Chapter 4

ALIGNMENT AND TYPICAL SECTION

Introduction

Horizontal and vertical alignment, or line and grade, are permanent design elements which warrant thorough study. It is difficult and costly to correct alignment deficiencies after the highway is constructed. On most arterial streets, extensive development may take place along the property lines making it difficult or impractical to change the alignments in the future. Any compromise in the alignment design must be weighed carefully as initial savings may be offset by compromising safety and causing traffic delays.

Horizontal and vertical alignment should not be designed independently. Poorly designed horizontal to vertical relationships can create roadways that are unnecessarily difficult to negotiate and that can be unsafe in less than optimum driving conditions.

Typical sections show the basic information for cross section dimensions and pavement details. The cross section dimensions affect right-of-way requirements and earthwork which in turn influence the selection of alignment.

Horizontal and Vertical Alignment

Design Control

After the general location has been fixed, the design speed should be chosen which then identifies limitations of horizontal and vertical alignments. Physical characteristics of the area that influence alignment are the function of the highway, traffic volumes, topography, subsurface conditions, existing development, likely future developments, and location where the improvements match the existing roadway. As design progresses to detailed alignment and profile, the design speed chosen should fit the travel needs and should be consistent with the expected operating speed. Features such as horizontal and vertical curvature, superelevation, and sight distance, are directly related to, and vary appreciably with, design speed. Other features, such as widths of lanes and shoulders, and clearances to walls and rails, are not directly related to design speed, but they affect vehicle speed, and higher standards should be accorded these features for the higher design speeds. Thus, when a change is made in design speed, many design elements of the highway are subject to change.

Line-Grade Coordination

Coordination of line and grade should begin with preliminary design, when adjustments can readily be made.

Proper combination of line and grade is obtained by considering the following controls from AASHTO's *A Policy on Geometric Design of Highways and Streets* (the "Green Book") (5):

- Curvature and grades should be in proper balance. Tangent alignment with steep grades and excessive curvature with flat grades are both undesirable design.

- Vertical curvature superimposed upon horizontal curvature, or the reverse generally results in a more pleasing facility. Successive changes in profile not in combination with horizontal curvature may result in a series of humps and an undesirable condition.

- Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. This condition is such that the driver cannot perceive the horizontal change in alignment, especially at night. The horizontal curvature should lead the vertical curvature, i.e., the horizontal curve is made longer than the vertical curve.

- Similar to the above, sharp horizontal curvature should not be introduced at or near the low point of pronounced sag vertical curves. Because the road ahead is foreshortened, sharp horizontal curvature assumes a distorted appearance. Also, vehicle speeds are often high at the bottom of grades, causing erratic operation.

- On two-lane roads and streets, the need for safe passing sections at frequent enough intervals and for an appreciable percentage of the length of the roadway often emphasizes the desirability of good line and grade combination. In these cases, it is necessary to design longer tangent sections.

- Horizontal curvature and profile at intersections should be made as flat as practical. Sight distance along all approaches is important.

- On divided highways and streets, variation in width of median and the use of separate horizontal alignments and profiles is sometimes an advantage. Where traffic justifies four or more lanes, a superior design without additional cost generally results from the design of separated one-way roadways.

- In residential areas, the alignment should be designed to minimize nuisance factors to the neighborhood. Generally, a depressed facility makes a highway less visible and less noisy for adjacent residents. Minor horizontal adjustments can also be made sometimes to increase the buffer zone between the highway and clusters of homes.

- The alignment should be designed to expose attractive natural scenic views.

The designer should use working plans of a size, scale and arrangement so that long, continuous stretches of highway can be studied in both plan and profile and visualized as a unit in three dimensions. Working plans should be of sufficiently manageable scale, with the profile corresponding to the plan.

After study of the preliminary horizontal and vertical alignments, adjustments can be made to obtain the desired coordination. At this stage, the designer should be more concerned with a graphical analysis that matches physical controls rather than with mathematizing alignments. The use of highway curve templates and straight-edges are convenient for this purpose. The criteria and elements of design covered in the preceding chapter should be considered. For the selected design speed, the values for controlling horizontal curvature, grade, sight distance, and superelevation runoff length should be available and checked graphically to ensure the design meets specific needs. The general design controls in this chapter for horizontal and vertical alignment should be considered. All aspects of terrain, traffic operation, and appearance can be considered and the horizontal and vertical alignments adjusted and coordinated before preparing detailed construction plans.

The Bureau of Highway Design normally uses the Department's CAD/D system to define final alignments and some preliminary alignments.

Location Surveys

Location surveys are performed by the Bureau of Highway Design, Design Services Survey Section. The alignment to be surveyed is normally mathematized during the preliminary or pre-hearing design process. Often circumstances are encountered during the design process that require the construction line to be laid out in the field to confirm that the major controls are met and that the alignment matches the existing roadway. The field-staked alignment can be used as a reference by other Bureaus, such as for Materials and Research's geotechnical investigations. Chapter 12 provides additional information on survey issues.

Horizontal Alignment

General Controls

In addition to the foregoing design controls, there are a number of general controls which should be considered. Excessive curvature or poor combinations of curvature may create unsafe situations, limit capacity, and detract from a pleasing roadway appearance. To avoid this, particularly on rural highways, use these general controls:

- Alignment should be as directional as possible but should be consistent with the topography (avoiding excessive cuts and fills) and with preserving environmental resources, developed properties and community values.
- Maximum curvature should be avoided wherever possible. The designer should attempt to use generally flatter curves, retaining the maximum for the most critical conditions.
- Consistent alignment is important for driver expectation. Sharp curves should not be introduced at the ends of long tangents. Sudden changes from areas of flat curvature to areas of sharp curvature should be avoided.
- For small deflection angles, curves should be sufficiently long to avoid the appearance of a "kink" or angle point in the road. Curves should be at least 150 m long for a deflection angle of five degrees, and the minimum length should be increased 30 m for each one degree decrease in the deflection angle. The minimum length of horizontal curve (in meters) on main highways should be about 3 times the design speed. On high-speed controlled-access facilities that

use flat curvature, a desirable minimum length of curve for aesthetic reasons would be about 6 times the design speed.

- Sharp curvature should be avoided on high, long fills. It is difficult for drivers to see the extent of curvature and adjust their speed to the conditions.
- Compound circular curves may be used with caution. Preferably their use should be avoided where curves are sharp, as they sometimes are on interchange ramps. A maximum ratio of compound circular curve radii should be 1.5:1 on two-way roads; 2:1 on one-way roads, one-way ramps, and intersection approach curves. Compound curves with large differences in curvature introduce similar problems that arise from a tangent approach to a circular curve. On one-way roads such as ramps, the difference in radii of compound curves is not so important if the second curve is flatter than the first.
- Abrupt "S" curves should be avoided. Such curves make it difficult for a driver to keep within the traveled lane. It is also difficult to superelevate both curves appropriately. A reversal in alignment can be designed suitably by including at least a minimum length of superelevation runoff tangent between the two curves.
- The "broken-back" arrangement of curves, with a short tangent between two curves in the same direction, should be avoided. Most drivers don't expect succeeding curves to be in the same direction, and this arrangement is not pleasing in appearance when both curves are visible for some distance ahead.
- To avoid the appearance of inconsistent distortion, the horizontal and vertical alignment should be carefully coordinated. General controls for this coordination are discussed earlier in this Chapter.
- Ending or beginning a curve on a bridge is undesirable, as it adds complications to the bridge design, construction, and maintenance. Likewise, horizontal curves beginning or ending near a bridge should be positioned to avoid any part of the superelevation transition on the bridge. When curvature is unavoidable, the bridge should be entirely on a simple curve as flat as physical conditions permit. It is important to coordinate the line and grade with the Bureau of Bridge Design early in the process.

Sight Distance

An important element of horizontal alignment is the sight distance across the inside of curves. Where there are sight obstructions (such as walls, cut slopes, buildings, and longitudinal barriers) on the inside of curves, a design to provide adequate sight distance may require adjustment in the normal highway cross section or change in alignment if the obstruction cannot be removed. Horizontal alignment must provide at least the minimum stopping sight distance for the design speed at all points on the highway, as given in Figures III-24(A) and III-24(B) in the 1994 "Green Book".

On two-lane highways, avoid the use of long flat curves that reduce the passing sight distance to less than adequate. It is better to use shorter curves and increase the length of the tangent between the curves. The designer must consider the impact that curves approaching and departing from passing sections have on passing sight distance.

Passing Opportunities

Passing sections are used to provide opportunities to pass slower moving traffic on two-lane highways. Passing sections should be provided as frequently as possible in keeping with the terrain.

The extent of restrictive sight distance has a considerable effect on the design capacity of a two-lane highway. Sight distances to the road surface in the range of 450 to 600 m at frequent intervals are considered essential if the gaps in the traffic stream created by slow-moving vehicles are to be filled and a more desirable operating speed maintained. This measurement criteria is selected for the purpose of evaluating design capacity on two-lane highways. Both horizontal and vertical sight distance should be determined and the restricted portion expressed as a percentage of total length of highway for evaluation purposes. Refer to the "Green Book" for passing sight distance criteria.

Intersection Sight Distance

Proper development of line and grade through intersections is critical to provide a safe design. Figures 4-1 and 4-2 show three cases of intersection sight distance problems:

Cases I and II -- are not normally applicable to New Hampshire design practice. For complete discussions, see the "Green Book".

Case III (A,B,C) -- vehicles stopped on the minor road must have time to safely negotiate any of three basic maneuvers, including:

A. Travel across the intersecting roadway by clearing traffic on both the left and the right of the crossing vehicle;

B. Turn left into the crossing roadway by first clearing traffic on the left and then to enter the traffic stream with vehicles from the right; or,

C. Turn right into the intersecting roadway by entering the traffic stream with vehicles from the left.

These conditions are shown by the Cases A, B and C, respectively, on Figure 4-2. Figure 4-3 and 4-4 provide design values.

Required sight distances for trucks will be substantially longer than for passenger cars. These relationships for trucks (SU, WB-15) can be derived using appropriate assumptions for vehicle acceleration rates and turning paths. Refer to the "Green Book" for a detailed discussion.

Case IV -- Signal control sight distance based on Case III should be achieved but it is not required due to operational characteristics (flows move at separate times).

With particular regard to Case III at ramp terminals, sight distance is very often affected by the proximity of a bridge and by snow-covered slopes intruding into the line of sight.

Therefore, all parts of the bridge structure including the bridge railing should be examined for sight distance obstruction, and an allowance of 0.6 m minimum snow depth on all slopes should be included as part of the sight distance evaluation. In areas of the State that experience heavier snowfalls, one may consider increasing the snow depth accordingly.

Grades up to 3 percent have little effect on stopping sight distances. Grades on an intersection leg should be limited to 3 percent unless the sight distances are considerably in excess of the minimums for stopping on a level grade, in which case the grades should not exceed 6 percent.

In Case III-A the time required to cross the major highway is materially affected by the grade of crossing on the minor road. Normally, the grade across an intersection is so flat that it need not be considered, but when curvature on the major road requires the use of superelevation, the grade across the superelevated area may be significant, in which case the sight distance along the major road should be increased. The "Green Book" gives recommended grade adjustment factors.

Intersections with skew angles less than 60 degrees may warrant individual calculation of sight distances due to the increased turning path length. Refer to the "Green Book" for a detailed discussion.

The designer should realize that sight distance problems rarely have one obvious solution. Careful consideration is necessary when designing for sight distance. The foregoing explanation is basic and the "Green Book" should be consulted to fully understand the ramifications of the problem.

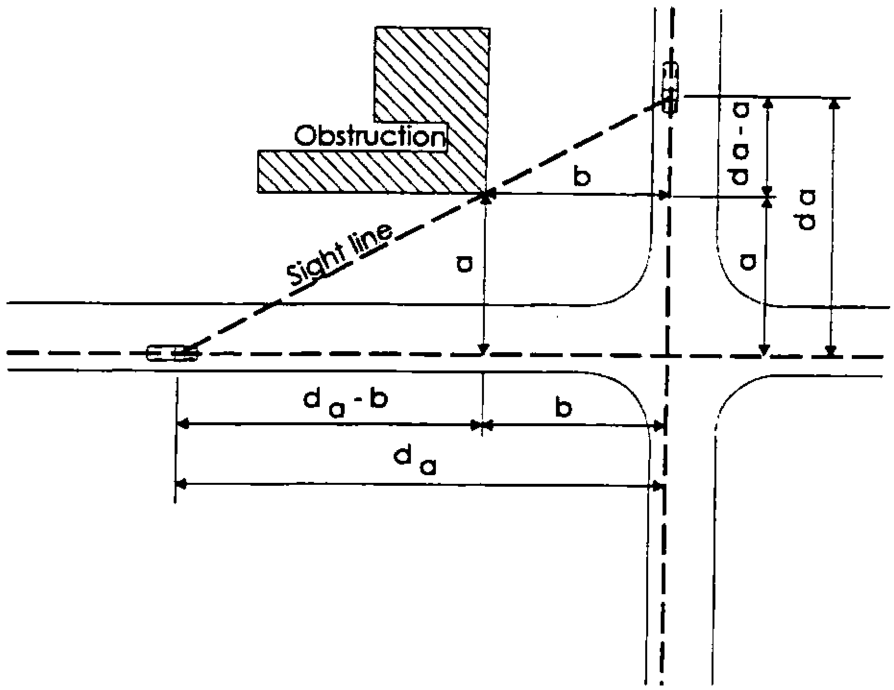
Measuring Intersection Sight Distance

Measuring Case III intersection sight distance is done by locating the driver's eye at 6.0 m behind the edge of the major highway's traveled way. The height of eye is 1070 mm (for passenger cars) and the height of object is 1300 mm above the pavement.

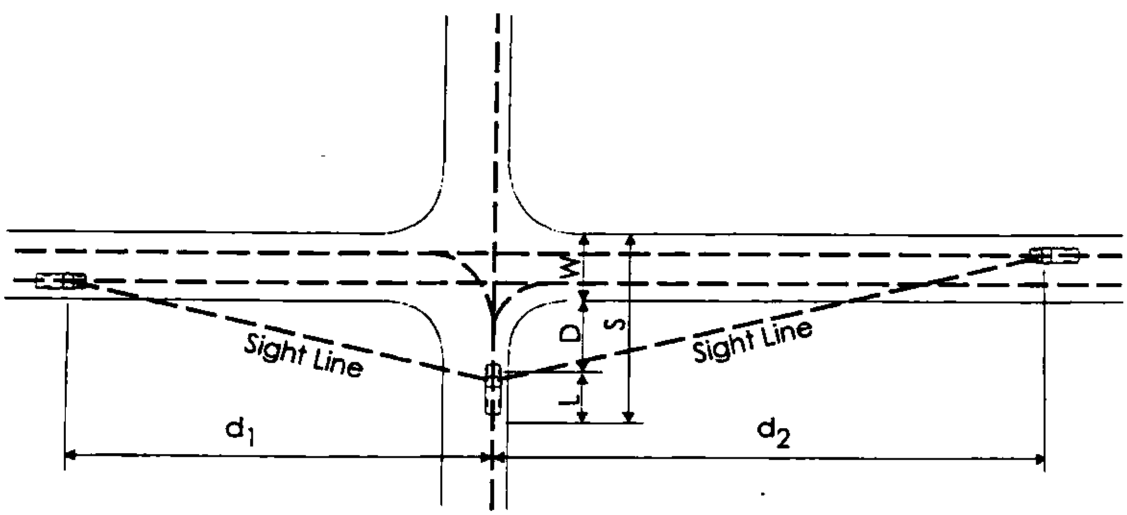
Figure 4-1

SIGHT DISTANCE AT INTERSECTIONS, Minimum Sight Triangle

Variables defined in the "Green Book"



A. CASE I & II
NO CONTROL OR YIELD CONTROL ON MINOR ROAD

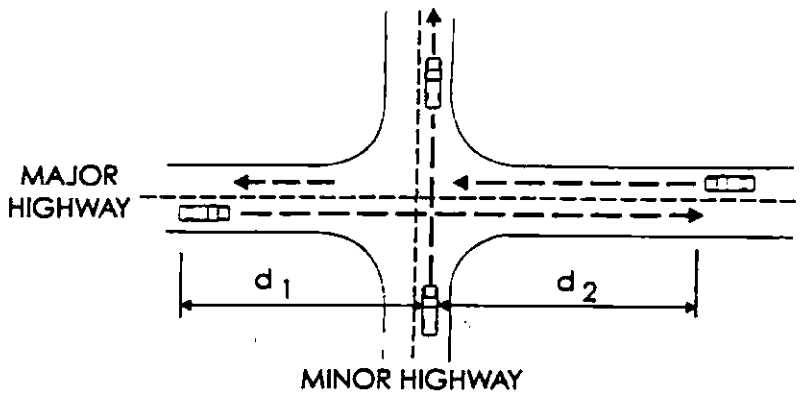


B. CASE III
STOP CONTROL ON MINOR ROAD

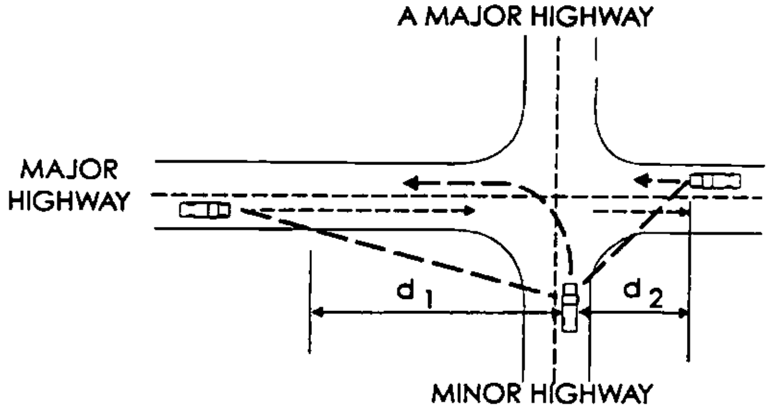
Figure 4-2

INTERSECTION SIGHT DISTANCE

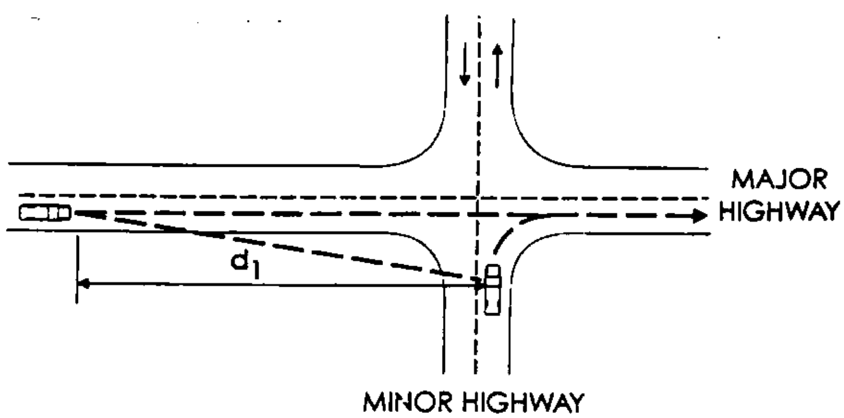
Variables defined in the "Green Book"



CASE IIIA - STOPPED VEHICLE CROSSING
A MAJOR HIGHWAY



CASE IIIB - STOPPED VEHICLE TURNING LEFT INTO
MAJOR HIGHWAY

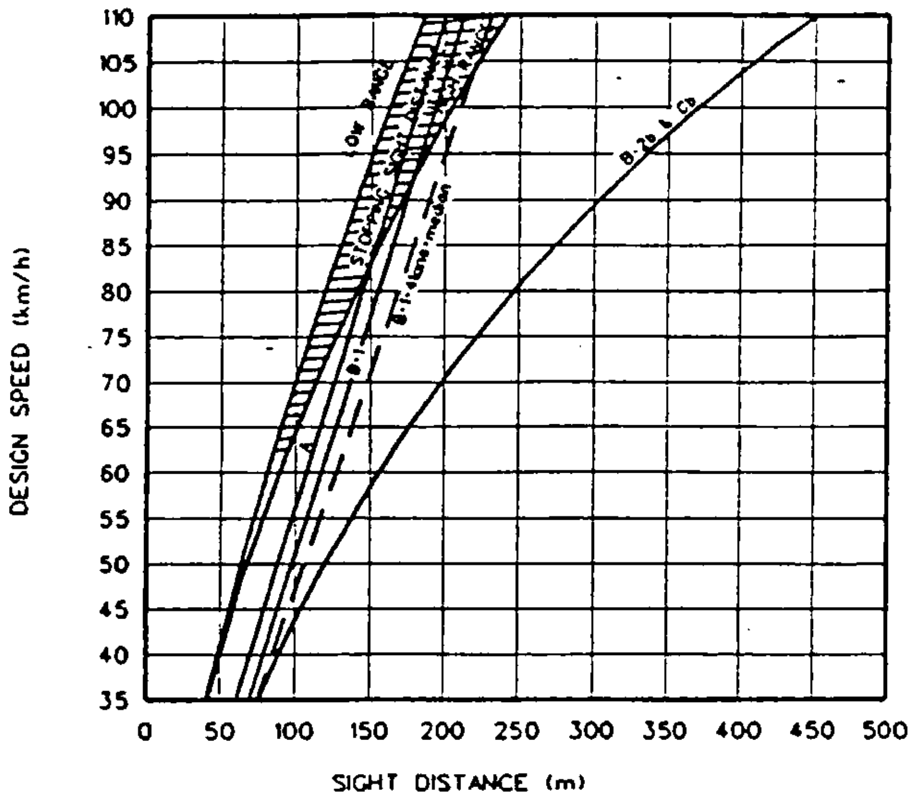


CASE IIIC - STOPPED VEHICLE TURNING RIGHT INTO
MAJOR HIGHWAY OR RIGHT TURN ON RED SIGNAL

Figure 4-3

CASE III INTERSECTION SIGHT DISTANCE

(Passenger vehicles, level grades, right angle intersection)



A- SIGHT DISTANCE FOR P VEHICLE CROSSING 2-LANE HIGHWAY FROM STOP. (SEE DIAGRAM).

B-1- SIGHT DISTANCE FOR P VEHICLE TURNING LEFT INTO 2-LANE HIGHWAY ACROSS P VEHICLE APPROACHING FROM LEFT. (SEE DIAGRAM).

B-1-4Lane-median- SIGHT DISTANCE FOR P VEHICLE TURNING LEFT INTO 4-LANE HIGHWAY ACROSS P VEHICLE APPROACHING FROM LEFT. (SEE DIAGRAM).

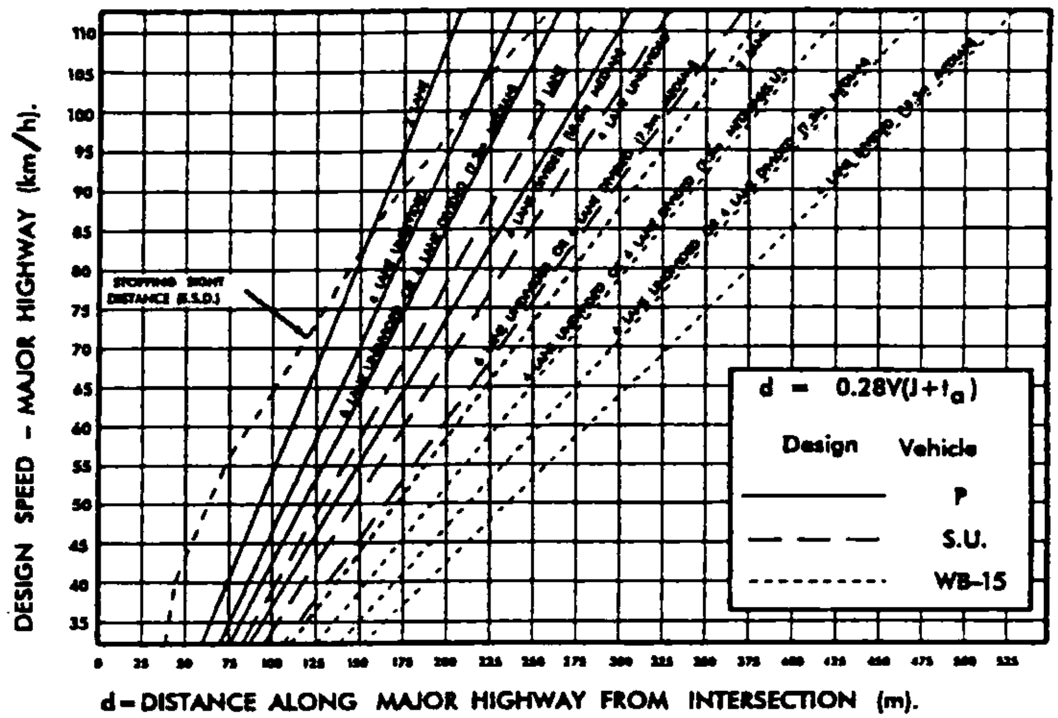
B-2b- SIGHT DISTANCE FOR P VEHICLE TO TURN LEFT INTO 2-LANE HIGHWAY AND ATTAIN 85% OF DESIGN SPEED WITHOUT BEING OVERTAKEN BY A VEHICLE APPROACHING FROM THE RIGHT REDUCING SPEED FROM DESIGN SPEED TO 85% OF DESIGN SPEED. (SEE DIAGRAM).

Cb- SIGHT DISTANCE FOR P VEHICLE TO TURN RIGHT INTO 2-LANE HIGHWAY AND ATTAIN 85% OF DESIGN SPEED WITHOUT BEING OVERTAKEN BY A VEHICLE APPROACHING FROM THE LEFT AND REDUCING FROM DESIGN SPEED TO 85/ OF DESIGN SPEED. (SEE DIAGRAM).

Figure 4-4

SIGHT DISTANCE AT INTERSECTIONS FOR CROSSING MANEUVER

(Case IIIA, required sight distance along a major highway).



Note: For 3.6m Lanes, Level Conditions and Right Angle Intersections only.

Horizontal Curves

Design of horizontal curvature is based primarily on the established design speed and terrain conditions. Design criteria for maximum permissible curvature and procedure for defining curvature are covered in this chapter.

Simple Curves

Simple curves are defined as an arc segment of a circle. The design elements of a simple horizontal curve and their terminology are shown in Figure 4-5.

Usually the P.I. station, and the deflection angle (Δ) are established. The remaining curve data may be computed using the formulas below.

L =	$(\Delta R \pi)/180$
T =	R tan $\Delta/2$
E =	T tan $(\Delta/4)$
M =	R $(1 - \cos \Delta/2)$
P.C. Station =	P.I. Station - T
P.T. Station =	P.C. Station + L

Units and accuracy of curve data shown on plans should be:

R =	meters, previously defined curves (in English units) will be soft converted to the nearest 0.001 m, proposed curves will be expressed in increments of 5 m up to 250 m, and in increments of 10 m above 250 m.
Δ =	degrees, minutes and seconds consistent with closure calculations and survey order of accuracy
L, T, E, M, Stations =	meters, to the nearest 0.001 m.

Spiral Transition Curves

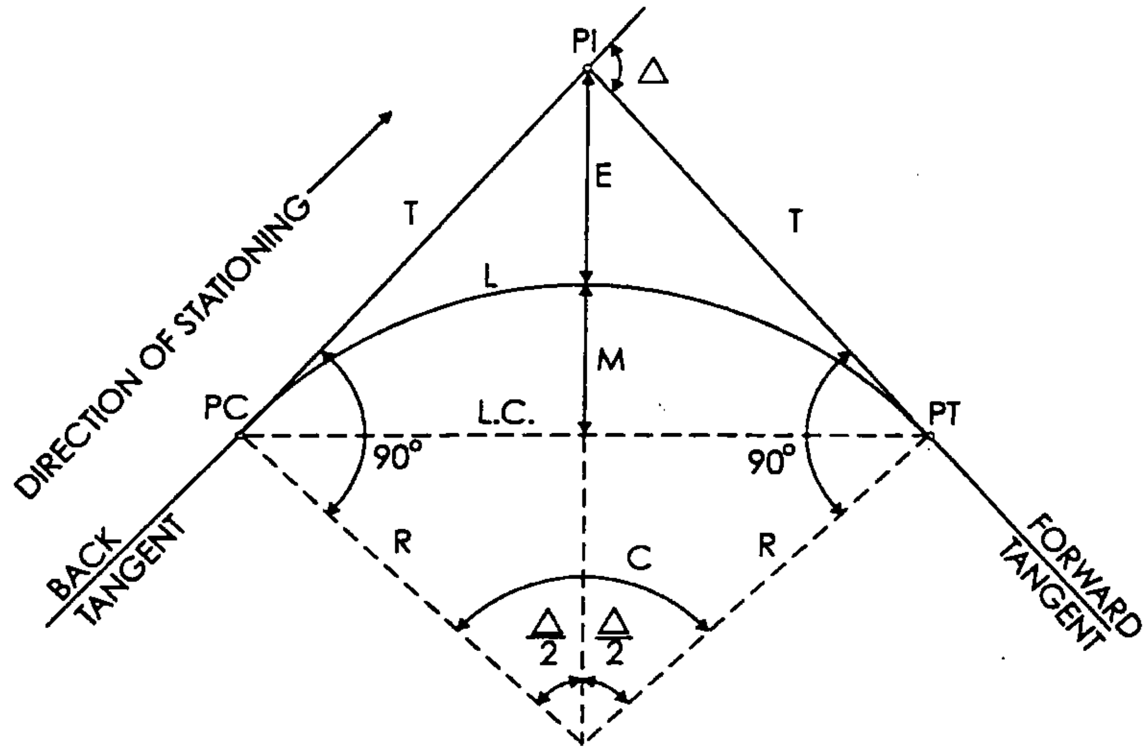
The purpose of spiral curves is to transition the driver into and out of circular curves without immediate departure from the tangent sections.

Spiral transition curves are used on the interstate system as well as certain major divided arterials and certain stage construction projects.

Values for spiral curve lengths are taken directly from the Tables 4-1, 4-2, and 4-3 in this chapter. The treatment of superelevation through simple curves and spiral curves is described later in this chapter.

Figure 4-5

SIMPLE CURVE DEFINITION



LEGEND

PI - POINT OF INTERSECTION
PC - POINT OF CURVATURE
PT - POINT OF TANGENCY
Δ - DEFLECTION ANGLE BETWEEN TANGENTS (DEGREES)
T - TANGENT DISTANCE
E - EXTERNAL DISTANCE
R - RADIUS OF THE CIRCULAR ARC
M - MIDDLE ORDINATE
LC - LONG CHORD
L - LENGTH OF CURVE
C - CENTRAL ANGLE (EQUAL TO Δ ANGLE)

Line Equations

Occasionally equations in stationing are introduced to avoid changing stationing throughout the project. The equation identifies two station numbers, one that is correct when measuring on the line back of the equation, and one that is correct when measuring ahead. Detour alignments and concentric alignments for divided highways are two examples where line equations may be used.

Concentric Curves on Divided Highways

When the median width of a divided highway is constant and narrow, the stationing and all other alignment computations can be based on a single alignment, normally along the center of the median. A common profile grade and one set of curve data serve for both roadways.

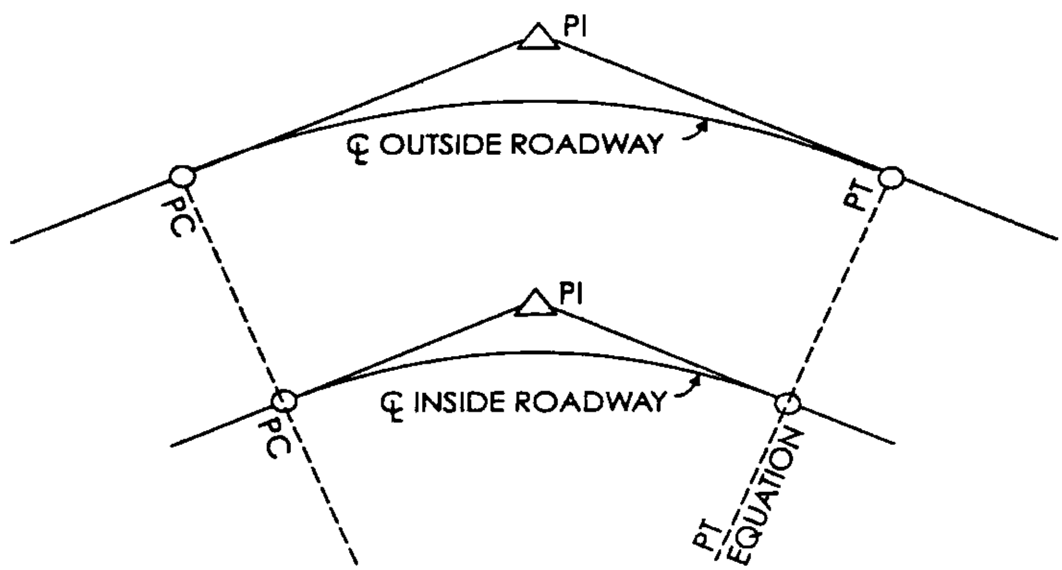
When the median width is variable and fairly wide, each roadway should have separate horizontal and vertical alignments.

Two problems arise when dealing with independent horizontal curve controls:

- The distance along the outside roadway is always longer than the distance along the inside roadway. In order to use parallel stationing on the tangents following the P.T., it is necessary to introduce an equation in stationing at the P.T. of one of the two curves.

Figure 4-6

CONCENTRIC ALIGNMENTS



- If the curves are to be concentric they must have a common center of radii and this means the radius of curvature of the outside roadway will be larger than the radius of the inside roadway. Each must be defined separately.

Concentric curves are illustrated in Figure 4-6. The deflection angle (Δ) and the P.C. stationing will be the same for the two curves. All other curve data will be different for each curve. The equation will reestablish common stationing ahead from the P.T.

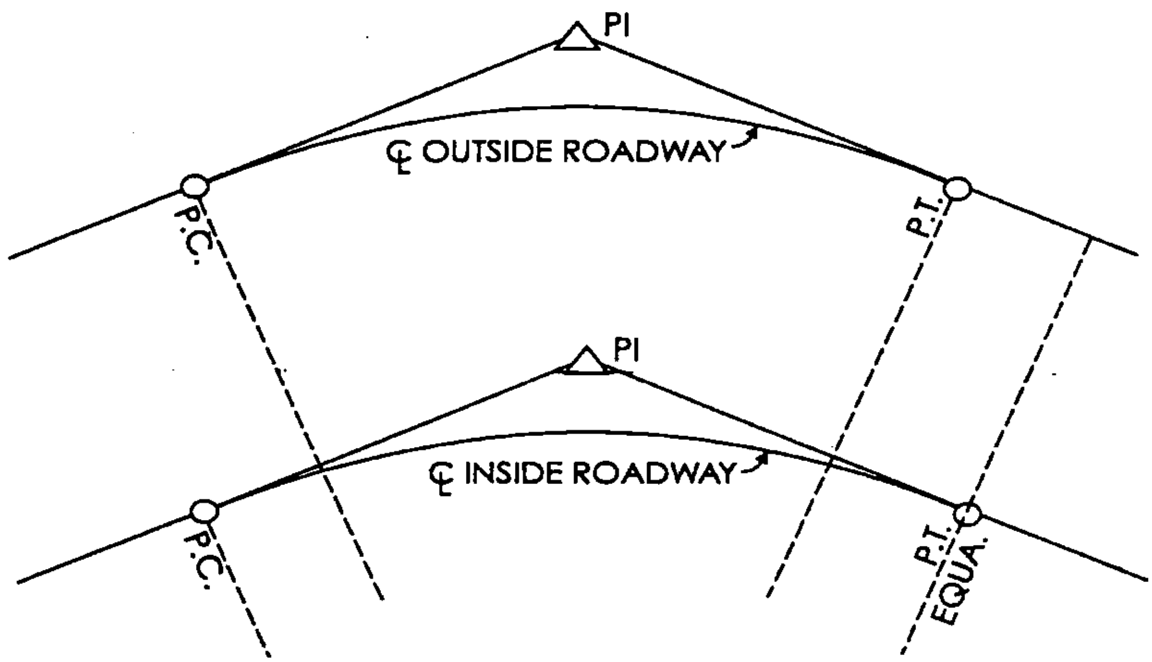
<u>Non-Concentric Curves on Divided Highways</u>

Another option is available for the situation described in the preceding section if it is not essential that the curves be precisely concentric. A reasonable approximation to concentric conditions can be attained if the radii for both roadways are the same. This situation is illustrated in Figure 4-7.

For this alternative, the basic curve data for the two curves will be identical, except that the stationing of the P.C., P.I., and P.T. will be different for each curve. The equation should be placed at the P.T. of the inside roadway to reestablish common stationing from that point ahead.

Figure 4-7

NON-CONCENTRIC ALIGNMENTS



Superelevation

When a vehicle moves in a circular path, it is forced radially outward by centrifugal force. The centrifugal force is counterbalanced by the vehicle mass component related to the roadway superelevation or the side friction developed between the tires and surface or by the combination of the two. If the vehicle is not skidding, these forces are in equilibrium as represented by the following equation:

$$(e/100) + f = 0.0079V^2/R = V^2/127R$$

Where: e = rate of roadway superelevation, percent;

f = side friction factor;

V = vehicle speed, km/h; and

R = radius of curve, m.

This formula is the basis for designing a curve for safe operation at a particular speed.

Superelevation <u>runoff</u> (L) is the length required to raise the pavement edge from the crown removed position (0 % cross slope) to full superelevation, or to lower the pavement edge from full superelevation to the crown removed position. Full superelevation is gradually attained by using a spiral transition curve, or in a simple curve by positioning the runoff length about the P.C. or P.T. Typically, 70% of the runoff length is positioned on the tangent. Note Figure 4-8. See the "Green Book" for additional guidance on positioning the transition for a simple curve.

Tables 4-1, 4-2, and 4-3, taken from the "Green Book", should be used to select appropriate superelevation rates and superelevation runoff lengths (L) for various radii and design speeds. The designer must also be aware of the specific recommendations in the project's Engineering Report before selecting superelevation rates.

On a purely empirical basis, minimum design superelevation runoff lengths for pavements wider than two lanes should be as follows:

- Three-lane pavements, 1.2 times the corresponding length for two-lane highways
- Four-lane undivided pavements, 1.5 times the corresponding length for two-lane highways
- Six-lane undivided pavements, 2.0 times the corresponding length for two-lane highways.
- Eight-lane undivided pavements, 2.5 times the corresponding length for two-lane highways

These factors assume the point of rotation is at the center of the pavement width.

TABLE 4-1

DESIGN ELEMENTS FOR HORIZONTAL CURVATURE

R	V_d = 30 km/h			V_d = 40 km/h			V_d = 50 km/h			V_d = 60 km/h			V_d = 70 km/h		
(m)	e (%)	L (m) 2 Lns	4 Lns	e (%)	L (m) 2 Lns	4 Lns	e (%)	L (m) 2 Lns	4 Lns	e (%)	L (m) 2 Lns	4 Lns	e (%)	L (m) 2 Lns	4 Lns
7000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
5000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
3000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
2500	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
2000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
1500	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	39	59
1400	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	39	59
1300	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	39	59
1200	NC	0	0	NC	0	0	NC	0	0	RC	33	50	RC	39	59
1000	NC	0	0	NC	0	0	NC	0	0	RC	33	50	2.2	39	59
900	NC	0	0	NC	0	0	RC	28	42	RC	33	50	2.4	39	59
800	NC	0	0	NC	0	0	RC	28	42	2.1	33	50	2.5	39	59
700	NC	0	0	NC	0	0	RC	28	42	2.3	33	50	2.7	39	59
600	NC	0	0	RC	22	33	2.1	28	42	2.5	33	50	2.9	39	59
500	NC	0	0	RC	22	33	2.3	28	42	2.7	33	50	3.1	39	59
400	NC	0	0	2.1	22	33	2.5	28	42	2.9	33	50	3.4	39	59
300	RC	17	26	2.4	22	33	2.8	28	42	3.3	33	50	3.8	39	59
250	RC	17	26	2.6	22	33	3.0	28	42	3.6	33	50	3.9	39	59
200	2.3	17	26	2.8	22	33	3.3	28	42	3.8	33	50	R_{min} = 215		
175	2.4	17	26	2.9	22	33	3.5	28	42	3.9	33	50			
150	2.5	17	26	3.1	22	33	3.7	28	42	4.0	33	50			
140	2.6	17	26	3.2	22	33	3.8	28	42	R_{min} = 150					
130	2.6	17	26	3.3	22	33	3.8	28	42						
120	2.7	17	26	3.4	22	33	3.9	28	42						
110	2.8	17	26	3.5	22	33	4.0	28	42						
100	2.9	17	26	3.6	22	33	4.0	28	42						
90	3.0	17	26	3.7	22	33	R_{min} = 100								
80	3.2	17	26	3.8	22	33									
70	3.3	17	26	3.9	22	33									
60	3.5	17	26	4.0	22	33									
50	3.7	18	27	R_{min} = 60.											
40	3.9	19	28												
	R_{min} = 35														

R	V_d = 80 km/h			V_d = 90 km/h			V_d = 100 km/h			V_d = 110 km/h			V_d = 120 km/h		
(m)	e (%)	L (m) 2 Lns	4 Lns	e (%)	L (m) 2 Lns	4 Lns	e (%)	L (m) 2 Lns	4 Lns	e (%)	L (m) 2 Lns	4 Lns	e (%)	L (m) 2 Lns	4 Lns
7000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
5000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
3000	NC	0	0	NC	0	0	RC	56	84	RC	61	92	2.1	67	101
2500	NC	0	0	RC	50	75	RC	56	84	2.1	61	92	2.4	67	101
2000	RC	44	66	RC	50	75	2.2	56	84	2.4	61	92	2.8	67	101
1500	RC	44	66	2.3	50	75	2.6	56	84	2.9	61	92	3.3	67	101
1400	2.1	44	66	2.4	50	75	2.7	56	84	3.0	61	92	3.5	67	101
1300	2.2	44	66	2.5	50	75	2.8	56	84	3.1	61	92	3.6	67	101
1200	2.3	44	66	2.6	50	75	2.9	56	84	3.3	61	92	3.7	67	101
1000	2.5	44	66	2.8	50	75	3.2	56	84	3.6	61	92	3.9	67	101
900	2.7	44	66	3.0	50	75	3.4	56	84	3.7	61	92	4.0	67	101
800	2.8	44	66	3.2	50	75	3.5	56	84	3.9	61	92	R_{min} = 870		
700	3.0	44	66	3.4	50	75	3.7	56	84	4.0	61	92			
600	3.2	44	66	3.6	50	75	3.9	56	84	R_{min} = 635					
500	3.5	44	66	3.8	50	75	4.0	56	84						
400	3.7	44	66	4.0	50	75	R_{min} = 490								
300	4.0	44	66	R_{min} = 375											
250	R_{min} = 280														

e_{max} = 4.0 % **URBAN**

R = radius of curve
V = assumed design speed
e = rate of superelevation
L = minimum length of runoff (does not include tangent runout)
NC = normal crown
RC = remove adverse crown, superelevate at normal crown slope
Note: Lengths rounded in multiples of 10 m permit simpler calculations.

TABLE 4-2

DESIGN ELEMENTS FOR HORIZONTAL CURVATURE

R (m)	V_d = 30 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 40 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 50 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 60 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns
7000	NC	0	0	NC	0	0	NC	0	0	NC	0	0
5000	NC	0	0	NC	0	0	NC	0	0	NC	0	0
3000	NC	0	0	NC	0	0	NC	0	0	NC	0	0
2500	NC	0	0	NC	0	0	NC	0	0	NC	0	0
2000	NC	0	0	NC	0	0	NC	0	0	NC	0	0
1500	NC	0	0	NC	0	0	NC	0	0	NC	0	0
1400	NC	0	0	NC	0	0	NC	0	0	NC	0	0
1300	NC	0	0	NC	0	0	NC	0	0	RC	33	50
1200	NC	0	0	NC	0	0	NC	0	0	RC	33	50
1000	NC	0	0	NC	0	0	RC	28	42	2.1	33	50
900	NC	0	0	NC	0	0	RC	28	42	2.3	33	50
800	NC	0	0	NC	0	0	RC	28	42	2.5	33	50
700	NC	0	0	RC	22	33	2.1	28	42	2.8	33	50
600	NC	0	0	RC	22	33	2.4	28	42	3.1	33	50
500	NC	0	0	2.1	22	33	2.8	28	42	3.5	33	50
400	RC	17	26	2.5	22	33	3.3	28	42	4.0	33	50
300	RC	17	26	3.1	22	33	3.9	28	42	4.6	33	50
250	2.3	17	26	3.5	22	33	4.2	28	42	5.0	33	50
200	2.8	17	26	3.9	22	33	4.7	28	42	5.5	33	50
175	3.0	17	26	4.1	22	33	5.0	28	42	5.8	35	52
150	3.3	17	26	4.4	23	34	5.3	29	43	5.9	35	53
140	3.5	17	26	4.5	23	35	5.4	29	44	6.0	36	54
130	3.6	17	26	4.6	24	36	5.6	30	45	R_{min} = 135		
120	3.8	18	27	4.8	25	37	5.7	31	46			
110	3.9	19	28	5.0	26	39	5.8	31	47			
100	4.1	20	29	5.2	27	40	5.9	32	48			
90	4.2	20	30	5.4	28	42	6.0	32	49			
80	4.5	22	32	5.6	29	43	R_{min} = 90					
70	4.7	23	34	5.8	30	45						
60	5.0	24	36	6.0	31	46						
50	5.4	26	39	R_{min} = 55								
40	5.8	28	42									
30	6.0	29	43									
	R_{min} = 30											

R (m)	V_d = 70 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 80 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 90 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 100 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 110 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 120 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns
7000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
5000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
3000	NC	0	0	NC	0	0	NC	0	0	RC	56	84	RC	61	92	2.3	67	101
2500	NC	0	0	NC	0	0	RC	50	75	RC	56	84	2.3	61	92	2.7	67	101
2000	NC	0	0	RC	44	66	2.1	50	75	2.5	56	84	2.8	61	92	3.3	67	101
1500	RC	39	59	2.2	44	66	2.7	50	75	3.1	56	84	3.6	61	92	4.2	67	101
1400	RC	39	59	2.4	44	66	2.8	50	75	3.3	56	84	3.8	61	92	4.4	67	101
1300	2.1	39	59	2.5	44	66	3.0	50	75	3.5	56	84	4.0	61	92	4.7	67	101
1200	2.2	39	59	2.7	44	66	3.2	50	75	3.7	56	84	4.2	61	92	5.0	67	101
1000	2.6	39	59	3.1	44	66	3.6	50	75	4.2	56	84	4.8	61	92	5.5	67	101
900	2.8	39	59	3.4	44	66	3.9	50	75	4.5	56	84	5.1	61	92	5.8	67	101
800	3.1	39	59	3.6	44	66	4.2	50	75	4.9	56	84	5.4	61	92	6.0	67	101
700	3.4	39	59	4.0	44	66	4.6	50	75	5.2	56	84	5.7	61	92	R_{min} = 755		
600	3.8	39	59	4.3	44	66	5.0	50	75	5.6	56	84	6.0	61	92			
500	4.2	39	59	4.8	44	66	5.4	50	75	5.9	56	84	R_{min} = 560					
400	4.7	39	59	5.3	44	66	5.9	50	75	R_{min} = 435								
300	5.4	39	59	5.9	44	66	R_{min} = 335											
250	5.7	39	59	6.0	44	66												
200	6.0	39	59	R_{min} = 250														
175	R_{min} = 195																	

e_{max} = 6.0 %

RAMPS

R = radius of curve
V = assumed design speed
e = rate of superelevation
L = minimum length of runoff (does not include tangent runout)
NC = normal crown
RC = remove adverse crown, superelevate at normal crown slope
Note: Lengths rounded in multiples of 10 m permit simpler calculations.

TABLE 4-3

DESIGN ELEMENTS FOR HORIZONTAL CURVATURE

R (m)	V_d = 30 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 40 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 50 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 60 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 70 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns
7000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
5000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
3000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
2500	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
2000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
1500	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	39	59
1400	NC	0	0	NC	0	0	NC	0	0	RC	33	50	2.1	39	59
1300	NC	0	0	NC	0	0	NC	0	0	RC	33	50	2.2	39	59
1200	NC	0	0	NC	0	0	NC	0	0	RC	33	50	2.4	39	59
1000	NC	0	0	NC	0	0	RC	28	42	2.2	33	50	2.8	39	59
900	NC	0	0	NC	0	0	RC	28	42	2.4	33	50	3.1	39	59
800	NC	0	0	NC	0	0	RC	28	42	2.7	33	50	3.4	39	59
700	NC	0	0	RC	22	33	2.2	28	42	3.0	33	50	3.8	39	59
600	NC	0	0	RC	22	33	2.6	28	42	3.4	33	50	4.3	39	59
500	NC	0	0	2.2	22	33	3.0	28	42	3.9	33	50	4.9	39	59
400	RC	17	26	2.7	22	33	3.6	28	42	4.7	33	50	5.7	39	59
300	2.1	17	26	3.4	22	33	4.5	28	42	5.6	34	51	6.7	44	66
250	2.5	17	26	4.0	22	33	5.1	28	42	6.2	37	56	7.3	48	72
200	3.0	17	26	4.6	24	36	5.8	31	47	7.0	42	63	7.9	52	78
175	3.4	17	26	5.0	26	39	6.2	33	50	7.4	44	67	8.0	52	79
150	3.8	18	27	5.4	28	42	6.7	36	54	7.8	47	70	R_{min} = 175		
140	4.0	19	29	5.6	29	43	6.9	37	56	7.9	47	71			
130	4.2	20	30	5.8	30	45	7.1	38	58	8.0	48	72			
120	4.4	21	32	6.0	31	46	7.3	39	59	R_{min} = 125					
110	4.7	23	34	6.3	32	49	7.6	41	62						
100	4.9	23	35	6.5	33	50	7.8	42	63						
90	5.2	25	37	6.9	36	53	7.9	43	64						
80	5.5	26	40	7.2	37	56	8.0	43	65						
70	5.9	28	42	7.5	39	58	R_{min} = 80								
60	6.4	31	46	7.8	40	60									
50	6.9	33	50	8.0	41	62									
40	7.5	36	54	R_{min} = 50											
30	8.0	38	57												
	R_{min} = 30														

R (m)	V_d = 80 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 90 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 100 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 110 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns	V_d = 120 km/h e (%)	L (m) 2 Lns	L (m) 4 Lns
7000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
5000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
3000	NC	0	0	NC	0	0	RC	56	84	2.1	61	92	2.4	67	101
2500	NC	0	0	RC	50	75	2.1	56	84	2.4	61	92	2.9	67	101
2000	RC	44	66	2.2	50	75	2.6	56	84	3.0	61	92	3.5	67	101
1500	2.4	44	66	2.8	50	75	3.4	56	84	3.9	61	92	4.6	67	101
1400	2.5	44	66	3.0	50	75	3.6	56	84	4.1	61	92	4.9	67	101
1300	2.7	44	66	3.2	50	75	3.8	56	84	4.4	61	92	5.2	67	101
1200	2.9	44	66	3.4	50	75	4.1	56	84	4.7	61	92	5.6	67	101
1000	3.4	44	66	4.0	50	75	4.8	56	84	5.5	61	92	6.5	67	101
900	3.7	44	66	4.4	50	75	5.2	56	84	6.0	61	92	7.1	67	101
800	4.1	44	66	4.8	50	75	5.7	56	84	6.5	61	92	7.6	68	103
700	4.5	44	66	5.3	50	75	6.3	56	84	7.2	62	93	8.0	72	108
600	5.1	44	66	6.0	50	75	6.9	56	84	7.7	66	99	R_{min} = 655		
500	5.8	44	66	6.7	51	76	7.6	61	91	8.0	69	103			
400	6.6	48	71	7.5	57	85	8.0	64	96	R_{min} = 500					
300	7.6	55	82	R_{min} = 305			R_{min} = 395								
250	7.9	57	85												
200	R_{min} = 230														

e_{max} = 8.0 %

OPEN HIGHWAY CONDITIONS

R = radius of curve

= assumed design speed

e = rate of superelevation

L = minimum length of runoff (does not include tangent runout)

NC = normal crown

RC = remove adverse crown, superelevate at normal crown slope

Note: Lengths rounded in multiples of 10 m permit simpler calculations.

Sprials not required above heavy lines.

It is important to note that four-lane values are required for divided highways (two lanes in each direction) regardless of median width because the pavement is revolved about the median traveled way.

The four-lane lengths shown in the tables are determined on this empirical basis. Proper design attention is needed to obtain smooth-edge profiles and to avoid distorted appearances. To meet the requirements of comfort and safety, superelevation should be introduced and removed uniformly over a length adequate for the expected operating speeds. To be pleasing in appearance, the pavement edges should not be distorted as the driver sees them during the transition.

Runoff lengths for divided pavements may require greater lengths than shown on the tables. To ensure that superelevation properly transitions from normal crown to full bank, a graphic representation is performed showing the edge line plot. If necessary, adjustments can then be made to correct abnormalities. This plot should be done on 1:250 or 1:500 horizontal scale and a 1:10 vertical scale.

The L values given in Tables 4-1, 4-2, and 4-3 are the desired minimum superelevation runoff lengths, however, shorter lengths may be acceptable in constrained conditions if approved by the Section Head. An absolute minimum runoff length can be calculated using the maximum relative gradient controls given in Table 4-4. For example:

Given: Design Speed = 80 km/h, R = 1000 m, e(max) = 8.0 %, two-lane roadway, 3.6 m lanes

Then: Design e = 3.4 %, L = 44 m (from Table 4-3)

Using the maximum turnover rate from Table 4-4 gives:

L = (change in cross slope) x (pavement width being rotated) x (maximum relative slope ratio)

L = (0.034) x (3.6 m) x (200)

L = 25 m

So the desired runoff length is 44 m and the absolute minimum runoff length is 25 m.

It is particularly important to check the edge line profiles graphically if the maximum turnover rate is used to insure a smooth transition.

Tangent <u>runout</u> is the length necessary to remove or restore normal pavement crown when the alignment enters or leaves a superelevated curve or spiral. In other words, it is the length required to go from normal crown to the crown removed position (0 % cross slope). The transition rate for tangent runout should be the same rate as that used in the superelevation runoff.

Figures 4-9 and 4-10 show a pavement edge view which graphically illustrates how superelevation transition is achieved. Note that three positions of rotation are shown for each of the two methods of superelevating pavements.

Although there are advantages to rotating about either edge, particularly for multi-lane pavements, two-lane, two-way pavements are rotated about the center line. Interstate highway pavements are typically rotated about the median traveled way for aesthetic reasons. The "Green Book" should be consulted when multi-lane pavements are to be superelevated. The Section Head should be consulted if there is any question on the proper point of rotation.

Table 4-4

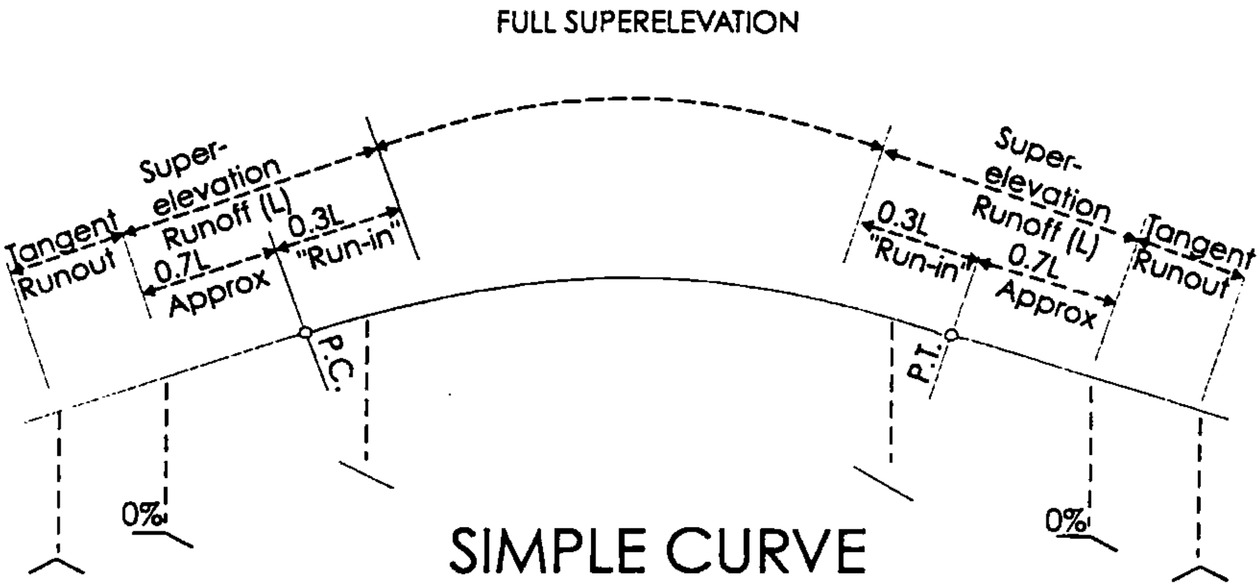
RELATIONSHIP OF DESIGN SPEED TO MAXIMUM RELATIVE PROFILE GRADIENTS

Design Speed V_D (km/h)	Maximum Relative Gradients (and Equivalent Maximum Relative Slopes) for Profiles Between the Edge of Two-Lane Traveled Way and the Centerline (%)
30	0.75 (1:133)
40	0.70 (1:143)
50	0.65 (1:150)
60	0.60 (1:167)
70	0.55 (1:182)
80	0.50 (1:200)
90	0.48 (1:210)
100	0.45 (1:222)
110	0.42 (1:238)
120	0.40 (1:250)

An EXCEL spreadsheet (Figure 4-11) is available to assist the designer with routine superelevation calculations. The spreadsheet is only a tool, full understanding of the preceeding discussion on superelevation is essential to produce meaningful results. The shaded boxes denote required input data.

Figure 4-8

SUPERELEVATION TRANSITION DIAGRAM



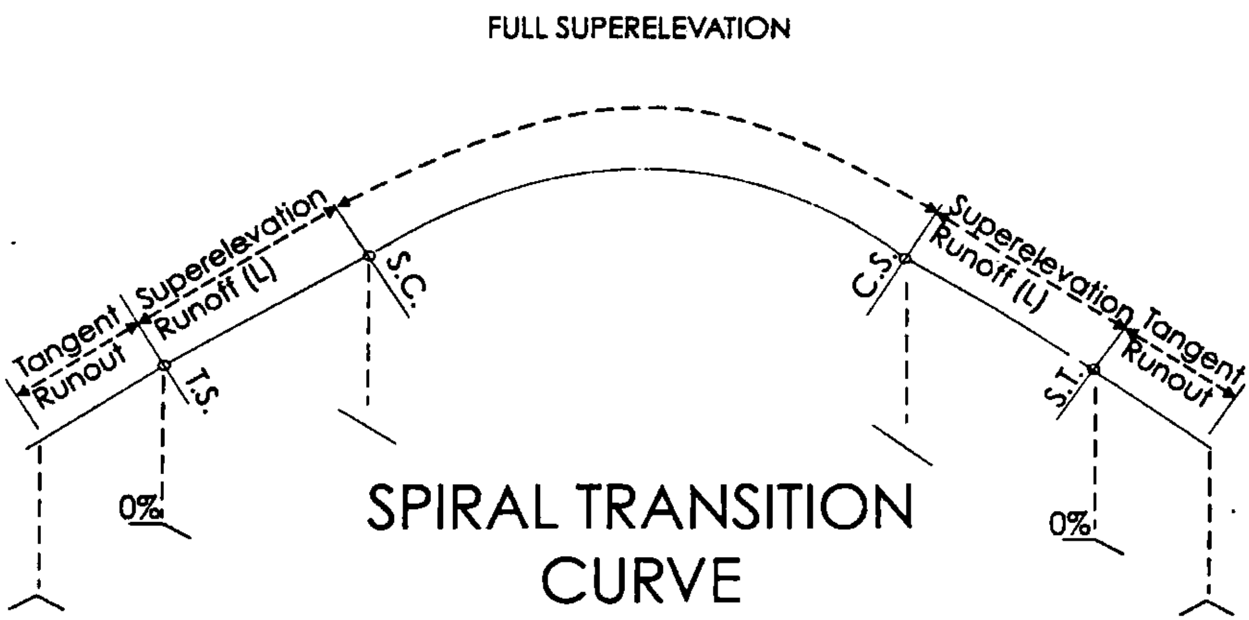


Figure 4-9

SPIRAL CURVE SUPERELEVATION TRANSITIONS

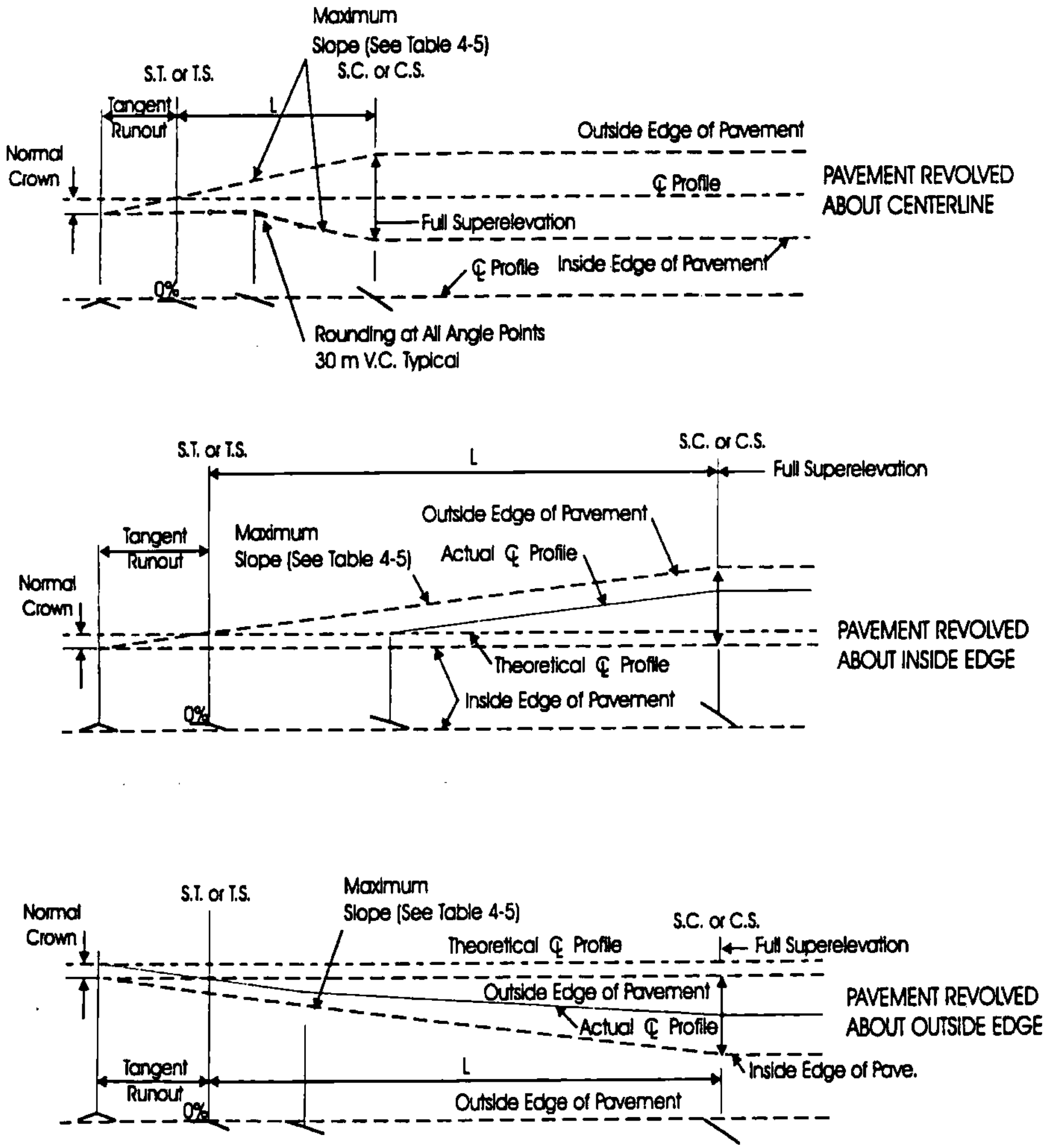
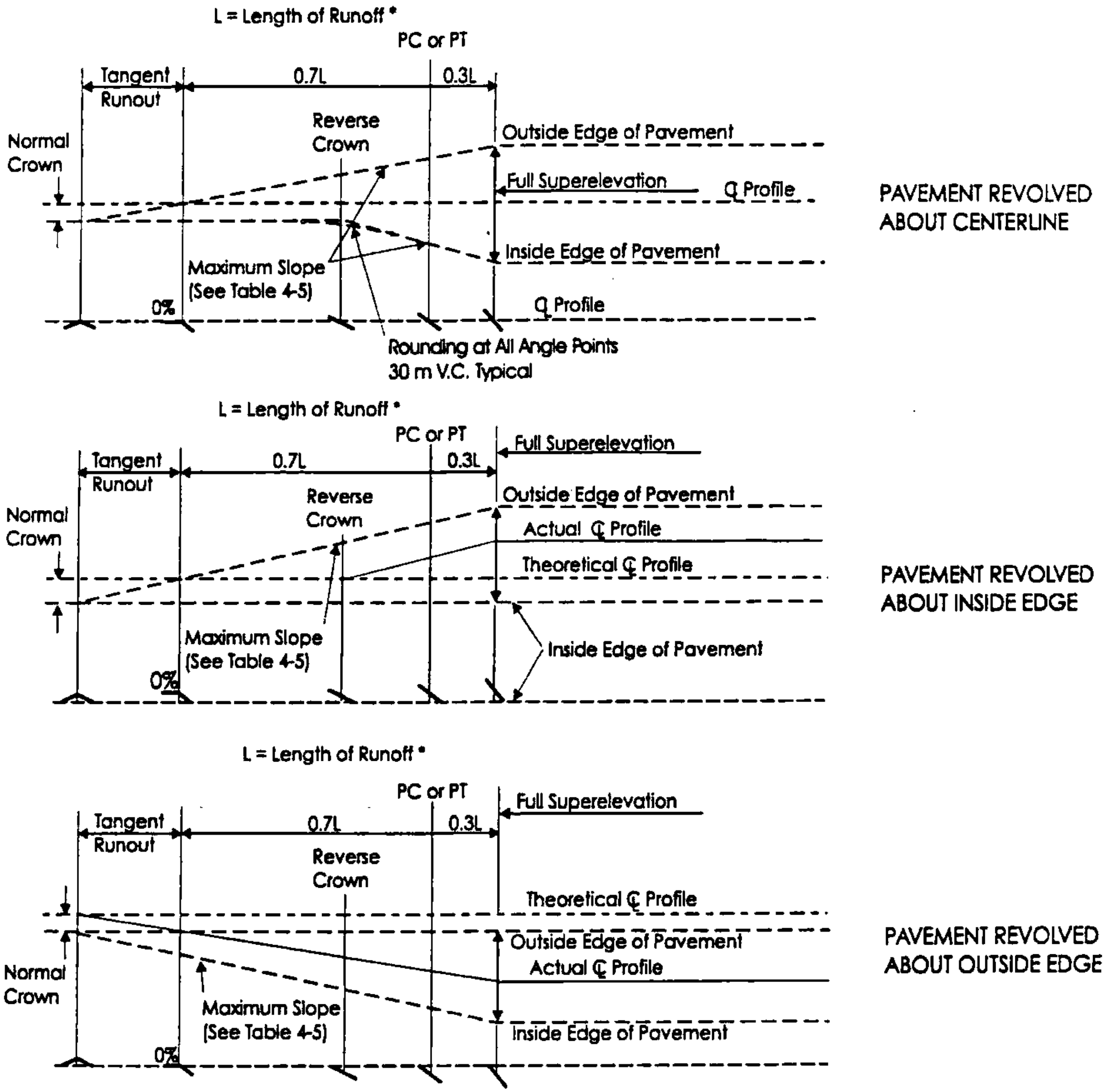


Figure 4-10

SIMPLE CURVE SUPERELEVATION TRANSITIONS



* A 70% - 30% split about the P.C. or P.T. is the normal distribution.
See the "Green Book" (8) for additional guidance.

Figure 4-11

SUPERELEVATION DATA
(FOR SIMPLE CURVES, NO SPIRALS)

PROJECT

NAME: ____________________
FEDERAL NO.: ____________________
STATE NO.: ____________________
ROAD NAME: ____________________

CURVE NO. ____________________
e_{max} = ____________
P.I. = ____________________
P.C = ____________________
P.T = ____________________

For use as spreadsheet, input information in shaded cells only.

R =	____________ Curve Radius (meters)	
V_{des} =	____________ Design Speed (km/h)	
e =	____________ Full Bank Cross-slope (%)	
L-IN =	____________ Superelevation Runoff Length (m)	L-OUT = ____________ Superelevation Runoff Length (m)
RUN-IN (in) =	____________ Dist. From P.C. to F.B. = (0.3L)	RUN-IN (out) = ____________ Dist. From P.T. to F.B. = (0.3L)
TAN RUN-OUT (in) =	____________ Dist. from N.C. to "0" = 2(L/e)	TAN RUN-OUT (out) = ____________ Dist. from N.C. to "0" = 2(L/e)
LANE WIDTH-IN (W-in) =	INSIDE OUTSIDE ____________ Width of Rotated Pavement (meters)	LANE WIDTH-OUT (W-out) = INSIDE OUTSIDE ____________ Width of Rotated Pavement (meters)

	STATION	INSIDE (mm)	C L	OUTSIDE (mm)
N.C. (in) =	Normal Crown="0"-Tan. Runout	(-2*W-in)10		(-2*W-in)10
"0"-sect. (in) =	N.C. Removed=(F.B.-L)	(-2*W-in)10		0 0
R.C. (in) =	Reverse Crown = "0"+2(L/e)	(-2*W-in)10		(2*W-in)10
P.C. =	Point of Curvature	(-0.70*e*W-in)10		(0.70*e*W-in)10
F.B. (in) =	P.C. + Run-in (in)	-(e*W-in)10		(e*W-in)10
F.B. (out) =	P.T. - Run-in (out)	-(e*W-out)10		(e*W-out)10
P.T. =	Point of Tangency	-(0.70)(e*W-out))10		(0 70*e*W-out)10
R.C. (out) =	Reverse Crown = "0"-2(L/e)	(-2*W-out)10		(2*W-out)10
"0"-sect. (out) =	N.C. Removed (F.B.+L)	(-2*W-out)10		0 0
N.C. (out) =	Normal Crown="0"+Tan. Runout	(-2*W-out)10		(-2*W-out)10

Vertical Alignment

The proper design of grade depends upon the choice of vertical curves which connect straight grades as well as the straight grade characteristics. Vertical curves are discussed later in this chapter. Desirable grade criteria is discussed next.

General Controls

General controls for grades which are to be observed:

- Attempt to design a smooth grade with no abrupt changes.
- The "roller-coaster" (undulating) or the "hidden-dip" type of profile should be avoided. Such profiles generally occur on relatively straight horizontal alignment where the roadway profile closely follows a rolling natural ground line. Hidden dips can hide vehicles from view, causing safety concerns for passing maneuvers. This type of profile is avoided by use of horizontal curves or by more gradual grades.
- Undulating grade lines, involving substantial lengths of down grades, should be avoided. Such profiles may encourage trucks to operate at higher speeds than desirable.
- A broken-back line (two vertical curves in the same direction separated by short sections of tangent grade) generally should be avoided. This situation is particularly unpleasant aesthetically on divided highways with open medians.
- On long upgrades it may be preferable to place the steepest grades at the bottom and flatten the grades near the top of the ascent or to break the sustained 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 design speed roads and streets.
- Gradient through intersections should be as smooth and flat as drainage and topographic conditions will permit.
- Sag vertical curves should be avoided in cuts unless adequate drainage can be provided.

Vehicle Operating Characteristics

Vehicle-operating characteristics on grades are the major factor affecting design. Research recognized by the "Green Book" regarding vehicle speed/grade relationships shows:

- Passenger cars show no speed loss at 3 percent upgrades and no appreciable loss in speed on 7 to 8 percent upgrades. Down grade speed is slightly higher than the average speed.
- Average speed studies for trucks show an increase in speed of about 5 percent on downgrades and about a 7 percent or more decrease in speed on upgrades.

The mass/power ratio is the standard evaluation for truck performance. The mass/power ratio of 180 kg/kW is normally used to determine critical lengths of grade.

- Recreation vehicles are affected by grade but unless there are many, additional climbing lanes will not be needed.

Maximum Grades

Maximum grades are dependent on design speed, highway functional class, and type of terrain. Refer to the appropriate functional class chapter in the "Green Book" for maximum values.

Minimum Grades

Flat tangent grades (less than 0.4%) on uncurbed pavements are permissible (although not desirable) when the pavement is adequately crowned to drain the surface laterally and side ditch drainage is adequately provided. With curbed pavements, longitudinal grades should be provided to facilitate surface drainage. The minimum grade is 0.4 percent where curbing is used. Use of flatter grades (less than 0.4% curbed and uncurbed) must be approved by the Highway Design Administrator. Particular attention should be given to the design of storm-water inlets and their spacing to keep the spread of water on traveled way within tolerable limits. Roadside channels and median swales frequently require grades steeper than the roadway profile for adequate drainage. Drainage is discussed in Chapter 6.

Climbing Lanes

Climbing lanes offer a comparatively inexpensive means of overcoming losses in capacity and provide improved operation where congestion on grades is caused by slow trucks in combination with high traffic volumes. A two-lane highway may be adequate with climbing lanes whereas a much more costly multi-lane highway would be necessary without them.

The term "critical length of grade" is used to indicate the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed. The "Green Book" suggests a 15 km/h reduction as being reasonable and on this basis, the critical length of an upgrade when approached by a level or nearly level section of roadway is shown in Table 4-5. For a given grade, lengths less than critical ones result in acceptable operation in the desired range of speeds. If the desired freedom of operation is to be maintained on grades longer than critical ones, capacity analysis should be made and additional lanes should be considered.

The maximum grade in itself is not a complete design control. See the "Green Book" for more detailed graphs on truck speed characteristics.

Upgrade, Percent	3	4	5	6	7	8
Critical Length of Upgrade (meters)	400	275	215	180	160	135

Table 4-5

CRITICAL LENGTH OF GRADE

The above critical lengths of grade are based on a heavy truck with a 180 kg/kW mass/power ratio. Some studies have indicated that mass/power ratios have been decreasing over the years, resulting in trucks with greater power and better climbing ability on upgrades. This means the values given in Table 4-5 may be shorter than the distances actually required to affect a 15 km/h speed reduction. The FHWA publication *New Methods for Determining Requirements for Truck-Climbing Lanes* (18) provides some additional guidance on critical length of grade and climbing lane warrants based on more recent mass/power ratio studies.

On moderate- to high-volume roads, where critical lengths of up-grade are substantially exceeded, consideration should be given to providing climbing lanes, particularly where truck volume is high. If a truck lane can be easily constructed, it may be proposed without meeting warrants, if convenience and safety are increased. Otherwise the conditions described below must be considered.

There are so many variables involved that hardly any given set of conditions can be described as "typical". If there is reason to believe a climbing lane may be justified, a detailed capacity analysis should be made. A truck speed simulation program is available through the Bureau of Information Technology Services, however, it is somewhat outdated and the results should be used only as a guide. The program uses English units.

Several charts in the "Green Book", Figure III-25(A),(B),(C), and Figure III- 29, show the characteristics of speed/grade relationships for trucks. Volume is not considered, however, and a capacity analysis is needed if the climbing lane warrants are to be substantiated. The charts are useful in preliminary design where grades may be adjusted to minimize the length of critical grade.

The following three conditions and criteria, reflecting economic considerations, should be satisfied to justify a climbing lane:

1. Upgrade traffic flow rate in excess of 200 vehicles per hour.
2. Upgrade truck flow rate in excess of 20 vehicles per hour.
3. One of the following conditions exist:
   - A 15 km/h or greater speed reduction is expected for a typical heavy truck.
   - Level-of-service E or F exist on the grade.

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

Climbing lanes, when warranted, should be 3.6 m wide. The cross slope should be a continuation (on tangent sections) of the adjacent traffic lane. On superelevated sections, it is generally necessary to limit the cross slope of climbing lanes to 4 percent maximum, however, drainage and maintenance (snow plowing) issues must be considered. An important consideration in designing the cross slope of any climbing lane is selecting a superelevation which is not hazardous to slow-moving traffic during icing conditions. In all cases where the combination of horizontal curvature and design speed require superelevation rates in excess of 4 percent, the designer must consider alternatives such as reducing the curvature or reducing the posted speed. The shoulder width adjacent to climbing lanes is normally 1.2 m.

Climbing lanes should begin at a point where trucks will have slowed to 15-25 km/h below the posted speed limit. The beginning of the lane should be preceded by a tapered section at least 60 m long (or 25:1 desirable). The climbing lane should extend to a point where a typical truck can attain a speed that is within 15 km/h of the posted speed limit. The desirable lane drop taper is 50:1 or flatter.

Emergency Escape Ramps

Long down-grades which lead to a curve, congested area, or intersection may warrant an escape ramp. The purpose of the ramp is to provide a safe emergency deceleration area for runaway trucks.

The essentials of the escape ramp are:

- the entrance must be obvious and accessible to a vehicle moving up to 140 km/h;
- the entrance must be at a location on the down-grade far enough from the hill crest to allow detection of failure, decision and reaction time to use the escape ramp;
- a sag vertical curve into the deceleration trap should not be abrupt;
- tow trucks must be provided with a way to retrieve trapped vehicles; and
- the deceleration trap must be positive but gradual in its entrapment.

Warrants for the construction of escape ramps are based on judgment and careful consideration. If the decision is made that an escape ramp should be considered, refer to the "Green Book", Chapter III, and perform a detailed analysis. Other mechanical devices may be available to help stop runaway vehicles.

Vertical Curves

General

Vertical curves are used to effect gradual change between tangent grades. They may be any one of the crest or sag types shown in Figure 4-12. Vertical curves should be simple in application and should result in a design that is safe, comfortable in operation, pleasing in appearance, and adequate for drainage. The major control for safe operation on crest vertical curves is the provision of ample sight distances for the design speed. Minimum stopping sight distance should be provided in all cases. Wherever practical, greater stopping sight distances should be used. Additional sight distance should be provided at decision points.

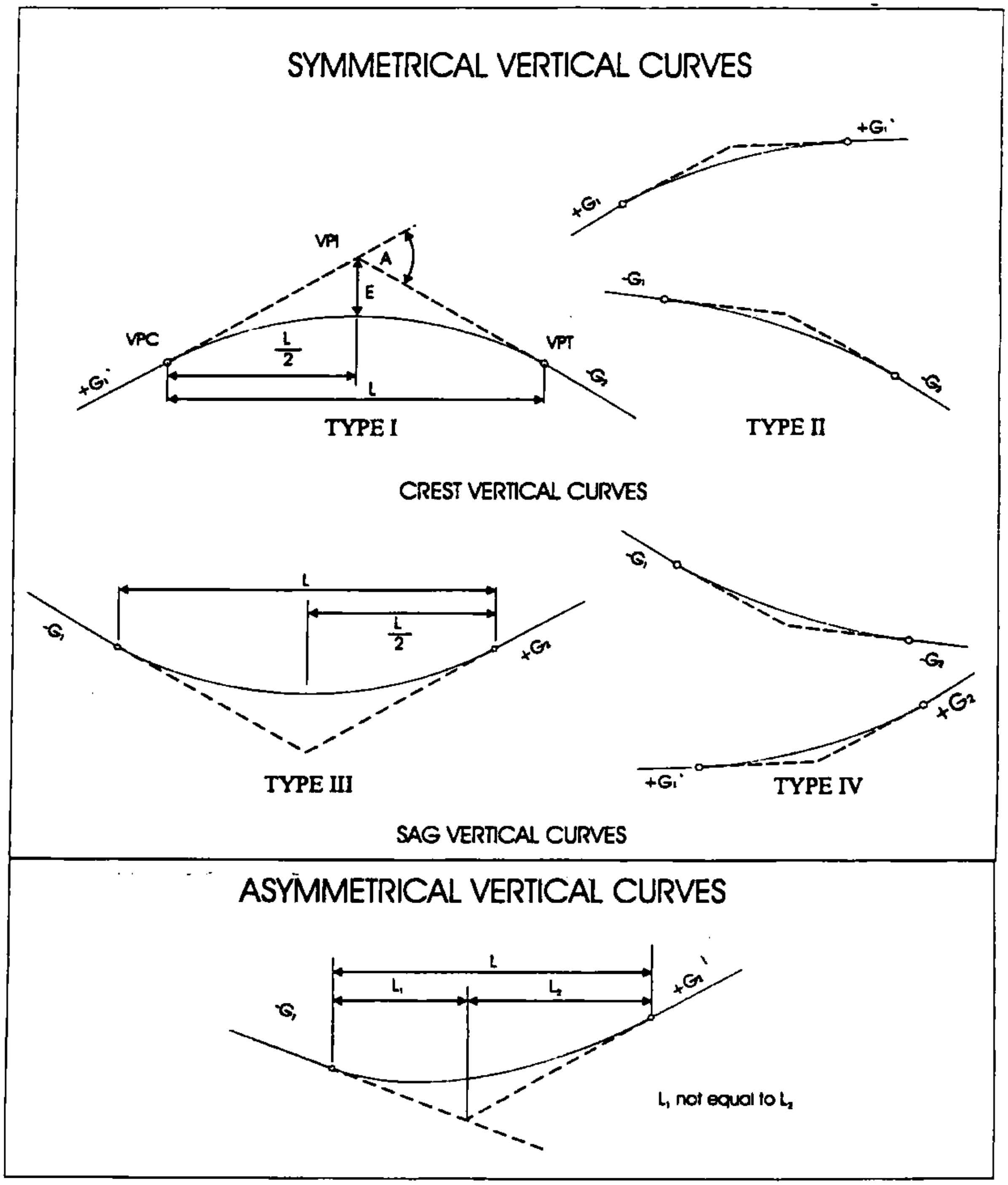
Consideration of motorists' comfort requires that the rate of change of grade be kept within tolerable limits. This consideration is most important in sag vertical curves where gravitational and vertical centrifugal forces act in the same direction. A long curve also has a more pleasing appearance than a short one.

For simplicity, the symmetrical parabolic curve with an equivalent vertical axis centered on the vertical point of intersection (VPI) is usually used in roadway profile design. The vertical offsets from the tangent vary as the square of the horizontal distance from the curve end (point of tangency). The vertical offset from the tangent grade at any point along the curve is calculated as a proportion of the vertical offset at the VPI (See Figure 4-13). The rate of change of grade at successive points on the curve is a constant amount for equal increments of horizontal distance, and equals the algebraic difference between intersecting tangent grades (in percent) divided by the length of curve in meters or A/L in percent per meter. The reciprocal, L/A, is the horizontal distance in meters required to effect a one-percent change in gradient and is, therefore, a measure of curvature. The ratio, L/A, termed "K," is useful in determining the horizontal distance from the vertical point of curvature (VPC) to the apex of Type I curves or to the low point of Type III curves. This point where the slope is zero occurs at a distance from the VPC equal to K times the approach gradient. The K value, discussed later in this chapter, is also useful in determining minimum lengths of vertical curves for various design speeds.

On certain occasions, due to critical clearance or other controls, the use of asymmetrical vertical curves may be required. An example is shown in Figure 4-12. Because the conditions dictating the need for these curves are infrequent, the derivation and use of the appropriate formulas have not been included here. For use in such limited instances, refer to asymmetrical curve computations found in standard highway engineering texts.

Figure 4-12

TYPES OF VERTICAL CURVES



G_1 AND G_2 = TANGENT GRADES (PERCENT)
A = ALGEBRAIC DIFFERENCE IN GRADES
L = LENGTH OF VERTICAL CURVE

Curve Computations

After the tangent grades have been established, appropriate vertical curves must be designed for each VPI. Standards and criteria for sight distance and minimum curvature should be adhered to.

Conceptual level design can use a graphically derived profile. The profile is mathematized as the design is refined and plans for a Public Hearing are prepared. The preliminary designer will select a vertical curve of adequate K value, graphically establish the midpoint and draft the curve as described later in this chapter. Matching of control points can usually be verified from this graphic line, however, when control is critical, the curve should be mathematically computed.

The diagram in Figure 4-13, illustrates a crest vertical curve. The formulas are also applicable to a sag vertical curve. It is important to recognize the plus or minus characteristics of the grades when computing algebraic differences and whether the offset distances are to be added to or subtracted from the tangent elevations.

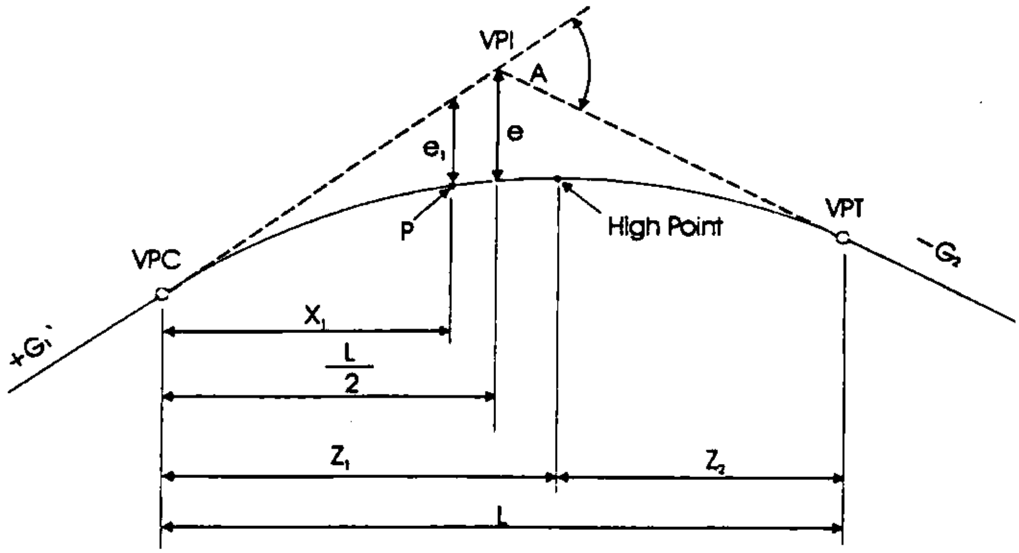
Vertical curves can be stored in CAD/D once the VPI stations and curve lengths have been established. Reports can be generated showing the elevation at each point along the alignment, VPC and VPT stations, and high or low point of the curve.

For manual calculations use the following procedure:

- Compute the minimum curve length (L) which will satisfy the sight distance requirements for the design speed (the product of the algebraic difference in grades and the appropriate value for K). Refer to Figures 4-14 or 4-15 for the minimum K value. Table 4-6 shows AASHTO requirements (ranges) for the values of K for various design speeds.
- Select a vertical curve length, preferably in even multiples of 20 m, which fits the terrain conditions and equals or exceeds the prescribed minimum length.
- Compute "e", the middle ordinate.
- Compute the vertical offset distance e_1 at each station. For points between the VPC and VPI, the distance X is measured ahead from the VPC. For points between the VPI and VPT, the distance X is measured back from the VPT.
- Establish curve elevations by subtracting e_1 distance from tangent elevation (crest vertical curves) or adding e_1 distance to tangent elevations (sag vertical curves).
- Compute high point for crest vertical curves and low point for sag vertical curves.

Figure 4-13

CREST VERTICAL CURVE



$$e = \frac{(G_1 - G_2)\,L}{800} \qquad Z_1 = G_1\left(\frac{L}{A}\right)$$

$$e_1 = e\,\frac{X_1^2}{(L/2)^2} \qquad Z_2 = G_2\left(\frac{L}{A}\right)$$

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

e = MIDDLE ORDINATE DISTANCE at V.P.I. (meters)
L = LENGTH OF VERTICAL CURVE (meters)
G_1 AND G_2 = TANGENT GRADES ($\pm$ percent)
P = ANY POINT ON THE VERTICAL CURVE
X_1 = HORIZONTAL DISTANCE FROM V.P.C. TO P (meters)
e_1 = VERTICAL OFFSET DISTANCE (meters)
A = ALGEBRAIC DIFFERENCE IN GRADES (percent)
K = RATE OR VERTICAL CURVATURE
Z_1 AND Z_2 = DISTANCE TO HIGH OR LOW POINT

Example: If $G_1 = +3.0\%$, $G_2 = -2.5\%$, and $L = 300$ m

Then $K = 300/5.5 = 55$

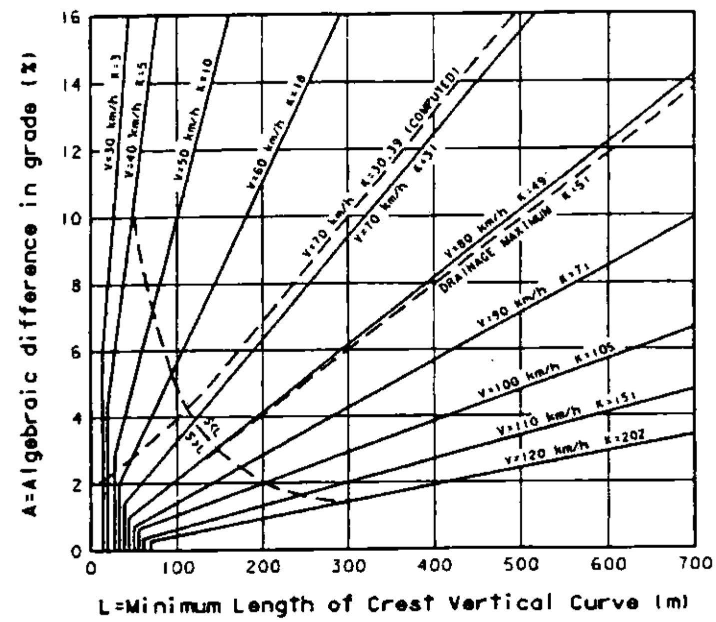
$$e = \frac{(3.0 - (-2.5))\,(300)}{800} = 2.063 \text{ m}$$

e_1 at 100 m from the VPC = 2.063 $((100)^2/(150)^2)$ = 0.917 m

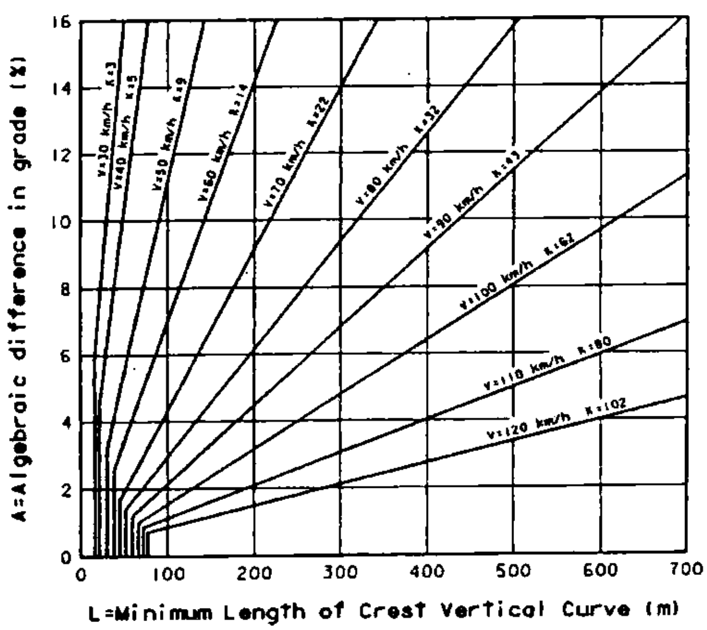
$Z_1 = 3.0\ (55) = 165$ m

Figure 4-14

CREST VERTICAL CURVES



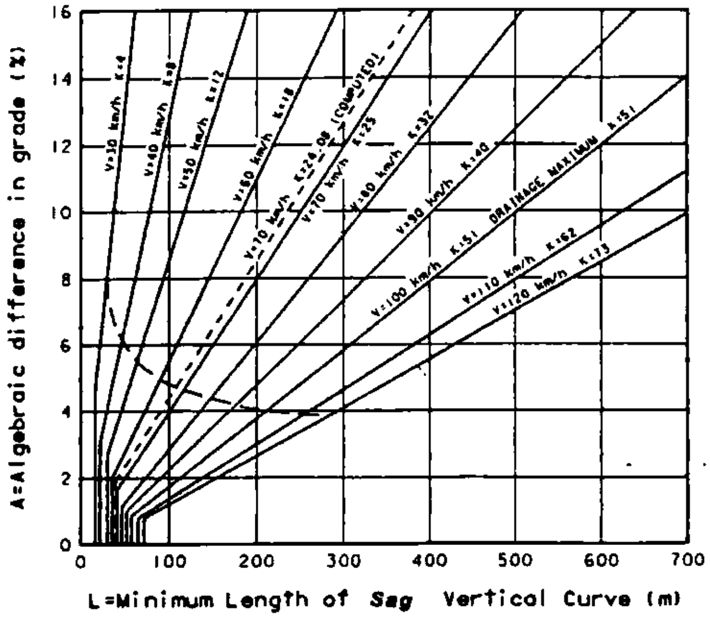
DESIGN CONTROLS FOR CREST VERTICAL CURVES FOR STOPPING SIGHT DISTANCE - UPPER RANGE



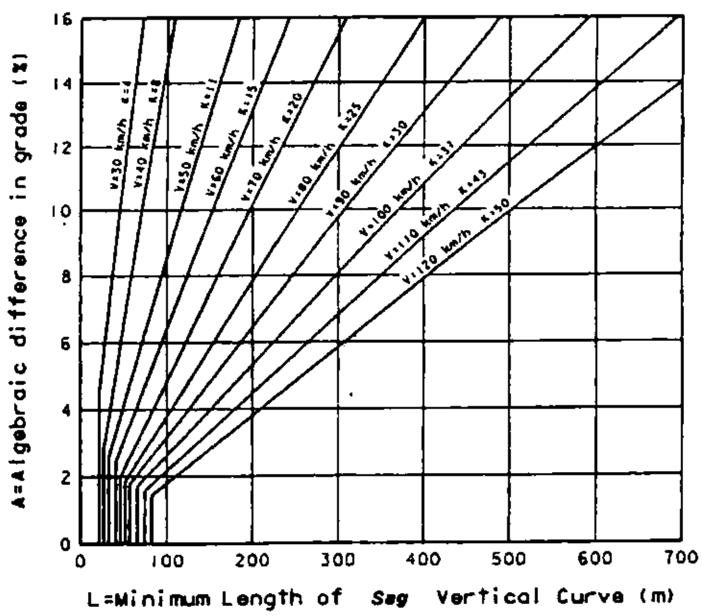
DESIGN CONTROLS FOR CREST VERTICAL CURVES FOR STOPPING SIGHT DISTANCE - LOWER RANGE

Figure 4-15

SAG VERTICAL CURVES



DESIGN CONTROLS FOR SAG VERTICAL CURVES - UPPER RANGE



DESIGN CONTROLS FOR SAG VERTICAL CURVES - LOWER RANGE

Table 4-6

RANGE OF DESIGN CONTROLS FOR VERTICAL CURVES

DESIGN CONTROLS FOR CREST VERTICAL CURVES BASED ON STOPPING SIGHT DISTANCE

DESIGN SPEED (km/h)	ASSUMED SPEED FOR CONDITION (km/h)	COEFFICIENT OF FRICTION f	STOPPING SIGHT DISTANCE FOR DESIGN (m)	RATE OF VERTICAL CURVATURE, K (LENGTH (m) PER % OF A) COMPUTED	ROUNDED FOR DESIGN
30	30 - 30	0.40	29.6 - 29.6	2.17 - 2.17	3 - 3
40	. 40 - 40	0.38	44.4 - 44.4	4.88 - 4.88	5 - 5
50	47 - 50	0.35	57.4 - 62.8	8.16 - 9.76	9 - 10
60	55 - 60	0.33	74.3 - 84.6	13 .66 - 17.72	14 - 18
70	63 - 70	0.31	94.1 - 110.8	21.92 - 30.39	22 - 31
80	70 - 80	0.30	112.8 - 139.4	31.49 - 48.10	32 - 49
90	77 - 90	0.30	131.2 - 168.7	42.61 - 70.44	43 - 71
100	85 - 100	0.29	157.0 - 205.0	61.01 - 104.02	62 - 105
110	91 - 110	0.28	179.5 - 246.4	79.75 - 150.28	80 - 151
120	98 - 120	0.28	202.9 - 285.6	101.90 - 201.90	102 - 202

DESIGN CONTROLS FOR CREST VERTICAL CURVES BASED ON PASSING SIGHT DISTANCE

DESIGN SPEED (km/h)	MINIMUM PASSING SIGHT DISTANCE FOR DESIGN (m)	RATE OF VERTICAL CURVATURE, K, ROUNDED FOR DESIGN (length (m) per % of A)
30	217	50
40	285	90
50	345	130
60	407	180
70	482	250
80	541	310
90	605	390
100	670	480
110	728	570
120	792	670

DESIGN CONTROLS FOR SAG VERTICAL CURVES BASED ON STOPPING SIGHT DISTANCE

DESIGN SPEED (km/h)	ASSUMED SPEED FOR CONDITION (km/h)	COEFFICIENT OF FRICTION f	STOPPING SIGHT DISTANCE FOR DESIGN (m)	RATE OF VERTICAL CURVATURE, K (LENGTH (m) PER % OF A) COMPUTED	ROUNDED FOR DESIGN
30	30 - 30	0.40	29.6 - 29.6	3.88 - 3.88	4 - 4
40	40 - 40	0.38	44.4 - 44.4	7.11 - 7.11	8 - 8
50	47 - 50	0.35	57.4 - 62.8	10.20 - 11.54	11 - 12
60	55 - 60	0.33	74.3 - 84.6	14.45 - 17.12	15 - 18
70	63 - 70	0.31	94.1 - 110.8	19.62 - 24.08	20 - 25
80	70 - 80	0.30	112.8 - 139.4	24.62 - 31.86	25 - 32
90	77 - 90	0.30	131.2 - 168.7	29.62 - 39.95	30 - 40
100	85 - 100	0.29	157.0 - 205.0	36.71 - 50.06	37 - 51
110	91 - 110	0.28	179.5 - 246.4	42.95 - 61.68	43 - 62
120	98 - 120	0.28	202.9 - 285.6	49.47 - 72.72	50 - 73

Crest Vertical Curves

Minimum lengths of crest vertical curves as determined by sight distance requirements generally are satisfactory from the standpoint of safety, comfort, and appearance. An exception may be at decision areas, such as sight distance to ramp exit gores, where longer lengths are necessary. Refer to the "Green Book", Chapter III, for details concerning decision sight distance.

Design Controls -- Stopping Sight Distance. Crest vertical curves must meet the minimum K values (lower value of the range) given in Table 4-6, with the upper value of the range being the desirable condition. The required lengths of vertical curves to provide the upper and lower values of the range of stopping sight distance for each design speed are shown in Figure 4-14. The solid lines give the required lengths, on the basis of rounded values of K as determined from computations. The short dashed curve at the lower left, crossing these lines, indicates where sight distance equals length. Note that to the right of the S = L line, the value of K, or length of vertical curve for the percent change in A, is a simple and convenient expression of the design control. For each design speed, this single value is a positive whole number that is indicative of the rate of vertical curvature. The design control in terms of K covers all combinations of A and L for any one design speed; thus, A and L need not be indicated separately in a design value tabulation. The selection of design curves is simplified because the required length of curve in meters is equal to K times the algebraic difference in grades in percent (L=KA). Conversely, checking plans is made easier by comparing curves with the design K values given in Table 4-6.

The primary concern in designing vertical curves is to assure that at least the minimum stopping sight distance is provided. The values set forth in the design criteria for desirable and minimum sight distance apply to vertical curves.

Basic assumptions are illustrated in Figures 4-16, 4-17 and 4-18 for conditions related to stopping sight distance on crest and sag vertical curves, and to passing sight distance on crest vertical curves.

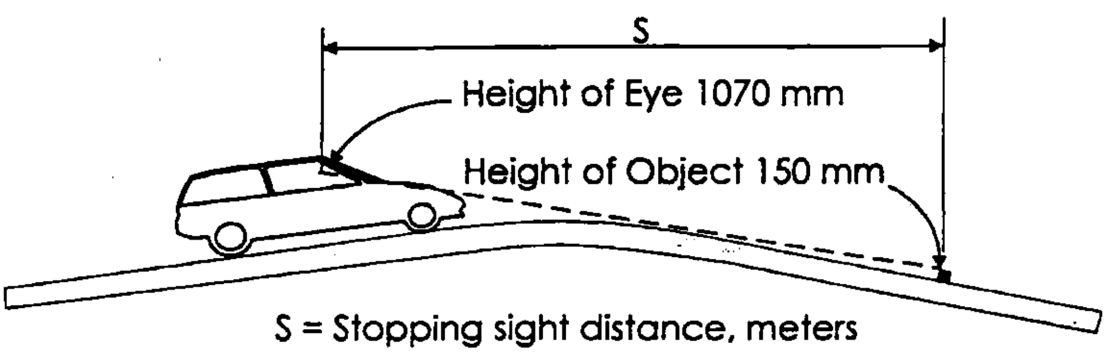


Figure 4-16

STOPPING SIGHT DISTANCE

Design Controls -- Passing Sight Distance. Design values of crest vertical curves for passing sight distance differs from those for stopping sight distance because of the height criteria illustrated in Figure 4-17 below.

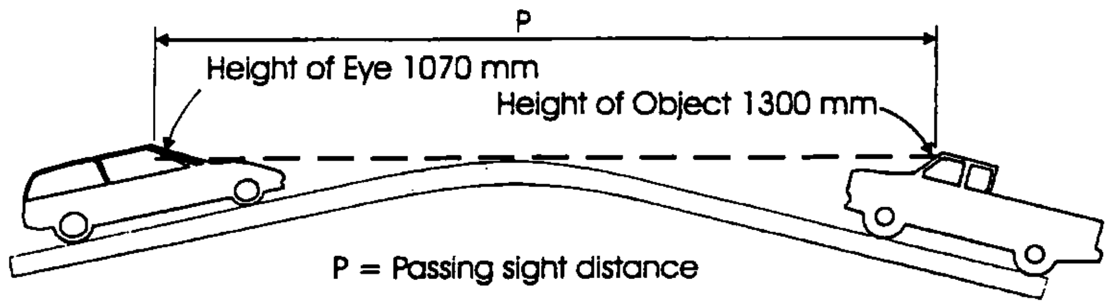


Figure 4-17

PASSING SIGHT DISTANCE

For minimum passing sight distances, the required lengths of crest vertical curves are substantially longer than those for stopping sight distances. The extent of difference is evident by the values of K for passing sight distances shown in Table 4-6. These lengths are 7 to 17 times the lengths necessary for stopping sight distances.

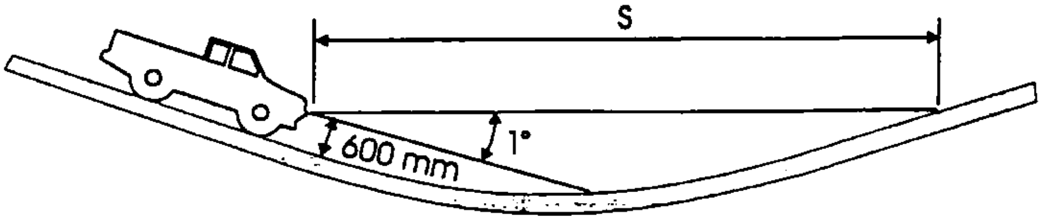
Generally, it is impractical to design crest vertical curves to provide for passing sight distance because of high cost where crest cuts are involved and the difficulty of fitting the required long vertical curves to the terrain, particularly for high-speed roads. Passing sight distance on crest vertical curves may be feasible on roads with an unusual combination of low design speed and gentle grades or higher design speeds with very small algebraic differences in grades. Ordinarily, passing sight distance is provided only at places where combinations of alignment and profile do not require the use of crest vertical curves.

Sag Vertical Curves

At least four different criteria for establishing lengths of sag vertical curves are recognized to some extent. These are 1) headlight sight distance, 2) rider comfort, 3) drainage control, and 4) a rule-of-thumb for general appearance.

When a vehicle traverses a sag vertical curve at night, the portion of highway illuminated ahead is dependent upon the position of the headlights and the direction of the light beam. General use is being given to a headlight height of 600 mm and a 1 degree upward divergence of the light beam from the longitudinal axis of the vehicle. The upward spread of the light beam beyond the 1 degree angle provides some additional visible length but this is generally ignored.

Figure 4-18 illustrates the criteria for headlight sight distance for sag vertical curves.



S = headlight beam distance = stopping sight distance (meters)

Figure 4-18

HEADLIGHT SIGHT DISTANCE

Sag vertical curves must generally meet the minimum K values (lower value of the range) given in Table 4-6, with the upper value of the range being the desirable condition. In some restrictive cases the minimum K value can be compromised if permanent lighting is provided.

For overall safety on highways, a sag vertical curve should be long enough so that the light beam distance is nearly the same as the stopping sight distance. The resulting lengths of vertical curves for the upper and lower values of the range of stopping sight distance for each design speed are shown in Figure 4-15 with solid lines using rounded K values as was done for crest vertical curves.

The comfort effect of change in vertical direction is greater on sag than on crest vertical curves because gravitational and centrifugal forces are combining rather than opposing forces. Comfort due to change in vertical direction is not measured readily because it is affected appreciably by vehicle body suspension, tire flexibility, weight carried and other factors.

But the length of vertical curve required to satisfy this comfort factor at the various design speeds is only about 50 percent of that required to satisfy the headlight sight-distance requirement for the normal range of design conditions.

Drainage affects design of vertical curves of Type III, Figure 4-12 where curbed sections are used. An approximate criterion for sag vertical curves is the same as for the crest conditions, that is, providing a minimum grade of 0.30 percent within 15 m of the level point, which corresponds to a K value of 51. Care should be exercised when using K's greater than 51 (crests and sags), especially with curbed sections, to ensure that water ponding will not be a problem.

NHDOT endorses minimum desirable lengths of vertical curves although shorter ones will comply with the "Green Book" criteria. These minimum lengths are shown in the following Table, 4-7.

Design Speed (km/h)	Length (m)
50	30
60	50
80	70
100	80
110	100

Table 4-7

MINIMUM VERTICAL CURVE LENGTHS

Plotting Vertical Curves

Drafting vertical curves by hand is done with circular curve templates which are manufactured with radii in millimeters. To use the circular curve templates for vertical curve drafting at the normal scales refer to Table 4-8.

Storing vertical curves on CAD/D alignments is done by specifying the VPI station, elevation and the curve length.

Establishing Grade

The profile gradeline usually will represent the finished surface elevation at the centerline of the roadway, the median travelway line for divided highways, or the base line designated on the typical section.

The established profile elevations should be recorded to the nearest 0.001 m along the bottom of the profile sheets and on the cross section sheets.

Also to be shown on the profiles are the stationing and elevation at each VPC, VPI, and VPT, the grades between VPIs, and the length and K value of each vertical curve.

Profiles are normally established graphically at the preliminary design stage, then mathematized in CAD/D prior to the Public Hearing. Computation procedures are described in the following paragraphs.

A smooth transition is needed to match existing roadways. Existing horizontal and vertical alignment should be considered for a sufficient distance beyond the limit of work to ensure adequate sight distance is provided. Connections should be made which are compatible with the design speed of the new project and which can be utilized when the adjoining road section is reconstructed.

	Scale: Horizontal = 1:250 Vertical = 1:50	Scale: Horizontal = 1:500 Vertical = 1:100
Metric K Value	Approx. Curve Radius (mm)	Approx. Curve Radius (mm)
3	240	120
5	400	200
7	560	280
10	800	400
15	1200	600
20	1600	800
25	2000	1000
30	2400	1200
35	2800	1400
40	3200	1600
45	3600	1800
50	4000	2000
55	4400	2200
60	4800	2400
65	5200	2600
70	5600	2800
80	6400	3200
90	7200	3600
100	8000	4000
125	10,000	5000
150	12,000	6000

Table 4-8

PLOTTING VERTICAL CURVES

Computations

Establishing a profile graphically is done by selecting VPI stations and approximate elevations, calculating the approximate grades between VPIs. and drafting the vertical curves for the desired K values. Establishing a profile in CAD/D will produce exact elevations at each station on the profile.

Initial drafts of the gradeline are recorded on a study profile. Using design controls and adhering to standards and criteria in this manual, the designer will plot a tentative gradeline on the study profile with the stationing and approximate elevation of each VPI clearly defined.

Starting at the beginning of the project, a grade should be established which, when extended forward, will match reasonably close to the plotted elevation of the first VPI. The percent grade and VPI elevations should be computed and recorded to three decimal places. It is desirable to locate VPIs at even stations or halfway between stations for ease of computation. After the first grade and VPI elevation have been established, the process is repeated between subsequent VPIs.

Frequently, it is necessary to adjust a gradeline after the initial grade computations have been made. When making this adjustment, it may be necessary to compute the new grade to more than three decimal places to accurately match a previously established VPI elevation; however, the final computed grades will be shown to the nearest 0.001 m.

String Line Method

To assist in developing a study profile, a convenient way to manually manipulate grades and VPI elevations is to use a string along with stringline weights. These specially designed weights have a heavy gauge wire bent out at a 90 degree angle which places the wire end down onto the table surface holding the string taught in the plane of the paper at an upward or downward angle representing a crest or sag VPI, respectively. When changes to the gradeline are necessary to meet vertical controls, simply move the weights accordingly and recompute a new "e" (middle ordinate) based on the algebraic difference in grades and the length of curve. Refer to Figure 4-13 for additional information.

Drainage Controls and Flood Elevations

Where there are controls affecting drainage flow lines, consideration must be used during gradeline design to ensure at least minimum cover over drainage structures. Criteria for minimum cover are set forth in Chapter 6, Drainage.

Profile grades should also be designed to assure adequate elevation above high-water levels. The minimum freeboard above the 100 year flood elevation at the pavement edge should be 300 mm plus the total depth of pavement and base courses. Under certain conditions, the freeboard requirement may be reduced. The Highway Design Administrator should make this decision.

Major Structures

Design of profile gradelines must be carefully coordinated at an early stage with the Bureau of Bridge Design and with the controls related to vertical clearances and high water.

During the bridge design process, there should be verification that the required clearance will be provided and that the grade controls shown on the bridge plans and on the road plans are identical. Remember, allowance must be made for future resurfacing for underpasses to perpetuate the required vertical clearance. Refer to Chapter 3, Design Considerations and Criteria, for clearance standards. Refer to Chapter 7, Highway Structures, for additional bridge criteria.

Railroad Grade Crossings

The profile gradeline must match the top of the rails at railroad grade crossings. In most instances, minor "warping" of the cross slopes will be required in the immediate area of the crossing. An acceptable cross slope turnover rate must be used. If there are significant grade and alignment differences, a grading plan should be considered. Any adverse or substandard superelevation must be approved by the Highway Design Administrator.

Special Field Conditions

Field inspections by the designer may result in recommendations with regard to the design of the profile. Special considerations may be required for items such as high-water levels, ditch flow lines, unusual soil conditions, road intersections, adjacent property development, or other unusual conditions which may control profile grade elevations.

Urban Grade Design

The design of vertical alignment on urban projects frequently involves special considerations such as adjacent property development, and matches with numerous existing side streets and driveways.

Where outside controlling factors are not severe, the normal practice of establishing the profile on the center line, or on the median travelway, will work satisfactorily. When the controls are significant, it may be necessary to supplement the main profile with other elevation controls such as gutterline profiles or top-of-curb profiles. When this is necessary, these supplemental controls should be clearly shown on the typical section and on the profile sheets.

Care must be taken to avoid flat spots where water will pond. Gutter lines should have a gradient of at least 0.4 percent and preferably 0.5 percent.

Vertical curves are not required when the algebraic difference in grades is 0.2 percent or less. Where vertical curves are required, they should be long enough to provide required sight distance, but should not be flattened to such an extent as to cause a drainage problem. Frequently it is desirable to locate VPIs and crest vertical curves at or near the centerline of intersecting cross streets to provide maximum sight distance at the intersecting street.

Street intersections, driveways, and existing utilities are dominant controls. Although they must be fully considered, they should not override the desirable features of a well designed gradeline. Even urban street design requires well designed, continuous gradelines rather than a connected series of block-by-block sections. Adequate cover over utilities is required. Grades should be adjusted to meet cover requirements or the utility must be moved or lowered. See Chapter 9, Utilities.

Balancing Earthwork

If possible on rural projects, it is desirable to design the profile gradeline to achieve a balance of earthwork or a slight borrow condition. Excavation from the normal cut sections should be adequate to construct the fills with no need for large additional fill quantities or for wasting excess material. Balancing earthwork is often outweighed by other more serious considerations and controls.

Refer to Chapter 8, Cross Sections and Earthwork, for more explanation about balancing earthwork.

Erosion Prevention

The erosion potential involved in highway construction may be minimized by choosing an alignment that will:

- Minimize the size of cut and fill sections.
- Avoid locations having high erosion potential.
- Minimize the amount of disturbance.
- Make use of natural land barriers and contours to confine runoff and limit erosion and sedimentation.
- Make use of existing vegetation.
- Conform to the contour and drainage patterns of the area.
- Use naturally stabilized waterways and channels to accept water from roadway ditches wherever feasible. Avoid undercutting existing channels. As a general practice, water should not be diverted from one watershed to another.
- Allow proper outfall locations and underdrain outlets.

Typical Sections

Typical sections show in cross section view how the roadway will be constructed. They are developed from design criteria and pavement structural requirements.

Typical sections establish the template to be used in drawing cross sections and computing earthwork. A new typical section should be developed for each unique segment of a project (i.e., highway, ramps, side roads), however, minor variations in cross section features do not necessarily warrant additional typical sections.

Information to be Shown

The "Model Plans" section of this manual shows a number of typical section sheets to be used as a guide.

The basic information to be shown is:

- Traveled way and shoulder pavement type, thickness, width, and slope;
- Type and depth of select materials (i.e., sand and gravels);
- Steepness of cut and fill slopes;
- Drainage, if it is longitudinal and typical;
- Ditches;
- Cross slopes and superelevation;
- Center line location;
- Profile grade location;
- Limiting Stations;
- Guardrail position and affect on shoulder break;
- Rock cuts;
- Slope rounding and clearing limits; and
- Specific notes related to the typical section.

Rural and Urban Sections

The basic difference between urban and rural typical sections is caused by the restriction of lateral clearance and the need to match adjacent areas. Curbs are used more frequently and sidewalks or other pedestrian and bicycle facilities are provided more often on urban projects.

Special Projects

Special projects or ones which do not involve pavement work sometimes require typical sections to show adjacent existing sections, such as for a lighting or signalization project.

Characteristics of the section should be shown for clarification particularly if cross section sheets are not a part of the plans.

Pavement Design

Pavement design depends on the volume of vehicles, the percentage of heavy trucks and the supporting soils in the project area. The design is a cooperative effort involving the Bureau of Transportation Planning (Traffic Research Section), which supplies the average daily load factor (ADL) upon request during the preliminary phase of project development, the Bureau of Highway Design which recommends structural section design, and the Bureau of Materials and Research which comments on the design and reports on the general availability of base course materials (crushed gravel versus crushed stone).

The customary pavement design process begins with an evaluation of the following considerations by the Bureau of Highway Design:

- Performance of the pavement on adjacent sections of highway ;
- The possibility of future construction which will allow less thickness initially and an anticipated overlay at a later time;
- The practicality of maintaining adequate curb reveal after subsequent resurfacing improvements; and
- Cost and implications of future overlays.

After evaluation of the considerations, a pavement thickness is designed and base course average thickness is assumed. Occasionally, Materials and Research will recommend additional sand thickness for frost protection requirements, in addition to the base courses needed for structural requirements. Preliminary estimates and preliminary design are based on the best information available prior to receiving complete reports. Refer to Appendix 4-A for an example of pavement design methods.

Geotechnical Report

A geological investigation or Geotechnical Report is requested from the Bureau of Materials and Research for most projects. The findings may affect the pavement design and typical section.

The Geotechnical Report is typically requested at the end of the preliminary design phase. The roadway Geotechnical Report provides recommendations for extra base course sand, underdrain recommendations, soil and rock slope recommendations in cut sections, special embankment considerations, detention basin recommendations, foundation recommendations for retaining walls, overhead sign structures, and traffic signal mast arms, muck removal limits, bedrock and boulder estimates as well as other design and construction items.

Materials Availability Report

On some large projects, the Preliminary Design Section may request an initial report concerning the availability of select materials. Materials availability is a factor in selecting the type of material (natural gravel vs. crushed stone base) to use in the structural section. The request for a Materials Availability Report should include a rough estimate of the volume of base course materials required for the project.

Computation

Structural section quantities are computed from constants and thicknesses that are shown on the typical section sheet. For preliminary engineering work, it is sufficient to use average depths and widths. The assembly and computation of plan quantities are discussed in Chapter 13, Plans, Specifications, and Estimates (PS&E).

Early Selection of Typical Section

The typical section must be established with reasonable assurance before preliminary line and grade can be approved, and before meaningful estimates of project cost can be prepared. Final plan or design phase typical sections must be complete to show pavement surface, aggregate base courses, and typical cross section geometry.