



Dave Heineman
Governor

STATE OF NEBRASKA

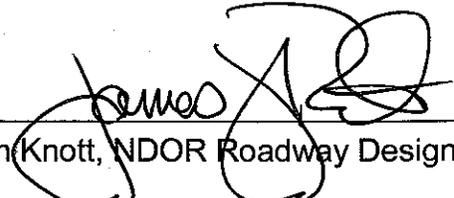
DEPARTMENT OF ROADS

Monty W. Fredrickson, P.E., Director - State Engineer

1500 Highway 2 • PO Box 94759 • Lincoln NE 68509-4759

Phone (402) 471-4567 • FAX (402) 479-4325 • www.transportation.nebraska.gov

The Nebraska Department of Roads Roadway Design Manual
Chapter Three, "Roadway Alignment", April 2011, has been approved
for use.

Approved by:  6/7/11
Jim Knott, NDOR Roadway Design Engineer Date

Approved by:  6/17/2011
Nick Finch, Nebraska FHWA Date

The information contained in Chapter Three: Roadway Alignment, dated April 2011, has been updated to reflect the June 2016 Errata. The errata addresses errors, changes in procedure, changes in NDOR department titles, changes in other Roadway Design Manual chapters and other reference material citations which have occurred since the latest publication of this chapter.

This chapter replaces Chapter Three: Roadway Alignment, dated July 2006. This chapter was approved by the Nebraska Division of the FHWA for use on the National Highway System and other federal projects on June 17, 2011.

Chapter Three

Roadway Alignment

This chapter presents the **Nebraska Department of Roads (NDOR)** 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 **Planning and Project Development Division (P&PD)** 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 the **NDOR's** design guidance (See Chapter One: Design Criteria).

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

2.A Horizontal Curvature

The **NDOR** 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. As a general guide, any change in direction with a deflection angle of 1° or greater will require a horizontal curve. For small deflection angles, curves should be long enough to avoid the appearance of kinks. For further information see Chapter Three of the *Green Book* (Ref. 3.1), "General Controls for Horizontal Alignment".

2.A.1 Simple Curves

The **NDOR** 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.A.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.B).

2.A.3 Compound Curves

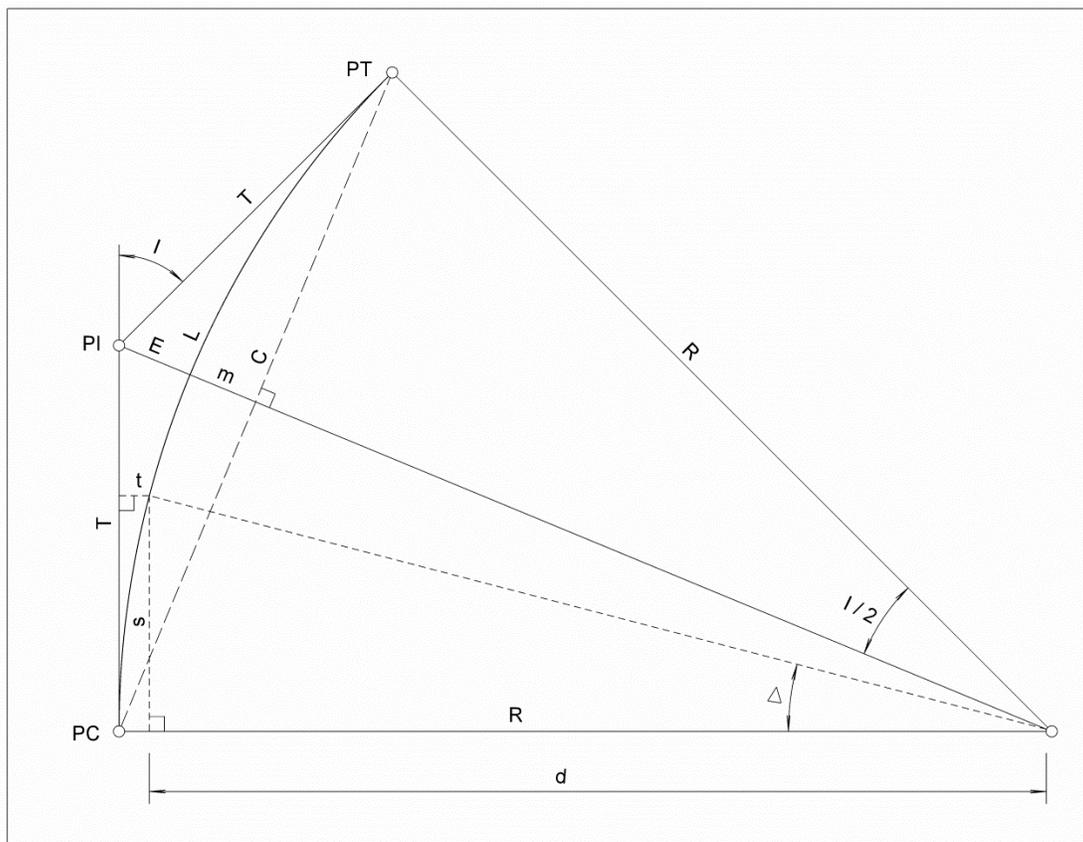
A compound curve consists of two consecutive curves of differing radii deflecting in the same direction with no tangent length between the curves. When field conditions dictate that a compound curve be utilized (e.g. for locations where an obstruction cannot be mitigated otherwise) the ratio of the flatter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

2.A.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.A.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 the **NDOR** 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 Three of the *Green Book* (Ref. 3.1), "Spiral Curve Transitions".



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) $R = 5729.57795/D$ $= (4m^2 + C^2)/8$ $s = R \sin \Delta$ $d = R \cos \Delta$ $t = R - d$ $\pi = 3.141592653$ $PC \text{ Sta.} = PI \text{ Sta.} - T$ $PT \text{ Sta.} = PC \text{ Sta.} + L$

Exhibit 3.1 Elements of a Simple Curve

2.B 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 Appendix H, "Application of Design Standards"). An 8% superelevation rate should be designed in accordance with Chapter Three of the *Green Book* (Ref. 3.1), "Horizontal Alignment".

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

* Requires **ADE** approval and a decision letter to the project file.

** The use of **TABLE 3-13b** (Ref. 3.1) requires **ADE** approval and a decision letter to the project file (See Appendix H, "Application of Design Standards").

Exhibit 3.2 Superelevation Rates

2.B.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 3-16** of the *Green Book* (Ref. 3.1) to determine the minimum superelevation runoff lengths for multilane undivided roadways.

For simple curves 60 to 90% of the superelevation runoff length (L_r) should be placed on the tangent prior to the curve. This percentage will vary based on the design speed and the number of lanes being rotated. To determine the appropriate percentage see **TABLE 3-18** of the *Green Book* (Ref. 3.1).

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.B.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 the **NDOR** 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 "Transition Design Controls" in Chapter 3 of the *Green Book* (Ref. 3.1) for methods of attaining superelevation where the **NDOR** 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 40 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 40 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 40 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 40 feet the axis of rotation and the profile grade point may be the centerline of the individual lanes.

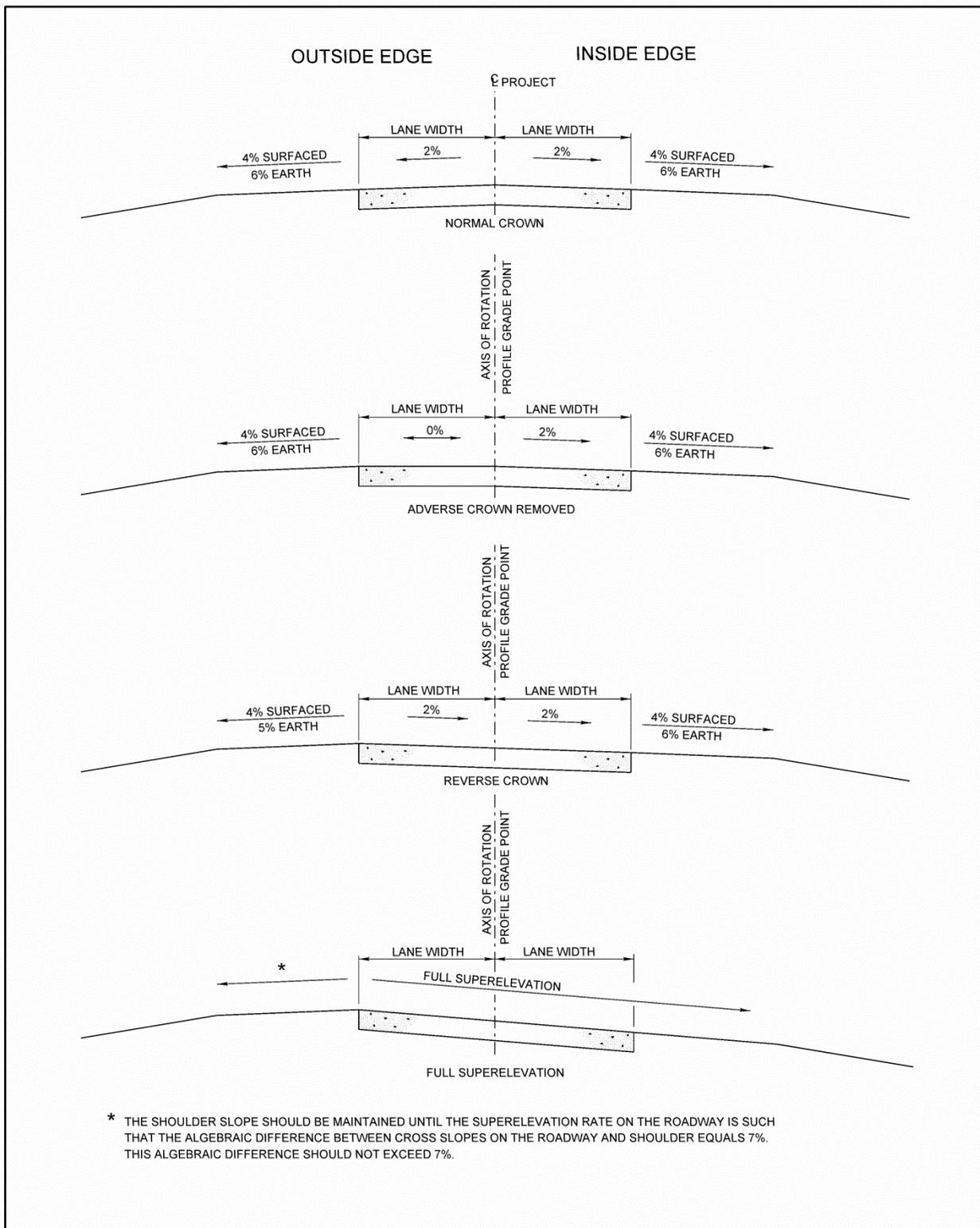
For multi-lane facilities with raised medians, the axis of rotation and the profile grade point should be located at the inside edge of the 12 foot inside lane (See [EXHIBIT 3.6a](#)).

2.B.3 Smoothing of Pavement Edge Profile

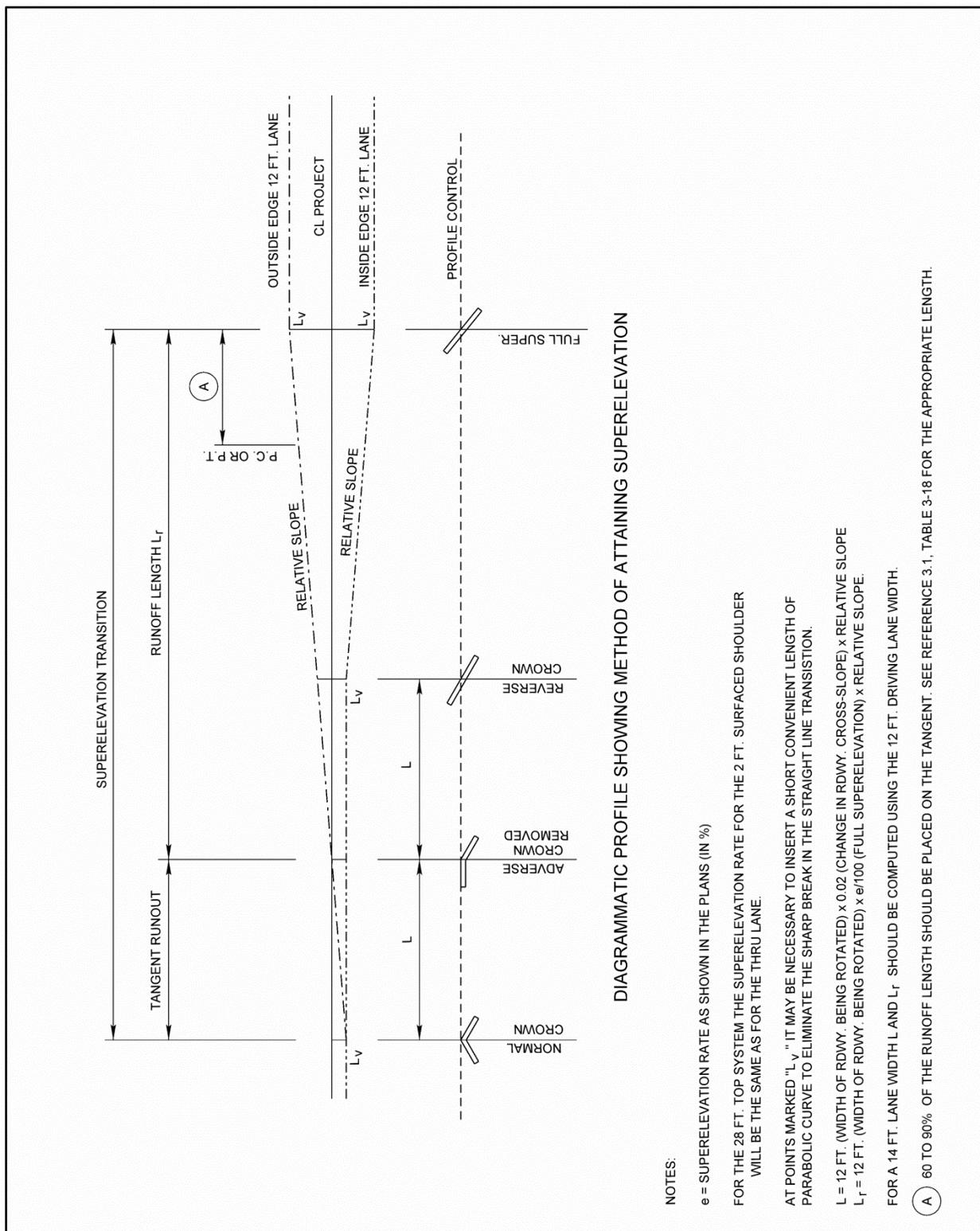
Angular breaks in the vertical profile of the pavement edge through the superelevation transition length should be rounded in the final design. For general appearance, the **NDOR** softens these sharp angular breaks by the insertion of short vertical curves along the pavement edge. As an approximate 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 practical.

2.C 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. For additional information see "Traveled-Way Widening on Horizontal Curves" in Chapter 3 of the *Green Book* (Ref. 3.1).



**Exhibit 3.3a Superelevation Data for Crowned Highways
 Typical Sections**



**Exhibit 3.3b Superlevation 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		e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope	
		L (ft.)	L _r (ft.)																		
23000	NC	0	0																		
20000	NC	0	0																		
17000	NC	0	0																		
14000	NC	0	0																		
12000	NC	0	0																		
10000	NC	0	0																		
8000	NC	0	0																		
6000	2.0	48	48	2.2	51	51	2.6	51	51	3.1	51	51	3.6	51	51	4.1	51	51	4.6	51	51
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			
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	6.0	60	189						
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												
1400	5.2	48	125	5.7	51	146	6.0	53	160												
1200	5.5	48	132	5.9	51	151															
1000	5.9	48	142																		
900	6.0	48	144																		

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, EXHIBIT 3-26.

KEY:

- V = Assumed design speed
- e = Rate of superelevation
- L = Tangent runoff based on a 12 ft. lane
- L_r = 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		e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope	
		L (ft.)	L _r (ft.)															
8000	NC	0	0															
6000	NC	0	0															
5000	NC	0	0															
4000	NC	0	0															
3500	NC	0	0															
3000	NC	0	0															
2500	NC	0	0	2.0	36	36	2.0	39	39	2.0	39	39	2.0	41	41	2.0	44	44
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															

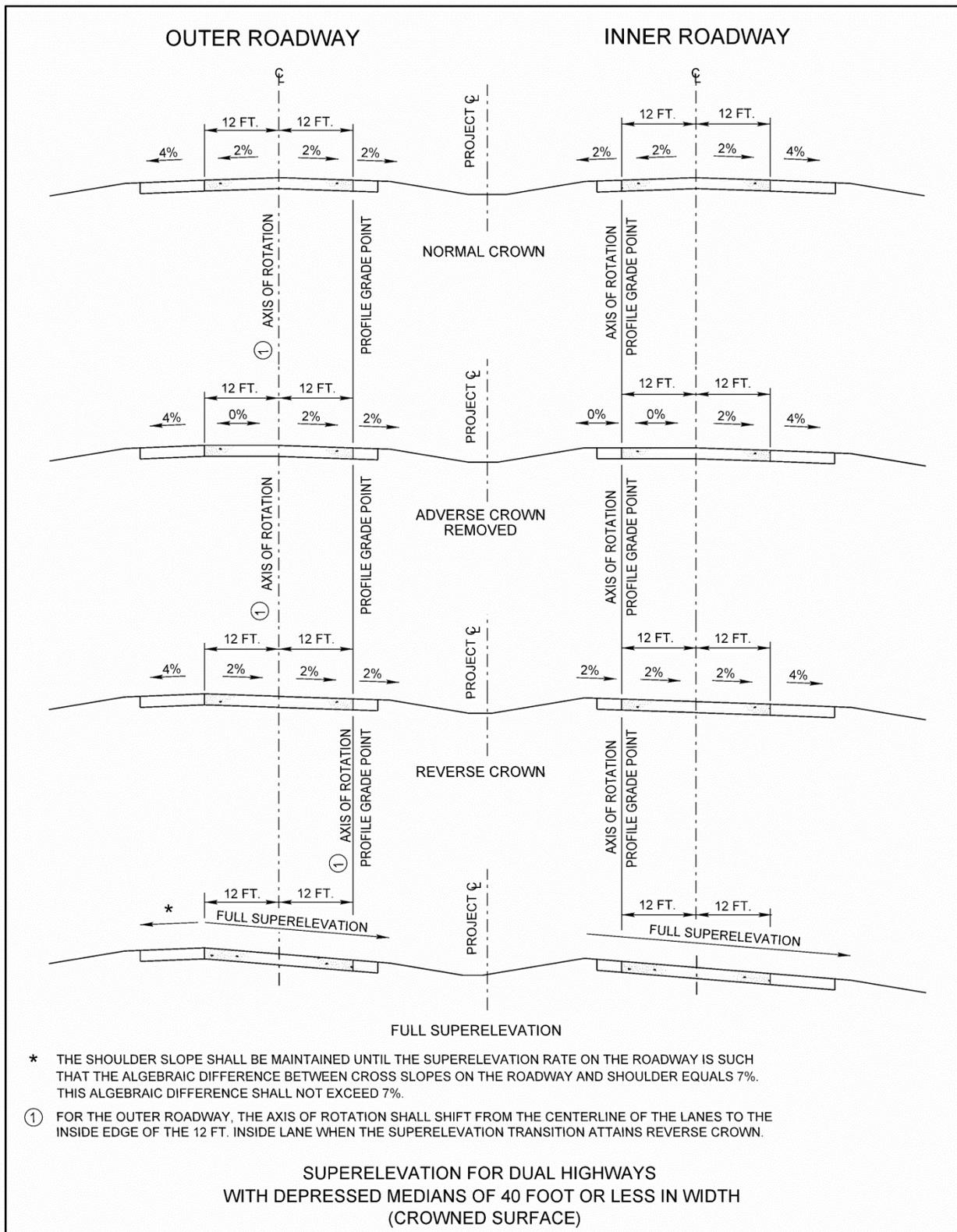
e_{max} = 4%

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, EXHIBIT 3-25.

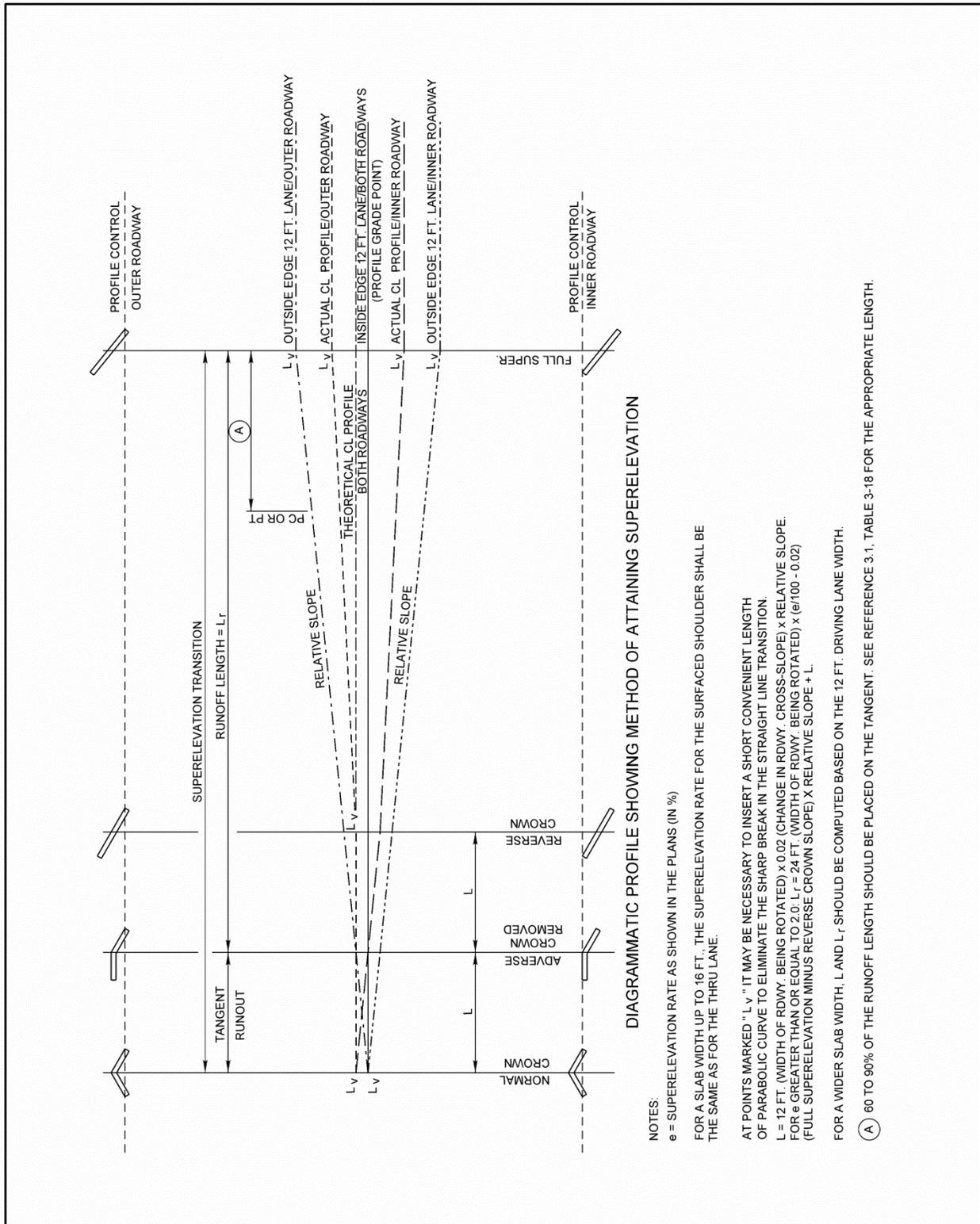
KEY:

- V = Assumed design speed
- e = Rate of superelevation
- L = Tangent runoff based on a 12 ft. lane
- L_r = 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%)



**Exhibit 3.4a Superelevation Data for Dual Highways (Crowned Surface)
 Depressed Median Width = 40 Foot or Less
 Typical Sections**



**Exhibit 3.4b Super-elevation Data for Dual Highways (Crowned Surface)
 Depressed Median Width = 40 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 (%)	L (ft.)	L _r (ft.)	e (%)	L (ft.)	L _r (ft.)	e (%)	L (ft.)	L _r (ft.)	e (%)	L (ft.)	L _r (ft.)	e (%)	L (ft.)	L _r (ft.)	e (%)	L (ft.)	L _r (ft.)	e (%)	L (ft.)	L _r (ft.)
23000	NC	0	0																		
20000	NC	0	0																		
17000	NC	0	0																		
14000	NC	0	0																		
12000	NC	0	0																		
10000	NC	0	0																		
8000	NC	0	0	2.0	51	51	2.0	51	51	2.0	51	51	2.0	51	51	2.0	51	51	2.0	51	51
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			
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									
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												
1400	5.2	48	202	5.7	51	240	6.0	53	266												
1200	5.5	48	216	5.9	51	250															
1000	5.9	48	235																		
900	6.0	48	240																		

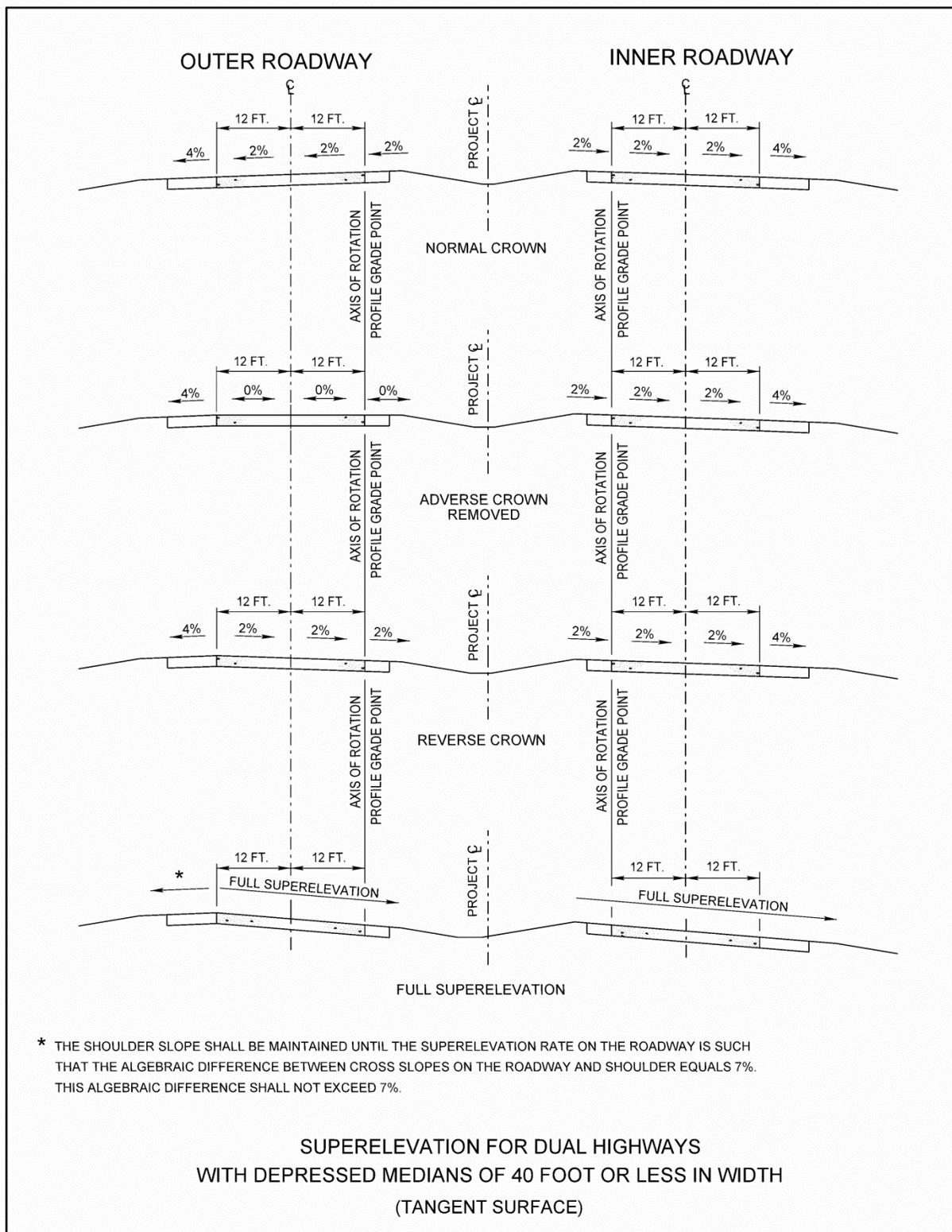
e_{max} = 6%

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, EXHIBIT 3-26.

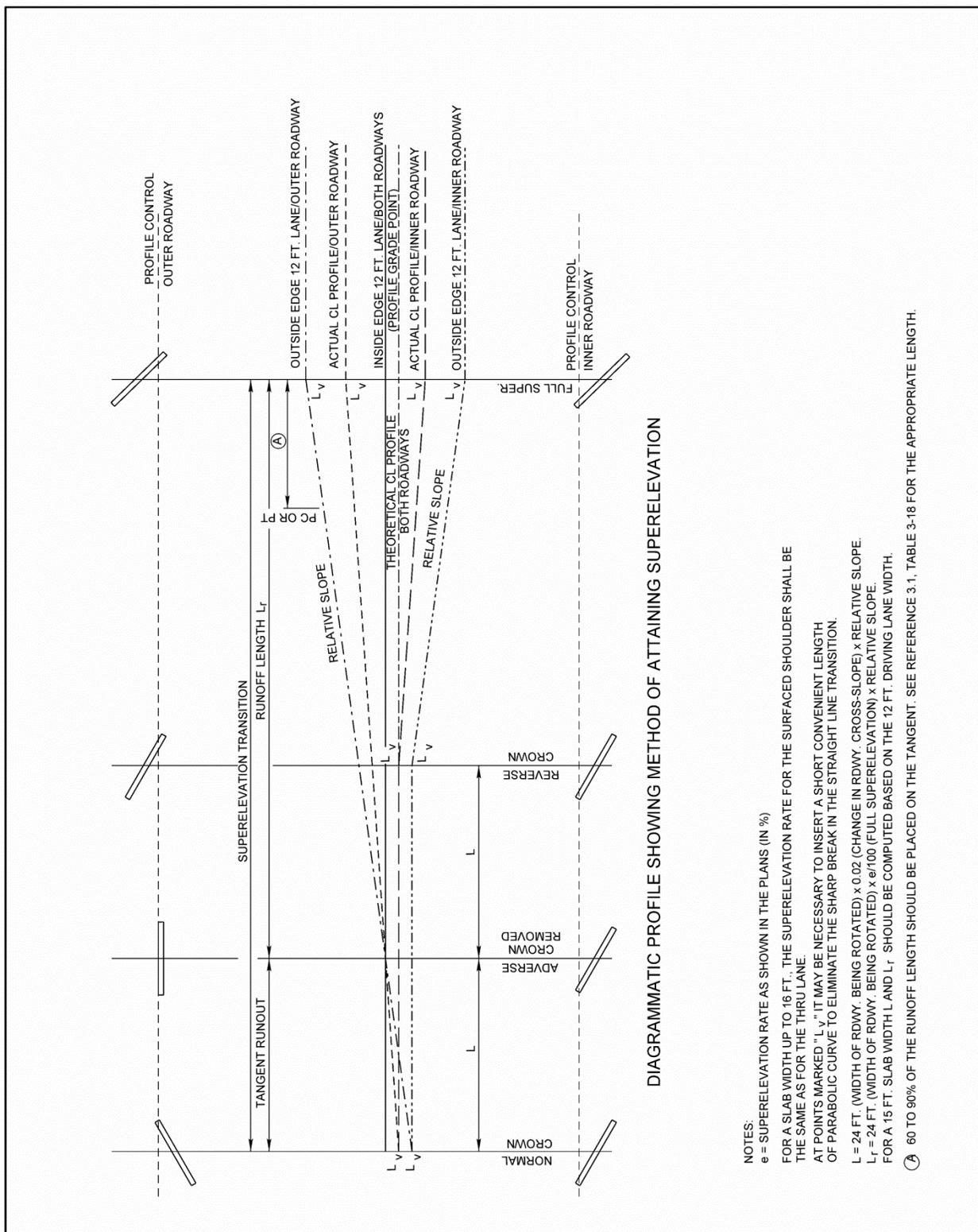
KEY:

- V = Assumed design speed
 - e = Rate of superlevation
 - L = Tangent runoff based on a 12 ft. lane
 - L_r = Minimum length of superlevation 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 Superlevation Data for Dual Highways (Crowned Surface)
 Depressed Median Width = 40 Foot or Less
 Values for Design Elements Related to Design Speed and Horizontal Curvature
 (e_{max}=6%)**



**Exhibit 3.5a Superlevation Data for Dual Highways (Tangent Surface)
 Depressed Median Width = 40 Foot or Less
 Typical Sections**



**Exhibit 3.5b Super-elevation Data for Dual Highways (Tangent Surface)
 Depressed Median Width = 40 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												
		L (ft.)		L _r (ft.)		L (ft.)		L _r (ft.)		L (ft.)		L _r (ft.)		L (ft.)
23000	NC	0												
20000	NC	0												
17000	NC	0												
14000	NC	0												
12000	NC	0	NC	0	NC	0	2.0	84	2.0	90	NC	95	2.1	103
10000	NC	0	NC	0	2.0	80	2.0	84	2.1	90	2.3	95	2.5	103
8000	NC	0	2.0	77	2.0	80	2.3	84	2.5	90	2.8	95	3.1	103
6000	2.0	72	2.2	77	2.6	80	2.9	84	3.2	90	3.5	95	4.0	103
5000	2.2	72	2.6	77	3.0	80	3.3	84	3.7	90	4.1	95	4.6	103
4000	2.7	72	3.1	77	3.6	80	4.0	84	4.4	90	4.9	95	5.5	103
3500	3.0	72	3.5	77	3.9	80	4.4	84	4.9	90	5.3	95	5.9	103
3000	3.4	72	3.8	77	4.3	80	4.8	84	5.4	90	5.7	95	6.0	103
2500	3.8	72	4.3	77	4.8	80	5.3	84	5.8	90	6.0	95	6.0	103
2000	4.3	72	4.9	77	5.4	80	5.8	84	6.0	90	6.0	95	6.0	103
1800	4.6	72	5.1	77	5.6	80	6.0	84	6.0	90	6.0	95	6.0	103
1600	4.9	72	5.4	77	5.8	80	6.0	84	6.0	90	6.0	95	6.0	103
1400	5.2	72	5.7	77	6.0	80	6.0	84	6.0	90	6.0	95	6.0	103
1200	5.5	72	5.9	77	6.0	80	6.0	84	6.0	90	6.0	95	6.0	103
1000	5.9	72	6.0	77	6.0	80	6.0	84	6.0	90	6.0	95	6.0	103
900	6.0	72	6.0	77	6.0	80	6.0	84	6.0	90	6.0	95	6.0	103

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, EXHIBIT 3-26.

KEY:

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

R_{min} = 2040

R_{min} = 2500

R_{min} = 1660

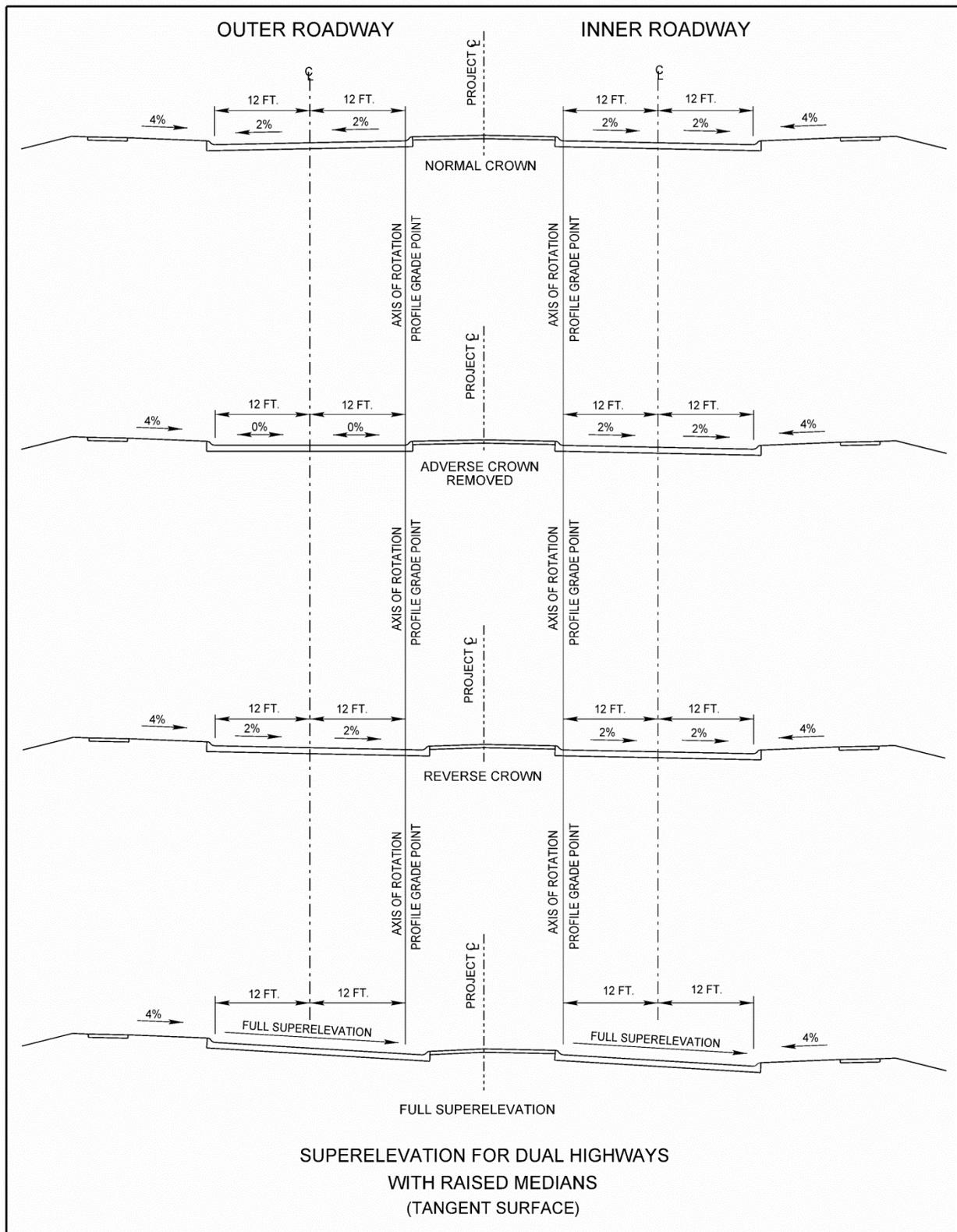
R_{min} = 1330

R_{min} = 1060

R_{min} = 833

5

**Exhibit 3.5c Superlevation Data for Dual Highways (Tangent Surface)
 Depressed Median Width = 40 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**

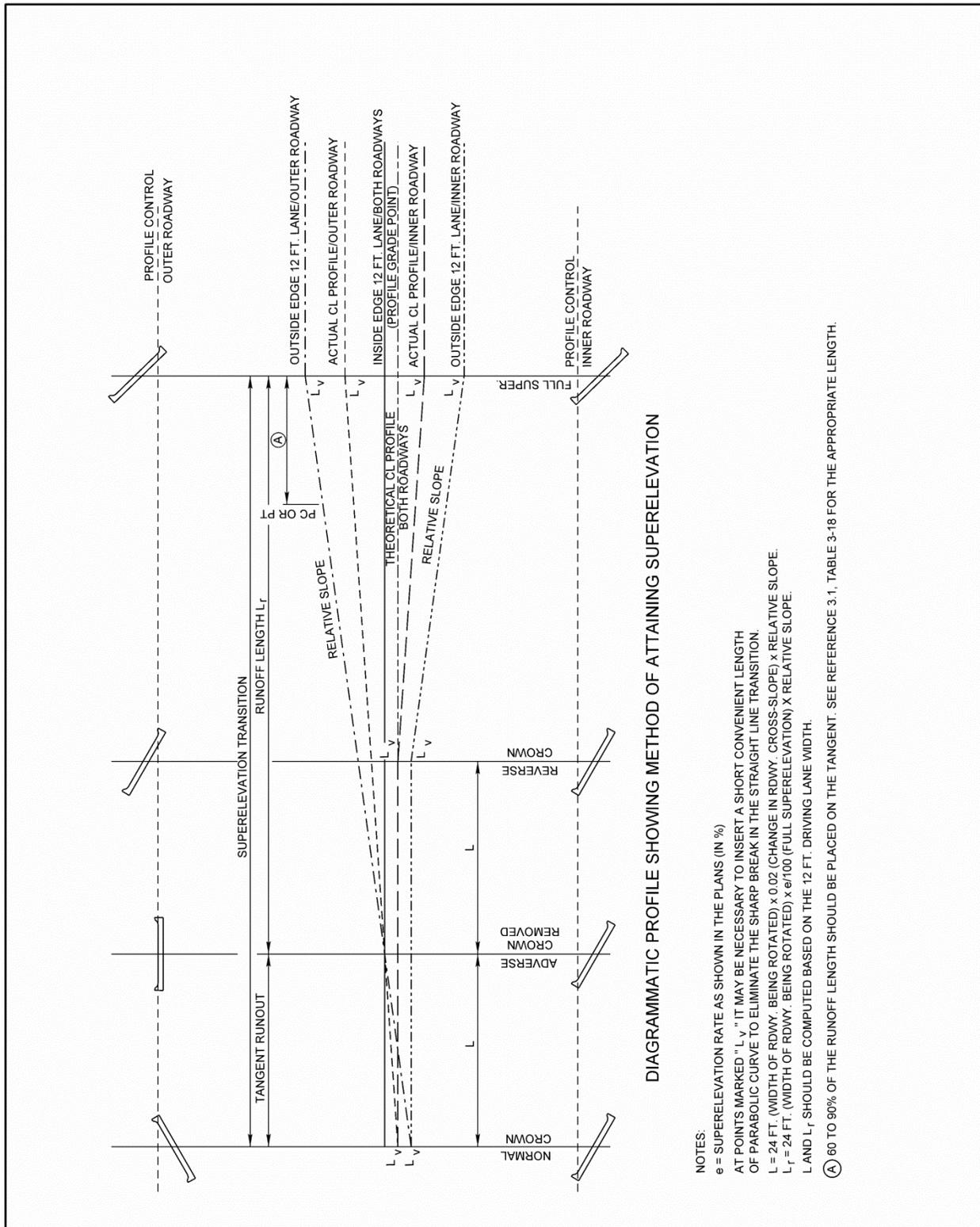


Exhibit 3.6b Super-elevation Data for Dual Highways w/Raised Median Diagrammatic Profile

Radius of Curve (ft.)	V=25 mph			V=30 mph			V=35 mph			V=40 mph			V=45 mph		
	e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope		e (%)	Maximum Relative Slope	
		107.25:1			114:1			120.75:1			129:1			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.)	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	Rmin = 371					
350	3.2	51	82	3.8	55	104	Rmin = 250								
300	3.4	51	88	3.9	55	107							Rmin = 154		
250	3.6	51	93	4.0	55	109									
200	3.8	51	98	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, EXHIBIT 3-25.

KEY:

- V = Assumed design speed
 - e = Rate of superelevation
 - L = Tangent runout based on a 24 ft. roadway
 - L_r = 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 **P&PD** in the development of, and in any subsequent alteration to, the vertical alignment. The vertical alignment should not be changed during the Final Design Phase (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: Earthwork, Section 1.

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 practical, to address the sight distance beyond the proposed project limits.

For additional information see "Vertical Alignment" in Chapter Three of the *Green Book* (Ref. 3.1).

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.transportation.nebraska.gov/gov-aff/gov-aff-design-standards.html>). 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 Appendix H, "Application of Design Standards"). Grades which are less than the maximum should be used whenever practical.

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)** 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 the **NDOR** standards for the design

of climbing lanes. See “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 2 feet turf. (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)	N/A	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). 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 3-28**, 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**.
4. If there is a mailbox located within the climbing lane, the surfaced 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).

Exhibit 3.7 Standards for Climbing Lanes

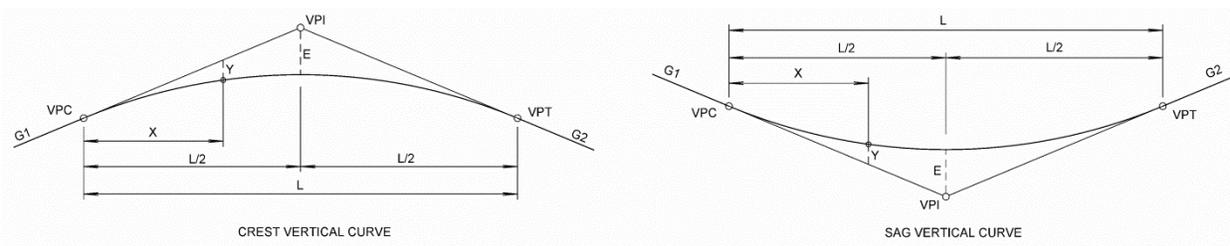
3.B Vertical Curves

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

The **NDOR** 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 **NDOR** practice to design vertical curves so that the vertical PC and PT fall on even stationing.



Vertical Curve Symbols and Abbreviations	Vertical Curve Formulas
<p>VPC Vertical point of curvature (beginning of curve)</p> <p>VPI Vertical point of intersection of grades</p> <p>VPT Vertical point of tangency (end of curve)</p> <p>G1 Grade at beginning of curve, ft/ft</p> <p>G2 Grade at end of curve, ft/ft</p> <p>E External distance from VPI to curve</p> <p>L Horizontal length of curve</p> <p>R Rate of change of grade, ft/ft</p> <p>E_0 Elevation of VPC</p> <p>V Elevation of VPI</p> <p>E_T Elevation of VPT</p> <p>X Distance of any point on the curve from the VPC</p> <p>Y Tangent offset to curve at X distance from the VPC</p> <p>E_X Elevation of any point X distance from the VPC</p> <p>X_S Distance from the VPC to lowest point of a sag curve or highest point of a crest curve</p> <p>E_S Elevation of lowest point on a sag curve or highest point on a crest curve</p> <p>A Algebraic difference of grades, %</p>	<p>$V = E_0 + [L/2 \times G1]$</p> <p>$E_T = V - [L/2 \times G2]$</p> <p>$R = (G2 - G1)/L$</p> <p>$E_X = E_0 + G1X + \frac{1}{2}RX^2$</p> <p>$E_S = E_0 - [G1^2 \div 2R]$</p> <p>$X_S = -G1/R$ (if X_S is negative or if $X_S > L$, the curve doesn't have a high point or a low point)</p> <p>$A = G1 - G2$</p> <p>$E = AL/800$</p> <p>$Y = (X \div L/2)^2 \times E$</p>

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.

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 Appendix H, "Application of Design Standards"). 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 Chapter 9 of the *Green Book* (Ref. 3.1), "Intersection Sight Distance".

The minimum K values shown in EXHIBITS 3.9 & 3.14 are from 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 Chapter 9 of the *Green Book* (Ref. 3.1), "Intersection Sight Distance". The use of the minimum K values will require **Unit Head** approval and a decision letter to the project file (See Appendix H, "Application of design Standards").

The use of K values below the minimum 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 Appendix H, "Application of design Standards").

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 Appendix H, "Application of Design Standards").

Special attention to pavement drainage must be exercised where a K value in excess of 143 is used, a minimum roadway cross-slope of 1.5% should be maintained. For additional information see Chapter Three of the *Green Book* (Ref. 3.1), "Design Controls – Stopping Sight Distance".

U.S. Customary								
Crest Vertical Curve								
Design Speed (mph)	Minimum Stopping Sight Distance ^①		Desirable Stopping Sight Distance ^② (2-Lane, Left-Turn Condition and 2-Lane w/TWTL **, Left-Turn Condition)		Desirable Stopping Sight Distance ^③ (5-Lane and 4-Lane Divided w/18' Median, Left-Turn Condition)		Desirable Stopping Sight Distance ^④ (4-Lane Divided w/40' 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 *
35	250	29	453	93	500	128	540	125
40	305	44	517	121	570	167	617	163
45	360	61	582	153	642	212	695	207
50	425	84	647	189	713	262	772	255
55	495	114	711	228	784	316	849	308
60	570	151	776	272	855	376	926	367
65	645	193	841	319	927	442	1003	430
70	730	247	906	370	998	512	1080	499

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

** TWTL = Two -Way Left-Turn Lane, See Chapter Four, Section 5.C.

@ Check Intersection Sight Distance if Building Tangent Lanes.

- ① Based on a 3.5' eye height and an object height of 2.0', (See "A Policy on Geometric Design of Highways and Streets", Reference 3.1, EXHIBIT 3-72). 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 Reference 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 and Reference 3.1, Chapter 9). Based on an 8.8 second gap in traffic, a 2.33' eye height and an object height of 3.25'. See Exhibit 3.10a 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 and Reference 3.1, Chapter 9). Based on a 9.7 second gap in traffic, a 2.0' eye height and an object height of 2.9'. See Exhibit 3.10b 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 and Reference 3.1, Chapter 9). Based on a 10.5 second gap in traffic, a 2.50' eye height and an object height of 3.38'. See Exhibit 3.10c 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 NDOR time gap is arrived at by adding an initial time gap of 7.5 sec. (left turn from "A Policy on Geometric Design of Highways and Streets", Reference 3.1, Chapter 9) plus 0.8 sec. (1.2 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 3.9 Design Controls for Crest Vertical Curves

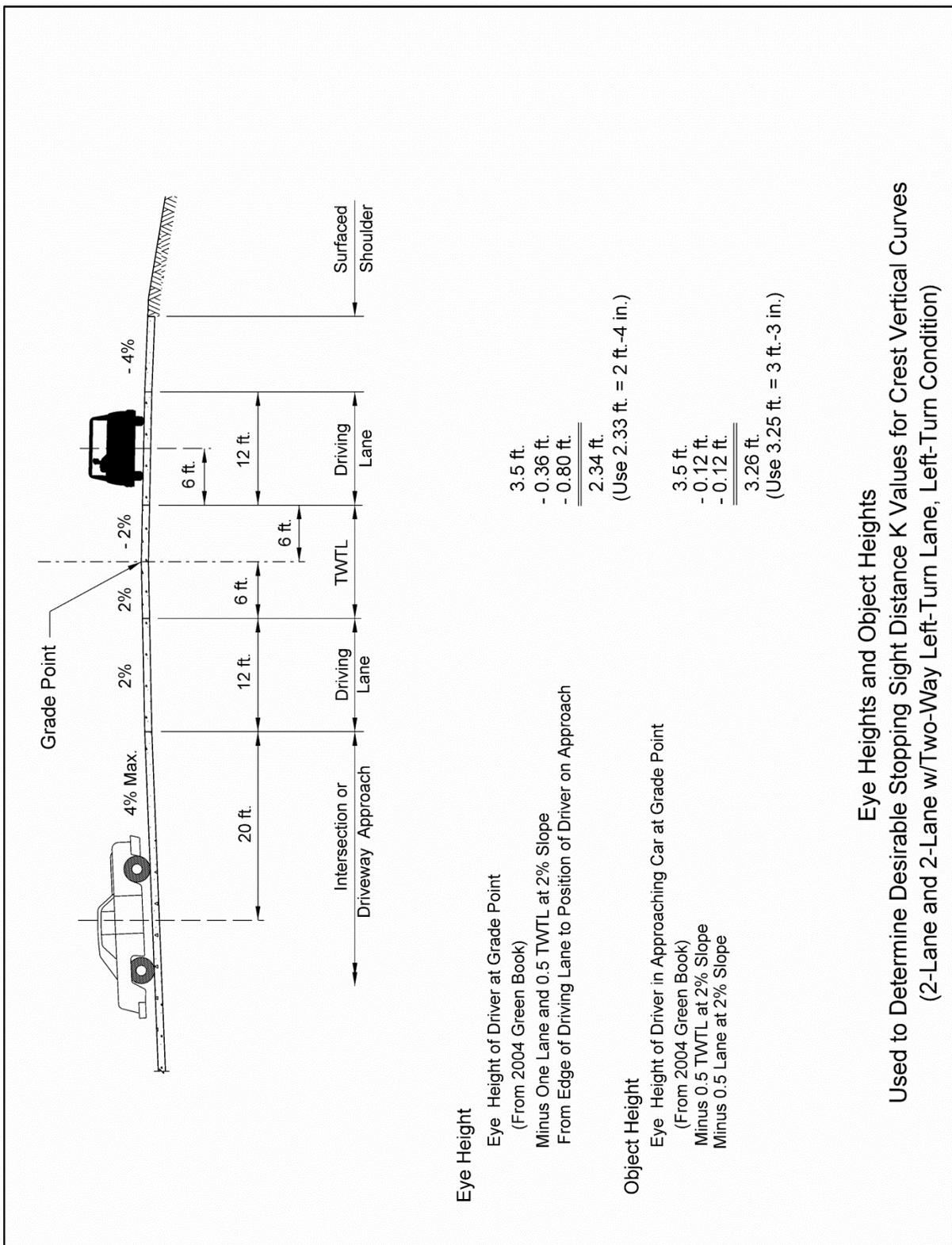


Exhibit 3.10a Eye and Object Heights used to Determine Desirable K Values
 (Crest Vertical Curves)

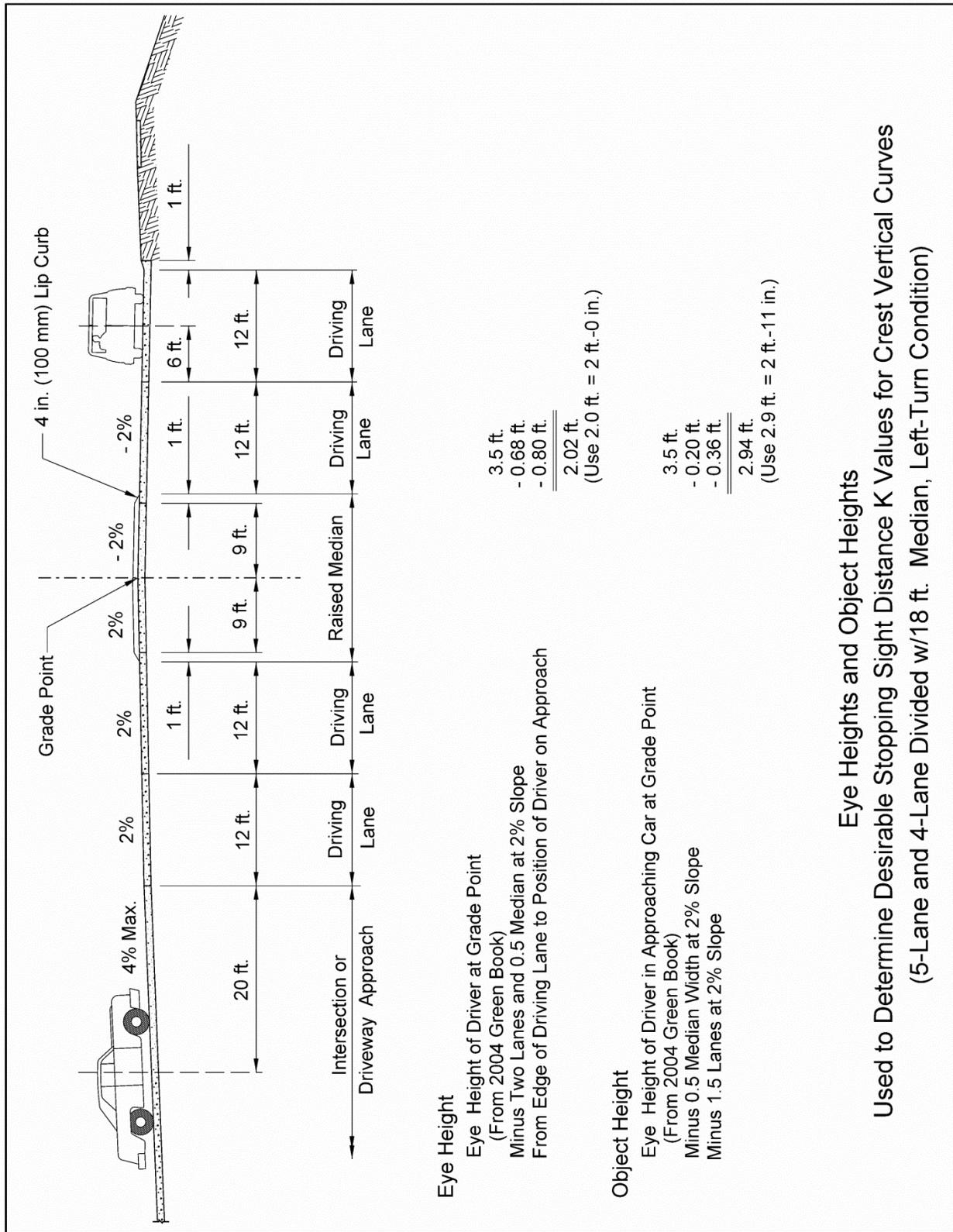


Exhibit 3.10b Eye and Object Heights used to Determine Desirable K Values
 (Crest Vertical Curves)

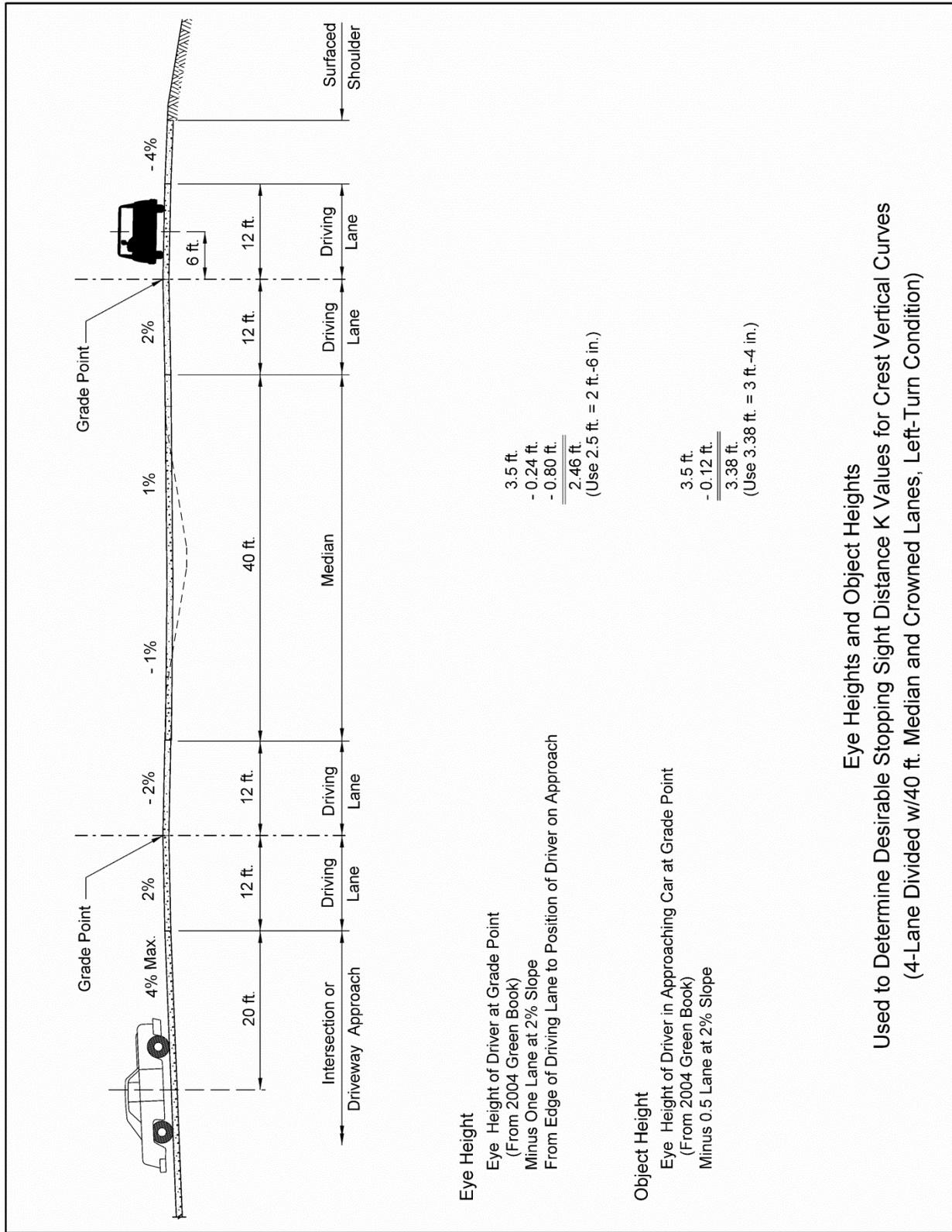


Exhibit 3.10c Eye and Object Heights used to Determine Desirable K Values (Crest Vertical Curves)

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 **NDOR desirable** stopping sight distance at each design speed (desirable stopping sight distance includes intersection sight distance).

$$S_d = 1.47 \times \text{speed (mph)} \times \text{time (sec)}$$

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 and Chapter 3 of the *Green Book*, Ref. 3.1). The **NDOR** uses a time gap of 8.8 sec. for the two-lane left-turn condition (the **NDOR** time gap is arrived at by adding an initial time gap of 7.5 sec. (left-turn condition from Chapter 9 of the *Green Book*, Ref. 3.1) plus 0.8 sec. (1.2 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 of 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).

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 Appendix H, "Application of Design Standards").

The minimum K values shown in EXHIBIT 3.9 are from 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 Chapter 9 of the *Green Book* (Ref. 3.1), "Intersection Sight Distance". 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 Appendix H, "Application of design Standards").

The use of K values below the stopping sight distance values given in EXHIBIT 3.9 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 Application H, "Application of Design Standards").

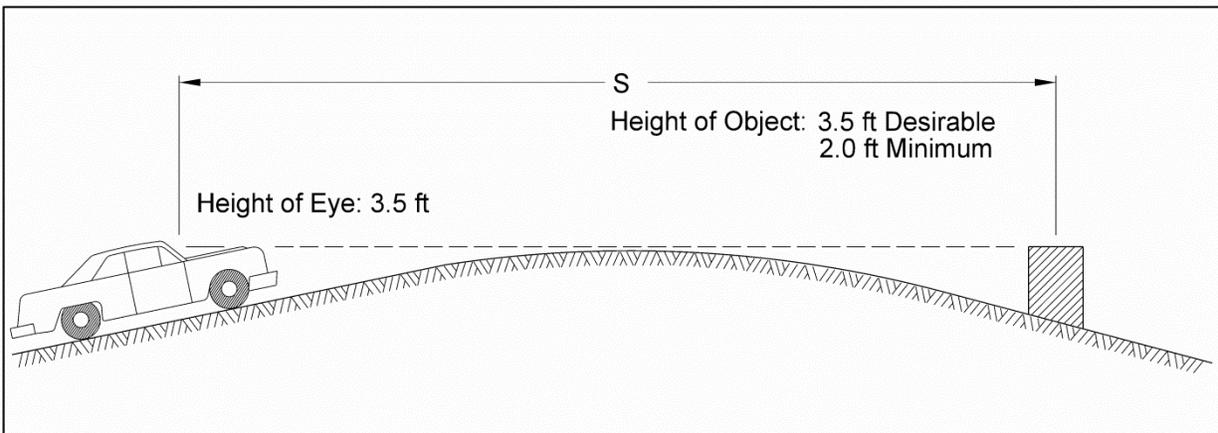


Exhibit 3.11 Crest Vertical Curve Design Elements

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

Passing sight distance is not one of the thirteen principal controlling design criteria (See Appendix H, "Application of Design Standards"). It is not practical to design crest vertical curves to provide for passing sight distance. For evaluation, **TABLE 3-34** and Equations 3-45 & 3-46 of the Green Book (Ref. 3.1) may be used to determine the minimum length of vertical curve for passing sight distance. For additional information see Chapter 3 of the Green Book (Ref. 3.1), "Passing Sight Distance for Two-Lane Highways".

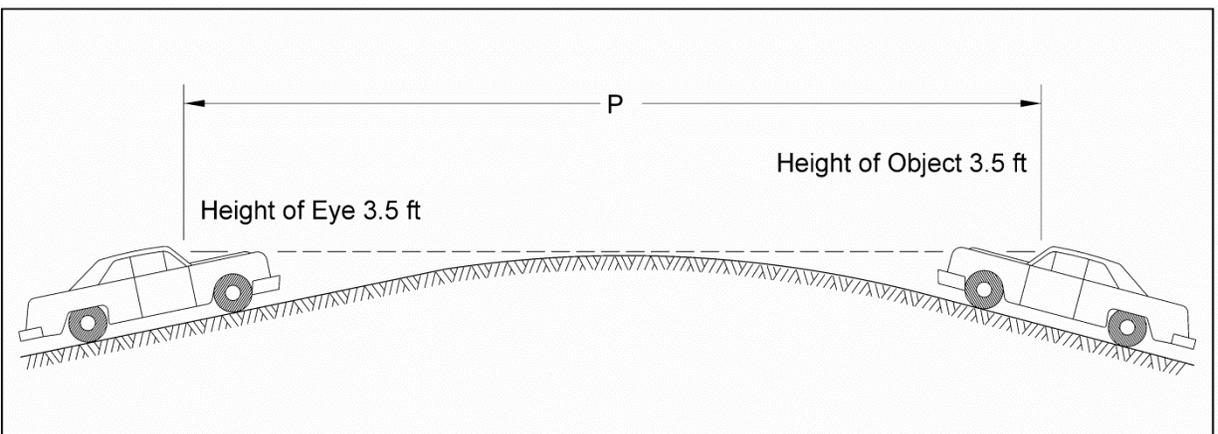


Exhibit 3.12 Passing Sight Distance Design Considerations

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 **NDOR desirable** stopping sight distance at each design speed (desirable stopping sight distance includes intersection sight distance).

$$S = 1.47 \times \text{speed (mph)} \times \text{time (sec)}$$

Where:

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

Time = acceptable time gap in traffic (in seconds) based on intersection conditions (See Chapter 3 of the *Green Book*, Ref. 3.1). The **NDOR** uses a time gap of 8.8 sec. for the two-lane, left-turn condition.

Equations 3-48 & 3-50 of 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 Appendix H, "Application of Design Standards").

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 Chapter 9 of the *Green Book* (Ref. 3.1), "Intersection Sight Distance". 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 Appendix H, "Application of Design Standards").

The use of K values below the stopping sight distance values given in EXHIBIT 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 Appendix H, "Application of Design Standards").

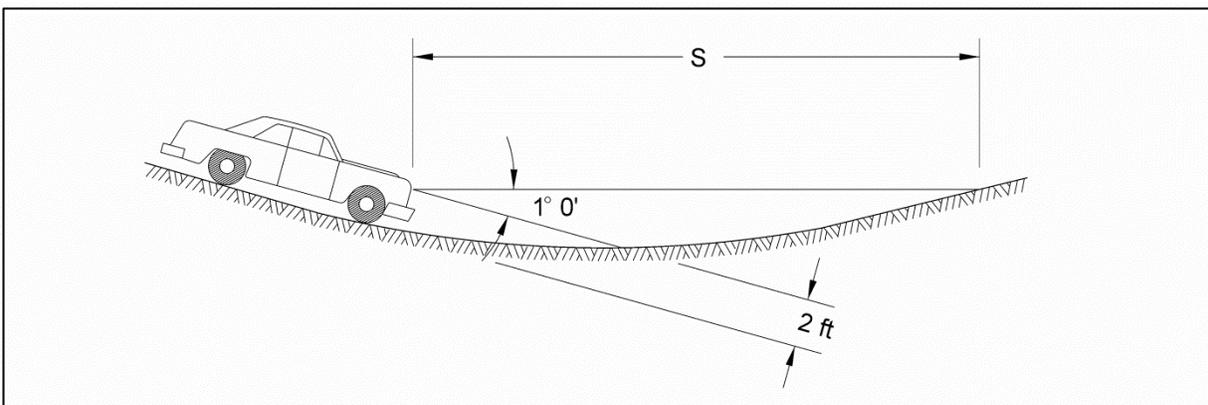


Exhibit 3.13 Sag Vertical Curve Design Elements

U.S. Customary								
Sag Vertical Curve								
Design Speed (mph)	Minimum Stopping Sight Distance ①		Desirable Stopping Sight Distance ② (2-Lane, Left-Turn Condition and 2-Lane w/TWTL **, Left-Turn Condition)		Desirable Stopping Sight Distance ③ (5-Lane and 4-Lane Divided w/18' Median, Left-Turn Condition)		Desirable Stopping Sight Distance ④ (4-Lane Divided w/40' 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 *
35	250	49	453	103	500	116	540	127
40	305	64	517	121	570	136	617	149
45	360	79	582	139	642	156	695	170
50	425	96	647	157	713	176	772	192
55	495	115	711	175	784	196	849	214
60	570	136	776	193	855	215	926	236
65	645	157	841	212	927	236	1003	257
70	730	181	906	230	998	256	1080	279

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

** TWTL = Two -Way Left-Turn Lane, See Chapter Four, Section 5.C.

- ① Based on a 3.5' eye height, a 2.0' headlight height and a 1 degree upward divergence of the light beam, (See "A Policy on Geometric Design of Highways and Streets", Reference 3.1, EXHIBIT 3-75). 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 Reference 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 and Reference 3.1, Chapter 9). Based on an 8.8 second gap in traffic, a 3.5' eye height, a headlight height of 2.0' and a 1 degree upward divergence of the light beam, (See Reference 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 and Reference 3.1, Chapter 9). Based on a 9.7 second gap in traffic, a 3.5' eye height, a headlight height of 2.0' and a 1 degree upward divergence of the light beam, (See Reference 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 and Reference 3.1, Chapter 9). Based on a 10.5 second gap in traffic, a 3.5' eye height, a headlight height of 2.0' and a 1 degree upward divergence of the light beam, (See Reference 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 NDOR time gap is arrived at by adding an initial time gap of 7.5 sec. (left turn from "A Policy on Geometric Design of Highways and Streets", Reference 3.1, Chapter 9) plus 0.8 sec. (1.2 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 3.14 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 3-51 of 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 3-52 & 3-53 of 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 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.

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 turf shoulders), which may or may not be at the profile grade point, and should allow sufficient clearance for a future 6 inch overlay, if possible.

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 turf shoulders. For additional discussion see Chapter Thirteen: Planning and Project Development, Section 6.
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.dor.state.ne.us/traffeng/mutcd.htm>).
3. Bridges: For information on allowable vertical clearances for bridge structures, see Chapter Ten: Miscellaneous Design Issues, Section 2.
4. Airfields: For information on allowable airspace at airfields, see **EXHIBIT 'R'** of the Design Process Outline (Ref. 3.9) (<http://www.nebraskatransportation.org/roadway-design/downloads.htm>).

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.

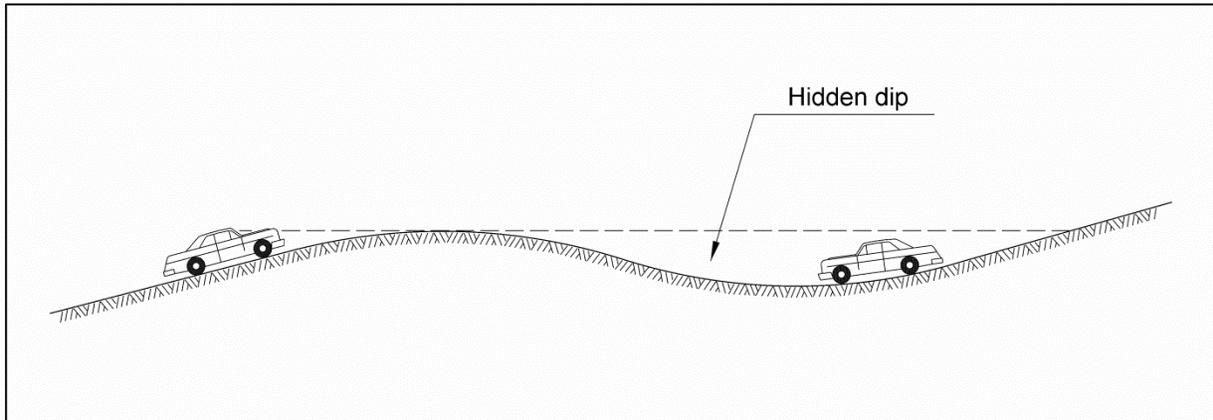


Exhibit 3.15 Roller Coaster Profile

4. **COMBINATION OF HORIZONTAL AND VERTICAL ALIGNMENT**

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

5. ALIGNMENT DESIGN VALUES

A summary of the basic alignment design values used by the **NDOR** is presented below. Chapter Eleven: Highway Plans Assembly, Section 4, 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 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*), 2011.
- 3.2 Board of Public Roads Classifications and Standards, Nebraska Minimum Design Standards (*MDS*), Current Edition.
(<http://www.transportation.nebraska.gov/gov-aff/gov-aff-design-standards.html>)
- 3.3 Nebraska Department of Roads, Standard/Special Plan Book (*Standard Plans*), Current Edition. (<http://www.nebraskatransportation.org/roadway-design/stan-spec-man.html>)
- 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.dor.state.ne.us/traffeng/mutcd.htm>)
- 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 Roads, Design Process Outline, Current Edition
(<http://www.nebraskatransportation.org/roadway-design/downloads.htm>)

