The information contained in Chapter Three: Roadway Alignment dated May 2022, has been updated to reflect the January 2023 Errata. The errata incorporates DES 22-02: "Maximum Allowable Deflection on a Horizontal Alignment Without a Curve", which was approved by the Nebraska Division of the FHWA on July 20, 2022, addresses errors, changes in procedure, changes in NDOT department titles, changes in other Roadway Design Manual chapters and other reference material citations occurring since the latest publication of this chapter.

Chapter Three presents guidance for the design of New and Reconstructed projects; design guidance for 3R projects is provided in Chapter Seventeen.

Chapter Three
Roadway Alignment

This chapter presents the **Nebraska Department of Transportation** (**NDOT**) policies and criteria for the design of roadway horizontal and vertical alignments, generally based on passenger car performance unless otherwise stated. The **American Association of State Highway and Transportation Officials** (**AASHTO's**) A Policy on Geometric Design of Highways and Streets (the *Green Book*) (Ref. 3.1) provides a more comprehensive description of the alignment considerations presented in this chapter and should be referred to for further details.

1. SIGHT DISTANCE

A driver must be able to see an object on the roadway ahead with sufficient time and distance to select and complete the appropriate action (avoid the object or stop). This is referred to as sight distance. The different types of sight distance include:

- Decision sight distance
- Stopping sight distance
- Passing sight distance
- Sight distance on horizontal curves
- Intersection sight distance

For additional information see Chapters 3 and 9 of the *Green Book* (Ref. 3.1).

2. HORIZONTAL ALIGNMENT DESIGN

Major considerations in horizontal alignment include:

- Design speed
- Topography
- The environment
- Economics

The **Project Development Division** (**PDD**) will usually recommend an approximate horizontal alignment for the project during the engineering review. It is the responsibility of the roadway designer to check the alignment and to verify that it is in compliance with **NDOT's** design guidance (See Chapter One: Roadway Design Standards of this manual).

Environmental considerations and impacts are a vital component of the design process; the roadway designer shall coordinate with the **Environmental Section** in **PDD** in the development of, and in any subsequent alteration to, the horizontal alignment. The horizontal alignment should not be changed during the Plan Details Phase (Clarity Activity 5500).

2.A Maximum Allowable Deflection on a Horizontal Alignment Without a Curve

As a general guide, any change in direction of the horizontal alignment with a deflection angle ≥ 0°30´ on high-speed roadways (≥ 50 mph) or ≥ 1° on low-speed (≤ 45 mph) and urban roadways will require a horizontal curve. Section 3.3.13, "General Controls for Horizontal Alignment", in Chapter 3 of the *Green Book* (Ref. 3.1) contains the following guidance:

- For small deflection angles, curves should be sufficiently long to avoid the appearance of a kink. Curves should be at least 500 feet long for a central angle of 5°, and the minimum length should be increased 100 feet for each 1° decrease in the central angle. The minimum length for horizontal curves on main highways, ($L_{c\ min}$) should be 15 times the design speed expressed in mph (V), or $L_{c\ min}$ = 15V. On high-speed controlled access facilities that use flat curvature for aesthetic reasons, the desirable minimum length for curves ($L_{c\ des}$) should be double the minimum length described above, or $L_{c\ des}$ = 30V.

For 3R projects, an improvement to the horizontal alignment may be considered if there is a relevant crash history. See Chapter Seventeen: Resurfacing, Restoration and Rehabilitation (3R) Projects, Section 3.B, of this manual for additional information.

2.B Horizontal Curvature

NDOT designs and designates horizontal curves based on the radius of the curve (for conversion from/ to degree of curvature see Appendix G, "Degree of Curvature"). The use of the minimum horizontal curve radius should be avoided unless economically or environmentally necessary.

2.B.1 Simple Curves

NDOT uses arc definition for curve computation. Usually the PI station, the intersection angle (I), and the circular curve radius (R) are established; the remaining curve data must be computed (See EXHIBIT 3.1).

2.B.2 Reverse Curves

A reverse curve consists of two curves on opposite sides of a common tangent with a relatively short tangent length between the curves. The tangent length between the curves is usually dictated by superelevation requirements for each curve (See Section 2.C of this chapter).

2.B.3 Compound Curves

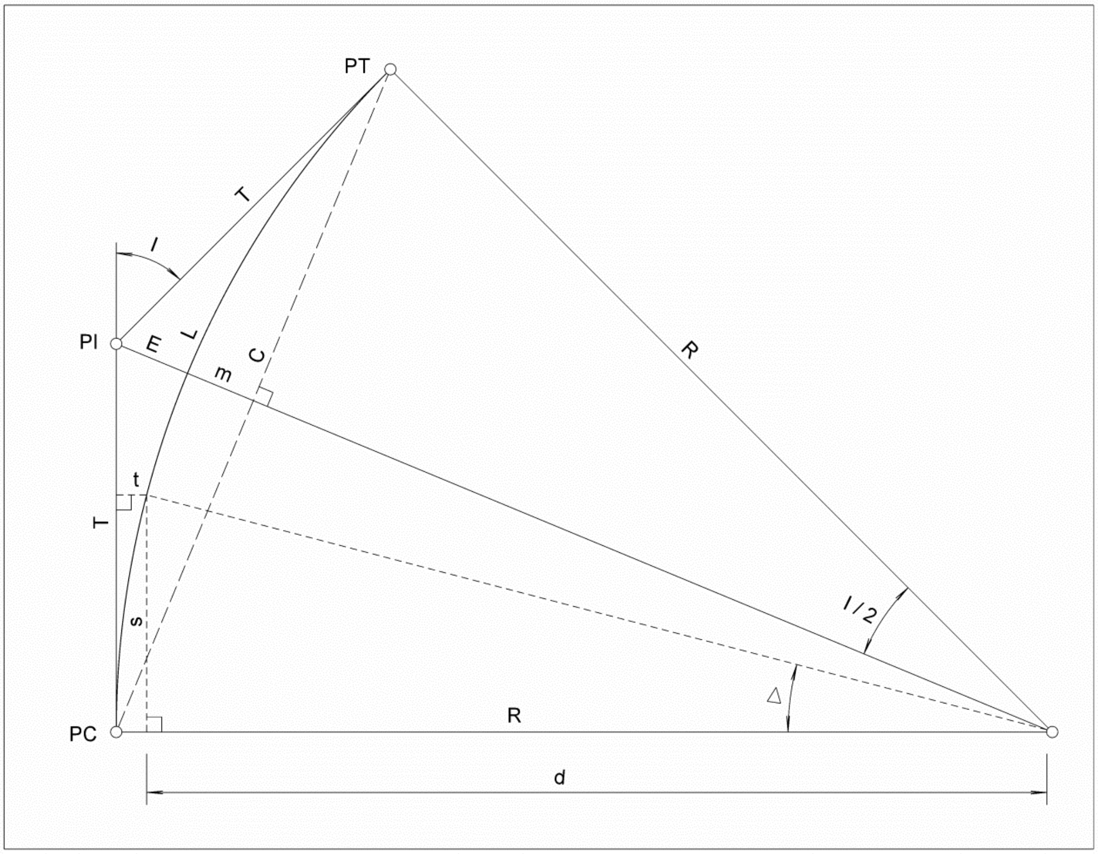
A compound curve consists of two consecutive curves of differing radii deflecting in the same direction with no tangent length between the curves. When field conditions dictate that a compound curve be utilized (e.g. for locations where an obstruction cannot be mitigated otherwise) the ratio of the flatter radius to the sharper radius should not exceed 1.5:1.

2.B.4 Broken Back Curves

A broken back curve consists of two consecutive curves deflecting in the same direction joined by a short tangent section.

2.B.5 Spiral Transition Curves

A spiral curve provides a transition between a tangent roadway section and a circular curve. The degree of curvature on a spiral gradually varies from zero at the tangent end to the degree of circular arc at the curve end. While **NDOT** sees only marginal benefits in the design of spiraled transition curves for new roadway alignments in general, spiral transition circular curves are preferred on Interstate ramps due to the higher percentage of truck traffic. For additional information see Chapter Five: Interstates, Grade Separations, and Interchanges, Section 3.C of this manual and Section 3.3.8.3, “Spiral Curve Transitions”, in Chapter 3 of the *Green Book* (Ref. 3.1).



Curve Symbols and Abbreviations	Curve Formulas
I Intersection angle of the curve C Length of long chord from PC to PT D Degree of curve E External distance (PI to mid-point of curve) L Length of curve from PC to PT m Middle ordinate (distance from the mid-point of the curve to the mid-point of the long chord) PC Point of curvature (beginning of curve) PI Point of intersection of tangents PT Point of tangency (end of curve) R Radius of curve T Tangent distance (distance from PC to PI, distance from PI to PT) s Distance along the tangent to any point on curve t Offset from the tangent to any point on curve Δ Intersection angle to any point on curve	$I = 2\sin^{-1}(L/2R) = (L/R) \times (180/\pi)$ $C = 2R\sin(I/2)$ (I in degrees) $D = 5729.57795/R$ $E = [\sqrt{(R^2 + T^2)}] - R$ $L = RI$ (I in radians) $= 2\pi R(I/360)$ (I in degrees) $m = R(1 - \cos(I/2))$ (I in degrees) $R = 5729.57795/D$ $= (4m^2 + C^2)/8$ $s = R\sin\Delta$ $d = R\cos\Delta$ $t = R - d$ $\pi = 3.141592653$ PC Sta. = PI Sta. – T PT Sta. = PC Sta. + L

Exhibit 3.1 Elements of a Simple Curve

2.C Superelevation

The minimum horizontal curve radius is determined by the design speed of the facility (V) and by the maximum allowable superelevation rate (e_{max}). Selection of a maximum superelevation rate depends on several factors, including:

- Design speed
- Location
- Climatic conditions
- Roadside conditions
- Future or ultimate development
- Roadway characteristics
- Facility type
- Driver expectations

EXHIBIT 3.2 summarizes the superelevation rates used in Nebraska.

For rural highways and for bridge structures a desirable maximum superelevation rate of 6% (EXHIBIT 3.3c) should be used unless design constraints dictate the use of the 8% maximum superelevation rate. Bridge structures should not be constructed in the transition zones of superelevated horizontal curves. Due to prevailing snow and ice conditions, the maximum superelevation rate shall not exceed 8%. The use of the maximum superelevation rate of 8% requires **Assistant Design Engineer** (**ADE**) approval and a decision letter to the project file (See Chapter One: Roadway Design Standards, Section 10.C, of this manual). An 8% superelevation rate should be designed in accordance with Section 3.3, "Horizontal Alignment", in Chapter 3 of the *Green Book* (Ref. 3.1).

Location	**Max. Allowable Superelevation**	**Desirable Max. Superelevation**
Rural Roadways	8%* (See Ref. 3.1, Chapter 3)	6% (EXHIBITS 3.3c, 3.4c & 3.5c)
Bridges	8%* (See Ref. 3.1, Chapter 3)	6% (EXHIBITS 3.3c, 3.4c & 3.5c)
High-Speed Urban Roadways V ≥ 50 mph	6% (EXHIBIT 3.3c)	6% (EXHIBIT 3.3c)
Desirable Design, Low-Speed Urban Roadways V ≤ 45 mph	4% (EXHIBIT 3.3d)	4% (EXHIBITS 3.3d & 3.6c)
Minimum Design, Low-Speed Urban Roadways V ≤ 45 mph	4% (Ref. 3.1, **TABLE 3-13** **)	4% (Ref. 3.1, **TABLE 3-13** **)

* Requires **ADE** approval and a decision letter to the project file.
** The use of **TABLE 3-13** (Ref. 3.1) requires **ADE** approval and a decision letter to the project file (See Chapter One: Roadway Design Standards, Section 10.C, of this manual).

Exhibit 3.2 Superelevation Rates

2.C.1 Transition Lengths

- **Tangent Runout** - The distance required to transition the roadway from a normal crown section to a section with the adverse crown removed, or vice versa.
- **Superelevation Runoff Length** (**L_r**) - The distance required to transition the roadway from a section with the adverse crown removed to a fully superelevated section, or vice versa.
- **Superelevation Transition Length** - The sum of the tangent runout and the superelevation runoff (L_r).

EXHIBITS 3.3b, 3.4b, 3.5b, and 3.6b illustrate the relationship between superelevation transition, tangent runout, and superelevation runoff length for two-lane roadways, for dual lane highways with crowned surface, for dual lane highways with tangent surface, and for dual lane highways with raised medians.

The minimum superelevation runoff lengths shown in EXHIBITS 3.3c AND 3.3d were calculated for two-lane and four-lane undivided roadways, see **TABLE 3-15** in the *Green Book* (Ref. 3.1) to determine the minimum superelevation runoff lengths for multilane undivided roadways.

For simple curves 50 to 80% of the superelevation runoff length (L_r) should be placed on the tangent prior to the curve (the majority of agencies use 67%). See Section 3.3.8, “Transition Design Controls”, in Chapter 3 of the *Green Book* (Ref. 3.1) for additional information.

To facilitate pavement drainage, a minimum profile grade of 1.5% shall be maintained through the area where the adverse crown has been removed. A flatter profile grade, down to and including a grade of 0.5%, may be used with **Unit Head** approval.

2.C.2 Axis of Rotation

The axis of rotation is the point on the roadway cross-section about which the roadway is rotated to attain the desired superelevation through the horizontal curve. EXHIBITS 3.3a, 3.4a, 3.5a, and 3.6a illustrate **NDOT** standard procedures for the application of the axis of rotation in superelevation development for two-lane highway sections and for four-lane divided highway sections. See Section 3.3.8, “Transition Design Controls”, in Chapter 3 of the *Green Book* (Ref. 3.1) for methods of attaining superelevation where **NDOT** procedures do not apply.

For two-lane roadways the superelevation should be rotated about the highway centerline, which is normally the profile grade line (See EXHIBIT 3.3a). This method minimizes the elevation differential between the pavement edges and their normal profiles. Rotation about the inside or outside axis of the roadway is acceptable when required to satisfy field conditions, such as surface drainage on a curbed facility.

For multi-lane facilities with crowned surfaces and depressed medians of 54 feet or less in width the profile grade point should be about the inside (median) edge of the 12 foot inside lane of each roadway (See EXHIBIT 3.4a). The axis of rotation for the outer roadway should be located at the centerline of the lanes until the superelevation transition attains reverse crown, at which point the axis of rotation will shift to the inside edge of the 12 foot inside lane. The axis of rotation for the inner roadway should be located at the inside edge of the 12 foot inside lane of the roadway, the same location as the profile grade point. This method results in the maintenance of approximately a 2-foot median ditch depth. When the median width is greater than 54 feet the axis of rotation and the profile grade point may be the centerline of the individual lanes.

For multi-lane facilities with tangent surfaces and depressed medians of 54 feet or less in width the axis of rotation should be about the profile grade line, which is the inside (median) edge of the 12 foot inside lane of each roadway. This method maintains the median in a horizontal plane throughout the curve (See EXHIBIT 3.5a). When the median width is greater than 54 feet the axis of rotation and the profile grade point may be the centerline of the individual lanes.

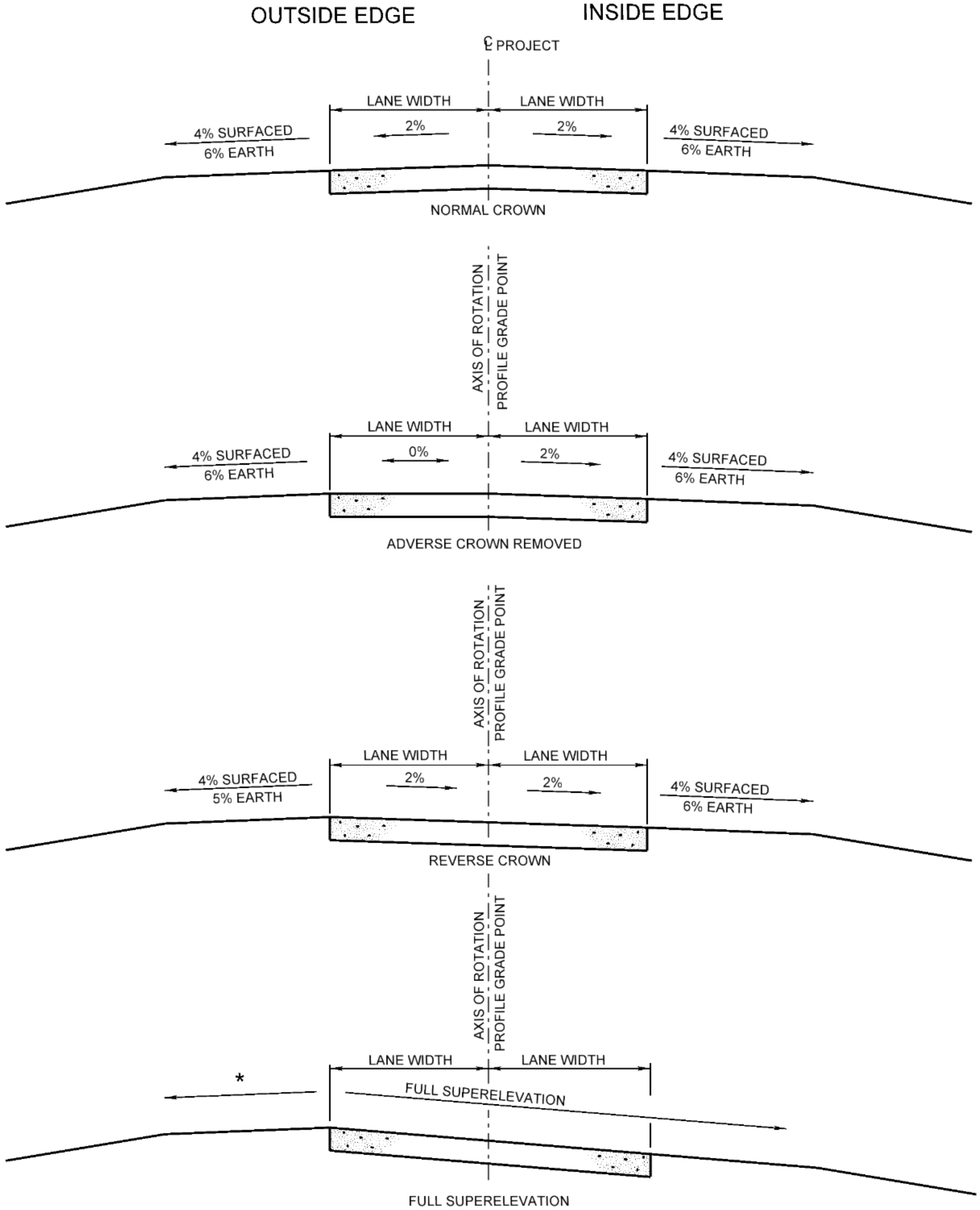
For multi-lane facilities with raised medians, the axis of rotation and the profile grade point should be located at the inside edge of the 12 foot inside lane (See EXHIBIT 3.6a).

2.C.3 Smoothing of Pavement Edge Profile

Angular breaks in the vertical profile of the pavement edge through the superelevation transition length should be rounded in Roadway Design Details (Clarity Task 5508). For general appearance, **NDOT** softens these sharp angular breaks by the insertion of short vertical curves along the pavement edge. As an approximate guide, the minimum vertical curve length in feet can be set as numerically equal to the design speed in miles per hour; greater lengths should be used where practicable.

2.D Pavement Widening on Curves (Off-tracking)

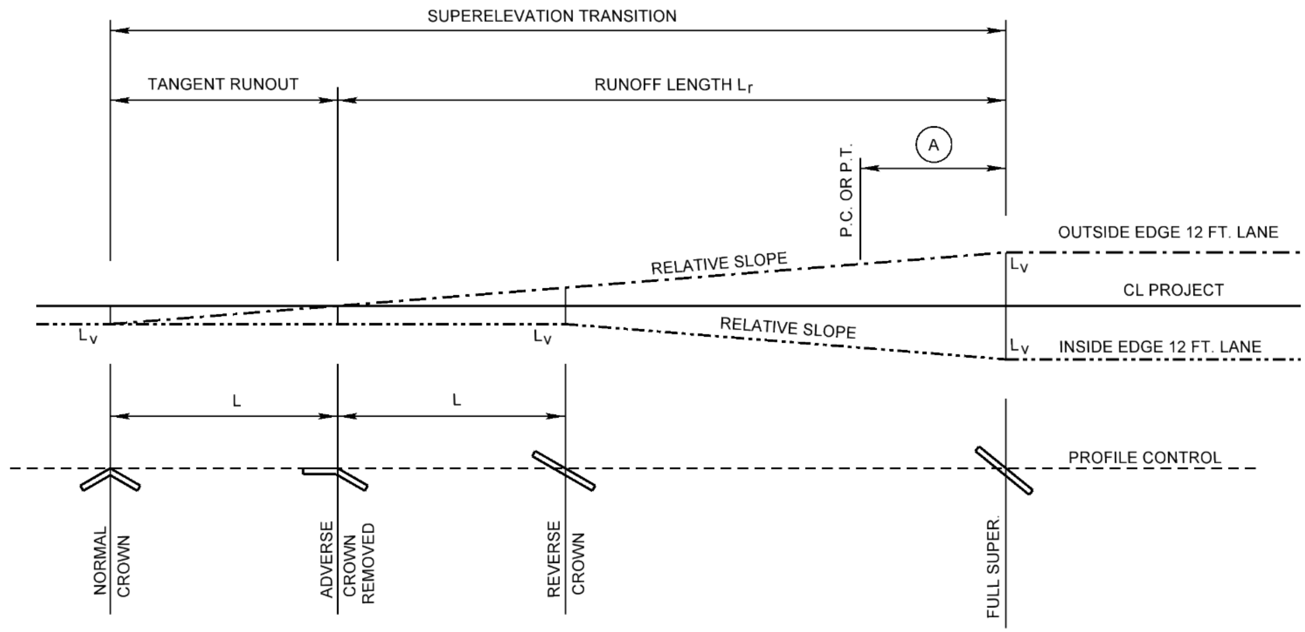
Pavement widening may be warranted if a vehicle or truck occupies a greater width due to the rear wheels tracking inside of the front wheels while rounding curves. See Section 3.3.10, "Traveled-Way Widening on Horizontal Curves", in Chapter 3 of the *Green Book* (Ref. 3.1) for additional information.



* THE SHOULDER SLOPE SHOULD BE MAINTAINED UNTIL THE SUPERELEVATION RATE ON THE ROADWAY IS SUCH THAT THE ALGEBRAIC DIFFERENCE BETWEEN CROSS SLOPES ON THE ROADWAY AND SHOULDER EQUALS 7%. THIS ALGEBRAIC DIFFERENCE SHOULD NOT EXCEED 7%.

FOR ADDITIONAL DETAILS, SEE EXHIBITS 6.3 THROUGH 6.6

Exhibit 3.3a Superelevation Data for Crowned Highways
Typical Sections



DIAGRAMMATIC PROFILE SHOWING METHOD OF ATTAINING SUPERELEVATION

NOTES:

e = SUPERELEVATION RATE AS SHOWN IN THE PLANS (IN %)

FOR THE 28 FT. TOP SYSTEM THE SUPERELEVATION RATE FOR THE 2 FT. SURFACED SHOULDER WILL BE THE SAME AS FOR THE THRU LANE.

AT POINTS MARKED "L_v" IT MAY BE NECESSARY TO INSERT A SHORT CONVENIENT LENGTH OF PARABOLIC CURVE TO ELIMINATE THE SHARP BREAK IN THE STRAIGHT LINE TRANSITION.

L = 12 FT. (WIDTH OF RDWY. BEING ROTATED) x 0.02 (CHANGE IN RDWY. CROSS-SLOPE) x RELATIVE SLOPE
L_r = 12 FT. (WIDTH OF RDWY. BEING ROTATED) x e/100 (FULL SUPERELEVATION) x RELATIVE SLOPE.

FOR A 14 FT. LANE WIDTH L AND L_r SHOULD BE COMPUTED USING THE 12 FT. DRIVING LANE WIDTH.

(A) 50 TO 80% OF THE RUNOFF LENGTH SHOULD BE PLACED ON THE TANGENT. SEE REFERENCE 3.1, CHAPTER 3, SECTION 3.3.8 FOR ADDITIONAL INFORMATION.

Exhibit 3.3b Superelevation Data for Crowned Highways
Diagrammatic Profile

Radius of Curve (ft.)	V=50 mph			V=55 mph			V=60 mph			V=65 mph			V=70 mph			V=75 mph			V=80 mph		
	e (%)	Maximum Relative Slope 200:1		e (%)	Maximum Relative Slope 213:1		e (%)	Maximum Relative Slope 222:1		e (%)	Maximum Relative Slope 233:1		e (%)	Maximum Relative Slope 250:1		e (%)	Maximum Relative Slope 263:1		e (%)	Maximum Relative Slope 286:1	
		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)
23000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
20000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
17000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
14000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	2.0	63	63	2.0	69	69
12000	NC	0	0	NC	0	0	NC	0	0	2.0	56	56	2.0	60	60	2.0	63	63	2.1	69	72
10000	NC	0	0	NC	0	0	2.0	53	53	2.0	56	56	2.1	60	63	2.3	63	73	2.5	69	86
8000	NC	0	0	2.0	51	51	2.0	53	53	2.3	56	64	2.5	60	75	2.8	63	88	3.1	69	106
6000	2.0	48	48	2.2	51	56	2.6	53	69	2.9	56	81	3.2	60	96	3.5	63	110	4.0	69	137
5000	2.2	48	53	2.6	51	66	3.0	53	80	3.3	56	92	3.7	60	111	4.1	63	129	4.6	69	158
4000	2.7	48	65	3.1	51	79	3.6	53	96	4.0	56	112	4.4	60	132	4.9	63	155	5.5	69	189
3500	3.0	48	72	3.5	51	89	3.9	53	104	4.4	56	123	4.9	60	147	5.3	63	167	5.9	69	202
3000	3.4	48	82	3.8	51	97	4.3	53	115	4.8	56	134	5.4	60	162	5.7	63	180	Rmin = 3050		
2500	3.8	48	91	4.3	51	110	4.8	53	128	5.3	56	148	5.8	60	174	6.0	63	189			
2000	4.3	48	103	4.9	51	125	5.4	53	144	5.8	56	162	Rmin = 2040			Rmin = 2500					
1800	4.6	48	110	5.1	51	130	5.6	53	149	6.0	56	167									
1600	4.9	48	118	5.4	51	138	5.8	53	155	Rmin = 1660											
1400	5.2	48	125	5.7	51	146	6.0	53	160												
1200	5.5	48	132	5.9	51	151	Rmin = 1330														
1000	5.9	48	142	Rmin = 1060																	
900	6.0	48	144																		
	Rmin = 833																				

$e_{max} = 6\%$

Source: Adapted from "A Policy on Geometric Design of Highways and Streets", (Reference 3.1), Eq. 3-12 thru 3-24.
For additional information see Reference 3.1, Tables 3-9 & 3-16.

KEY:
V = Assumed design speed
e = Rate of superelevation
L = Tangent runout based on a 12 ft. lane
Lr = Minimum length of superelevation runoff based on a 12 ft. lane
NC = Normal crown section

Exhibit 3.3c Superelevation Data for Crowned Highways
Values for Design Elements Related to Design Speed and Horizontal Curvature
(e_{max}=6%)

Radius of Curve (ft.)	V=25 mph			V=30 mph			V=35 mph			V=40 mph			V=45 mph			V=50 mph		
	e (%)	Maximum Relative Slope 143:1		e (%)	Maximum Relative Slope 152:1		e (%)	Maximum Relative Slope 161:1		e (%)	Maximum Relative Slope 172:1		e (%)	Maximum Relative Slope 185:1		e (%)	Maximum Relative Slope 200:1	
		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)
8000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
6000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	2.0	48	48
5000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	2.0	44	44	2.0	48	48
4000	NC	0	0	NC	0	0	NC	0	0	2.0	41	41	2.0	44	44	2.3	48	55
3500	NC	0	0	NC	0	0	2.0	39	39	2.0	41	41	2.2	44	49	2.5	48	60
3000	NC	0	0	NC	0	0	2.0	39	39	2.1	41	43	2.4	44	53	2.7	48	65
2500	NC	0	0	2.0	36	36	2.0	39	39	2.3	41	47	2.6	44	58	2.9	48	70
2000	2.0	34	34	2.0	36	36	2.3	39	45	2.6	41	54	2.8	44	62	3.2	48	77
1800	2.0	34	34	2.0	36	36	2.4	39	46	2.7	41	56	3.0	44	67	3.3	48	79
1600	2.0	34	34	2.2	36	40	2.5	39	48	2.8	41	58	3.2	44	71	3.5	48	84
1400	2.0	34	34	2.3	36	42	2.6	39	50	3.0	41	62	3.3	44	73	3.7	48	89
1200	2.1	34	36	2.4	36	44	2.8	39	54	3.2	41	66	3.6	44	80	3.9	48	94
1000	2.3	34	39	2.6	36	47	3.0	39	58	3.4	41	70	3.8	44	84	4.0	48	96
900	2.4	34	41	2.7	36	49	3.1	39	60	3.6	41	74	3.9	44	87			
800	2.4	34	41	2.8	36	51	3.3	39	64	3.7	41	76	4.0	44	89			
700	2.5	34	43	3.0	36	55	3.4	39	66	3.9	41	80						
600	2.7	34	46	3.2	36	58	3.6	39	70	4.0	41	83						
500	2.8	34	48	3.4	36	62	3.9	39	75									
450	2.9	34	50	3.5	36	64	4.0	39	77									
400	3.1	34	53	3.6	36	65	4.0	39	77									
350	3.2	34	55	3.8	36	69												
300	3.4	34	58	3.9	36	71												
250	3.6	34	62	4.0	36	73												
200	3.8	34	65															

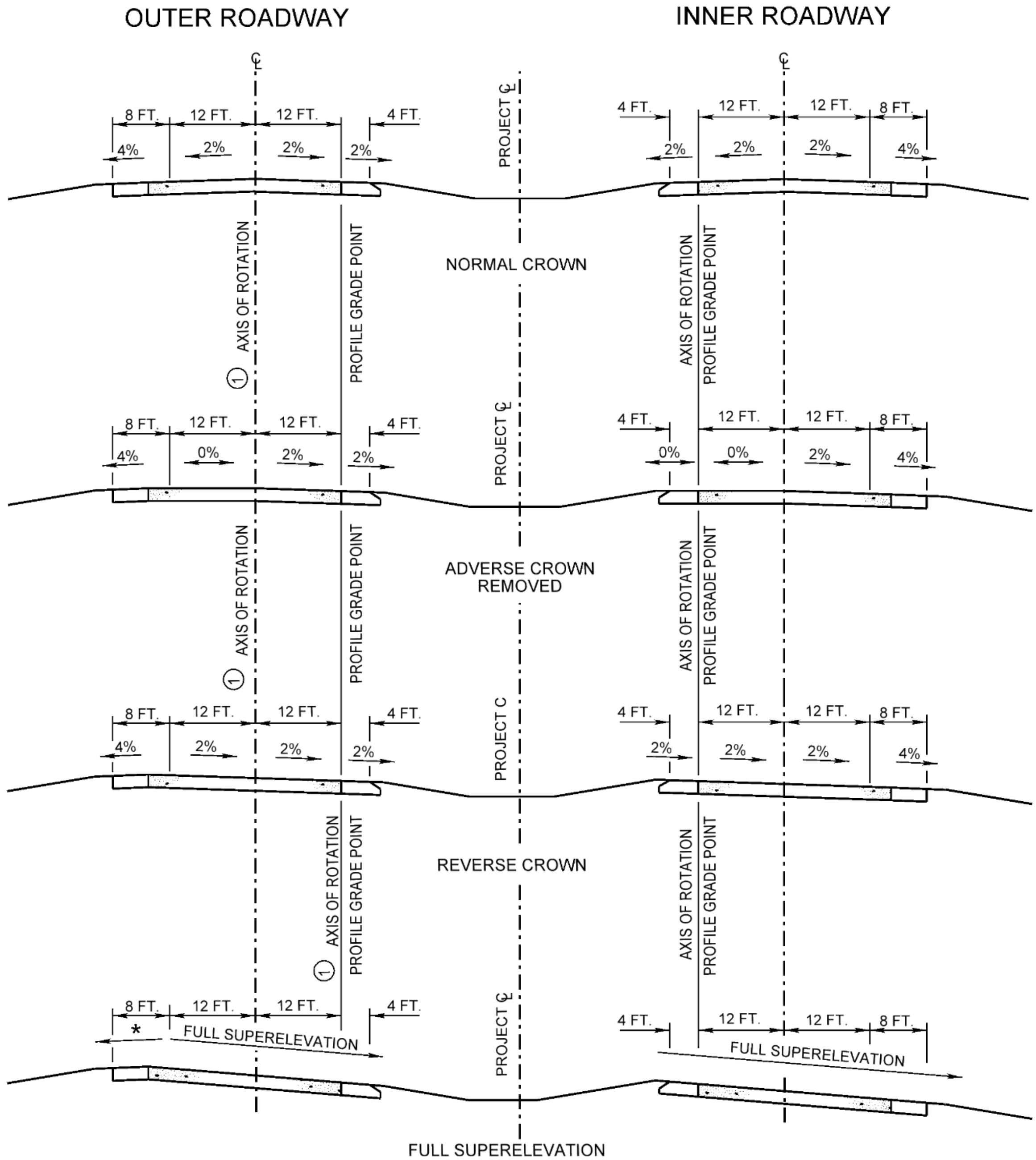
Rmin = 154 (V=25 mph); Rmin = 250 (V=30 mph); Rmin = 371 (V=35 mph); Rmin = 533 (V=40 mph); Rmin = 711 (V=45 mph); Rmin = 926 (V=50 mph)

$e_{max} = 4\%$

Source: Adapted from "A Policy on Geometric Design of Highways and Streets", (Reference 3.1), Eq. 3-12 thru 3-24.
For additional information see Reference 3.1, Tables 3-13 & 3-16.

KEY:
V = Assumed design speed
e = Rate of superelevation
L = Tangent runout based on a 12 ft. lane
Lr = Minimum length of superelevation runoff based on a 12 ft. lane
NC = Normal crown section

Exhibit 3.3d Superelevation Data for Crowned Highways
Values for Design Elements Related to Design Speed and Horizontal Curvature
(e_{max}=4%)



* THE SHOULDER SLOPE SHALL BE MAINTAINED UNTIL THE SUPERELEVATION RATE ON THE ROADWAY IS SUCH THAT THE ALGEBRAIC DIFFERENCE BETWEEN CROSS SLOPES ON THE ROADWAY AND SHOULDER EQUALS 7%. THIS ALGEBRAIC DIFFERENCE SHALL NOT EXCEED 7%.

(1) FOR THE OUTER ROADWAY, THE AXIS OF ROTATION SHALL SHIFT FROM THE CENTERLINE OF THE LANES TO THE INSIDE EDGE OF THE 12 FT. INSIDE LANE WHEN THE SUPERELEVATION TRANSITION ATTAINS REVERSE CROWN.

FOR ADDITIONAL DETAILS, SEE EXHIBIT 6.1

SUPERELEVATION FOR DUAL HIGHWAYS
WITH DEPRESSED MEDIANS OF 54 FOOT OR LESS IN WIDTH
(CROWNED SURFACE)

Exhibit 3.4a Superelevation Data for Dual Highways (Crowned Surface)
Depressed Median Width = 54 Foot or Less
Typical Sections

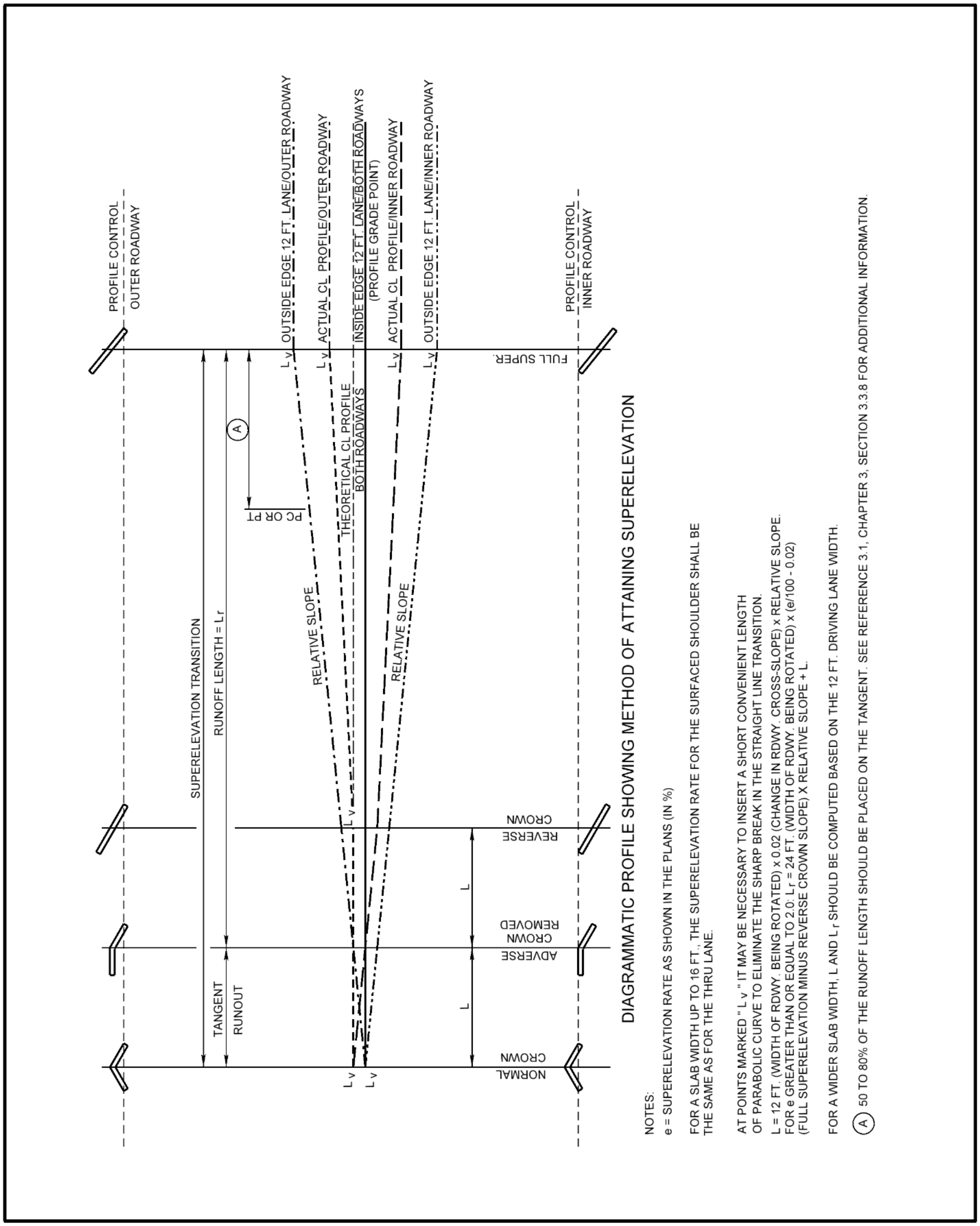


Exhibit 3.4b Superelevation Data for Dual Highways (Crowned Surface)
Depressed Median Width = 54 Foot or Less
Diagrammatic Profile

Radius of Curve (ft.)	V=50 mph			V=55 mph			V=60 mph			V=65 mph			V=70 mph			V=75 mph			V=80 mph		
	e (%)	Maximum Relative Slope 200:1		e (%)	Maximum Relative Slope 213:1		e (%)	Maximum Relative Slope 222:1		e (%)	Maximum Relative Slope 233:1		e (%)	Maximum Relative Slope 250:1		e (%)	Maximum Relative Slope 263:1		e (%)	Maximum Relative Slope 286:1	
		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)
23000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
20000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
17000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
14000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	2.0	63	63	2.0	69	69
12000	NC	0	0	NC	0	0	NC	0	0	2.0	56	56	2.0	60	60	2.0	63	63	2.1	69	76
10000	NC	0	0	NC	0	0	2.0	53	53	2.0	56	56	2.1	60	66	2.3	63	82	2.5	69	103
8000	NC	0	0	2.0	51	51	2.0	53	53	2.3	56	73	2.5	60	90	2.8	63	113	3.1	69	144
6000	2.0	48	48	2.2	51	61	2.6	53	85	2.9	56	106	3.2	60	132	3.5	63	158	4.0	69	206
5000	2.2	48	58	2.6	51	82	3.0	53	106	3.3	56	129	3.7	60	162	4.1	63	196	4.6	69	247
4000	2.7	48	82	3.1	51	107	3.6	53	138	4.0	56	168	4.4	60	204	4.9	63	246	5.5	69	309
3500	3.0	48	96	3.5	51	128	3.9	53	154	4.4	56	190	4.9	60	234	5.3	63	271	5.9	69	337
3000	3.4	48	115	3.8	51	143	4.3	53	176	4.8	56	212	5.4	60	264	5.7	63	297	Rmin = 3050		
2500	3.8	48	134	4.3	51	168	4.8	53	202	5.3	56	240	5.8	60	288	6.0	63	315			
2000	4.3	48	158	4.9	51	199	5.4	53	234	5.8	56	268	Rmin = 2040			Rmin = 2500					
1800	4.6	48	173	5.1	51	209	5.6	53	245	6.0	56	280									
1600	4.9	48	187	5.4	51	225	5.8	53	255	Rmin = 1660											
1400	5.2	48	202	5.7	51	240	6.0	53	266												
1200	5.5	48	216	5.9	51	250	Rmin = 1330														
1000	5.9	48	235	Rmin = 1060																	
900	6.0	48	240																		
	Rmin = 833																				

$e_{max} = 6\%$

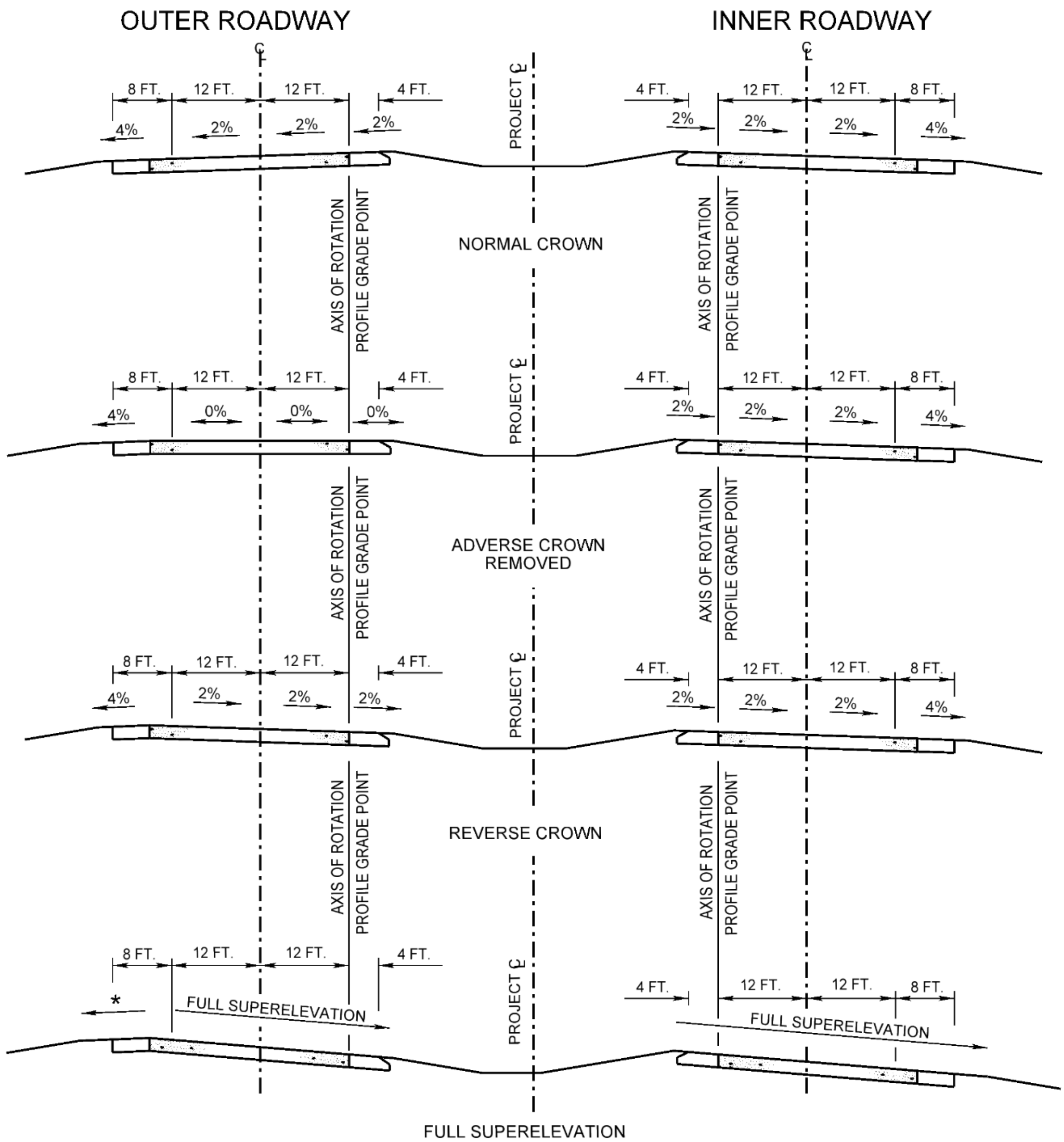
Source: Adapted from "A Policy on Geometric Design of Highways and Streets", (Reference 3.1), Eq. 3-12 thru 3-24.
For additional information see Reference 3.1, Tables 3-9 & 3-16.

KEY:

V = Assumed design speed
e = Rate of superelevation
L = Tangent runout based on a 12 ft. lane
Lr = Minimum length of superelevation runoff based on a 24 ft. roadway
NC = Normal crown section

Notes: For 4 lane divided roadways with medians greater than 40 ft. in width, use the relative gradient given for the 2 lane roadway in Exhibit 3.3

Exhibit 3.4c Superelevation Data for Dual Highways (Crowned Surface)
Depressed Median Width = 54 Foot or Less
Values for Design Elements Related to Design Speed and Horizontal Curvature
(e_{max}=6%)

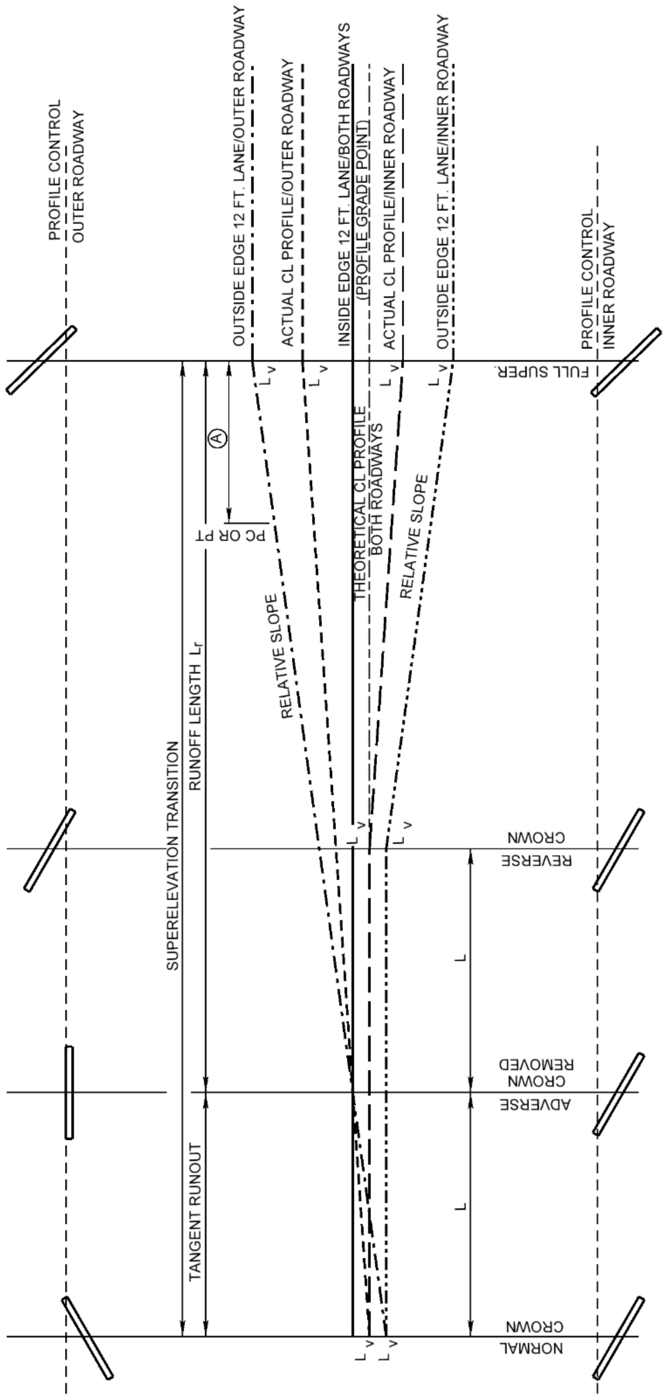


* THE SHOULDER SLOPE SHALL BE MAINTAINED UNTIL THE SUPERELEVATION RATE ON THE ROADWAY IS SUCH THAT THE ALGEBRAIC DIFFERENCE BETWEEN CROSS SLOPES ON THE ROADWAY AND SHOULDER EQUALS 7%. THIS ALGEBRAIC DIFFERENCE SHALL NOT EXCEED 7%.

FOR ADDITIONAL DETAILS, SEE EXHIBIT 6.2

SUPERELEVATION FOR DUAL HIGHWAYS
WITH DEPRESSED MEDIANS OF 54 FOOT OR LESS IN WIDTH
(TANGENT SURFACE)

Exhibit 3.5a Superelevation Data for Dual Highways (Tangent Surface)
Depressed Median Width = 54 Foot or Less
Typical Sections



DIAGRAMMATIC PROFILE SHOWING METHOD OF ATTAINING SUPERELEVATION

NOTES:
e = SUPERELEVATION RATE AS SHOWN IN THE PLANS (IN %)

FOR A SLAB WIDTH UP TO 16 FT., THE SUPERELEVATION RATE FOR THE SURFACED SHOULDER SHALL BE THE SAME AS FOR THE THRU LANE.

AT POINTS MARKED "L_V" IT MAY BE NECESSARY TO INSERT A SHORT CONVENIENT LENGTH OF PARABOLIC CURVE TO ELIMINATE THE SHARP BREAK IN THE STRAIGHT LINE TRANSITION.

L = 24 FT. (WIDTH OF RDWY. BEING ROTATED) x 0.02 (CHANGE IN RDWY. CROSS-SLOPE) x RELATIVE SLOPE.
L_r = 24 FT. (WIDTH OF RDWY. BEING ROTATED) x e/100 (FULL SUPERELEVATION) x RELATIVE SLOPE.
FOR A 15 FT. SLAB WIDTH L AND L_r SHOULD BE COMPUTED BASED ON THE 12 FT. DRIVING LANE WIDTH.

(A) 50 TO 80% OF THE RUNOFF LENGTH SHOULD BE PLACED ON THE TANGENT. SEE REFERENCE 3.1, CHAPTER 3, SECTION 3.3.8 FOR ADDITIONAL INFORMATION.

Exhibit 3.5b Superelevation Data for Dual Highways (Tangent Surface)
Depressed Median Width = 54 Foot or Less
Diagrammatic Profile

Radius of Curve (ft.)	V=50 mph			V=55 mph			V=60 mph			V=65 mph			V=70 mph			V=75 mph			V=80 mph		
	e (%)	Maximum Relative Slope 150:1		e (%)	Maximum Relative Slope 160:1		e (%)	Maximum Relative Slope 166.5:1		e (%)	Maximum Relative Slope 175:1		e (%)	Maximum Relative Slope 187.5:1		e (%)	Maximum Relative Slope 197:1		e (%)	Maximum Relative Slope 214.5:1	
		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)
23000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
20000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
17000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
14000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	2.0	95	95	2.0	103	103
12000	NC	0	0	NC	0	0	NC	0	0	2.0	84	84	2.0	90	90	NC	95	95	2.1	103	108
10000	NC	0	0	NC	0	0	2.0	80	80	2.0	84	84	2.1	90	95	2.3	95	109	2.5	103	129
8000	NC	0	0	2.0	77	77	2.0	80	80	2.3	84	97	2.5	90	112	2.8	95	132	3.1	103	160
6000	2.0	72	72	2.2	77	84	2.6	80	104	2.9	84	122	3.2	90	144	3.5	95	165	4.0	103	206
5000	2.2	72	79	2.6	77	100	3.0	80	120	3.3	84	139	3.7	90	166	4.1	95	194	4.6	103	237
4000	2.7	72	97	3.1	77	119	3.6	80	144	4.0	84	168	4.4	90	198	4.9	95	232	5.5	103	283
3500	3.0	72	108	3.5	77	134	3.9	80	156	4.4	84	185	4.9	90	220	5.3	95	251	5.9	103	304
3000	3.4	72	122	3.8	77	146	4.3	80	172	4.8	84	202	5.4	90	243	5.7	95	269		Rmin = 3050	
2500	3.8	72	137	4.3	77	165	4.8	80	192	5.3	84	223	5.8	90	261	6.0	95	284			
2000	4.3	72	155	4.9	77	188	5.4	80	216	5.8	84	244		Rmin = 2040			Rmin = 2500				
1800	4.6	72	166	5.1	77	196	5.6	80	224	6.0	84	252									
1600	4.9	72	176	5.4	77	207	5.8	80	232		Rmin = 1660										
1400	5.2	72	187	5.7	77	219	6.0	80	240												
1200	5.5	72	198	5.9	77	226		Rmin = 1330													
1000	5.9	72	212		Rmin = 1060																
900	6.0	72	216																		
		Rmin = 833																			

$e_{max} = 6\%$

Source: Adapted from "A Policy on Geometric Design of Highways and Streets", (Reference 3.1), Eq. 3-12 thru 3-24.
For additional information see Reference 3.1, Tables 3-9 & 3-16.

KEY:
V = Assumed design speed
e = Rate of superelevation
L = Tangent runout based on a 24 ft. roadway
Lr = Minimum length of superelevation runoff based on a 24 ft. roadway
NC = Normal crown section

Exhibit 3.5c Superelevation Data for Dual Highways (Tangent Surface)
Depressed Median Width = 54 Foot or Less
Values for Design Elements Related to Design Speed and Horizontal Curvature
(e_{max}=6%)

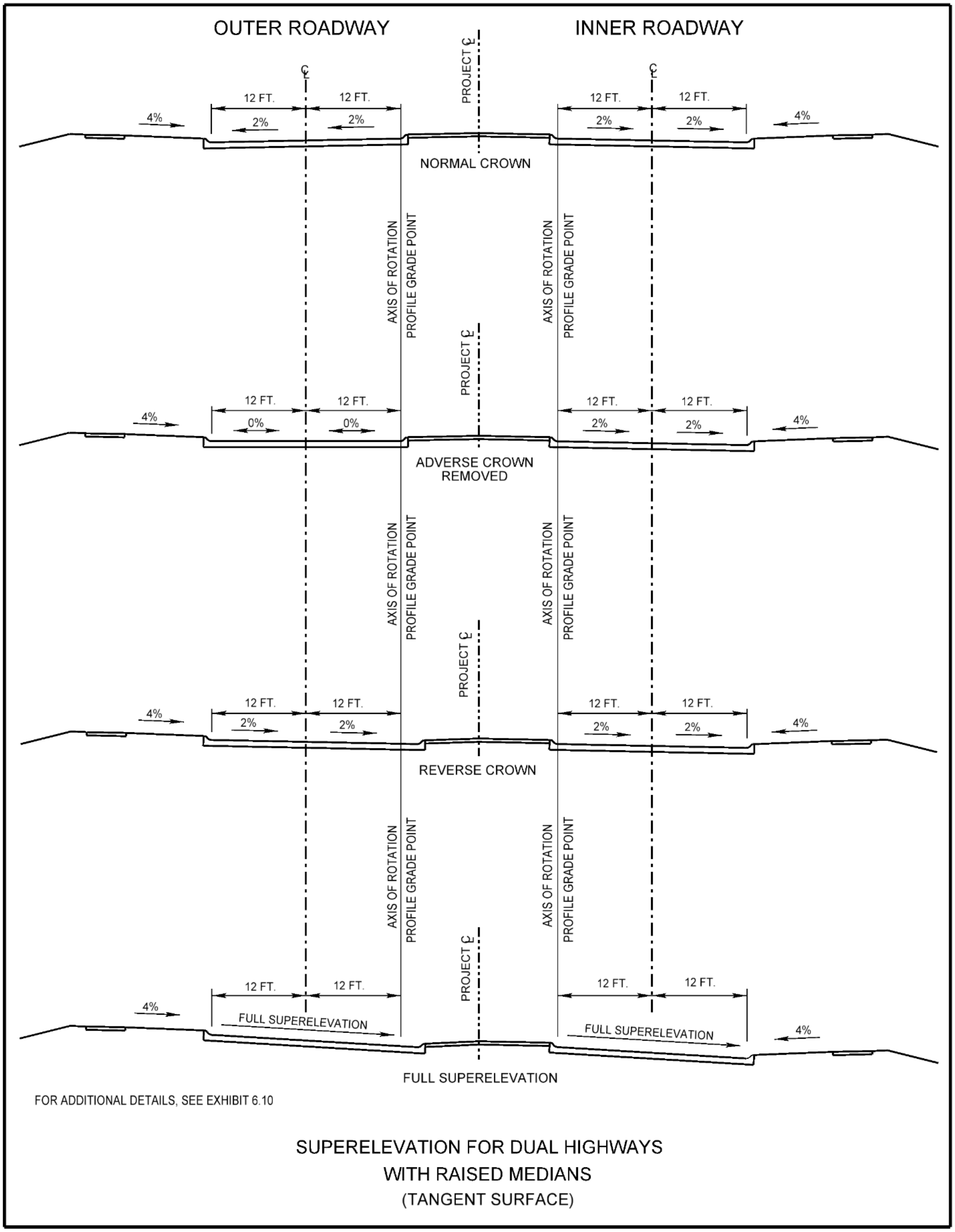


Exhibit 3.6a Superelevation Data for Dual Highways w/ Raised Median Typical Sections

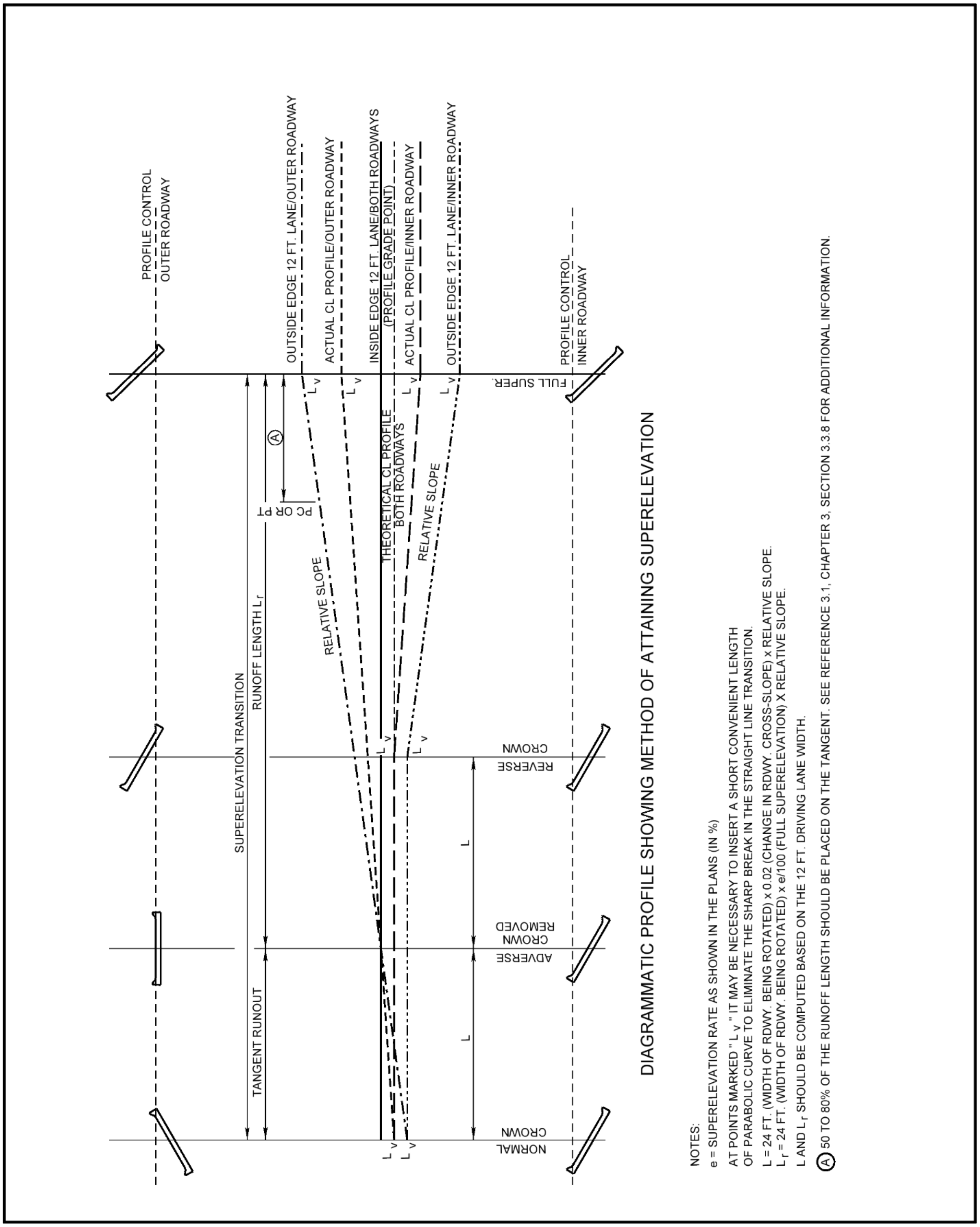


Exhibit 3.6b Superelevation Data for Dual Highways w/ Raised Median Diagrammatic Profile

Radius of Curve (ft.)	V=25 mph			V=30 mph			V=35 mph			V=40 mph			V=45 mph		
	e (%)	Maximum Relative Slope 107.25:1		e (%)	Maximum Relative Slope 114:1		e (%)	Maximum Relative Slope 120.75:1		e (%)	Maximum Relative Slope 129:1		e (%)	Maximum Relative Slope 138.75:1	
		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)		L (ft.)	L_r (ft.)
8000	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0
6000	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	2.0	67	67
4000	NC	0	0	NC	0	0	NC	0	0	2.0	62	62	2.0	67	67
3500	NC	0	0	NC	0	0	2.0	58	58	2.0	62	62	2.2	67	73
3000	NC	0	0	NC	0	0	2.0	58	58	2.1	62	65	2.4	67	80
2500	NC	0	0	2.0	55	55	2.0	58	58	2.3	62	71	2.6	67	87
2000	2.0	51	51	2.0	55	55	2.3	58	67	2.6	62	80	2.8	67	93
1800	2.0	51	51	2.0	55	55	2.4	58	70	2.7	62	84	3.0	67	100
1600	2.0	51	51	2.2	55	60	2.5	58	72	2.8	62	87	3.2	67	107
1400	2.0	51	51	2.3	55	63	2.6	58	75	3.0	62	93	3.3	67	110
1200	2.1	51	54	2.4	55	66	2.8	58	81	3.2	62	99	3.6	67	120
1000	2.3	51	59	2.6	55	71	3.0	58	87	3.4	62	105	3.8	67	127
900	2.4	51	62	2.7	55	74	3.1	58	90	3.6	62	111	3.9	67	130
800	2.4	51	62	2.8	55	77	3.3	58	96	3.7	62	115	4.0	67	133
700	2.5	51	64	3.0	55	82	3.4	58	99	3.9	62	121		Rmin = 711	
600	2.7	51	69	3.2	55	88	3.6	58	104	4.0	62	124			
500	2.8	51	72	3.4	55	93	3.9	58	113		Rmin = 533				
450	2.9	51	75	3.5	55	96	4.0	58	116						
400	3.1	51	80	3.6	55	98	4.0	58	116						
350	3.2	51	82	3.8	55	104		Rmin = 371							
300	3.4	51	88	3.9	55	107									
250	3.6	51	93	4.0	55	109									
200	3.8	51	98		Rmin = 250										
		Rmin = 154													

$e_{max} = 4\%$

Source: Adapted from "A Policy on Geometric Design of Highways and Streets", (Reference 3.1), Eq. 3-12 thru 3-24. For additional information see Reference 3.1, Tables 3-13 & 3-16.

KEY:

V = Assumed design speed
e = Rate of superelevation
L = Tangent runout based on a 24 ft. roadway
Lr = Minimum length of superelevation runoff based on a 24 ft. roadway
NC = Normal crown section

Note: For values for a high-speed roadway (> 45 mph), see Exhibit 3.5c.

Exhibit 3.6c Superelevation Data for Dual Highways w/ Raised Median Values for Design Elements Related to Design Speed and Horizontal Curvature (e_{max}=4%)

3. VERTICAL ALIGNMENT DESIGN

The vertical alignment (profile grade line) is a reference line which establishes the elevation of the pavement and other features of the highway. Vertical alignment is influenced by such factors as:

- Design speed
- Design year traffic volumes
- Environmental concerns
- Location (e.g. in a Flood Plain)
- Topography
- Earthwork
- Horizontal alignment
- Functional classification of the roadway
- Type of improvement
- Vertical clearances
- Geology
- Drainage control
- Construction costs
- Appearance considerations
- Vehicle operating characteristics (trucks)

The performance of heavy vehicles on grades is a significant factor in the development of the vertical alignment.

A practical vertical alignment design will be economically sound and minimize impacts to environmentally sensitive areas while meeting sight distance and other design requirements for the design classification of the highway. Environmental considerations and impacts are a vital component of the design process; the roadway designer shall coordinate with the **Environmental Section** in **PDD** in the development of, and in any subsequent alteration to, the vertical alignment. The vertical alignment should not be changed during the Plan Details Phase (Clarity Activity 5500).

Projects should be designed to produce balanced earthwork whenever practicable. Adjustments to the vertical alignment are usually preferred to ditch widening. For additional discussion see Chapter Seven: <u>Earthwork</u>, Section 1, of this manual.

There should be a smooth transition between the proposed profile grade line and the existing grade line of an adjacent highway section. Connections with previously constructed projects should be compatible with the design speed of the proposed project. A connecting profile grade line should be established which satisfactorily joins to the existing alignment. Existing grade lines should be considered for a distance of 2000 feet or more, if practicable, to address the sight distance beyond the proposed project limits.

See Section 3.4, “Vertical Alignment”, in Chapter 3 of the *Green Book* (Ref. 3.1) for additional information.

3.A Grades

3.A.1 Maximum Grades

A Policy on Design Standards Interstate System (Ref. 3.8) and the *Green Book* (Ref. 3.1) establish the applicable maximum grades for use on the National Highway System (NHS). The **Board of Public Roads Classifications and Standards** establishes the maximum allowable grades for State highways in the Nebraska Minimum Design Standards (*MDS*) (Ref. 3.2) (http://www.roads.nebraska.gov/media/5593/nac-428-rules-regs-nbcs.pdf). Grades steeper than those given shall only be used with an approved design exception from the **Federal Highway Administration** (**FHWA**) and/ or an approved design relaxation from the **Board of Public Roads Classifications and Standards** (See Chapter One: Roadway Design Standards, Section 10, of this manual). Grades which are less than the maximum should be used whenever practicable.

3.A.2 Minimum Grades For Drainage

1. Rural Curbed Roadways and Bridges: A minimum grade of 0.50% is acceptable. Flatter grades may cause stormwater runoff to spread across the traveled way.
2. Urban Curbed Roadways: A minimum grade of 0.35% is acceptable. Flatter grades, down to and including 0.20%, may be used with **Unit Head** approval. As an alternative to a grade flatter than 0.35%, rolling the gutterline and warping the centerline grade line at a minimum slope of 0.35% may be considered.
3. Non-curbed Roadways: Level longitudinal gradients are acceptable where the pavement is crowned 2% or more, provided that consideration is given to the need for special ditches.
4. Superelevation Runout: To facilitate pavement drainage, a minimum grade of 1.5% shall be maintained through the area where the adverse crown has been removed. A flatter grade, down to and including 0.5%, may be used with **Unit Head** approval.

3.A.3 Critical Length of Grade

Critical length of grade is the maximum length of a designated upgrade on which a loaded truck can operate without an unreasonable reduction in speed. For normal design purposes, a maximum speed reduction of 10 mph is acceptable. If this speed reduction is exceeded, the designer should consider shortening the length of grade, flattening the grade, or adding a climbing lane.

3.A.4 Climbing Lanes

The **Traffic Engineering Division** (**Traffic Engineering**) analyzes the need for climbing lanes based on capacity and operations characteristics. The type of roadway facility is usually taken into consideration in the climbing lane analysis. Climbing lanes may be provided on arterial highways based on speed differential and capacity analysis. Climbing lanes may be provided on collector two-lane highways based on speed reduction only. Climbing lanes should be used to provide uniformity of operation rather than to avoid extreme congestion and disruption. Climbing lanes are applicable on multilane highways where extreme grade conditions reduce the level of service below that provided in adjacent roadway sections. Climbing lanes on multilane facilities may not be warranted until several years after construction; for this condition there may be an economic advantage in designing and grading for, but deferring the pavement construction of, climbing lanes on multilane facilities. EXHIBIT 3.7 summarizes **NDOT** standards for the design of climbing lanes. See Section 3.4.3, “Climbing Lanes”, in Chapter 3 of the *Green Book* (Ref. 3.1) for additional information.

Design Element	Desirable	Minimum
Lane Width	Same as approach roadway.	Same as approach roadway.
Shoulder Width	Same as approach roadway. For turf shoulders, same as minimum requirement.	Expressway: Same as approach roadway. Other: 4 feet paved plus a 2 feet turf transition. (4)
Cross Slope on Tangent	Same as adjacent travel lane.	Same as adjacent travel lane.
Superelevation	(1)	(1)
Beginning of Full-Width Lane	Near the Vertical Point of Tangency (VPT) of the grade.	Where the truck speed is 10 mph below highway design speed or is at 45 mph, whichever is less. (2)
End of Full-Width Lane (3)	Where the truck has reached highway design speed.	Where the truck has reached 10 mph below highway design speed. (2)
Entering Taper	1:25	300 feet
Exiting Taper	1:50	600 feet
Minimum Full-Width Length (3)	NA	1000 feet

1. For horizontal curves on climbing lanes, determine the proper superelevation of the climbing lane on the high side by reading the applicable superelevation table for V = 40 mph or the design speed, whichever is less (See Section 2.B of this chapter). This reflects the slower operating speeds of the climbing lane. The maximum allowable difference in cross slope between the travel lane and the climbing lane is 4%.
2. See Chapter 3 of A Policy on Geometric Design of Highways and Streets (Ref. 3.1), **FIGURE 3-21**, to determine truck deceleration and acceleration rates.
3. The designer will provide sufficient decision sight distance and length of auxiliary lane past the crest of a vertical curve to allow for the completion of the merger maneuver. The designer should coordinate the design of the auxiliary lane with **Traffic Engineering**.
4. If there is a mailbox located within the climbing lane, the mailbox turnout width will be 10 feet, 8 feet of which will be surfaced. See the Standard/Special Plans Book (*Standard Plans*) (Ref. 3.3), Standard Plan No. 307 (http://www.roads.nebraska.gov/business-center/design-consultant/stand-spec-manual/).

Exhibit 3.7 Standards for Climbing Lanes

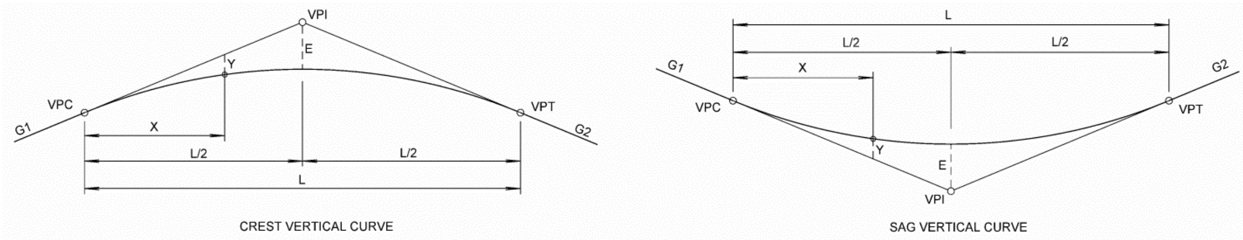
3.B Vertical Curves

Vertical curves join two intersecting tangents to provide a smooth transition between changes in grade. Vertical curves are not required on low-speed roadways (≤ 45 mph) where the algebraic difference in grades is less than 1%; high-speed roadways (≥ 50 mph) will generally require a vertical curve when the change in grade is greater than 0.5%. Curves must be long enough to provide desirable stopping sight distance but should not be flattened to such an extent as to make drainage a problem. A series of angular breaks in lieu of a vertical curve is not an acceptable design. For additional information see NCHRP Synthesis 299, Recent Geometric Design Research for Improved Safety and Operations (Ref. 3.7).

3.B.1 Vertical Curve Computations

NDOT uses the symmetrical parabolic curve. Symmetrical curves are those with equal tangent lengths at the point where the curve is divided by the vertical point of intersection (VPI) of the two tangents. Asymmetrical vertical curves (curves with unequal tangent lengths) are sometimes used in unique situations, such as to provide vertical clearance at a bridge structure where a symmetrical curve will not work. Equations for asymmetric curves may be found in many engineering manuals. Compound curves designed in the vertical plane may also be used in these situations.

Dimensions and equations for use in symmetrical vertical curve computations are shown in EXHIBIT 3.8. The symbols, abbreviations, and formulas apply to both crest and sag vertical curves. It is **NDOT** practice to design vertical curves so that the vertical PC and PT fall on even stationing.



Vertical Curve Symbols and Abbreviations	**Vertical Curve Formulas**
VPC Vertical point of curvature (beginning of curve) VPI Vertical point of intersection of grades VPT Vertical point of tangency (end of curve) G1 Grade at beginning of curve, ft./ft. G2 Grade at end of curve, ft./ft. E External distance from VPI to curve L Horizontal length of curve R Rate of change of grade, ft./ft. E_O Elevation of VPC V Elevation of VPI E_T Elevation of VPT X Distance of any point on the curve from the VPC Y Tangent offset to curve at X distance from the VPC E_X Elevation of any point X distance from the VPC X_S Distance from the VPC to lowest point of a sag curve or highest point of a crest curve E_S Elevation of lowest point on a sag curve or highest point on a crest curve A Algebraic difference of grades, %	$V = E_O + [L/2 \times G1]$ $E_T = V - [L/2 \times G2]$ $R = (G2 - G1) / L$ $E_X = E_O + G1X + {}^{1}/_{2}RX^{2}$ $E_S = E_O - [G1^2 \div 2R]$ $X_S = -G1/R$ *(if X_S is negative or if $X_S > L$, the curve doesn't have a high point or a low point)* $A = G1 - G2$ $E = AL/800$ $Y = (X \div L/2)^2 \times E$

Exhibit 3.8 Elements and Formulas for Symmetrical Parabolic Vertical Curves

3.B.2 Design

The primary design control for the minimum length of a vertical curve is stopping sight distance. The minimum vertical curve length in feet should be approximately three times the design speed of the roadway when the length of curve is less than the desirable stopping sight distance. See Section 3.2.2, “Stopping Sight Distance”, in Chapter 3 of the *Green Book* (Ref. 3.1) for additional information.

The relationship of the curve length (L) and the algebraic difference in percent of grades (A) or L/A, is termed the K value. The K value is a measure of curvature, defined as the horizontal distance in feet required to effect a 1% change in grade. K values for roadways on new alignment, with minimum and desirable stopping sight distance for crest and sag vertical curves, are shown in EXHIBITS 3.9 & 3.14 for a range of design speeds and intersection configurations.

The desirable K values shown in EXHIBITS 3.9 & 3.14 should be used for all New and Reconstructed projects. The desirable K values provide intersection stopping sight distance for passenger cars for various intersection conditions. If the desirable K values cannot be met, the vertical curve may be designed to any length down to and including stopping sight distance with **Unit Head** approval and a decision letter to the project file (See Chapter One: Roadway Design Standards, Section 10.C, of this manual). For intersection conditions other than listed in the exhibits, intersections and driveways (except for field entrances) will be evaluated for intersection sight distance according to the procedures presented in Section 9.5, “Intersection Sight Distance”, in Chapter 9 of the *Green Book* (Ref. 3.1).

The minimum K values shown in EXHIBITS 3.9 & 3.14 are from **TABLES 3-35 AND 3-37** in the *Green Book* (Ref. 3.1) and are for stopping sight distance **only** and do not include intersection sight distance. When the minimum K values are used for New and Reconstructed projects, intersections and driveways (except for field entrances) will be evaluated for intersection sight distance according to the procedures presented in Section 9.5, “Intersection Sight Distance”, in Chapter 9 of the *Green Book* (Ref. 3.1). The use of the minimum K values will require **Unit Head** approval and a decision letter to the project file (See Chapter One: Roadway Design Standards, Section 10.C, of this manual).

The use of K values below the stopping sight values given in EXHIBITS 3.9 & 3.14 for a New and Reconstructed project will require **Roadway Design Engineer** approval, a design exception from the **FHWA** for projects on the NHS, and/ or a relaxation of the *MDS* (Ref. 3.2) (See Chapter One: Roadway Design Standards, Section 10.C, of this manual).

The use of an intersection sight distance less than that given in **TABLE 9-7** in Chapter 9 of the *Green Book* (Ref. 3.1) will require **Unit Head** approval and a decision letter to the project file (See Chapter One: Roadway Design Standards, Section 10.C, of this manual).

Special attention to pavement drainage must be exercised where a K value in excess of 167 is used, a minimum roadway cross-slope of 1.5% should be maintained.

U.S. Customary								
Crest Vertical Curve								
Design Speed (mph)	① Minimum Stopping Sight Distance		② Desirable Stopping Sight Distance (2-Lane, Left-Turn Condition and 2-Lane w/ TWLTL, Left-Turn Condition)		③ Desirable Stopping Sight Distance (5-Lane and 4-Lane Divided w/ 18 ft. Median, Left-Turn Condition)		④ Desirable Stopping Sight Distance (4-Lane Divided w/ 40 ft. Median, Left-Turn Condition) Crowned Lanes **	
	Length (ft.)	Rate of Vertical Curvature K	Length (ft.)	Rate of Vertical Curvature K	Length (ft.)	Rate of Vertical Curvature K	Length (ft.)	Rate of Vertical Curvature K
15	80	3	195	18	215	21	240	27
20	115	7	260	31	285	38	320	47
25	155	12	325	49	355	58	400	74
30	200	19	390	70	430	86	485	109
35	250	29	455	96	500	116	565	148
40	305	44	520	125	570	151	645	193
45	360	61	580	156	640	190	725	244
50	425	84	645	193	715	237	805	300
55	495	114	710	234	785	286	890	367
60	570	151	775	278	855	339	970	436
65	645	193	840	327	925	396	1,050	510
70	730	247	905	380	1,000	463	1,130	592
75	820	312	970	436	1,070	531	1,215	684
80	910	384	1,035	496	1,140	602	1,290	770

K is the length of curve per percent algebraic difference in grades (A). K = L/ A

TWLTL = Two Way Left-Turn Lane, See Chapter Four: Intersections, Driveways and Channelization, Section 5.B.2, of this manual.

** Check intersection sight distance if building tangent lanes.

① Based on a 3.5 ft. eye height and an object height of 2 ft., (See A Policy on Geometric Design of Highways and Streets, Ref. 3.1, Section 3.2.6). These values do not meet intersection sight distance requirements. All intersections and driveways, except for field entrances, shall be evaluated for intersection sight distance according to the procedures outlined in Chapter 9 of Ref. 3.1. Minimum K values may be used on New and Reconstructed projects with **Design Unit Head** approval.

② Includes intersection sight distance for the given conditions (for other intersection conditions, see Chapter Four: Intersections, Driveways and Channelization of this manual and Chapter 9 of Ref. 3.1). Based on an 8.8 sec. gap in traffic, a 2.33 ft. eye height and an object height of 3.25 ft. See Exhibit 3.10a of this manual for the derivation of the eye and object heights. The use of K values for less than the desirable stopping sight distance requires **Design Unit Head** approval.

③ Includes intersection sight distance for the given conditions (for other intersection conditions, see Chapter Four: Intersections, Driveways and Channelization of this manual and Chapter 9 of Ref. 3.1). Based on a 9.7 sec. gap in traffic, a 2.0 ft. eye height and an object height of 2.9 ft. See Exhibit 3.10b of this manual for the derivation of the eye and object heights. The use of K values for less than the desirable stopping sight distance requires **Design Unit Head** approval.

④ Includes intersection sight distance for the given conditions (for other intersection conditions, see Chapter Four: Intersections, Driveways and Channelization of this manual and Chapter 9 of Ref. 3.1). Based on a 11.0 sec. gap in traffic, a 2.50 ft. eye height and an object height of 3.38 ft. See Exhibit 3.10c of this manual for the derivation of the eye and object heights. The use of K values for less than the desirable stopping sight distance requires **Design Unit Head** approval.

Note: The **NDOT** time gap is arrived at by adding an initial time gap of 7.5 sec. (Case B-1, Left-Turn from the Minor Road condition from Chapter 9 of Ref. 3.1) plus 0.8 sec. (0.2 sec. x 4 for a driveway/ intersection approach at a 4% grade) plus 0.5 sec. for each additional 12 ft. lane (or for each 12 ft. width of median).

Exhibit 3.9a Design Controls for Crest Vertical Curves

U.S. Customary						
Crest Vertical Curve						
Design Speed (mph)	⑤ Desirable Stopping Sight Distance (4-Lane Divided w/ 50 ft. Median, Left-Turn Condition) Crowned Lanes **		Intersection Sight Distance (4-Lane Divided w/ 54 ft. Median)			
			⑥ Case B-3 Crossing Maneuver (Traversing the Traffic & Rt.-Turn Lane to Enter the Median Refuge)		⑦ Case B-1 Left-Turn Condition (From the Median)	
	Length (ft.)	Rate of Vertical Curvature K	Length (ft.)	Rate of Vertical Curvature K	Length (ft.)	Rate of Vertical Curvature K
15	250	29	170	13	170	13
20	330	50	225	23	225	23
25	415	80	285	38	280	36
30	500	116	340	54	335	52
35	580	156	395	72	390	70
40	665	205	455	96	445	92
45	745	257	510	120	500	116
50	830	319	565	148	555	143
55	915	388	620	178	610	172
60	995	459	680	214	665	205
65	1,080	540	735	250	720	240
70	1,160	624	790	289	775	278
75	1,245	718	850	335	830	319
80	1,330	820	# 910	384	# 910	384

K is the length of curve per percent algebraic difference in grades (A). K = L/ A

TWLTL = Two Way Left-Turn Lane, See Chapter Four: Intersections, Driveways and Channelization, Section 5.B.2, of this manual.

** Check intersection sight distance if building tangent lanes.

The stopping sight distance was substituted for the intersection sight distance, which is less than the stopping sight distance.

⑤ Includes intersection sight distance for the given conditions (for other intersection conditions, see Chapter Four: Intersections, Driveways and Channelization of this manual and Chapter 9 of Ref. 3.1). Based on a 11.3 sec. gap in traffic, a 2.50 ft. eye height and an object height of 3.38 ft. See Exhibit 3.10c of this manual for the derivation of the eye and object heights. The use of K values for less than the desirable stopping sight distance requires **Design Unit Head** approval.

Note: The **NDOT** time gap is arrived at by adding an initial time gap of 7.5 sec. (Case B-1, Left-Turn from the Minor Road condition from Chapter 9 of Ref. 3.1) plus 0.8 sec. (0.2 sec. x 4 for a driveway/ intersection approach at a 4% grade) plus 0.5 sec. for each additional 12 ft. lane (or for each 12 ft. width of median).

⑥ The Crossing Maneuver time gap of 7.7 sec. (provided for the Type "A" Median Break) is arrived at by adding an initial time gap of 6.5 sec. (Case B-3, Crossing Maneuver from the Minor Road condition from Chapter 9 of Ref. 3.1) plus 1.2 sec. (4 ft. shoulder/ 8 ft. median/ 4 ft. shoulder and 12 ft. turn lane at 0.5 sec. per each additional 12 ft. of width).

⑦ Source: **Table 9-7** of the *Green Book* (Ref. 3.1). Based on a time gap of 7.5 sec. (**Table 9-6**)

Exhibit 3.9b Design Controls for Crest Vertical Curves

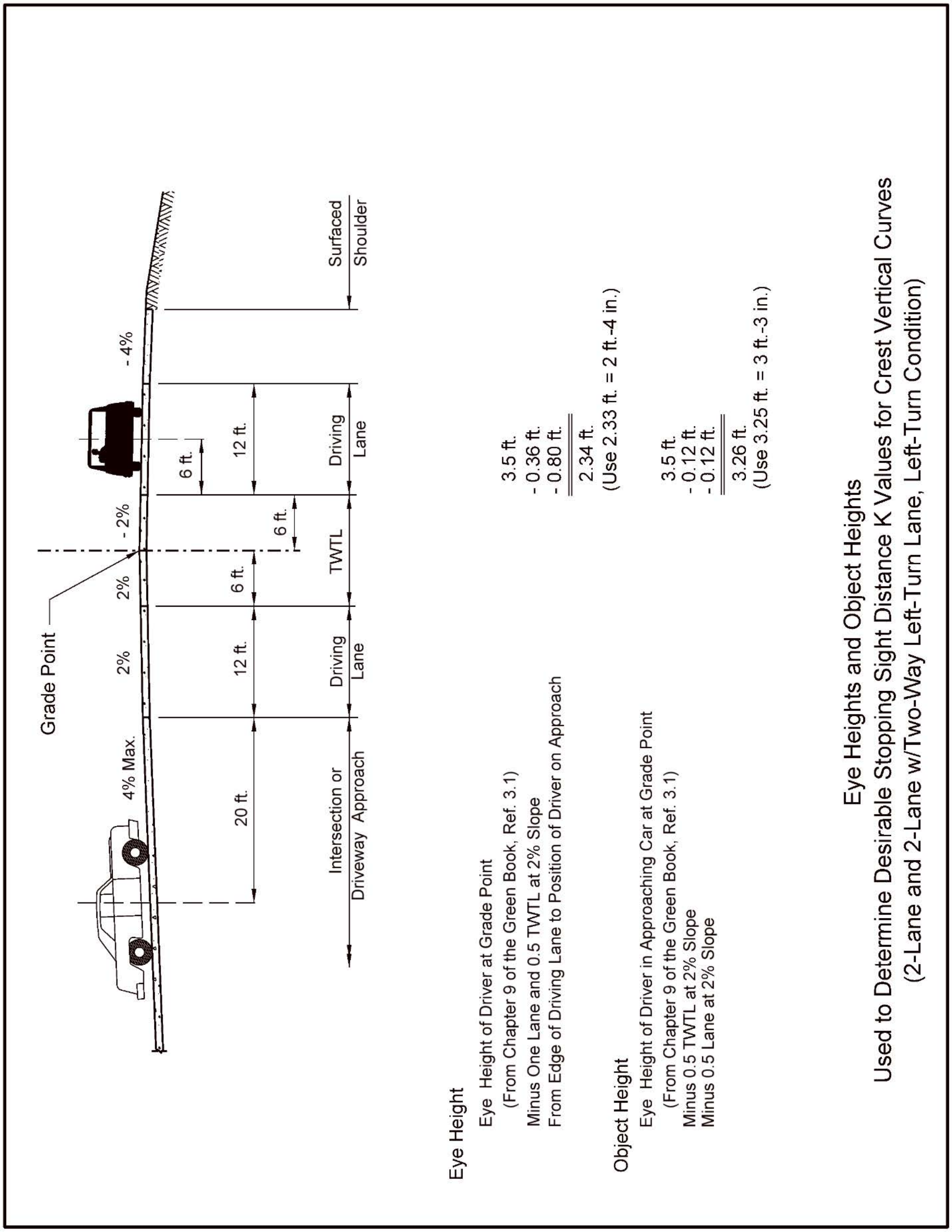


Exhibit 3.10a Eye and Object Heights used to Determine Desirable K Values (Crest Vertical Curves)

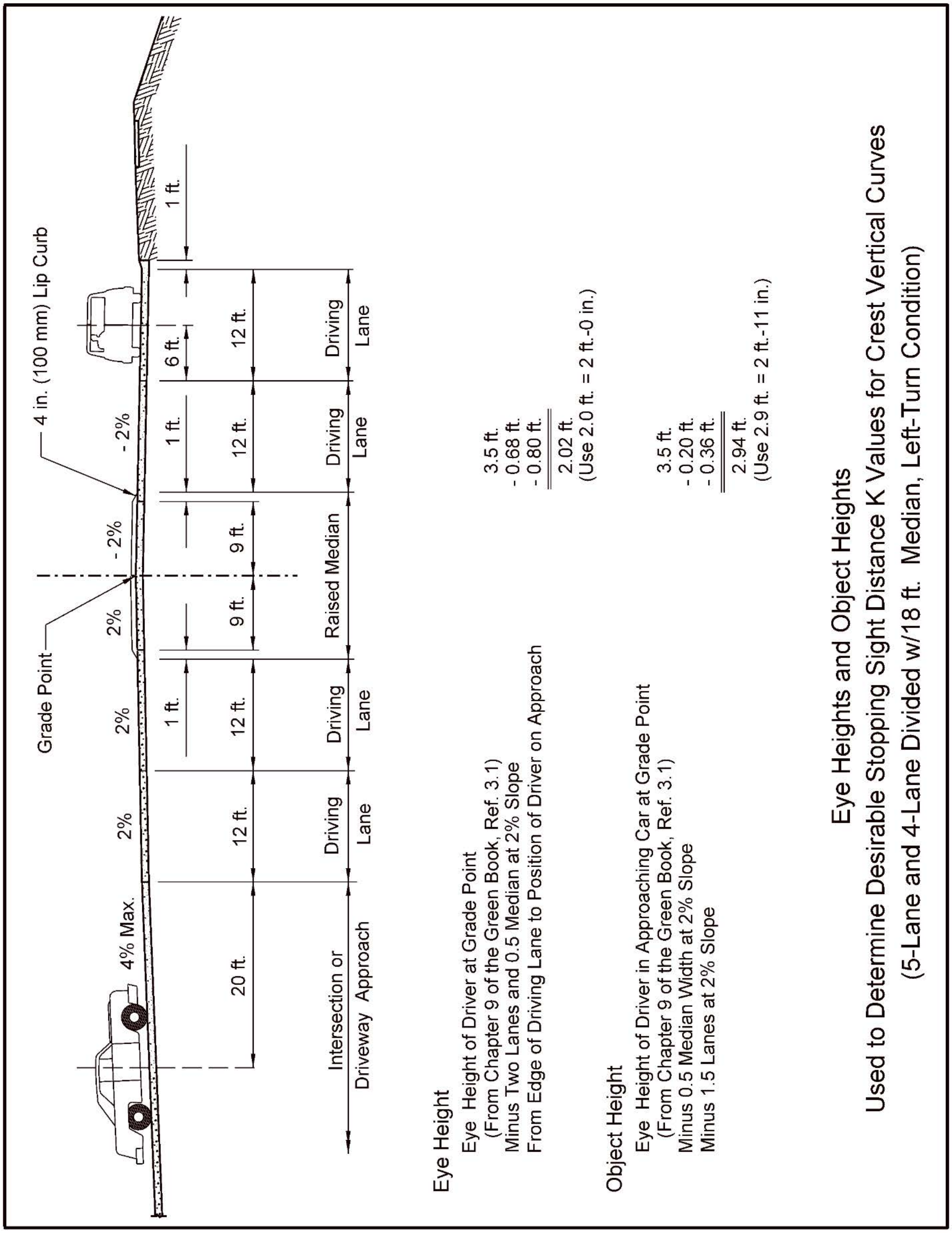


Exhibit 3.10b Eye and Object Heights used to Determine Desirable K Values (Crest Vertical Curves)

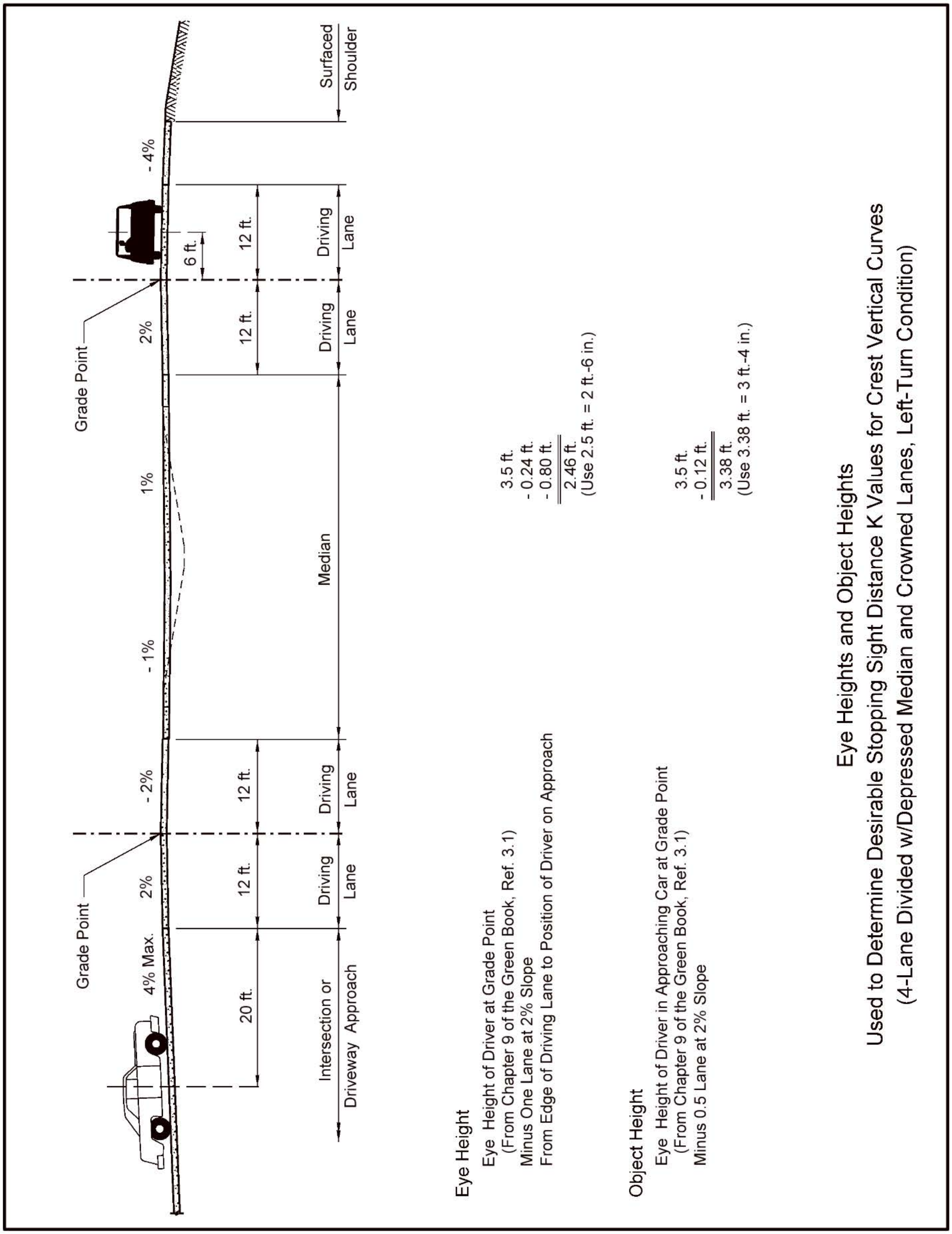


Exhibit 3.10c Eye and Object Heights used to Determine Desirable K Values (Crest Vertical Curves)

3.C Crest Vertical Curves

3.C.1 Stopping Sight Distance

EXHIBIT 3.11 depicts the stopping sight distance height of eye and height of object assumptions for crest vertical curves. The equation given below may be used to determine the length of crest vertical curve which provides the **NDOT desirable** stopping sight distance at each design speed (desirable stopping sight distance includes intersection sight distance).

S_d = 1.47 x speed (mph) x time (sec) (Eq. 9-1 in the *Green Book*, Ref. 3.1)

Where:

S_d = stopping sight distance = intersection sight distance in feet.
Time = acceptable time gap in traffic (in seconds) based on intersection conditions (See Chapter Four: Intersections, Driveways and Channelization of this manual and Chapter 3 of the *Green Book*, Ref. 3.1). **NDOT** uses a time gap of 8.8 sec. for the two-lane left-turn condition (the **NDOT** time gap is arrived at by adding an initial time gap of 7.5 sec. (Case B1, left-turn condition from Section 9.5.3 in Chapter 9 of the *Green Book*, Ref. 3.1) plus 0.8 sec. (0.2 sec. x 4 for a driveway/ intersection approach at a 4% grade) plus 0.5 sec. for each additional 12-foot lane (or for each 12 feet of median width).

Equations 3-43 & 3-44 in the *Green Book* (Ref. 3.1) may be used to determine the stopping sight distance for crest vertical curves.

When designing a crest vertical curve, the designer should refer to EXHIBIT 3.9 in choosing an appropriate K value. Multiply K by A (algebraic difference in grade) to obtain the desirable length of vertical curve (L).

The following equation is used to determine a K value for a condition not shown in EXHIBIT 3.9:

$K = S_d^2 / 2158$

This equation is derived from Eq. 3-44 in the *Green Book* (Ref. 3.1), $L = AS^2 / 2158$, substituting L / K for A and intersection sight distance for S.

When the desirable sight distance cannot be attained, the vertical curve may be designed to any length down to and including the stopping sight distance shown in EXHIBIT 3.9 with **Unit Head** approval and a decision letter to the project file (See Chapter One: Roadway Design Standards, Section 10.C, of this manual).

The minimum K values shown in EXHIBIT 3.9 are from **TABLE 3-35** in the *Green Book* (Ref. 3.1) and are for stopping sight distance **only** and do not include intersection sight distance. For intersection conditions other than listed in the exhibit, intersections and driveways on New and Reconstructed projects (except for field entrances) will be evaluated for intersection sight distance according to the procedures presented in Section 9.5, “Intersection Sight Distance”, in Chapter 9 of the *Green Book* (Ref. 3.1). The use of an intersection sight distance less than that given in Chapter 9 of the *Green Book* (Ref. 3.1) will require **Unit Head** approval and a decision letter to the project file (See Chapter One: Roadway Design Standards, Section 10.C, of this manual).

The use of K values below the sight distance values given in EXHIBIT 3.9 for a New and Reconstructed project will require **Roadway Design Engineer** approval and may require a relaxation of the *MDS* (Ref. 3.2) and/ or design exception from the **FHWA** (See Chapter One: Roadway Design Standards, Section 10.C, of this manual).

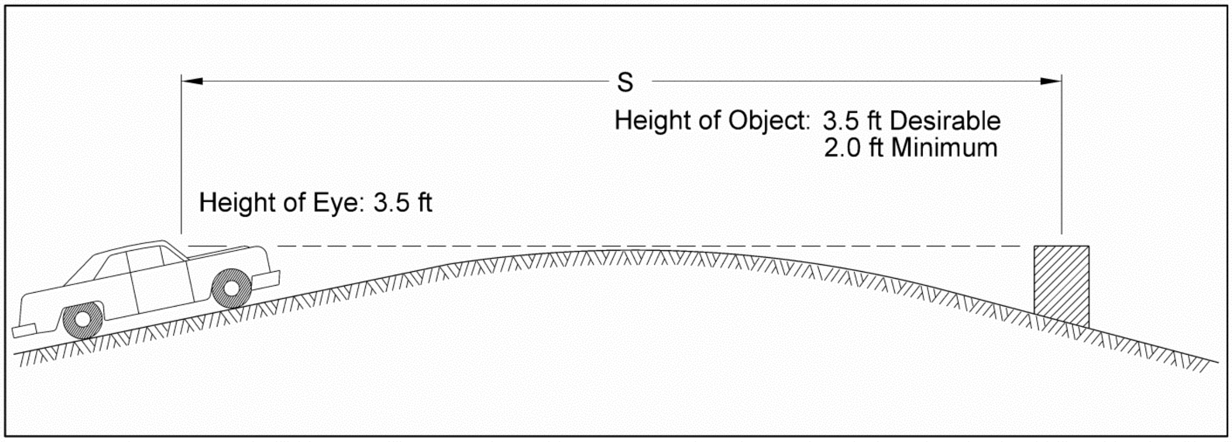


Exhibit 3.11 Crest Vertical Curve Design Elements

3.C.2 Two-Lane, Two-Way Roadways - Passing Sight Distance

Passing sight distance is not one of the principal controlling design criteria (See Chapter One: Roadway Design Standards, Section 8, of this manual). It is not practicable to design crest vertical curves to provide for passing sight distance. For evaluation, **TABLE 3-4** and Equations 3-46 and 3-47 in the Green Book (Ref. 3.1) may be used to determine the minimum length of vertical curve for passing sight distance. See Section 3.2.4, "Passing Sight Distance for Two-Lane Highways", in Chapter 3 of the Green Book (Ref. 3.1) for additional information.

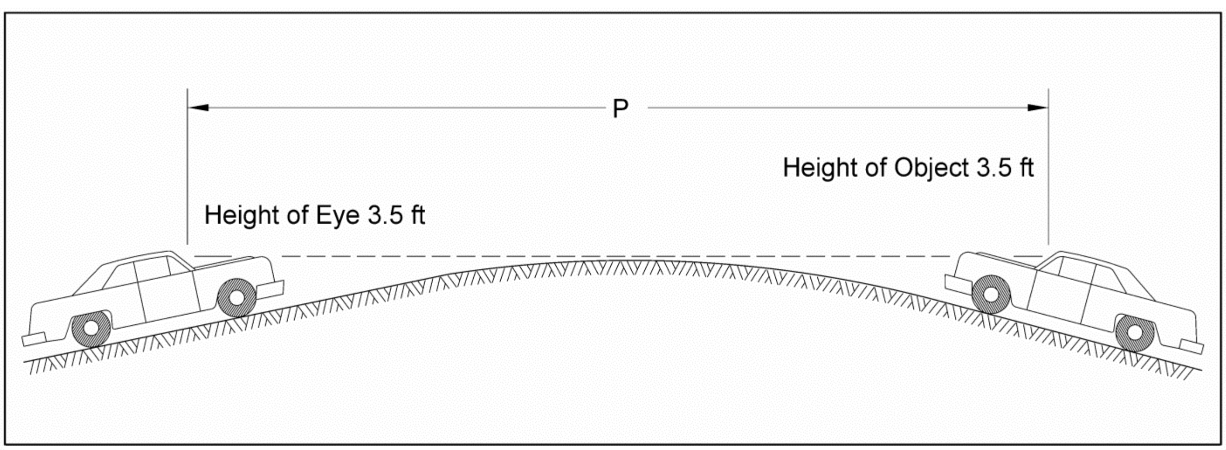


Exhibit 3.12 Passing Sight Distance Design Considerations

3.D Sag Vertical Curves

3.D.1 Stopping Sight Distance

The minimum length of a sag vertical curve depends mainly on headlight distance at night except in areas where roadway lighting is present (See EXHIBIT 3.13). For overall safety on highways, it is desirable that the stopping sight distance be nearly the same as the headlight beam distance.

The equation given below may be used to calculate the length of sag vertical curve which provides the **NDOT desirable** stopping sight distance at each design speed (desirable stopping sight distance includes intersection sight distance).

S_d = 1.47 x speed (mph) x time (sec) (Eq. 9-1 in the *Green Book*, Ref. 3.1)

Where:

S_d = headlight beam distance = stopping sight distance in feet.

Time = acceptable time gap in traffic (in seconds) based on intersection conditions (See Section 9.5.3 in Chapter 9 of the *Green Book*, Ref. 3.1). **NDOT** uses a time gap of 8.8 sec. for the two-lane, left-turn condition (the **NDOT** time gap is arrived at by adding an initial time gap of 7.5 sec. (Case B-1, left-turn condition from Section 9.5.3 in Chapter 9 of the *Green Book*, Ref. 3.1) plus 0.8 sec. (0.2 sec. x 4 for a driveway/ intersection approach at a 4% grade) plus 0.5 sec. for each additional 12-foot lane (or for each 12 feet of median width).

Equations 3-48 & 3-50 in the *Green Book* (Ref. 3.1) may be used to determine the stopping sight distance for sag vertical curves.

EXHIBIT 3.14 lists the desirable sight distances and the stopping sight distances for various design speeds. When the desirable sight distance cannot be attained, the vertical curve may be designed down to and including the stopping sight distance with **Unit Head** approval and a decision letter to the project file (See Chapter One: Roadway Design Standards, Section 10.C, of this manual).

The minimum K values shown in EXHIBIT 3.14 are for stopping sight distance **only** and do not include intersection sight distance. For intersection conditions other than those listed in the exhibit, intersections and driveways on New and Reconstructed projects (except for field entrances) will be evaluated for intersection sight distance according to the procedures presented in Section 9.5, "Intersection Sight Distance", in Chapter 9 of the *Green Book* (Ref. 3.1). The use of an intersection sight distance less than that given in the *Green Book* (Ref. 3.1) will require **Unit Head** approval and a decision letter to the project file (See Chapter One: Roadway Design Standards, Section 10.C, of this manual).

The following equation is used to determine a K value for a condition not shown in EXHIBIT 3.14:

$$K = S_d^2 / (400 + 3.5S_d)$$

This equation is derived from Eq. 3-49 in the Green Book (Ref. 3.1), $L = AS^2 / 400 + 3.5S$, substituting L / K for A and intersection sight distance for S.

The use of K values below the sight distance values given in EXHIBIT 3.14 for a New and Reconstructed project will require **Roadway Design Engineer** approval and may require a relaxation of the *MDS* (Ref. 3.2) and/ or design exception from the **FHWA** (See Chapter One: Roadway Design Standards, Section 10.C, of this manual).

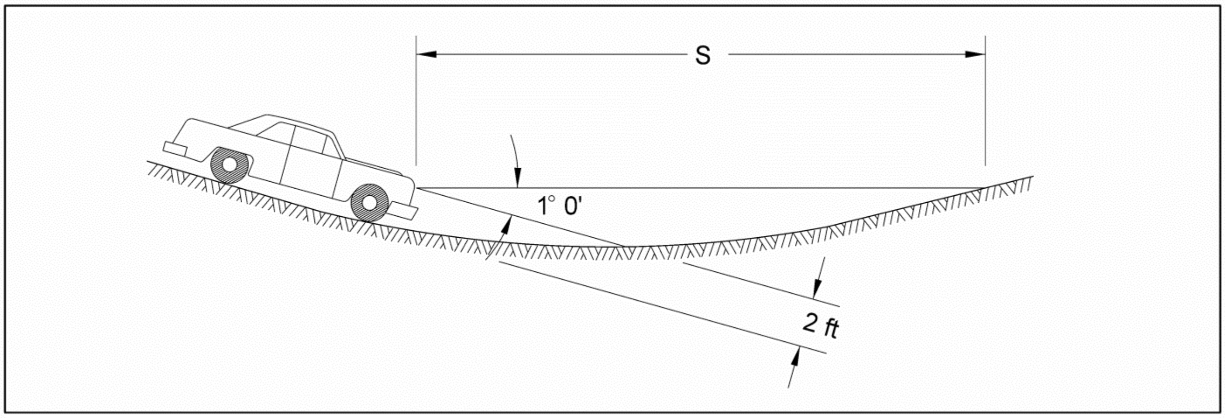


Exhibit 3.13 Sag Vertical Curve Design Elements

U.S. Customary								
Sag Vertical Curve								
Design Speed (mph)	① Minimum Stopping Sight Distance		② Desirable Stopping Sight Distance (2-Lane, Left-Turn Condition and 2-Lane w/ TWLTL, Left-Turn Condition)		③ Desirable Stopping Sight Distance (5-Lane and 4-Lane Divided w/ 18 ft. Median, Left-Turn Condition)		④ Desirable Stopping Sight Distance (4-Lane Divided w/ 40 ft. Median, Left-Turn Condition)	
	Length (ft.)	Rate of Vertical Curvature K	Length (ft.)	Rate of Vertical Curvature K	Length (ft.)	Rate of Vertical Curvature K	Length (ft.)	Rate of Vertical Curvature K
15	80	10	195	35	215	40	240	46
20	115	17	260	52	285	58	320	67
25	155	26	325	69	355	77	400	89
30	200	37	390	86	430	97	485	112
35	250	49	455	104	500	116	565	134
40	305	64	520	122	570	136	645	156
45	360	79	580	138	640	155	725	179
50	425	96	645	157	715	176	805	201
55	495	115	710	175	785	196	890	225
60	570	136	775	193	855	215	970	248
65	645	157	840	211	925	235	1,050	270
70	730	181	905	230	1,000	256	1,130	293
75	820	206	970	248	1,070	276	1,215	317
80	910	231	1,035	266	1,140	296	1,290	339

K is the length of curve per percent algebraic difference in grades (A). K = L/ A

TWLTL = Two Way Left-Turn Lane, See Chapter Four: Intersections, Driveways and Channelization, Section 5.B.2, of this manual.

① Based on a 3.5 ft. eye height, a 2 ft. headlight height and a 1° upward divergence of the light beam, (See A Policy on Geometric Design of Highways and Streets, Ref. 3.1, Section 3.4.6.3). These values do not meet intersection sight distance requirements. All intersections and driveways, except for field entrances, shall be evaluated for intersection sight distance according to the procedures outlined in Chapter 9 of Ref. 3.1. Minimum K values may be used on New and Reconstructed projects with **Design Unit Head** approval.

② Includes intersection sight distance for the given conditions (for other intersection conditions, see Chapter Four: Intersections, Driveways and Channelization of this manual and Chapter 9 of Ref. 3.1). Based on an 8.8 sec. gap in traffic, a 3.5 ft. eye height, a 2 ft. headlight height and a 1° upward divergence of the light beam (See Ref. 3.1, Chapter 3). The light beam distance has been set to the intersection sight distance. The use of K values for less than the desirable stopping sight distance requires **Design Unit Head** approval.

③ Includes intersection sight distance for the given conditions (for other intersection conditions, see Chapter Four: Intersections, Driveways and Channelization of this manual and Chapter 9 of Ref. 3.1). Based on a 9.7 sec. gap in traffic, a 3.5 ft. eye height, a 2 ft. headlight height and a 1° upward divergence of the light beam (See Ref. 3.1, Chapter 3). The light beam distance has been set to the intersection sight distance. The use of K values for less than the desirable stopping sight distance requires **Design Unit Head** approval.

④ Includes intersection sight distance for the given conditions (for other intersection conditions, see Chapter Four: Intersections, Driveways and Channelization of this manual and Chapter 9 of Ref. 3.1). Based on a 11.0 sec. gap in traffic, a 3.5 ft. eye height, a 2 ft. headlight height and a 1° upward divergence of the light beam (See Ref. 3.1, Chapter 3). The light beam distance has been set to the intersection sight distance. The use of K values for less than the desirable stopping sight distance requires **Design Unit Head** approval.

Note: The **NDOT** time gap is arrived at by adding an initial time gap of 7.5 sec. (Case B-1, Left-Turn from the Minor Road condition from Chapter 9 of Ref. 3.1) plus 0.8 sec. (0.2 sec. x 4 for a driveway/ intersection approach at a 4% grade) plus 0.5 sec. for each additional 12 ft. lane (or for each 12 ft. width of median).

Exhibit 3.14a Design Controls for Sag Vertical Curves

U.S. Customary Sag Vertical Curve						
Design Speed (mph)	⑤ Desirable Stopping Sight Distance (4-Lane Divided w/ 50 ft. Median, Left-Turn Condition) Crowned Lanes **		Intersection Sight Distance (4-Lane Divided w/ 54 ft. Median)			
			⑥ Case B-3 Crossing Maneuver (Traversing the Traffic & Rt.-Turn Lane to Enter the Median Refuge)		⑦ Case B-1 Left-Turn Condition (From the Median)	
	Length (ft.)	Rate of Vertical Curvature K	Length (ft.)	Rate of Vertical Curvature K	Length (ft.)	Rate of Vertical Curvature K
15	250	49	170	29	170	29
20	330	70	225	43	225	43
25	415	93	285	58	280	57
30	500	116	340	73	335	71
35	580	138	395	88	390	86
40	665	162	455	104	445	101
45	745	184	510	119	500	116
50	830	208	565	134	555	131
55	915	232	620	150	610	147
60	995	255	680	166	665	162
65	1,080	279	735	182	720	178
70	1,160	302	790	197	775	193
75	1,245	326	850	214	830	208
80	1,330	350	# 910	231	# 910	231

K is the length of curve per percent algebraic difference in grades (A). K = L/ A

TWLTL = Two Way Left-Turn Lane, See Chapter Four: Intersections, Driveways and Channelization, Section 5.B.2, of this manual.

** Check intersection sight distance if building tangent lanes.

\# The stopping sight distance was substituted for the intersection sight distance, which is less than the stopping sight distance.

⑤ Includes intersection sight distance for the given conditions (for other intersection conditions, see Chapter Four: Intersections, Driveways and Channelization of this manual and Chapter 9 of Ref. 3.1). Based on a 11.3 sec. gap in traffic, a 2.50 ft. eye height and an object height of 3.38 ft. See Exhibit 3.10c of this manual for the derivation of the eye and object heights. The use of K values for less than the desirable stopping sight distance requires **Design Unit Head** approval.

Note: The **NDOT** time gap is arrived at by adding an initial time gap of 7.5 sec. (Case B-1, Left-Turn from the Minor Road condition from Chapter 9 of Ref. 3.1) plus 0.8 sec. (0.2 sec. x 4 for a driveway/ intersection approach at a 4% grade) plus 0.5 sec. for each additional 12 ft. lane (or for each 12 ft. width of median).

⑥ The Crossing Maneuver time gap of 7.7 sec. (provided for the Type “A” Median Break) is arrived at by adding an initial time gap of 6.5 sec. (Case B-3, Crossing Maneuver from the Minor Road condition from Chapter 9 of Ref. 3.1) plus 1.2 sec. (4 ft. shoulder/ 8 ft. median/ 4 ft. shoulder and 12 ft. turn lane at 0.5 sec. per each additional 12 ft. of width).

⑦ Source: **Table 9-7** of the *Green Book* (Ref. 3.1). Based on a time gap of 7.5 sec. (**Table 9-6**)

Exhibit 3.14b Design Controls for Sag Vertical Curves

3.D.2 Comfort Criteria

Driver/ passenger comfort is affected more by the change in vertical direction on sag vertical curves than on crest vertical curves. This is because the gravitational and centripetal forces are combining rather than opposing. The length of a sag vertical curve required to meet the comfort criteria is approximately 50% of that required to satisfy the stopping sight distance requirement (headlight distance). The minimum length of a sag vertical curve based on comfort criteria may be calculated using Equation 3-52 in the *Green Book* (Ref. 3.1).

3.D.3 Underpass Sight Distance

The designer should verify that, when sag vertical curves are used on underpasses, the overhead structure does not obstruct driver visibility. Equations 3-53 and 3-54 in the *Green Book* (Ref. 3.1) should be used to calculate the appropriate overhead clearance to provide the stopping sight distance.

Underpasses become a design control when the minimum length of sag vertical curve required for unobstructed sight distance exceeds the minimum length of curve required for stopping sight distance (headlight sight distance). The longer of the two lengths will control design.

3.D.4 Vertical Curve with Obstructions

The required minimum vertical clearance over or under an existing or future obstruction of known elevation often dictates that the vertical profile must pass through a specific point to satisfy minimum clearance criteria. The minimum vertical clearance will be measured from the high point of the roadway, including the shoulders. If the minimum vertical clearance point on the profile is located on a vertical curve, the curve length will be dependent upon the required elevation of the given point. “Sight Distance at Undercrossings” in Section 3.4.6.4 in Chapter 3 of the *Green Book* (Ref. 3.1) presents a procedure for calculating the length of vertical curve required to pass the curve through a specific point. “General Controls for Vertical Alignment” in Section 3.4.6.5 in Chapter 3 of the *Green Book* (Ref. 3.1) presents additional information on vertical clearance at undercrossings.

3.D.4.a Minimum Vertical Clearances for Overhead Facilities

The roadway designer should always check the overhead clearance based on the high point of the roadway (including the shoulders), which may or may not be at the profile grade point, and should allow sufficient clearance for a future 6-inch overlay, if practicable.

1. Utilities: Vertical clearances for overhead utility facilities shall comply with all applicable state and national electrical codes. Utilities over the roadway shall never be less than 18 feet above the high point of the roadway, including the shoulders. For additional discussion see Chapter Ten: Miscellaneous Design Issues, Section 11, of this manual.
2. Signs or Signal Structures: Vertical clearances for overhead signs or signal structures shall comply with the Manual on Uniform Traffic Control Devices for Streets and Highways (Ref. 3.5) (http://www.roads.nebraska.gov/business-center/contractor/mutcd/).
3. Bridges: For information on allowable vertical clearances for bridge structures, see Chapter Ten: Miscellaneous Design Issues, Section 2, of this manual.
4. Airfields: For information on allowable airspace at airfields, see Chapter Ten: Miscellaneous Design Issues, Section 3, of this manual.

3.E Roller-Coaster Profile

“Roller-coaster” or “hidden-dip” profiles (See EXHIBIT 3.15) generally occur on relatively straight horizontal alignments where the roadway profile closely follows a rolling natural ground line. This type of profile should be avoided on New and Reconstructed projects.

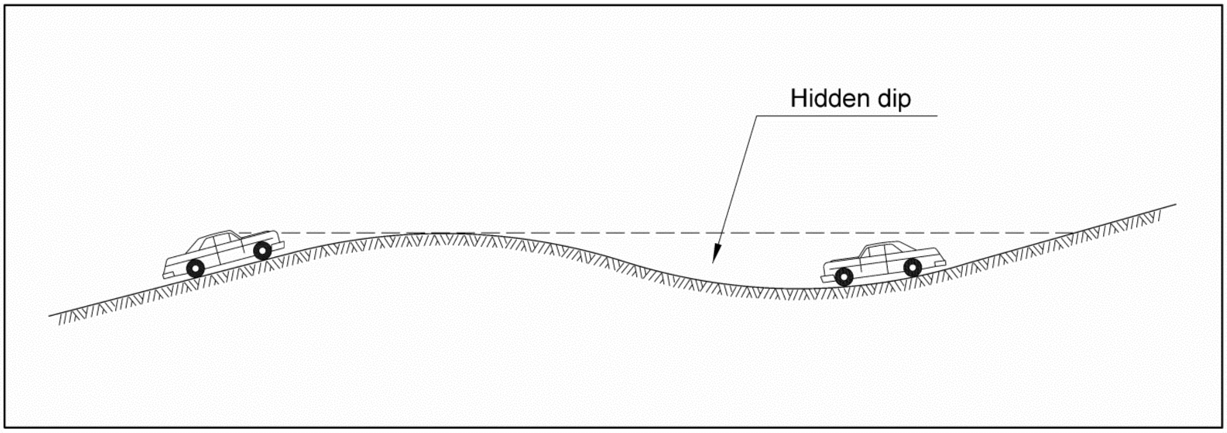


Exhibit 3.15 Roller Coaster Profile

4. COMBINATION OF HORIZONTAL AND VERTICAL ALIGNMENT

The design of horizontal and vertical roadway alignments must be carefully coordinated. See Section 3.5, “Combinations of Horizontal and Vertical Alignment”, in Chapter 3 of the *Green Book* (Ref. 3.1) for guidance.

5. ALIGNMENT DESIGN VALUES

A summary of the basic alignment design values used by **NDOT** is presented below. Chapter Eleven: Highway Plans Assembly discusses the format and content of the sheets that make up a typical set of roadway design plans.

5.A Horizontal Alignment

The common horizontal alignment design values and the degree of accuracy to be shown on the plans include:

- Stationing - to the nearest 0.01 foot
- Radius of curvature - to the nearest 0.01 foot
- Superelevation runoff lengths - rounded to the next appropriate 5 feet
- All other curve data - to the nearest 0.01 foot, or to the nearest 0.01 second, whichever is applicable

Computations for the design of horizontal alignment assume a horizontal plane.

5.A.1 Station Equations

Station equations may be utilized to avoid revising the stationing throughout the length of a project when changes are made from the original surveyed line in the field. The station equation links two station numbers, one that is correct when measuring on the line back of the equation and one that is correct when measuring on the line ahead of the equation.

5.B Vertical Alignment

The common vertical profile design values and the degree of accuracy to be shown on the plans include:

- Grades - expressed in percent rise (+) or fall (-) to the fourth decimal place
- Profile elevations - to the nearest 0.01 foot
- Vertical curve lengths - usually defined to even 100 feet

6. REFERENCES

3.1 American Association of State Highway and Transportation Officials, A Policy on Geometric Design of Highways and Streets (the *Green Book*), 2018.

3.2 Board of Public Roads Classifications and Standards, Nebraska Minimum Design Standards (*MDS*), Current Edition. (http://www.roads.nebraska.gov/media/5593/nac-428-rules-regs-nbcs.pdf)

3.3 Nebraska Department of Transportation, Standard/Special Plan Book (*Standard Plans*), Current Edition. (http://www.roads.nebraska.gov/business-center/design-consultant/stand-spec-manual/)

3.4 American Association of State Highway and Transportation Officials, Roadside Design Guide, Washington, DC, 2011.

3.5 United States Department of Transportation Federal Highway Administration, Manual on Uniform Traffic Control Devices for Streets and Highways, Washington, D.C., 2009. (http://www.roads.nebraska.gov/business-center/contractor/mutcd/)

3.6 Transportation Research Board, Highway Geometric Design and Operational Effects Issues, National Cooperative Highway Research Program (NCHRP) Report 1658, Washington, DC. 1999.

3.7 Transportation Research Board, Recent Geometric Design Research for Improved Safety and Operations, National Cooperative Highway Research Program (NCHRP) Synthesis 299, Washington, DC. 2001.

3.8 American Association of State Highway and Transportation Officials, A Policy on Design Standards Interstate System, Washington, D.C., 2005.

3.9 Nebraska Department of Transportation, Design Process Outline, Current Edition (https://dot.nebraska.gov/business-center/design-consultant/)