pavements, desirable values multiplied by 1.15 are to be used. Spiral parameter values are shown rounded to the nearest whole metre.

B.3.8 Providing Sight Distance on Horizontal Curves

The minimum lateral clearance required on a curve to provide minimum sight distance requirements is determined using the following equation:

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

where:

C = lateral clearance (m)
R = radius (m)
S = sight distance (m), refer to Tables B-2-3a (SSD), B-2-3b (SSD on grade), B-2-4a (PSD), B-2-5a (NPZSD) and B-2-6a (DSD).
π = 3.14159265

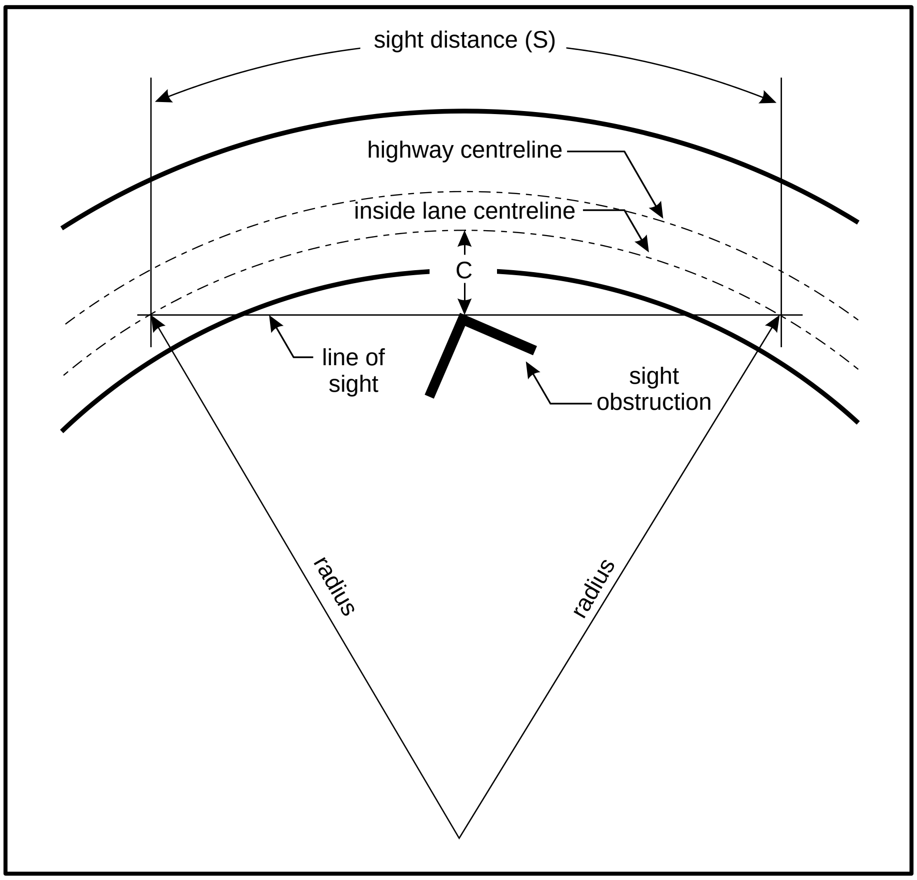
The equation for lateral clearance applies only to the circular portion of curves longer than the sight distance and when both the vehicle and sight obstructions are located within the limits of the simple horizontal curve. Otherwise, the results will be approximate only. The design must be checked by either a graphical procedure or a computational method.

Figure B-3-8a illustrates the application of the equation described above. R is measured to the center of the inside lane. Designers may refer to Figures 3.2.11 and 3.2.12 of TAC [8] for more information.

Horizontal stopping sight distance must be provided along the entire road length. This may only involve the changing of a backslope, the removal of vegetation or setback of a bridge rail or roadside barrier. When the horizontal stopping sight distance is not met, a design exception request may be required. Refer to the Alberta Transportation, “Design Exceptions Guideline” [2] for further details.

Figure B-3-8a Lateral Clearance on Horizontal Curves for Sight Distance

(Source: Figure 3.2.10 of TAC 2017 [8])



B.3.9 Urban Fringe Transition

Horizontal alignment plays an important role in providing visual cues to road users that they are transitioning from a rural highway to an urban highway or vice versa. Applying curvilinear alignments will require drivers to slow down, which is a good segue from driving on a high-speed rural roadway to a slower urban roadway. Other visual cues may be used; however, changes in horizontal alignment force a driver to change, whereas other types of visual cues are there to encourage change in driver behaviour.

B.4 VERTICAL ALIGNMENT

B.4.1 General Controls for Vertical Alignment

The following general controls should be considered in vertical alignment design, 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, is preferred over a gradeline with numerous breaks and short lengths of grade. The manner in which the maximum grade and the critical length of grade are applied and fit to the terrain, determines the suitability and appearance of the finished roadway.
2. The roller coaster or the hidden-dip type of profile, as shown in Figure B-4-1a, should be avoided. Such profiles generally occur on a relatively straight horizontal alignment where the roadway profile closely follows rolling natural ground. Examples of these undesirable profiles may be evident on many older highways. They are unpleasant aesthetically, hazardous and more difficult to drive. Hidden dips contribute to decreased passing opportunities and passing manoeuvre problems for drivers. The passing driver may be deceived by the view of the road beyond the dip that appears to be free of opposing vehicles. Even with shallow dips, the driver does not know if there are oncoming vehicles hidden in the dip. This type of profile is avoided by use of smaller grade changes and longer vertical curves. Particular attention should be paid to drainage on long, shallow curves.
3. Undulating gradelines involving substantial lengths of grade permit heavy trucks to operate at higher overall speeds than is possible when an upgrade is not preceded by a downgrade. These profiles may encourage excessive truck speeds, however, that result in conflicts with other vehicles.

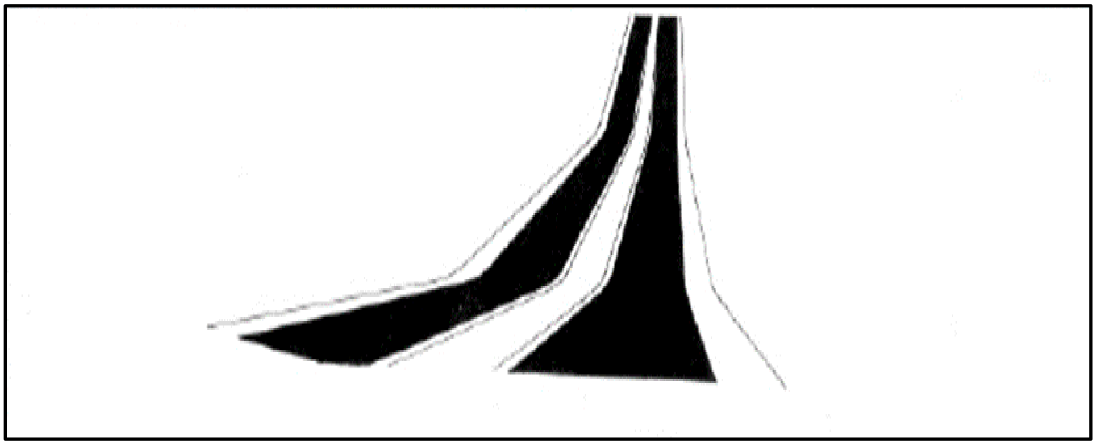
Figure B-4-1a Illustration of Design: Roller-Coaster / Hidden Dip Profile

Image from: “SantaBanta” (https://www.santabanta.com) [12]

The restricted vertical alignment, which attempts to match the minor humps and hollows in the surrounding terrain, is not consistent with the straighter, less restricted horizontal alignment.

4. A broken-back gradeline, as depicted in Figure B-4-1b, consists of two vertical curves in the same direction, separated by a short section of consistent grade. Generally, these gradelines are more noticeable in sags where both vertical curves are in full view. The effect is very noticeable on divided roadways with open median sections.

Figure B-4-1b Illustration of Design: Brocken-back Curve



This figure shows the broken-back arrangement of curves (two curves in the same direction connected with a tangent). Use of longer curves and removal of short sections of tangent grade may improve the aesthetics of a broken-back gradeline.

5. On long grades, an option is to break a steep grade by short intervals of flatter grade, instead of a uniform, sustained grade that might be only slightly below the allowable maximum. This is particularly applicable to low speed highways.
6. Sag vertical curves should be avoided in cuts. If sag curves occur in cuts, adequate drainage must be provided.
7. Generally, to ensure a smooth gradeline, a minimum Vertical Point of Intersection (VPI) spacing of 300 m is used. A minimum length of vertical curve of 120 m is used to ensure that the parabolic shape is achieved using a survey control with 20 m 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 (i.e., the K value will change). On low speed urban roadways (design speed less than or equal to 70 km/h), the minimum vertical curve length is not limited to 120 m and will depend on the design and sight distance requirements.
8. The provision of decision sight distance is desirable at all approaches to intersections or gore points for turning roadways and interchange ramps.
9. In a low seed urban environment, flattening the roadway cross-slope to 1% may be useful to avoid abrupt changes in grade through the intersection.
10. The profile and offset of existing and proposed utilities should be considered in urban areas.
11. Proposed grades must consider existing and future intersections.
12. When a major road and a minor road intersect, the profile and cross-slope of the major road is carried through the intersection and the minor road is adjusted accordingly (refer to Drawing No. CB6-2.3M4 and CB6-2.3M5 on the Alberta Transportation, “CB-6 Highway Standard Plates – Active” website [13]). If two similarly classified roads intersect, the profile and cross-slope of the road with the stop or yield condition is adjusted according to the figures above. At a low speed urban intersection, the profile and cross-slope of each roadway may be adjusted to balance the influence on the curb and gutter.

The following vertical clearances shall be provided on the underside of structures, signals, pedestrian crossing structures, etc.

Table B-4-1a Minimum Design Vertical Clearances below Structures

Obstacle	Minimum Clearance
High Load Corridor	9.0 m - 12.8 m
Typical Corridor	5.4 m
Pedestrian Crossings / Traffic Signal Heads	5.7 m
Train Tracks (trains under roadway)	7.1 m to underside of structure
Train Tracks (trains over roadway)	Same as roadway clearances above

The minimum vertical clearance for bikeways is 2.7 m, although it is desirable to provide 3.6 m in order to provide access for service vehicles. Similar clearances are normally provided for sidewalks since cyclists may use the sidewalk.

Increased clearance may be required where there is an increased level of risk or reduced structural redundancy. Alberta Transportation has a commitment to the trucking industry to maintain the posted vertical clearance of a bridge structure over the lifespan of the structure. This ensures route consistency and minimizes replacement of sign structures. Allowance of future pavement overlays are not considered beneath a bridge structure so the pavement design of the under-passing roadway should reflect this.

B.4.2 Maximum Gradient

Factors considered when selecting the maximum gradient include road classification, traffic operation, terrain, climate conditions, length of grade, benefit cost, property, environmental considerations, adjacent land use (urban), vehicle type and intersections.

Tables A-10-1a and A-10-1b provide the desirable maximum gradient for each design designation. The desirable maximum gradients are also summarized in Table B-4-2a. The desirable maximum gradient ranges from 3% to 13% depending on the design designation.

Table B-4-2a Desirable Maximum Gradient (%)

Functional Class	Design Speed (km/h)							
	60	70	80	90	100	110	120	130
RLU	10 – 13			7 – 9				
RCU				6	6	6		
RAU						5		
RAD							3	
RFD						3		3
ULU	8							
UCU		8						
UAU		8						
UAD		6	6					
UED			6	5				
UFD				5	3	3	3	

Where at-grade intersections occur on rural roadways with moderate to steep grades, it is desirable to reduce the gradient of the main road, through the intersection, to less than 3 percent. Such a profile change is beneficial for all turning vehicles.

The desirable maximum grade through intersections on lower speed roadways (design speed ≤ 70 km/h), in an urban environment, is 4 percent. Higher speed roadways (design speed > 70 km/h) should be limited to a 3% desirable maximum grade through intersections.

The desirable maximum gradients shown in Tables A-10-1a and A-10-1b provide maximum gradients that should not be exceeded whenever practical. The maximum gradient is site specific, however. On alignments where construction costs increase substantially depending on gradients and/or site-specific constraints, 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 Tables A-10-1a and A-10-1b. 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 gradient. The choice of maximum gradient may have a bearing on whether or not a climbing lane or truck runaway lane is required. This should be included in the economic analysis. For economic analysis purposes, please refer to the latest version of the Alberta Transportation, “Benefit Cost Model and User Guide” [14].

An additional consideration that will not be apparent specifically from the economic analysis is level of service. The level of service may be impacted considerably by the choice of a gradient exceeding 3% 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

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 4% to 5% without appreciable loss in speed below that normally maintained on level highways, except for cars with high mass/power ratios, including some compact and subcompact cars.

Generally, a 3% upgrade has only a slight effect on passenger car speed under uncongested conditions, compared with that on level roadways. On steeper grades, the speeds decrease progressively with an increase of the ascending grade. On downgrades, passenger car speeds generally are slightly higher than on level sections.

The effect of grade on truck speeds (and, in some cases, recreational vehicles) is much more pronounced than on passenger car speeds. The average speed of trucks on level sections of highway approximates the average speed of passenger cars. Trucks display up to about a 5% increase in speed on downgrades and about a 7% or more decrease in speed on upgrades, 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. Performance characteristics for various mass/power ratios are shown in Figure B-5-3-3b through Figure B-5-3-3g.

For Alberta Transportation highways, a mass/power ratio of 150 g/W is generally used for the design truck to simulate vehicle performance on grade. This recommendation is based on a 2018 survey and phone conversations with truck dealers. The additional performance charts are provided for special situations. When the design truck is a log haul truck, or when at least 20% of the heavy vehicles are long-combination vehicles (WB-28 or higher) as may be the case on divided highways, 180 g/W should be used. For light trucks or recreation vehicles, 120 g/W is appropriate.

B.4.3 Minimum Gradient

B.4.3.1 Rural Highways

Level grades (0%) on uncurbed rural highways are acceptable for short distances, if the roadway surface is adequately crowned to drain the surface laterally. A 2% crown is standard for paved roadways. A 3% crown is standard for gravel surfaces.

Although a level gradeline 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 an independent design ditch. This is achieved by adjusting the ditch depth relative to the subgrade shoulder.

For wide ditches (3 m or greater width), the desirable minimum longitudinal grade is 0.2% and absolute minimum is 0.05%. Steeper gradients are required on narrower ditches. The desirable minimum gradient on bridge structures is 1%, with an absolute minimum of 0.5%, due to deck drainage requirements (regardless of rural or urban). Table B-4-3-2a provides the desirable and absolute minimum gradients for narrow and wide roadside ditches.

B.4.3.2 Urban Curbed Roadways

In urban applications, the minimum grade should be 0.5%. An absolute minimum of 0.35% may be considered in a retrofit situation where it means the retention, rather than the removal, of existing pavement. A minimum grade of 0.6% should be maintained along curb returns and a minimum of 1.0% towards the curb line within the limits of an intersection (i.e., the resultant slope of the crossfall and longitudinal gradient should be a minimum of 1%).

Grades on flat crest and sag curves should be checked to ensure the slope is a minimum of 0.35% at a distance of 15 m from the crest or sag. On a crest curve with K ≤ 43, there is generally no problem if the 0.35% point is 15 m from the crest. If using a crest curve with K > 43, additional measures such as increasing the number of catch basins may be required. In sag curves, independent grading of the gutter at 0.35% to the low point is allowable.

Table B-4-3-2a indicates the minimum longitudinal gradients for ditches and gutters on all classes of roads.

Table B-4-3-2a Minimum Ditch or Gutter Gradient

Roadway Type	Gradient (%)	
	Desirable Minimum	Absolute Minimum
Rural	Longitudinal Gradient for Ditch	
Rural Wide Ditches (greater than 3 m)	0.2	0.05
Rural Narrow Ditches (less than 3 m)	0.5	0.2
Urban	Longitudinal Gradient for Roadway Surface	
With Curb and Gutter	0.5	0.35

B.4.3.3 Minimum Transition Grades and Drainage Considerations

Section 3.3.8.9 of the 2018 AASHTO [3] identifies two potential pavement surface drainage problems that are of concern in the superelevation transition section.

One problem relates to the potential lack of adequate longitudinal grade. This problem generally occurs when the superelevation is developing and the longitudinal grade of the pavement edge becomes negligible, because the longitudinal grade of the roadway is not steep enough. If the edge of pavement has little longitudinal grade, it can lead to poor pavement surface drainage, especially on curbed cross sections.

The other potential drainage problem relates to inadequate lateral drainage due to negligible cross-slope during development of superelevation. This problem occurs in the transition section, where the cross-slope of the outside lane varies from an adverse slope (on normal crown), to a flat slope (adverse crown removed), to a superelevated slope (at the normal cross-slope rate). The length of the transition section includes the tangent runout section and a portion of the spiral (for a spiral curve) or a portion of the length of runoff (for a simple curve). Within this length, the pavement cross-slope may not be sufficient to adequately drain the pavement laterally. Refer to Figure B-3-6a through Figure B-6-3c for examples of superelevation transition.

Two criteria can be used to alleviate the potential drainage problems:

1. maintain a minimum profile grade of 0.5% through the superelevation transition section; and
2. maintain a minimum edge-of-pavement grade of 0.2% (0.5% for curbed roadways) through the transition section.

The second grade criterion is equivalent to the following equations relating profile grade and effective maximum relative gradient:

Uncurbed	Curbed
$G \leq -\alpha - 0.2$	$G \leq -\alpha - 0.5$
$G \geq -\alpha + 0.2$	$G \geq -\alpha + 0.5$
$G \leq \alpha - 0.2$	$G \leq \alpha - 0.5$
$G \geq \alpha + 0.2$	$G \geq \alpha + 0.5$

with:

$$\alpha = \frac{(w\,n)\,e}{Lr}$$

where:

G = profile grade (%)
α = effective maximum relative gradient (%)
w = width of one traffic lane (m)
n = number of lanes that are rotated
e = design superelevation rate (%)
Lr = length of runoff (m)

B.4.4 Vertical Curves

B.4.4.1 K Value

The K value is a coefficient for defining the rate of gradient change. It is the length of vertical curve required for each percent algebraic difference in grade. For example, a K value of 90 means a horizontal distance of 90 m is required for every 1% gradient change.

For metric curve calculation, the length of a vertical curve is determined from the K value, using the following equation:

$$L = K\,A$$

where:

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

Curves of different K values adjacent to each other, with no tangent in between, are acceptable as long as sight distance requirements are met.

B.4.4.2 Crest Vertical Curves

Minimum K values for crest vertical curves that satisfy minimum stopping sight distance (SSD), passing sight distance (PSD) and no passing zone sight distance (NPZSD) requirements are given in Table B-4-4-2a. These minimum curvatures are expressed in terms of the K value for each design speed.

Minimum crest vertical curves that satisfy minimum SSD requirements were derived based on an eye height of 1.08 m, an object height of 0.6 m and using the minimum SSDs on level roadways (see Table B-2-3a). These values are the minimum K values for each design speed. In practice, a higher value should be used whenever possible.

Where practical on two-lane undivided highways, crest vertical curves that provide minimum PSD should be used. This will result in improved traffic flow, increased capacity and probably some reduction in the number of collisions. Minimum crest vertical curves that satisfy minimum PSD requirements were derived based on an eye height of 1.08 m, height of opposing vehicle of 1.30 m and using the minimum PSDs shown in Table B-2-4a.

Passing sight distance is desirable on crest vertical curves but, if the K value for the minimum PSD cannot be attained, effort should be made to provide at least the K value for the minimum NPZSD. Sections B.2.4 and B.2.5 include further information about PSD and NPZSD. Minimum crest vertical curves that satisfy minimum NPZSD requirements were derived based on an eye height of 1.15 m, height of opposing vehicle of 1.15 m and using the minimum NPZSDs shown in Table B-2-5a.

Table B-4-4-2a Minimum Vertical Curve Criteria for K Value

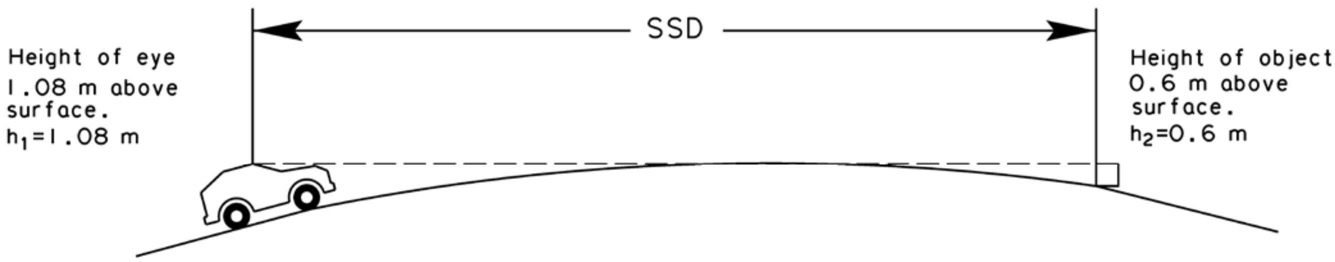
Design Speed (km/h)	Minimum K Values of Vertical Curves				
	Vertical Crest Curves			Vertical Sag Curves	
	SSD	PSD	NPZSD	Headlight Control	Comfort Control *
40	4	80		9	5
50	7	130		13	7
60	11	190		18	10
70	17	250	65	23	13
80	26	335	85	30	17
90	39	405	120	38	21
100	52	490	250	45	26
110	74	580	250	55	32
120	95	675	250	63	37
130	124	780		73	44
Eye Height (m)	1.08	1.08	1.15	1.08	n/a
Object Height (m)	0.60	1.30	1.15	0	n/a

Note:
* Comfort Control for vertical sag curves is to be used on illuminated roads only.
Designers should use vertical curve K values higher than the minimums whenever practical.

Figures B-4-4-2a, B-4-4-2b and B-4-4-2c show the models and equations used to determine the crest vertical curvature K values.

Figure B-4-4-2a Stopping Sight Distance on Crest Vertical Curves

(i) For use in design of two-lane undivided 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

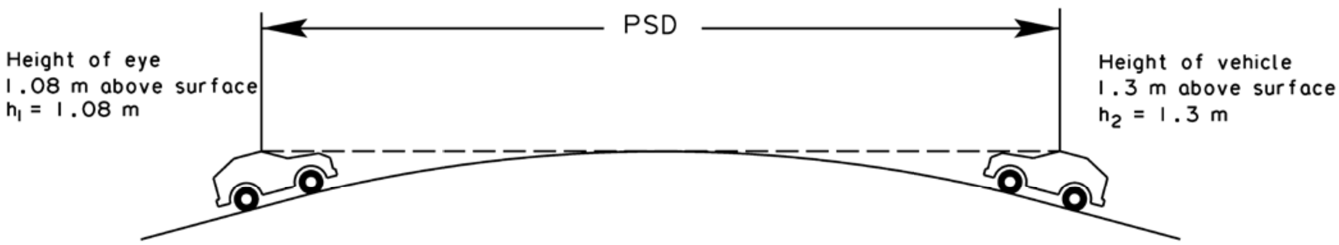
$$K = \frac{SSD^2}{200\left(\sqrt{h_1}+\sqrt{h_2}\right)^2} = \frac{SSD^2}{657.99}$$

When SSD > L

$$K = \frac{2\,SSD}{A} - \frac{200\left(\sqrt{h_1}+\sqrt{h_2}\right)^2}{A^2} = \frac{2\,SSD}{A} - \frac{657.99}{A^2}$$

Figure B-4-4-2b Passing Sight Distance on Crest Vertical Curves

Desirable for design of two-lane undivided 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

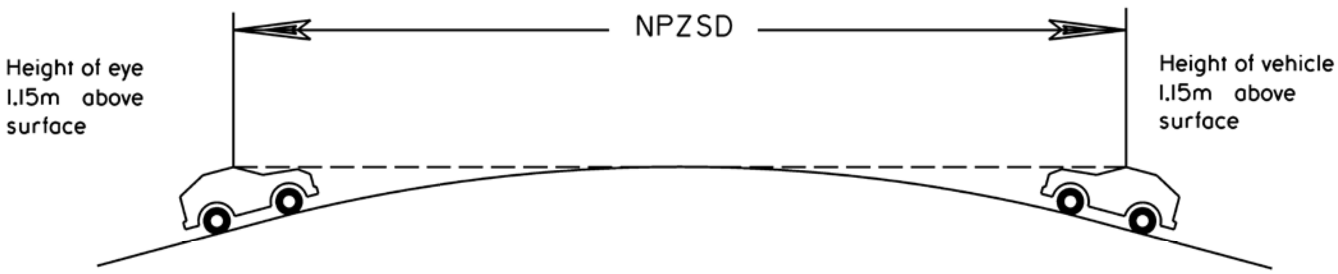
$$K = \frac{PSD^2}{200\,(\sqrt{h_1}+\sqrt{h_2})^2} = \frac{PSD^2}{949.96}$$

When PSD > L

$$K = \frac{2\,PSD}{A} - \frac{200\,(\sqrt{h_1}+\sqrt{h_2})^2}{A^2} = \frac{2\,PSD}{A} - \frac{949.96}{A^2}$$

Figure B-4-4-2c No Passing Zone Sight Distance on Crest Vertical Curves

For use in design of two-lane undivided highways carrying two-way traffic



L = Minimum length of vertical curve in metres

A = Algebraic difference in grades, percent

NPZSD = No passing zone sight distance in metres

When NPZSD < L

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

When NPZSD > L

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

B.4.4.3 Sag Vertical Curves

The minimum K values for sag vertical curves are shown in Table B-4-4-2a and are based on providing stopping sight distance within the headlight beam. This is described as headlight control and is required for roadways that are not illuminated. Headlight control is based on the following assumptions: the headlight beams slope upward at an angle of one degree from the plane of the vehicle, the headlight height is 0.6 m above the driving surface, and the object (to be stopped for) comes into view when the vehicle is the minimum stopping sight distance away.

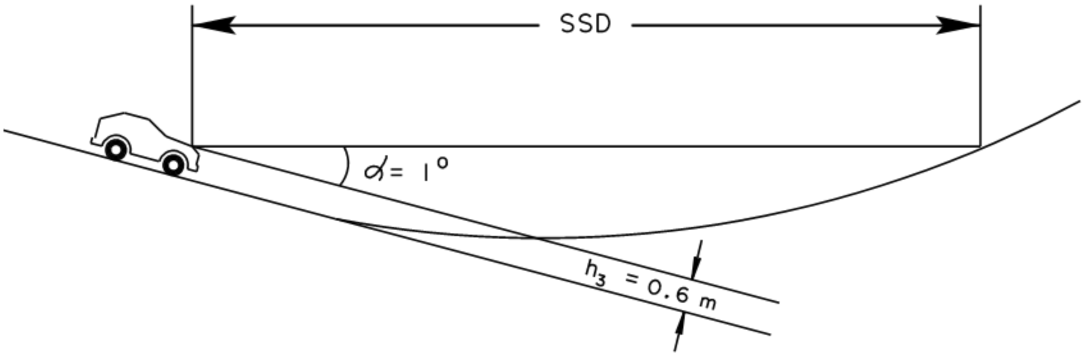
On illuminated sag curves, it is acceptable to use comfort control rather than headlight control to select K values for sag vertical curves. These values are normally exceeded where feasible, in consideration of possible power failures and other malfunctions of the street lighting systems. The comfort K value is based on the radial acceleration experienced by occupants of vehicles travelling at the design speed at the bottom of the sag vertical curve. The maximum acceptable radial acceleration adopted by Alberta Transportation is 0.3 m/s^2, as indicated by TAC [8]. The comfort K values shown in Table B-4-4-2a and Tables A-10-1a and A-10-1b are based on this allowable radial acceleration.

Placement of the lowest point of a sag vertical curve on a bridge deck should be avoided in order to minimize complications with deck drainage and bridge design and construction.

Figure B-4-4-3a shows the model and equations used to determine the sag vertical curvature K values.

Figure B-4-4-3a Stopping Sight Distance on Sag Vertical Curves

For use in design of all highways and interchanges.



h_3 = 0.6 m, 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"

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

Equations for minimum K with HEADLIGHT control:

- When SSD < L

$$K = \frac{SSD^2}{200\,[\,h_3 + SSD\,(tan\ 1°)\,]}$$

- When SSD > L

$$K = \frac{2\,SSD}{A} - \frac{200\,[\,h_3 + SSD\,(tan\ 1°)\,]}{A^2}$$

Equation for minimum K with COMFORT control:

$$\text{Radial acceleration} = \frac{v^2}{R}$$

where:

v = velocity (m/s)
R = rate of change of curvature, = 100 K

The radial acceleration is not to exceed 0.3 m/s², therefore:

$$0.3 \geq \frac{v^2}{100\ K} \quad \Rightarrow \quad K \geq \frac{V^2}{100 * 0.3}\left(\frac{1000}{3600}\right)^2 \quad \Rightarrow \quad K \geq \frac{V^2}{388.8}$$

where:

V = design speed (km/h)

B.4.4.4 Sight Distance at Underpasses

Refer to Section 3.3.4 of TAC [8] for information regarding the sight distance at underpasses, where the overhead structure may restrict the line of sight. Typically, Alberta Transportation uses an eye height of 2.3 m for trucks. For this particular application, designers should use an eye height of 2.4 m for trucks and an object height of 0.6 m, as indicated in the TAC guideline [8].

B.5 CLIMBING AND PASSING LANES

B.5.1 Introduction

Auxiliary lanes are additional lanes that are provided at select 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 are covered in this chapter.

B.5.2 Geometric Features of Climbing and Passing Lanes

The following geometric criteria are to be met when providing climbing or passing lanes.

B.5.2.1 Lane Width

The width of the auxiliary lane is the same as the through lane. This lane width is for the ultimate stage (i.e., after the two future overlays).

B.5.2.2 Shoulder Width

The shoulder adjacent to the auxiliary lane is equal to the lesser of 1.5 m or the standard shoulder width on that design designation of highway. The specified shoulder width is for the ultimate stage. The shoulder is built wider at the initial stage to accommodate the two future overlays.

B.5.2.3 Superelevation

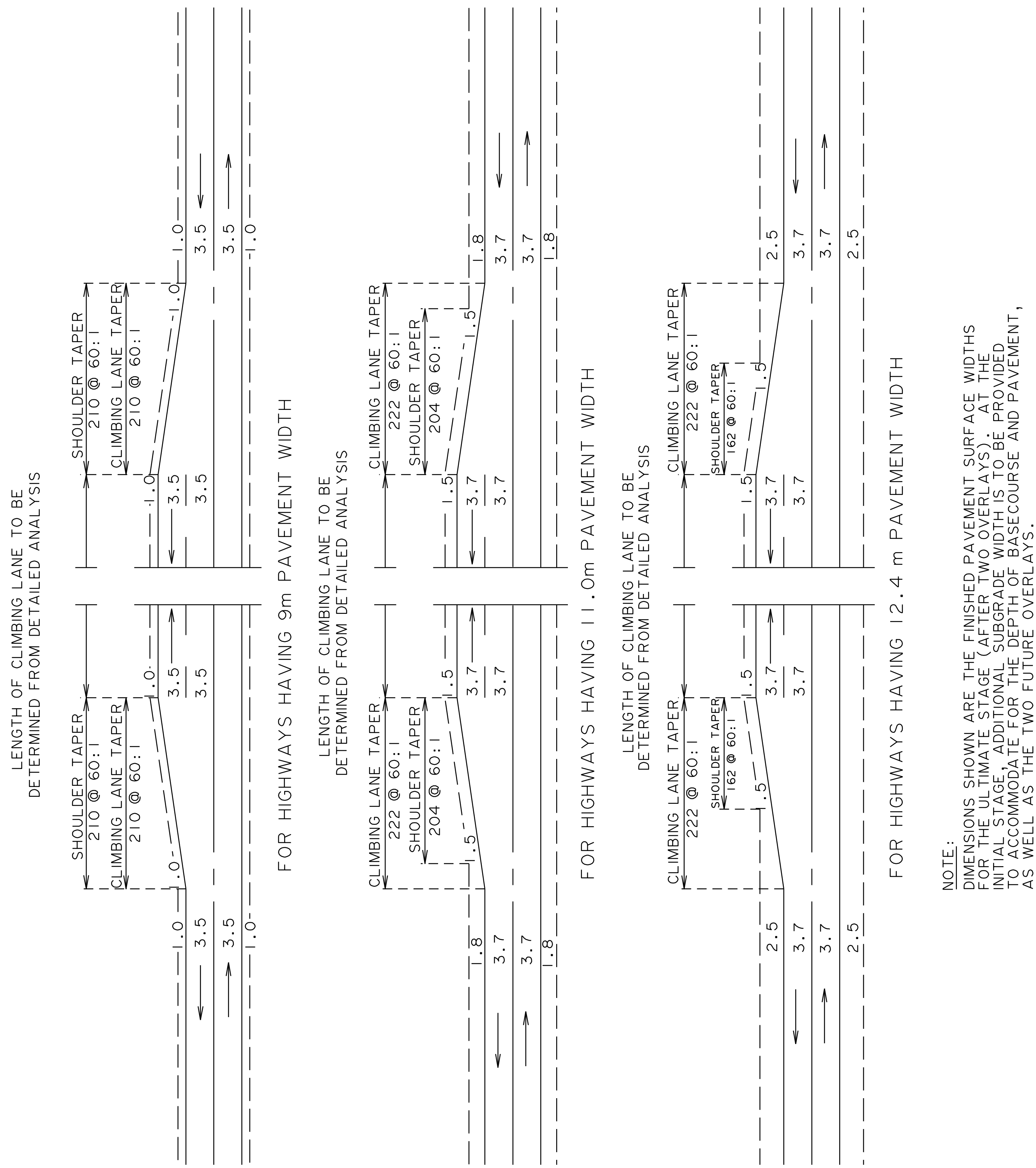
Superelevation on the climbing or passing lane portion of the roadway surface should generally be the same as on the adjacent through lane. On a climbing lane, however, where operating speeds of heavy vehicles are expected to be much lower than the design speed, the designer may use judgment in selecting a lower superelevation rate on the right (outside) lane. High superelevation on climbing lanes can cause a problem for trucks travelling slow during slippery conditions.

For tangent sections of climbing or passing lanes, the cross-slope is usually handled the same as for the addition of a lane to a two-lane undivided highway. The cross-slope of the additional lane matches the cross-slope of the through lanes.

B.5.2.4 Tapers

The taper at the beginning and end of a climbing or passing lane is 60:1. The 60:1 diverging taper promotes use of the right hand lane by all vehicles except those intending to overtake slower vehicles. Figure B-5-2-4a shows typical climbing and passing lanes for various pavement widths.

Figure B-5-2-4a Climbing/Passing Lanes for Various Pavement Widths



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 required intersection treatment. 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 merge speed (the same speed 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, e.g., 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 (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. When the passing demand is high, this generally results in a more efficient climbing lane and a higher level of service for the roadway. It is preferable that the length of climbing lane be less than two or three kilometres to provide greater cost effectiveness. Very long climbing lanes, especially on lower volume roads, tend to be underused.

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 decision sight distance at the end. 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, 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 ends 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, undivided highways because of the restricted passing for the opposing traffic stream. Current pavement marking guidelines for Alberta Transportation suggest that a double solid barrier line (prohibiting passing in the single lane direction) be painted at all passing and 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 if passing sight distance is available. This is illustrated in Figure B-5-2-7a.

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 located 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, an eye height of 1.08 m (corresponding to a passenger vehicle) and an object height of zero metres (corresponding to the roadway surface) are used. The decision sight distances suggested for the termination of an auxiliary lane are shown in Table B-2-6a. Condition C, D, or E should be applied based on context.

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

- A driver, seeing the pavement surface at this point, will know that there is a taper because the driver will already have seen the two arrows on the pavement, the end of the auxiliary lane and the narrowing pavement.

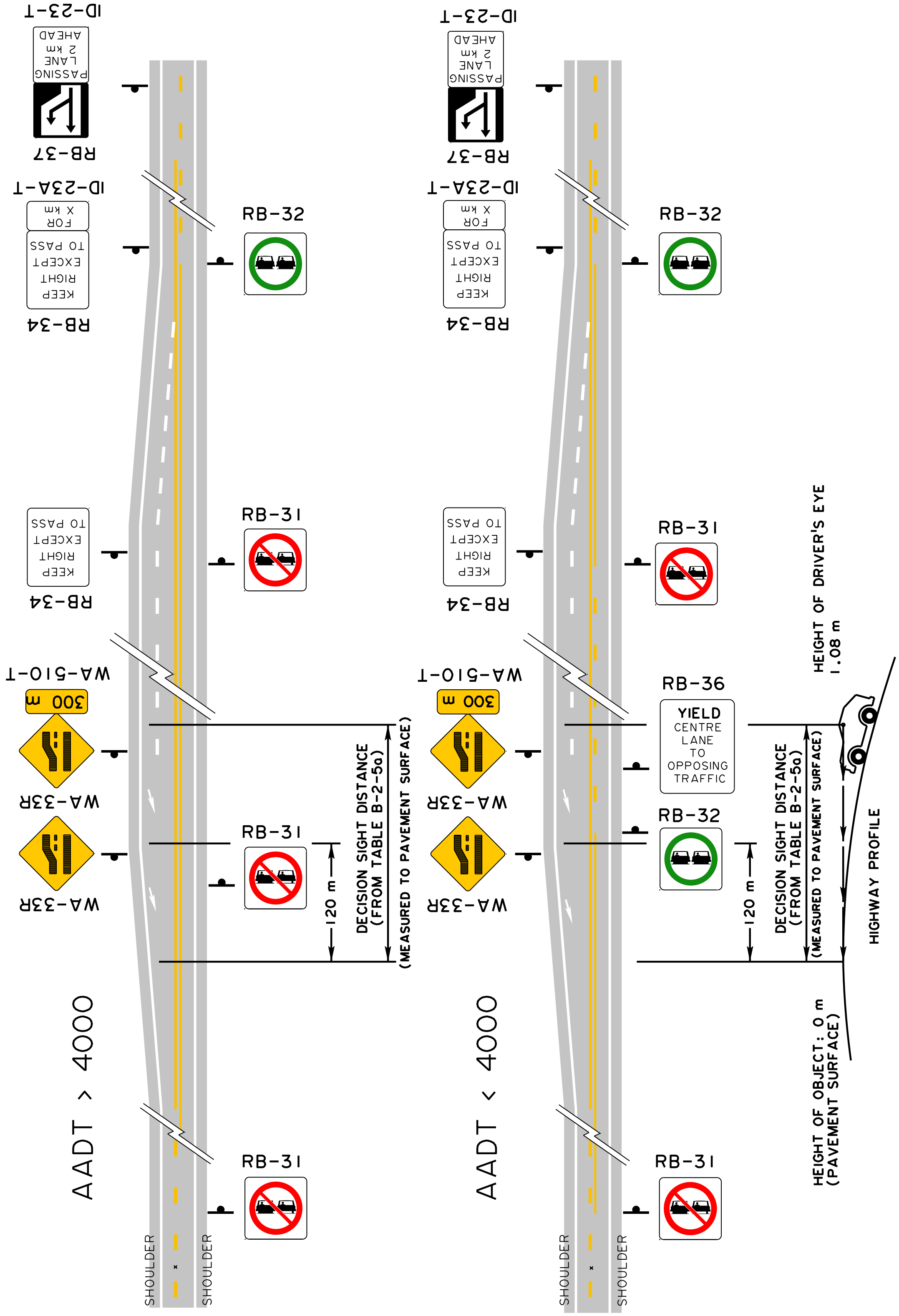
- The required decision sight distance includes four seconds for a manoeuvre (lane change) which could occur in the first half of the taper (a vehicle travelling at 110 km/h will travel approximately 120 m in four seconds).

For example, for a design speed of 110 km/h on a rural roadway, the driver of a passenger vehicle should be able to see the pavement surface over the first 120 m of taper, from a point 210 m before the taper begins. This is based on a required decision sight distance of 330 m for Avoidance Manoeuvre C, as indicated in Table B-2-6a. This should enhance the safety of the merging operation.

Figure B-5-2-7a illustrates the general layout of a climbing/passing lane and the requirement for decision sight distance at the merge taper.

Figure B-5-2-7a Passing & Climbing Lanes Decision Sight Distance at Merge Taper

The signing and pavement markings are for illustration only. Refer to the Alberta Transportation, "Highway Pavement Marking Guide" [7] for the signing and pavement marking requirements.

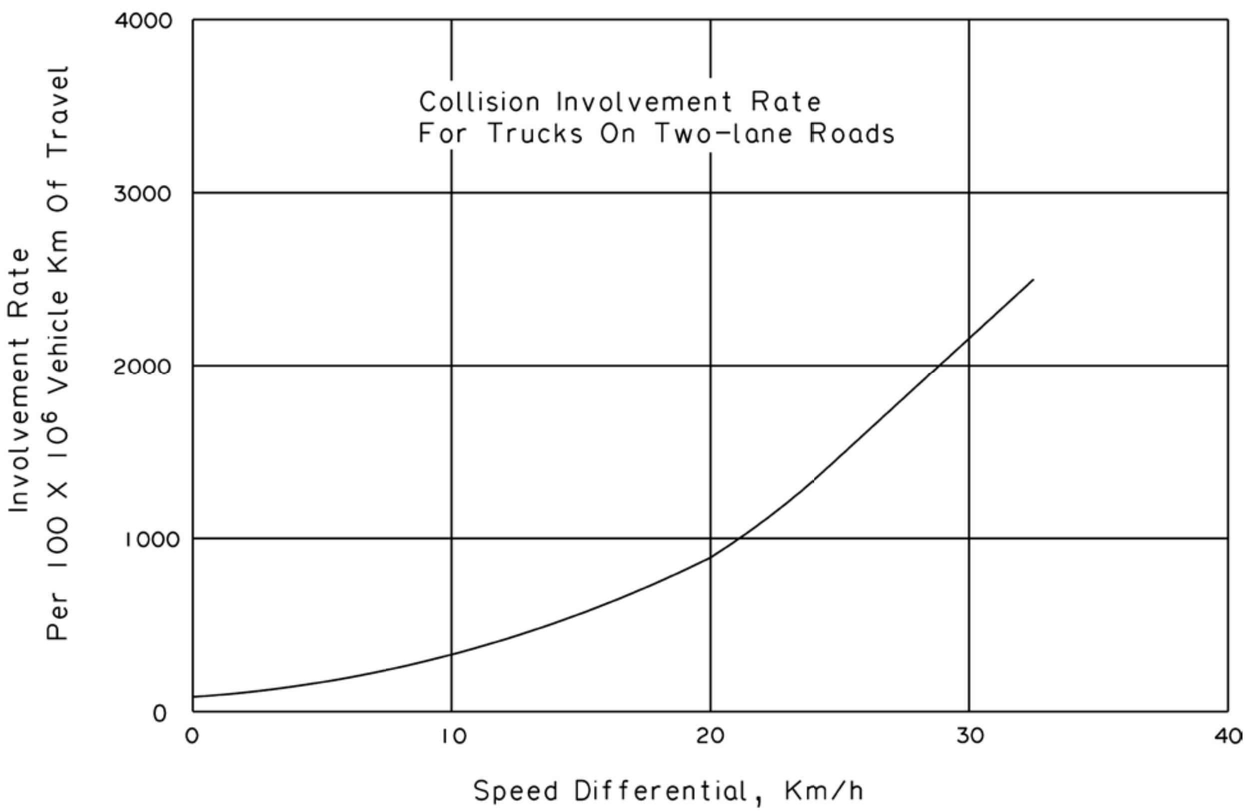


B.5.3 Climbing Lanes

Level of service and safety of operation on two-lane undivided 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 collisions occurring on grades involving heavy vehicles, climbing lanes are commonly included in new construction of busier highways. Additional lanes on existing highways are frequently built as safety improvement projects. The justification for these safety improvements is demonstrated by a plot of collision involvement rate for trucks on two-lane roads versus speed reduction. See Figure B-5-3a.

Figure B-5-3a Collision Involvement Rate for Trucks on Two-lane Roads

(Source: Figure 3-20, AASHTO 2018 [3])



It is desirable to provide a climbing lane as an extra lane on the upgrade side of a two-lane undivided 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 are present, 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.

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

All of the first three conditions should be satisfied to justify a climbing lane on two-lane undivided highways.

1. Upgrade traffic flow rate in excess of 200 veh/h;
2. Upgrade truck flow rate in excess of 20 veh/h; and
3. One of the following conditions exists:
   A. A 15 km/h or greater speed reduction is expected for a typical heavy truck;
   B. Level of Service D or lower exists on the grade; or
   C. A reduction of two or more levels of service is experienced when moving from the approach segment to the grade.
4. An economic analysis may justify the addition of a climbing lane regardless of grade or traffic volumes.

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

A review of the geometry and traffic conditions on existing climbing lanes on Alberta's provincial highway system shows that neither the volume nor the level of service criteria recommended in this warrant are too high.

If the traffic flow rate or LOS, required for the warrant, occurs in the first half of the design life, the condition shall be considered as being met (i.e., it is not necessary to justify a climbing lane based on the initial traffic volume).

Condition 1: Upgrade traffic flow rate in excess of 200 veh/h

The upgrade flow rate is determined by multiplying the predicted or existing design hour volume by the directional distribution factor for the upgrade direction and dividing the result by the peak hour factor. The design hour volume is determined by multiplying the design AADT and the Design Hour Factor (K).

For example, if K = 0.1, the peak hour factor is 0.9, the directional distribution factor is 0.5 and the two-way AADT is 2000, the upgrade traffic flow rate is 2000 x 0.5 x (0.1 / 0.9) = 111 veh/h.

The daily volume to be used is generally the AADT. If the ASDT or AWDT is more than 15% greater than the AADT, the higher number should be used.

Condition 2: Upgrade truck flow rate in excess of 20 veh/h

The number of upgrade trucks is obtained by multiplying the upgrade flow rate by the percentage of trucks in the upgrade direction. If truck percentage is 10%, the upgrade truck flow rate is 111 x 0.1 = 11 veh/h. Trucks are defined as the sum of tractor-trailer combinations (WB), single unit trucks (SU), half of the buses (B), and half of the recreational vehicles (RV) (i.e., trucks = WB + SU + 0.5 B + 0.5 RV). Buses and recreational vehicles generally perform better than trucks on grades.

The reasons for the volume warrants are as follows:

- It is necessary to choose a minimum volume for which climbing lanes will be built. Use of the level of service criteria alone could result in some relatively low volume roads warranting climbing lanes, even though they are not cost effective based on collision reduction or road user savings. A volume of 20 heavy vehicles per hour is in alignment with the 2018 AASHTO [3] recommendation. A 2013 Alberta truck survey indicated that appropriately 30% of trucks are empty. Twenty heavy vehicles per hour indicates 14 loaded heavy vehicles per hour are in the upgrade direction. The presence of one loaded heavy vehicle travelling in the upgrade direction every 4.2 minutes in the design hour (100th highest hour of the design year) does not represent a serious congestion problem, nor would it normally be a serious safety problem.

Condition 3:

<u>3A: A 15 km/h or greater speed reduction is expected for a typical heavy truck</u>

The speed reduction evaluation should be conducted first because, where the critical length of grade is exceeded, no further evaluation under Condition 3 will be needed.

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

- the entry speed of the design vehicle is 95 km/h; and
- the mass/power ratio of the design vehicle is 150 g/W (* refer to exceptions below).

The entry speed of the design vehicle is based on the posted speed for two-lane undivided highways in Alberta. The posted speed is 100 km/h for the majority of two-lane undivided highways in Alberta. Trucks are assumed to travel at a slightly lower speed than the posted speed. The mass/power ratio is based on a survey of the Alberta trucking industry together with phone conversations with several truck dealers; both conducted in 2018.

* Exceptions to the standard design vehicle mass/power ratio should only be made where records of the actual mass/power ratio of the heavy vehicles 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% 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 is used to determine if the speed reduction warrant is met on a particular grade. The truck performance curves are used, together with other considerations, to determine the start and end of the climbing lane.

Table B-5-3-1a Critical Length of Grade (Metres) for a 15 km/h Speed Reduction

Design Vehicle Mass/Power Ratio Metric	(Imperial)	Grade in Percentage						
		2	3	4	5	6	7	8
120 g/W	(197 lb / hp)	765	405	310	230	180	155	130
*** 150 g/W**	(247 lb / hp)	650	345	250	200	155	130	-
180 g/W	(296 lb / hp)	530	280	190	170	125	100	-

Note:

* 150 g/W is normally used for two-lane undivided highways.

- Length of specified grade at which the speed of the designated design vehicle is reduced by 15 km/h, from its assumed entry speed (95 km/h) to 80 km/h.

3B: Level of Service D or lower exists on the grade.

The level of service on the grade must be LOS D or lower in the design hour on the two-lane roadway. If the level of service on the grade in the design hour is LOS C, B or A, a climbing lane is not required. The level of service analysis is based on the methodology for undivided highways shown in the most current version of the Transportation Research Board, “Highway Capacity Manual” [15].

3C: A reduction of two or more levels of service is experienced when moving from the approach segment to the grade.

The remaining item to evaluate if condition 3A or 3B is not satisfied is whether there is a two-level reduction in the LOS between the approach and the upgrade.

Condition 4:

Although traffic and grade is set as a general warrant, inclusion of climbing lanes should be considered if shown to be cost effective, especially when the collision history is high and may be improved through the construction of a climbing lane. For example, 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:

- there is a high percentage of loaded trucks in the upgrade traffic stream;
- the geometry of the highway prior to the grade is very restrictive for passing (which may result in a high demand for passing); or
- there is a high collision history that may be improved by the addition of climbing lanes.

On particular projects, it may be possible to show that the construction of climbing lanes is cost effective (Condition 4) even though not all of the first three conditions have been met. In this case, the construction of climbing lanes would be considered justified.

The economic justification of a climbing lane must be established using the Alberta Transportation, “Benefit Cost Model and User Guide” [14]. Benefits should include predicted improvement in collision experience and reduced road user costs due to reduced delay. When all costs and benefits are considered and discounted by the standard methods, the Internal Rate of Return for the climbing lane must be at least 4% at year 20 to be considered justified.

The following are examples of the use of the climbing lane warrant for two-lane undivided highways.

Example 1: Use of Climbing Lane Warrant for Two-Lane Undivided 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-211-110	
Length of Grade:	1000 m	
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 line)
Design Truck (85th percentile):	150 g/W	
Design / Existing AADT:	2133 / 1422 (upgrade direction) based on 20-year design life and 2.5% annual growth, not compounded.	
Traffic Composition:	WB:	5%
	SU:	1%
	RV:	6%
	BUS:	2%
	PV:	86%
Design Hour Factor (K):	0.10	
Peak Hour Factor:	0.92	

Step 1: Check Upgrade Traffic Flow Rate (Condition 1)

Existing upgrade traffic flow rate = 1422 x (0.10 / 0.92) = 155 veh/h.

The upgrade traffic flow rate that would meet Condition 1 is 200 veh/h. Assuming a 2.5% non-compounding growth rate:

200 = 155 * (1 + 0.025 X)
X = 12 years

Since Condition 1 is not met in the first half of the design life (less than 10 years), the climbing lane is not warranted. No further evaluation is required.

Example 2: Use of Climbing Lane Warrant for Two-Lane Undivided 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-211-110	
Length of Grade:	1000 m	
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 line)
Design Truck (85th percentile):	150 g/W	
Design / Existing AADT:	2700 / 1800 (upgrade direction) based on 20-year design life and 2.5% annual growth, not compounded.	
Traffic Composition:	WB:	5%
	SU:	1%
	RV:	6%
	BUS:	2%
	PV:	86%
Design Hour Factor (K):	0.10	(i.e., Design Hour Volume = Design AADT x 0.10 = 270)
Peak Hour Factor:	0.92	

Step 1: Check Upgrade Traffic Flow Rate (Condition 1)

Existing upgrade traffic flow rate=1800 x (0.10 / 0.92) = 196 veh/h

The upgrade traffic flow rate that would meet Condition 1 is 200 veh/h. Assuming a 2.5% non-compounding growth rate:

200 = 196 * (1 + 0.025 X)
X = 1 year

Condition 1 is met in the first half of the design life (less than 10 years).

Step 2: Check Upgrade Truck Flow Rate (Condition 2)

Truck percentage = WB + SU + ½ (BUS + RV) = 10%
Existing upgrade truck flow rate = 196 x 0.10 = 20 veh/h

Condition 2 is met in the first half of the design life (less than 10 years).

Step 3: Check speed reduction due to grade (Condition 3A)

According to Table B-5-3-1a, a 15 km/h speed reduction would have occurred after 345 m at 3% using a 150 g/W design truck. Therefore, the speed reduction warrant is definitely met on a 1000 m long, 3% grade.

As all of the three conditions are met, a climbing lane is warranted for this example.

B.5.3.2 Climbing Lane Warrant for Multi-lane Highways

The addition of climbing lanes on multi-lane highways need not be considered if the upgrade volume is less than 1,000 veh/h/lane, regardless of grades or percentages of trucks, because of the generally high level of service provided by a multi-lane facility with this traffic volume. If the upgrade volume exceeds 1,000 veh/h/lane 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.

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 start, end and length are determined using the truck performance curves (Figures B-5-3-3b through B-5-3-3g for 180 g/W, 150 g/W, and 120 g/W mass/power ratios). The curves are adapted from those in the 1985 Transportation Research Board, “Highway Capacity Manual” (HCM) [16].

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

The vertical alignment and truck performance curves are shown in Figure B-5-3-3a. The design truck is assumed to have a mass/power ratio of 150 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:

- The assumed entry speed is 95 km/h at PI #1 (Station 1+000), where the 4% upgrade begins;
- Table B-5-3-1a indicates the critical length of grade on a 4% upgrade is 250 metres. Therefore, the truck decelerates to 80 km/h at Station 1+250, which is where the climbing lane begins;
- Enter the truck deceleration curve where the 4% curve intersects 80 km/h. Due to the remaining 550 m upgrade at 4%, the truck further decelerates to 60 km/h at PI #2 (Station 1+800);
- The truck enters the next section of upgrade (6%) which is followed by a downgrade of 2%. The algebraic difference of the grade change exceeds 4% (i.e., +6% - (-2%) = +8%).

  When the algebraic difference exceeds 4%, the vertical curve connecting the grades is approximated through the average grade that connects the quarter points of the vertical curve. These quarter points act as new PI's for estimating the design vehicle speed. In this example, the length of the vertical curve is 800 m. Therefore, the quarter points occur at 200 m on either side of the real PI #3 and the grade connecting the quarter points is estimated at 2%. This approximated grade, 400 m in length, reduces the length of the preceding and following grades by 200 m each.

- The truck decelerates to 31 km/h (in this case, the crawl speed) due to the 600 m upgrade at 6%;
- Now enter the truck acceleration curve where the 2% upgrade curve intersects 31 km/h. The truck accelerates from 31 km/h to 50 km/h on the 400 m, 2% upgrade;
- On the 400 m, 2% downgrade, the truck accelerates from 50 km/h to 75 km/h at PI #4 (Station 3+200);
- The design truck accelerates on a 350 m, 0% grade from 75 km/h to 80 km/h (the merge speed) at Station 3+550.

The climbing lane begins when the speed of the design truck slows to 80 km/h. As indicated in Figure B-5-3-3a, this occurs at 1+250. The 60:1 diverge taper is completed at this location. The end of the climbing lane can occur anywhere after the merge speed has been achieved (after 3+550), provided that the required decision sight distance is available. The merge speed is the same as the speed at the beginning of the climbing lane (80 km/h). The 60:1 merge taper begins after the end of the climbing lane.

Figure B-5-3-3a Climbing Lane Design Example

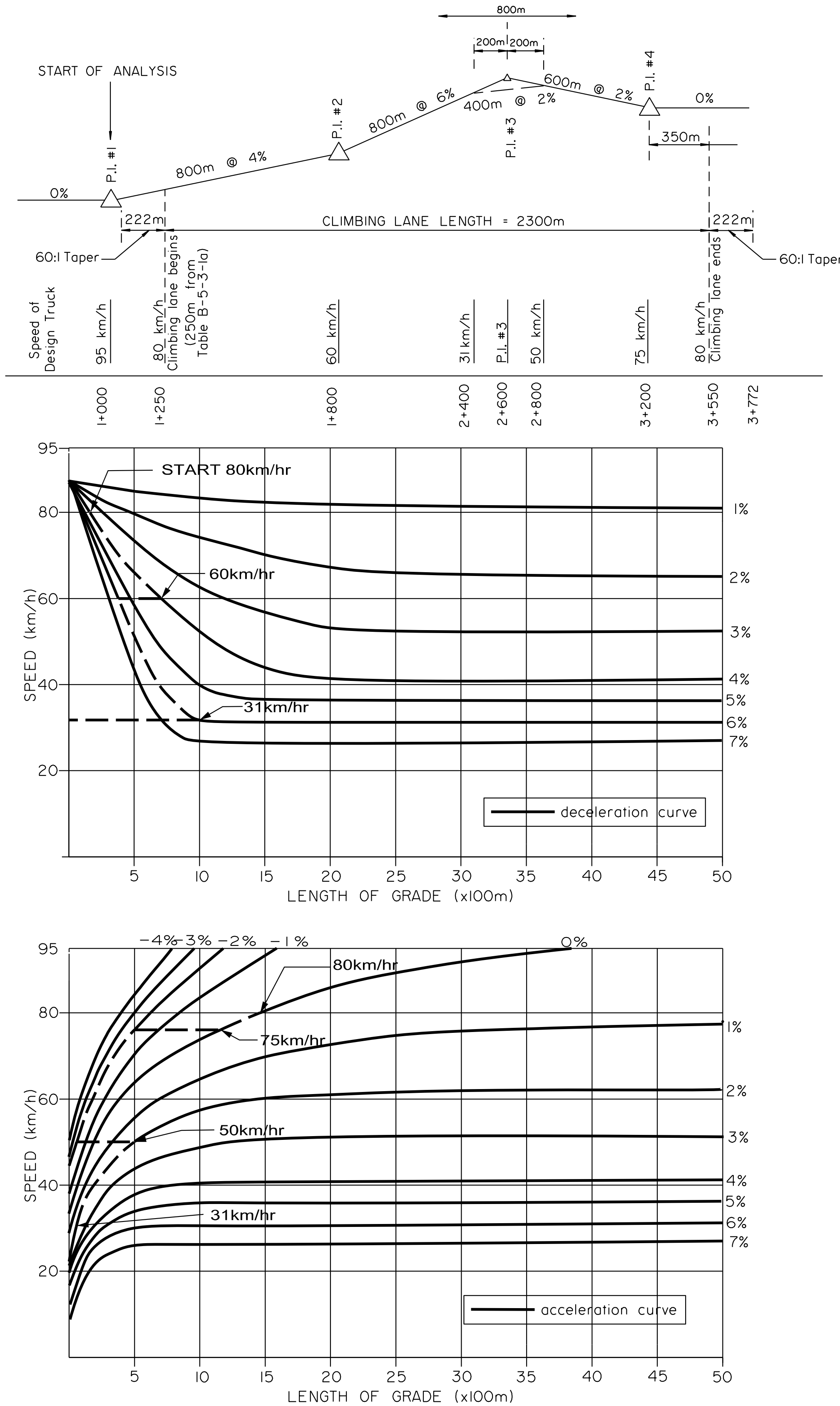
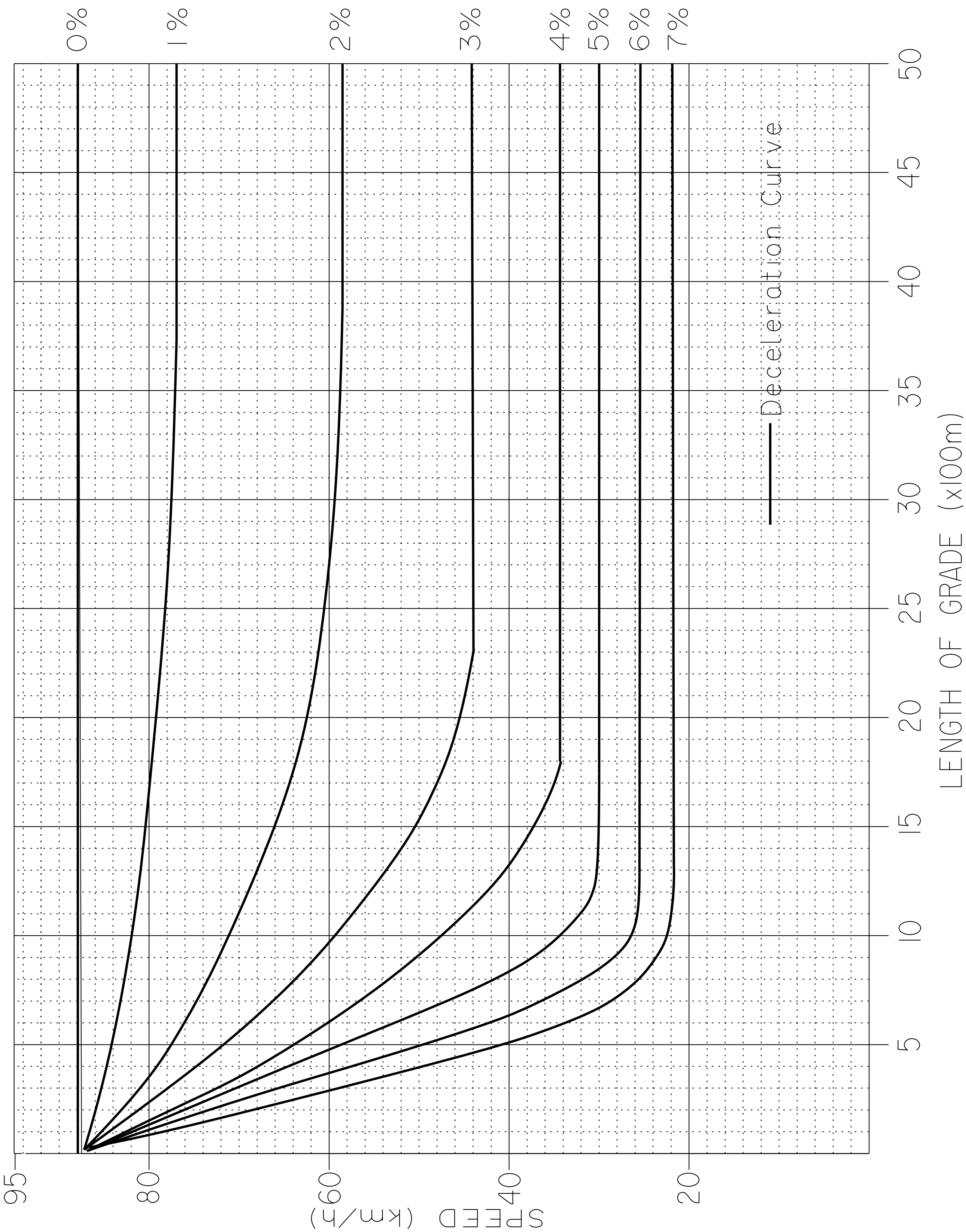


Figure B-5-3-3b Performance Curves for Heavy Trucks – 180 g/W Deceleration

(Source: Adapted from Figure I.3-4, “Highway Capacity Manual” [16])



Classification: Public

Figure B-5-3-3c Performance Curves for Heavy Trucks – 180 g/W Acceleration

(Source: Adapted from Figure I.3-4, “Highway Capacity Manual” [16])

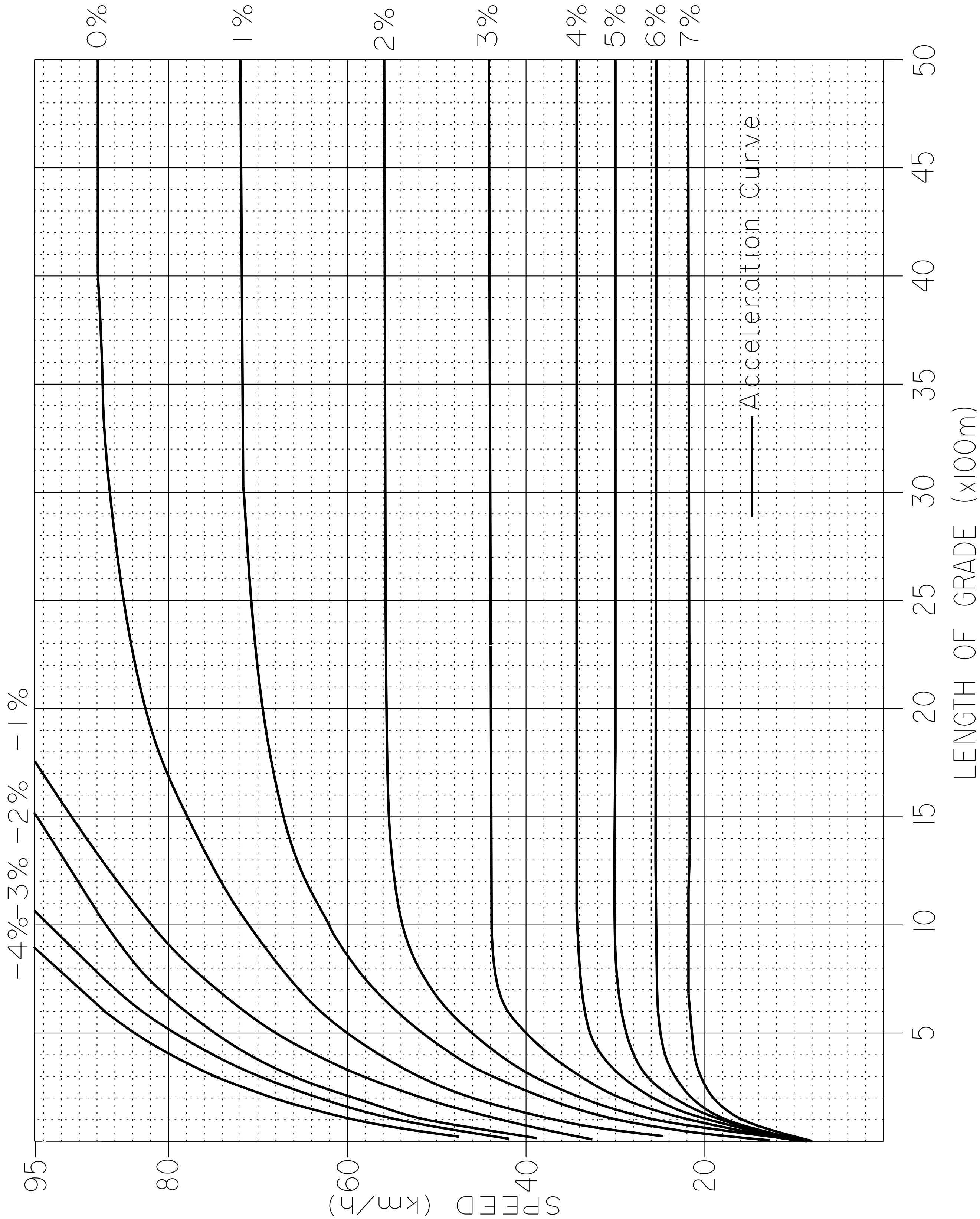


Figure B-5-3-3d Performance Curves for Heavy Trucks – 150 g/W Deceleration

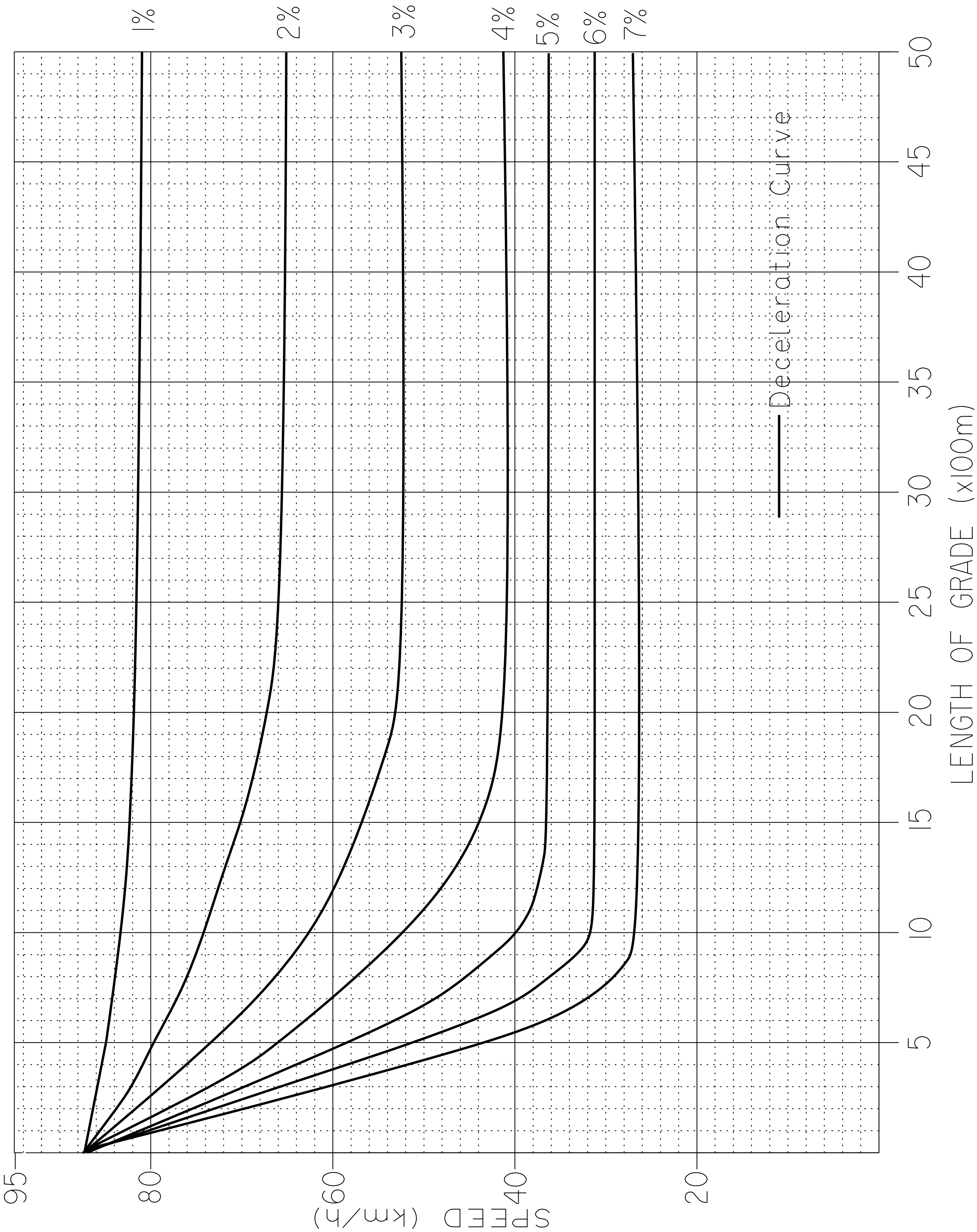


Figure B-5-3-3e Performance Curves for Heavy Trucks – 150 g/W Acceleration

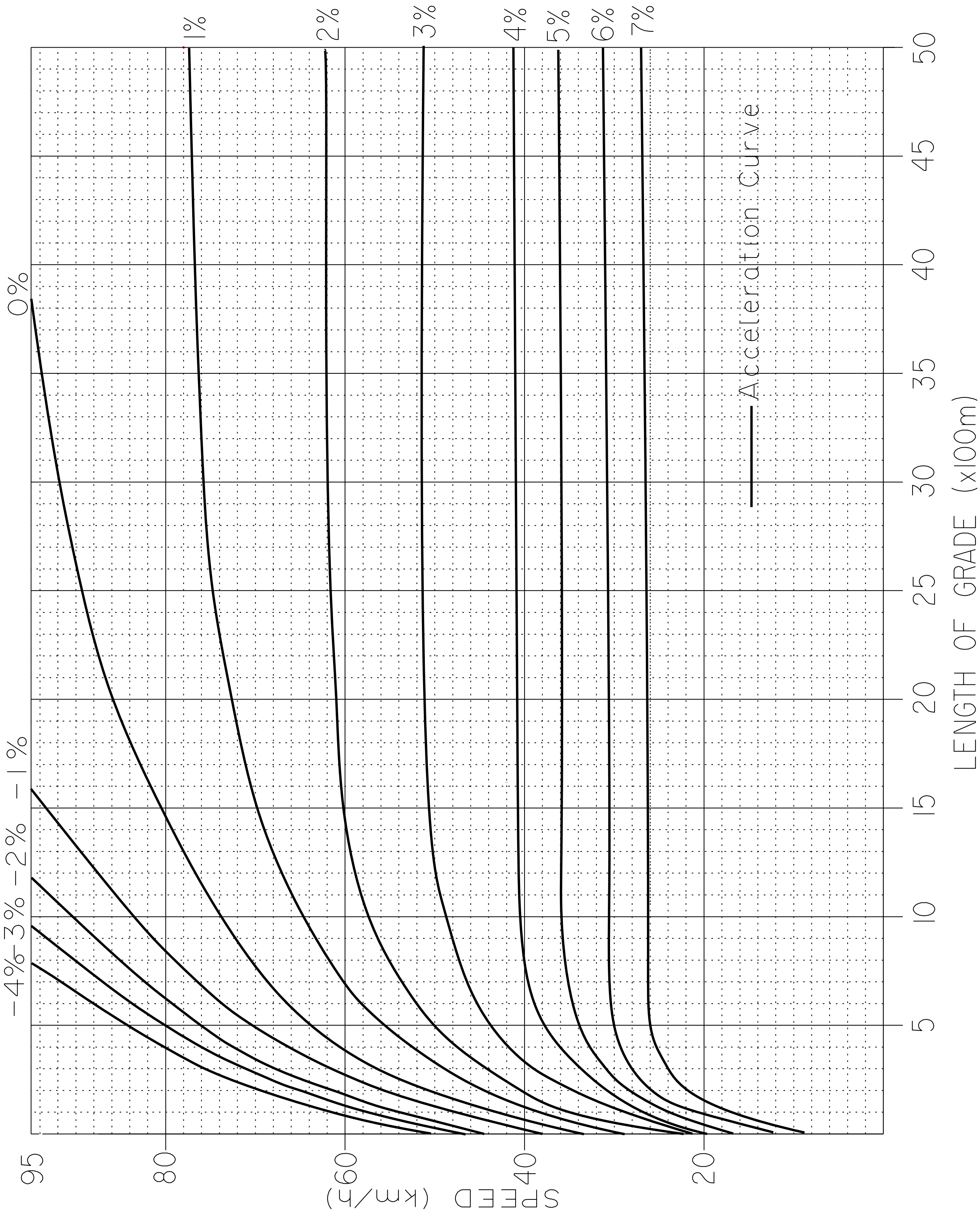


Figure B-5-3-3f Performance Curves for Heavy Trucks – 120 g/W Deceleration

(Source: Adapted from Figure I.3-2, “Highway Capacity Manual” [16])

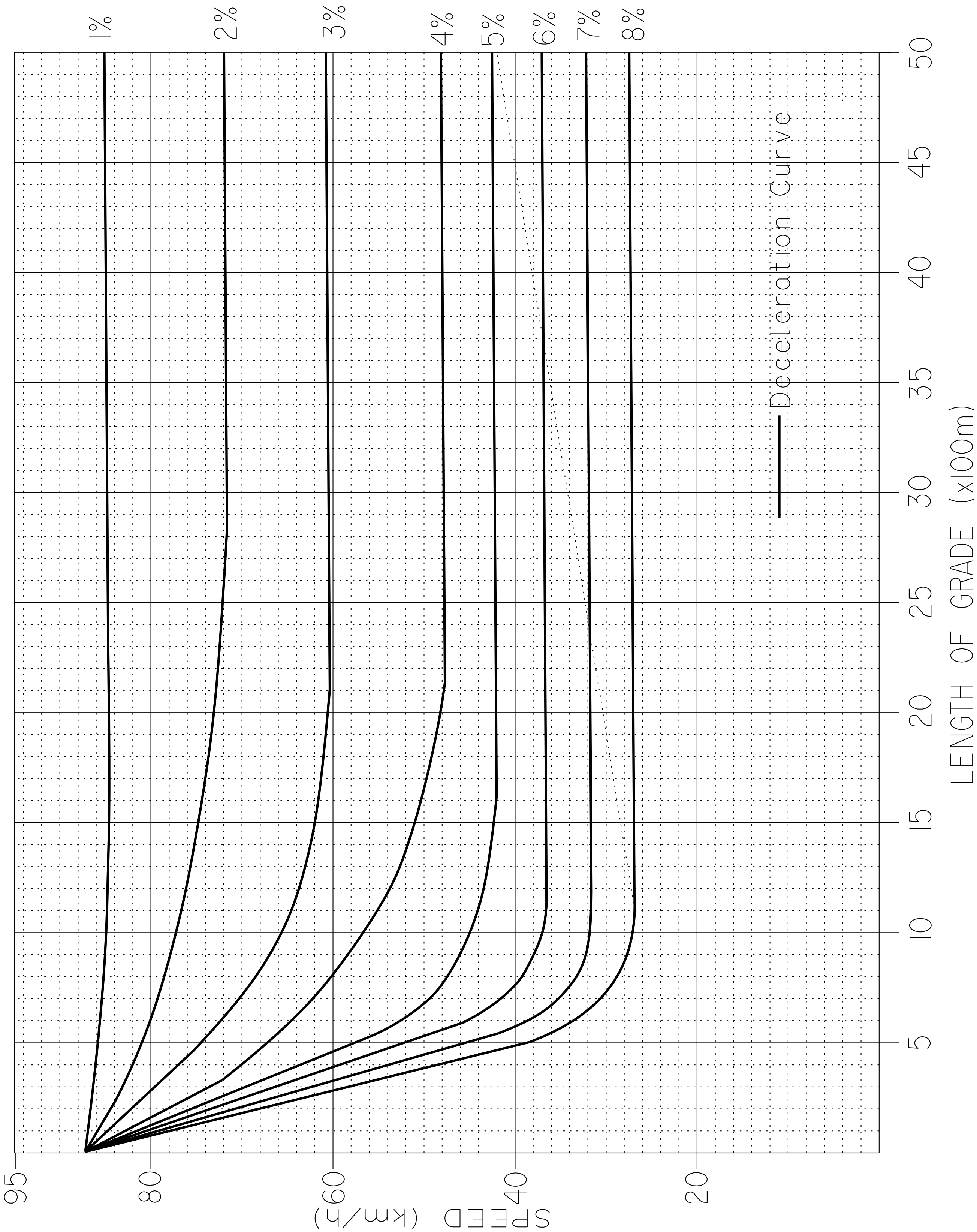
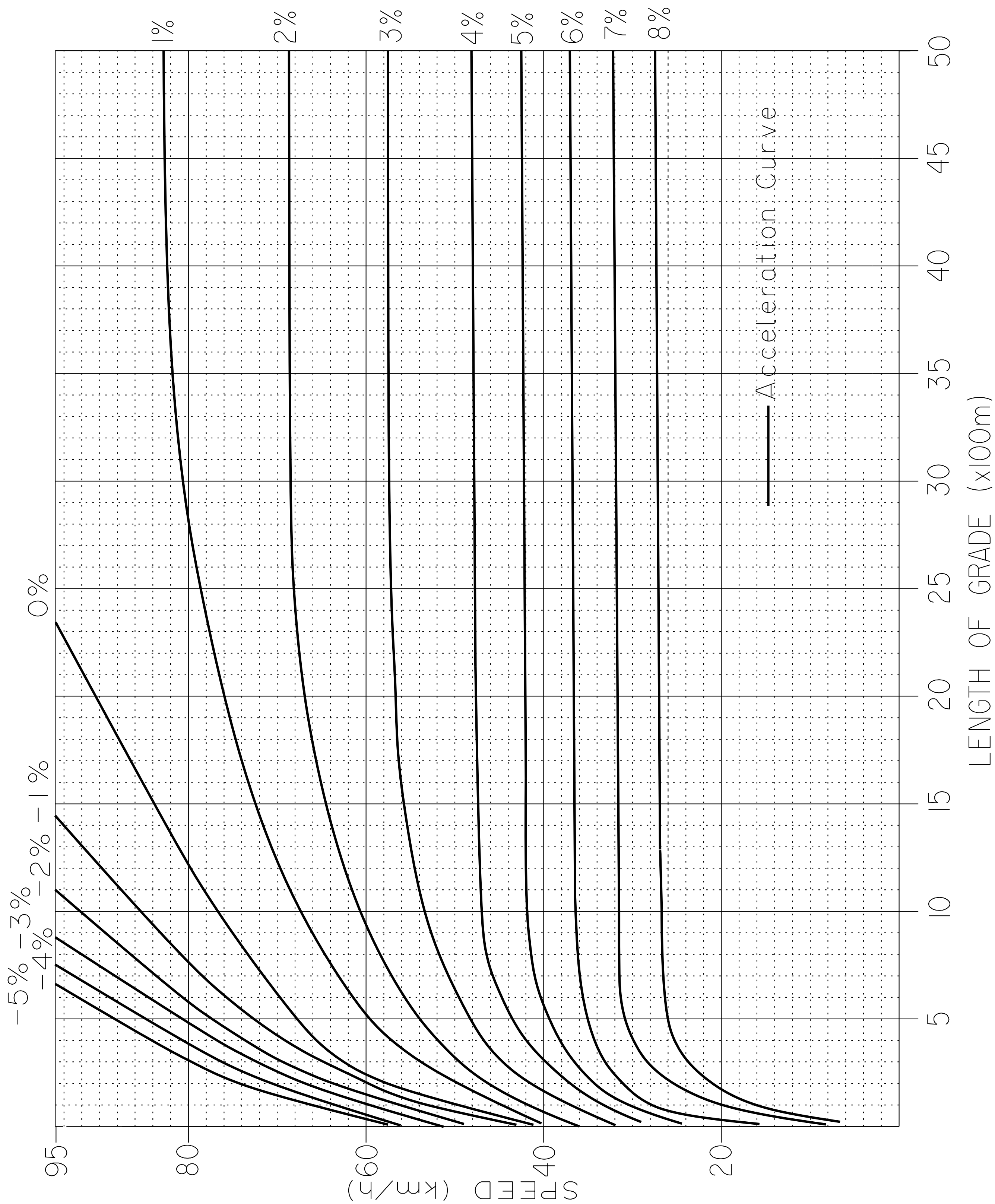


Figure B-5-3-3g Performance Curves for Heavy Trucks – 120 g/W Acceleration

(Source: Adapted from Figure I.3-2, "Highway Capacity Manual" [16])



B.5.4 Passing Lanes

Passing lanes are additional parallel auxiliary lanes provided on two-lane undivided highways for 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. Refer to Chapter A for information related to level of service targets. 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:

- volumes on a two-lane undivided highway are increasing and will soon warrant twinning; or
- the provision of passing lanes may postpone the construction of a divided facility; for example, for five to 10 years.

Aside from passing lanes, two other lane configurations (turnouts and 2+1 roadways) have been considered by Alberta Transportation to increase passing opportunities. Turnouts are addressed in Chapter F and have been used on Alberta provincial highways on oversize and overweight corridors where very slow-moving vehicles are present. The application of 2+1 roadways (a three-lane cross section where the road is striped in a manner to provide passing lanes in alternating directions), is not recommended for Alberta highways. The paper, “Feasibility of 2+1 Roads on Alberta Rural Highway Network” [17] analyzed 67 control sections in the province and concluded that 2+1 roadways are not an appropriate treatment on Alberta highways due to the large number of at-grade intersections. Alberta Transportation may consider the option of 2+1 roadways for case-specific projects, however, especially when there are no at-grade intersections within a long segment of corridor. This option is to be reviewed in accordance with the Alberta Transportation, “Design Exceptions Guideline” [2].

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 used. NPO is a function of both passing opportunities provided by highway geometry and the number of gaps in the opposing traffic stream. This concept is discussed in more detail in “Impact of Passing Lanes on the Quality of Service on Two-Lane Highways” [18].

The probability of time gaps greater than 30 seconds available for overtaking, known as P(GAO), is estimated using the following equation:

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

where:

$\mathrm{P(GAO)}$ = probability of time gaps (greater than 30 seconds) available for passing
Vopp = opposing traffic volume (veh/h)

For various values of Vopp, Table B-5-4-1a indicates the value of P(GAO).

The net passing opportunity (NPO) 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.

$$\mathrm{NPO} = \mathrm{P(GAO)}\ \mathrm{x}\ (\%\ \mathrm{PZ})$$

where:

$\mathrm{P(GAO)}$ = probability of time gaps (greater than 30 seconds) available for passing
$\%\ \mathrm{PZ}$ = percent of the road segment where passing is allowed, based on pavement markings (%)

For example:

- for an opposing volume of 100 veh/h, the probability of time gaps available for passing is 0.792.
- if a roadway has 73% passing zones, based on pavement markings,

$$\text{NPO} = 0.792 \times 73.0\% = 57.8\%$$

(i.e., the net passing opportunity is 57.8% on a roadway with 73% passing zones if the probability of time gaps available for passing is 0.792).

Each direction of travel should be examined separately to establish a warrant for provision of passing lanes. 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 Probability of Time Gaps Available for Overtaking

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 is used. Refer to Section A.4.2 for the calculation of DHV.

To obtain values for NPO based on opposing volume and percent no passing zones, Table B-5-4-1b may be used. 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 differ from above, make the appropriate adjustments.

The Alberta Transportation passing lane warrant is shown in Table B-5-4-1b and is 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%, passing improvement is warranted.

Table B-5-4-1b 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 420 m using an eye height of 1.08 m and an object height of 1.3 m 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 to 325)																
		Passing Lanes (150 to 500)																
		Twinning (250 to 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. <3 000) geometric improvements may be appropriate, where the volumes are at an intermediate level (i.e. 3 000 to 5 000) passing lanes may be the most effective method of providing the desirable N.P.O. and where the volume exceeds 5 000 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.

For warranting purposes, designers should consider the net passing opportunity derived for traffic in 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. Three rows are provided in 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 a function of traffic volume and speed distribution and is proportional to the square of the one-way stream flow. 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 realignment (to provide more passing zones) or twinning (which would provide unlimited 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. 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 (DHV > 300), it will be difficult to achieve a high level of service in the design hour without twinning. Passing lanes would have limited application on those roadways. 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 the roadway. Placement of passing lanes on an existing roadway, by altering the pavement markings without doing any grade widening (a pavement crown shift may be required), can be a very cost-effective improvement. Three lanes are placed on the existing roadway width, resulting in shoulders that are less than the desirable width. This type of improvement should be considered as an interim measure only.

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 is questionable.

B.5.4.2 Location, Spacing, and Length of Passing Lanes

"Planning and Design of Passing Lanes Using Simulation Model" [19] discussed a two-lane simulation model to determine the location and spacing of passing lane systems. Passing lane locations should be chosen with a view to minimizing unnecessary costs. Locations requiring large culverts, bridge widening, major cuts or major fills should be avoided where possible unless the construction of passing lanes is also beneficial in terms of balancing earthwork quantities.

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 required intersection treatment.

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

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 added to both sides of a two-lane undivided 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. Signs advising of the upcoming passing lane may alleviate driver frustration in the zone of restricted passing.

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.

Generally, passing lanes should not exceed about 25% 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 beginnings of successive passing lanes in the same direction.

A passing lane should be sufficiently long for a following vehicle to complete at least one passing manoeuvre. Short passing lanes, with lengths of 400 m or less, are not very effective in reducing platooning. As the length of a passing lane increases above 1.6 km, it will generally provide diminishing operational benefits. 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.

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 Figure B-6-1a.

B.6.2 Construction Practices at Typical Highway Transitions

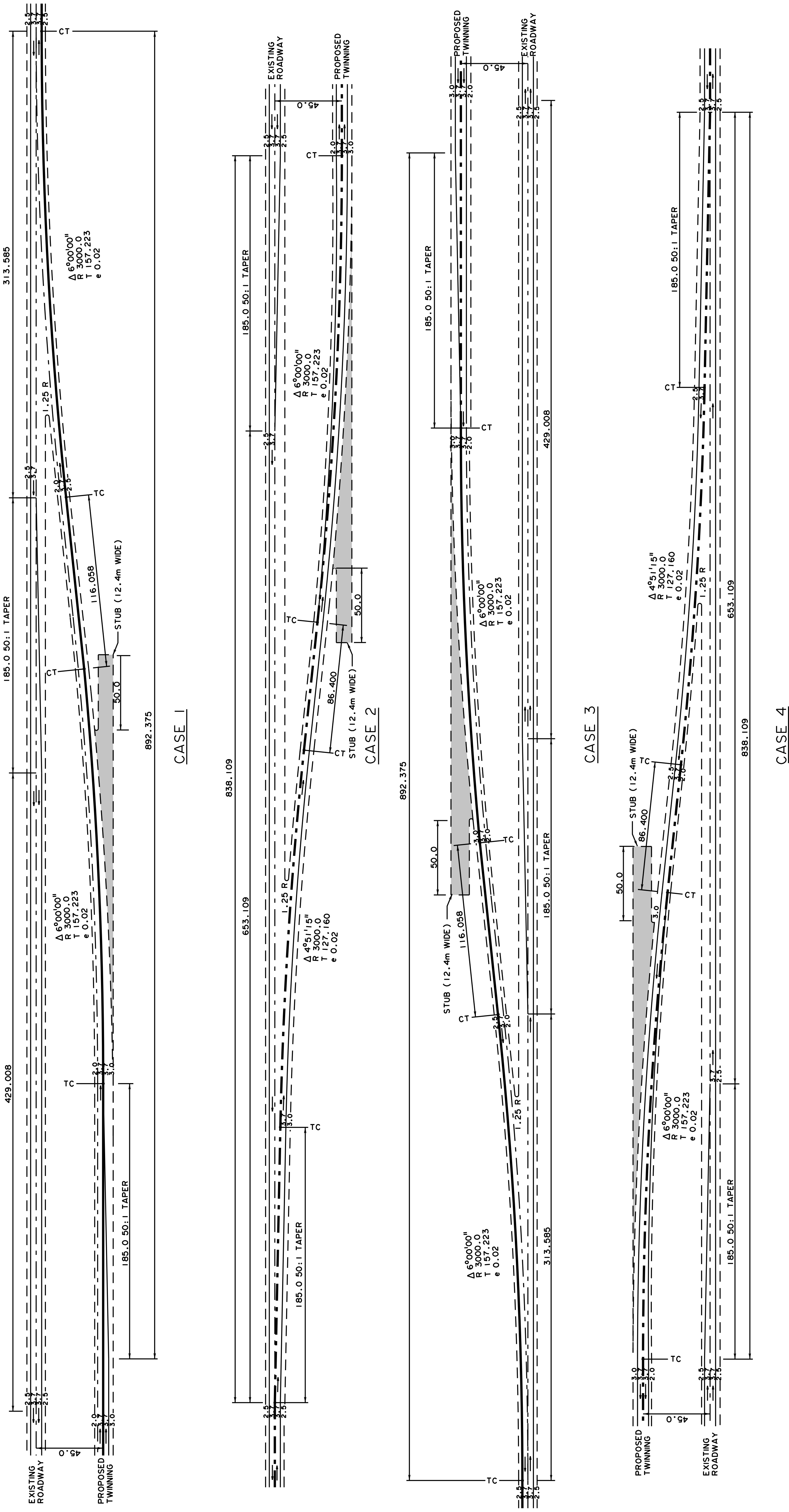
On projects where twinning of an existing two-lane facility is expected to occur in the near future, a parallel alignment section of the new two-lane, one-way roadway, should be built. This extension (subgrade and base course) beyond the point at which transition back to a two-lane undivided highway occurs, is called a stub.

As a minimum, the stub of the new roadway should be 50 m beyond the physical separation between the temporary transition sections (where the gore has occurred). In the event there are additional factors or circumstances such as grade, lighting conditions, etc., additional consideration for the minimum stub length should be given.

The stub maintains the -0.02 m/m cross-slope (normal crown) of the future through lanes along the entire length. The transition roadway section connects to the through lane at -0.02 m/m. Beyond the gore, the transition roadway section is rotated, attaining the -0.02 m/m cross-slope (normal crown) of the next roadway before joining it. The transition between the two roadways is then complete. Superelevation on the curved portion of the transition is kept to 2% to match the normal crown of each parallel roadway.

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

Figure B-6-1a Typical Highway Transitions (Two-lane Undivided to Four-lane Divided)



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 THE NEAR FUTURE. THE STUB MAY EXTEND MORE THAN 50 METRES. THE STUB MAY BE EXCLUDED IF DIRECTED BY ALBERTA TRANSPORTATION.
2. SUPERELEVATION (e) ON CURVES AND TRANSITIONS ARE KEPT AT 2% TO MATCH THE FUTURE CROWN. CONSTRUCTION IS TO ACCOMMODATE FINAL STAGE PAVING IF REQUIRED.
3. THE LINES SHOWN ON THIS DRAWING ARE FOR DESIGN ILLUSTRATION ONLY, THEY DO NOT REPRESENT PAVEMENT MARKINGS. REFER TO THE ALBERTA TRANSPORTATION "HIGHWAY PAVEMENT MARKING GUIDE" FOR TYPICAL PAVEMENT MARKINGS.
4. THE PLAN SHOWS A 45 m ℄ TO ℄ SPACING. DESIGNERS SHOULD USE A MINIMUM CURVE RADIUS OF 3000m AND A MINIMUM TANGENT OF 60m WHEN ADJUSTING FOR OTHER ℄ TO ℄ SPACINGS.

B.7 TEMPORARY HIGHWAY DETOURS

B.7.1 Introduction

Temporary highway detours (also known as “localized detours”) are temporary roadways that are built within or adjacent to the highway right-of-way to accommodate traffic through a work zone. They allow motorists to go around construction at an acceptable level of service without having an impact on workers. Temporary highway detours that are to be open for public use must be built from detailed drawings that have been signed and stamped by a Professional Engineer registered in the Province of Alberta. The detour plans indicate the proposed traffic accommodation measures and shall be drawn to scale, shall include the proposed vertical and horizontal alignments and shall be included in the Contractor’s “Traffic Accommodation Strategy”.

The following guidelines should be used where temporary detours are required to accommodate traffic on existing highways due to bridge construction, grading or other activities. Designers may exceed the standards shown here for efficiency or safety. The guidelines provide advice on the following topics:

- accommodation of pedestrians and cyclists;
- surfacing of the detour;
- appropriate design speed for the detour;
- minimum geometric parameters that should be used for each design speed on a detour; and
- horizontal alignment guidelines.

A typical cross-section plan and two typical detour plans are provided at the end of Section B.7.5. For other work zone aspects, such as signing of detours and barrier requirements, refer to the most recent edition of the Alberta Transportation, “Traffic Accommodation in Work Zones Manual” [20] and “Roadside Design Guide” [21].

B.7.2 Accommodation of Pedestrians and Cyclists

When constructing a temporary highway detour, it may be necessary to include specific accommodation for pedestrians and/or cyclists. If a highway has dedicated pedestrian and/or cyclist facilities (like sidewalks or multi-use pathways) that will be affected by construction, pedestrians and cyclists shall be provided with safe passage through or around the work areas. When passage is provided through the work area, suitable provisions shall be made to ensure pedestrians and cyclists are physically separated from workers and equipment. When pedestrian and cyclist traffic cannot be accommodated through the work area, an alternate route shall be made available. This may include additional signage to alert pedestrian and cyclist traffic of closures and the option of taking an alternate route to avoid passing through the work zone.

On provincial highways, specific accommodation for these users is most likely to be necessary in or near urban areas, but it may also be relevant in rural areas. Detailed guidance is not provided, as each scenario should be addressed on a case-by-case basis.

B.7.3 Guidelines for Surfacing of Detours

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 structurally adequate 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 on freeways, expressways and divided arterials (regardless of the project duration) or on undivided arterials and collectors (when the daily traffic volume exceeds 4000 and the detour will be in place for more than 4 months, or no alternate routes are available).

In some cases, Alberta Transportation may stipulate the type of surface required on a detour. Where this is not the case, Table B-7-3a is suggested as a guide. The surfacing options are asphalt concrete pavement, or road gravel with dust abatement treatment.

Table B-7-3a Guidelines for Surfacing of Detours

Functional Class	Project Duration < 4 Months		Project Duration ≥ 4 Months	
	AADT	Detour Surfacing	AADT	Detour Surfacing
Freeways, Expressways, Arterial Divided	All	Pavement *	All	Pavement
Arterial Undivided, Collector	≥ 4000	Gravel	≥ 4000	Pavement / Gravel
	< 4000	Gravel	< 4000	Gravel
Local	All	Gravel	All	Gravel

***** In emergencies, a gravel surface may be used.

B.7.4 Design Speed of Detours

Generally, at locations where the work results in a change to the existing road conditions (e.g., lane restrictions, reduced lane widths, detours, etc.), creates obstructions or requires the presence of workers/equipment in or adjacent to the normal path of travel, a reduced speed zone is warranted. Speeds shall be appropriate for accommodating traffic safely through or around the work zone with a minimum of inconvenience.

The reduced speed through the work zone is selected based on the type of roadway (two-lane or multi-lane), type of traffic control, type and nature of the disruption, physical layout and constraints, etc. as noted in the Alberta Transportation, “Traffic Accommodation in Work Zones Manual” [20].

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 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 time or where the detour itself is long.

B.7.5 Horizontal Alignment Guidelines for Detours

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

The maximum deflection angles and minimum radii are shown in Table B-7-5a. The minimum radii are 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 appropriate because detours do not have to provide the same level of comfort that is appropriate on the open highway; however, they must be safe. Using a 0.05 m/m superelevation rate safely accommodates 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 side-slipping.

Table B-7-5a Geometric Parameters of Detours

Design Speed of Detour (km/h)	50	60	70	80	90
Maximum Deflection (degrees) for first curve joining the highway	25	25	20	15	10
Maximum Grade (%) ≥ 200 AADT	8	8	8	8	7
Maximum Grade (%) < 200 AADT	10	10	9	8	7
Stopping Sight Distance (m) *	50	65	85	105	130
Minimum Crest K * (based on stopping sight distance)	4	7	11	17	26
Minimum Sag K * (based on comfort control)	5	7	10	13	17
Maximum Superelevation (m/m)	Use 0.05 m/m as a maximum on the detour				
Superelevation	Use Table B-7-5b Superelevation for Detours				
Minimum Radius (m) **	120 ***	120	185	250	350
Minimum Width (m) ****	9 m if AADT < 2000, 10 m if AADT ≥ 2000 See notes in Figure B-7-5a				
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", for low-speed urban design.

*** The 120 m radius is a practical minimum for tractor-trailer combinations on Alberta provincial highway detours.

**** Exceptions have been made to this where the project duration has been one month or less, and illumination has been provided. A basic lane width of 3.5 m, plus the suggested minimum shy line offset, as indicated in Section H5.4.1 of the Alberta Transportation, “Roadside Design Guide” [21] (based on design speed being equivalent to posted speed) has been permitted.

**** Where the detour is being constructed on an oversize and overweight corridor, consider increasing parameters like road width and radii from the suggested minimums.

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

Adequate warning and decision sight distance (based on the design speed of the highway) are to be provided on the approaches to the detour. The guidelines for construction zone signs and other traffic control devices are outlined in the Alberta Transportation, “Traffic Accommodation in Work Zones Manual” [20].

Table B-7-5b shows recommended superelevation values for each radius, for design speeds from 50 km/h to 90 km/h. For higher design speeds, the 0.06 m/m superelevation table in Section B.3.5 is used. The distribution of superelevation and side friction, through the range of curves shown in Table B-7-5b, is based on Method 5, described in the 2018 AASHTO, “A Policy on Geometric Design of Highways and Streets” [3]. Method 5 represents a practical distribution over the range of curvature.

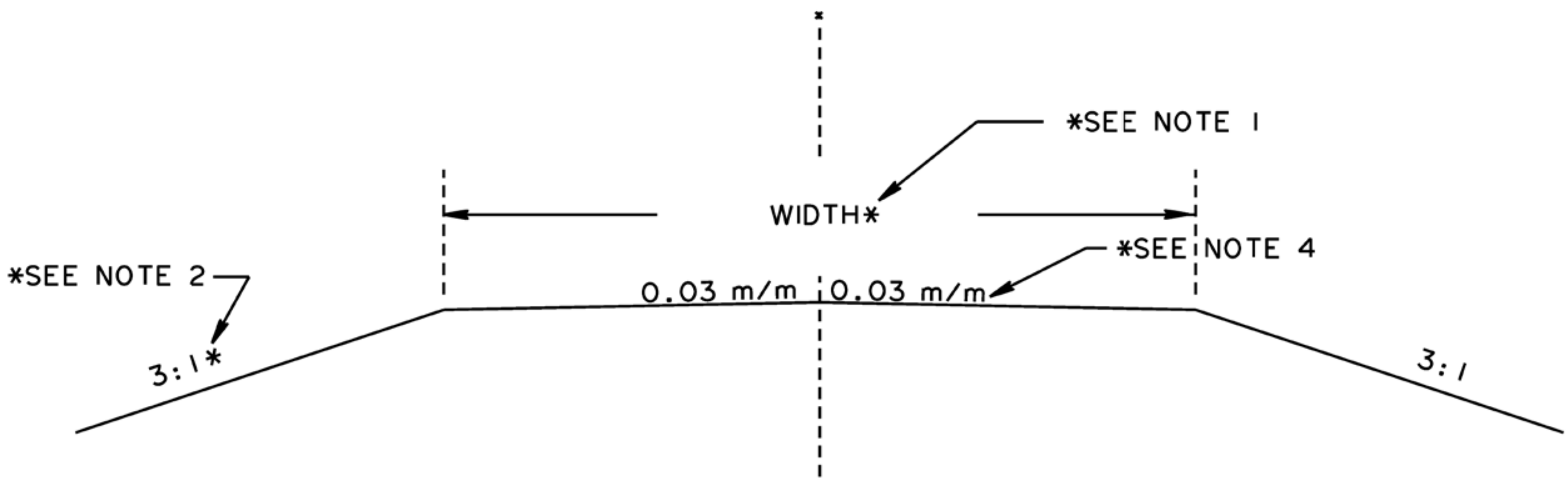
Table B-7-5b Superelevation for Detours

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

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

Figures B-7-5a, B-7-5b, and B-7-5c show a typical cross section as well as typical plan views of bridge detours with three-curve and four-curve layouts. The four-curve layout is appropriate where a temporary bridge structure (overhead truss type) requiring a tangent approach is utilized.

Figure B-7-5a Typical Cross-Section for Detour



GRADED EMBANKMENT WITH GRAVEL
DRIVING SURFACE

NOTES

1. WIDTH = 9 m FOR AADT LESS THAN 2000.
   WIDTH = 10 m FOR AADT GREATER THAN 2000 OR FOR LOG HAUL ROUTES.
   AN ADDITIONAL 0.5 m WIDTH IS TO BE PROVIDED ADJACENT TO THE 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 THE SLOPE, THE SLOPE MAY BE BUILT AS STEEP AS POSSIBLE BASED ON THE STABILITY OF THE SOIL.
3. TRAFFIC SHALL BE PROTECTED FROM ANY NON-TRAVERSABLE HAZARDS ADJACENT TO THE DETOUR. THIS INCLUDES CULVERT OPENINGS IN EXCESS OF 1.4 m DIAMETER WITHIN 5 m OF THE DRIVING SURFACE. MORE SEVERE HAZARDS MAY NEED TO BE MITIGATED EVEN AT GREATER OFFSETS.
4. IF THE DETOUR ROADTOP IS TO BE PAVED, A 0.02 m/m NORMAL CROWN MAY BE USED.

Figure B-7-5b Typical Plan View for Bridge Detour (3 Curves)

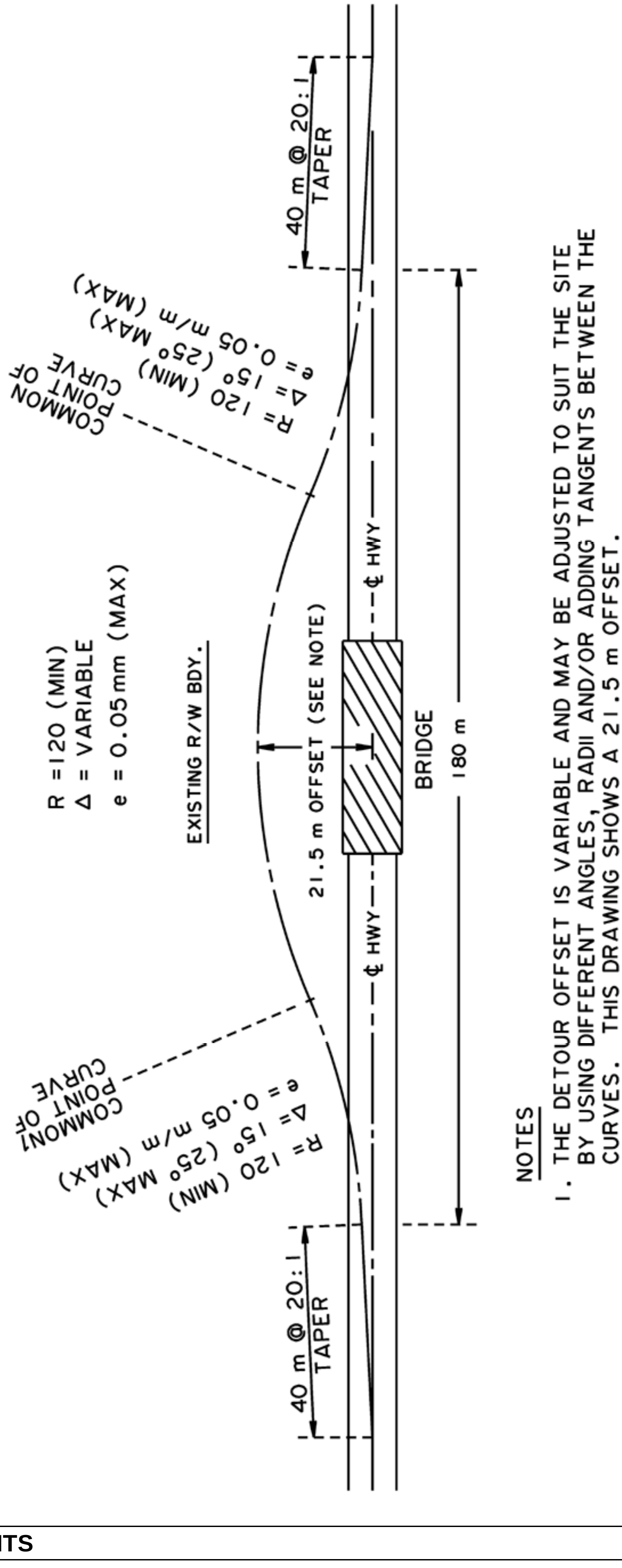
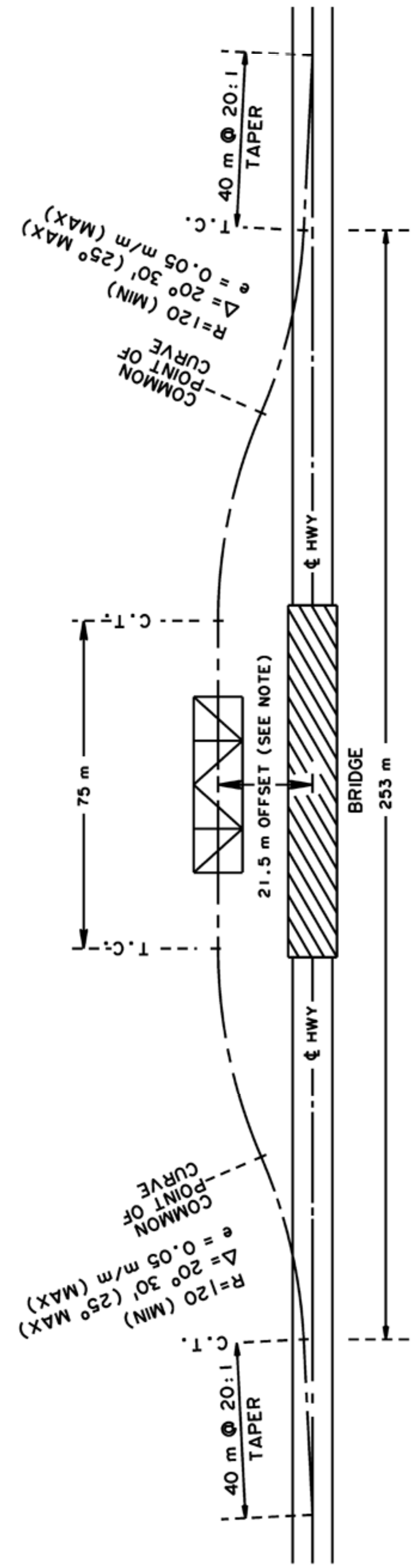


Figure B-7-5c Typical Plan View for Bridge Detour (4 Curves)



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 THE CURVES. THIS DRAWING SHOWS A 21.5 m OFFSET.

REFERENCES

[1] D. B. Fambro, K. Fitzpatrick and R. J. Koppa, "NCHRP 400: Determination of Stopping Sight Distances," Transportation Research Board, National Research Council, Washington, D.C., 1997.

[2] Alberta Transportation, "Design Exceptions Guideline," September 2018. [Online]. Available: https://open.alberta.ca/publications/design-exceptions-guideline. [Accessed July 2020].

[3] American Association of State Highway Transportation Officials, A Policy on Geometric Design of Highways and Streets, 7th ed., Washington, D.C., 2018.

[4] B. Kenny, A. Kwan and J. Morrall, "Design and Operational Considerations to Accommodate Long Combination Vehicles and Log Haul Trucks on Rural Highways in Alberta, Canada," in *Proceedings - 7th International Symposium on Heavy Vehicle Weights and Dimensions*, Delft, The Netherlands, June 2002.

[5] American Association of State Highway Transportation Officials, A Policy on Geometric Design of Highways and Streets, 5th ed., Washington, D.C., 2004.

[6] Transportation Association of Canada, Manual of Uniform Traffic Control Devices for Canada, 4th ed., Ottawa, ON, 1998.

[7] Alberta Transportation, "Highway Pavement Marking Guide," March 2003. [Online]. Available: https://open.alberta.ca/publications/highway-pavement-marking-guide-2nd-edition. [Accessed July 2020].

[8] Transportation Association of Canada, Geometric Design Guide for Canadian Roads, 3rd ed., Ottawa, ON, 2017.

[9] C. Seams, "Driving.ca," 14 March 2014. [Online]. Available: http://driving.ca/auto-news/news/five-great-canadian-driving-roads. [Accessed 22 July 2020].

[10] Sam, "OZROADS: The Australian Roads Website," November 2005. [Online]. Available: http://www.ozroads.com.au/NSW/Highways/Pacific/karuah-bulahdelah.htm. [Accessed 22 July 2020].

[11] C. Liu and T. J. Ye, "Report No. DOT HS 811 500: Run-Off-Road Crashes: An On-Scene Perspective," U.S. Department of Transportation, National Highway Traffic Safety Administration, Washington, D.C., July 2011.

[12] "SantaBanta," [Online]. Available: http://www.santabanta.com/photos/roads/2120029.htm. [Accessed 22 July 2020].

[13] Alberta Transportation, "CB-6 Highway Standard Plates - Active," [Online]. Available: https://www.alberta.ca/cb-6-highway-standard-plates-active.aspx. [Accessed July 2020].

[14] Alberta Transportation, "Benefit Cost Model and User Guide," [Online]. Available: https://www.alberta.ca/benefit-cost-model-and-user-guide.aspx.

[15] Transportation Research Board, Highway Capacity Manual, 6th ed., Washington, D.C.: Transportation Research Board, National Academy of Sciences, 2016.

[16] Transportation Research Board, Highway Capacity Manual, 3rd ed., Washington D.C.: Transportation Research Board, National Academy of Sciences, 1985.

[17] R. V. McGregor, M. Hassan and B. Kenny, "Feasibility of 2+1 Roads on Alberta Rural Highway Network," in *Proceedings of the 2009 Annual Conference and Exhibition of the Transportation Association of Canada*, Vancouver, Canada, October 2009.

[18] J. Morrall, "Impact of Passing Lanes on the Quality of Service on Two-Lane Highways," in *Proceedings of the Third International Symposium on Highway Capacity*, Copenhagen, Denmark, June 1998.

[19] J. Morrall, E. Miller Jr., G. A. Smith, J. Feuerstein and F. Yazdan, "Planning and Design of Passing Lanes Using Simulation Model," *ASCE Journal of Transportation Engineering,* vol. 12, no. 1, pp. 50-62, 1995.

[20] Alberta Transportation, "Traffic Accommodation in Work Zones Manual," December 2018. [Online]. Available: https://manuals.transportation.alberta.ca/tas/Pages/Home.aspx. [Accessed July 2020].

[21] Alberta Transportation, "Roadside Design Guide," 2007. [Online]. Available: https://www.alberta.ca/roadside-design-guide.aspx. [Accessed July 2020].