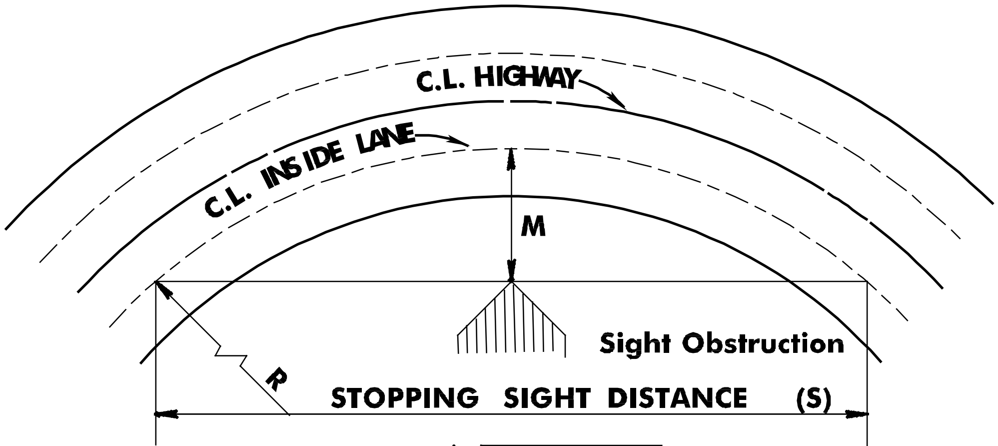
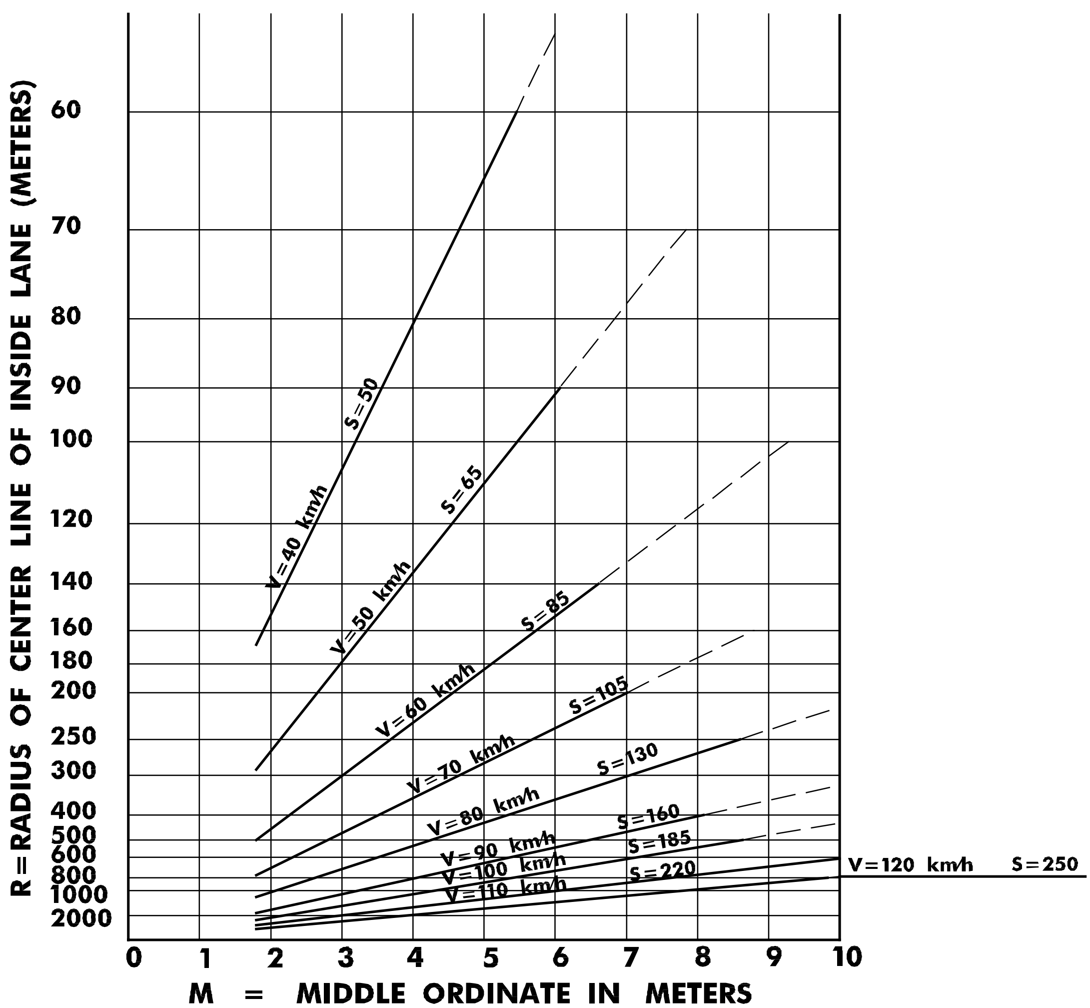
MINIMUM STOPPING SIGHT DISTANCE ON HORIZONTAL CURVES

FIGURE: 4–A
BDC02MR–4



$$M = R - \sqrt{R^2 - (S/2)^2}$$

R = Radius



VALUES OF SUPERELEVATION FOR RURAL HIGHWAYS AND RURAL OR URBAN FREEWAYS

FIGURE: 4-B

BDC02MR-4

RADIUS (METERS)	SUPERELEVATION (PERCENT) FOR DESIGN SPEEDS OF							
	40 km/h	50 km/h	60 km/h	70 km/h	80 km/h	90 km/h	100 km/h	110 km/h
60	6.0							
70	5.8							
80	5.6							
90	5.4	6.0						
100	5.2	6.0						
110	5.0	5.8						
120	4.8	5.7						
130	4.6	5.6						
140	4.5	5.4	6.0					
150	4.4	5.3	6.0					
175	4.1	5.0	5.8					
200	3.9	4.7	5.5	6.0				
250	3.5	4.2	5.0	5.8	6.0			
300	3.1	3.9	4.6	5.4	5.9			
400	2.5	3.3	4.0	4.7	5.3	5.9		
500	2.1	2.8	3.5	4.2	4.8	5.4	5.9	
600	1.8	2.4	3.1	3.8	4.3	5.0	5.6	6.0
700	1.6	2.1	2.8	3.4	4.0	4.6	5.2	5.8
800	NC	1.9	2.5	3.1	3.6	4.2	4.9	5.4
900		1.7	2.3	2.8	3.4	3.9	4.5	5.1
1000		1.6	2.1	2.6	3.1	3.6	4.2	4.8
1200		NC	1.8	2.2	2.7	3.2	3.7	4.2
1300			1.6	2.1	2.5	3.0	3.5	4.0
1400			NC	2.0	2.4	2.8	3.3	3.8
1500				1.9	2.2	2.7	3.1	3.6
2000				NC	1.7	2.1	2.5	2.8
2500					NC	1.7	2.0	2.3
3000						NC	1.7	2.0
5000							NC	NC
7000								

NC = NORMAL CROWN

NO SUPERELEVATION REQUIRED WHEN RADIUS IS GREATER THAN:

	40km/h	50km/h	60km/h	70km/h	80km/h	90km/h	100km/h	110km/h
6% MAX:	750 m	1044 m	1438 m	1897 m	2350 m	2862 m	3504 m	4049 m

NOTE: SUPERELEVATION RATES LESS THAN 1.5% SHALL NOT BE USED.

VALUES OF SUPERELEVATION FOR URBAN HIGHWAYS

FIGURE: 4–C

BDC02MR–4

RADIUS (METERS)	SUPERELEVATION (PERCENT) FOR DESIGN SPEEDS OF									
	40 km/h		50 km/h		60 km/h		70 km/h	80 km/h	90 km/h	100 km/h
	4% MAX.	6% MAX.	4% MAX.	6% MAX.	4% MAX.	6% MAX.	4% MAX.	4% MAX.	4% MAX.	4% MAX.
40		6.0								
45		2.5								
50		NC								
60	4.0									
70	3.9			6.0						
80	3.8			2.6						
90	3.7			NC						
100	3.6		4.0							
110	3.5		4.0							
120	3.4		3.9			5.0				
130	3.3		3.8			3.2				
140	3.2		3.8			1.6				
150	3.1		3.7		4.0	NC				
175	2.9		3.5		3.9					
200	2.8		3.3		3.8					
250	2.6		3.0		3.6		3.9			
300	2.4		2.8		3.3		3.8	4.0		
400	2.1		2.5		3.0		3.4	3.7	4.0	
500	1.9		2.3		2.7		3.1	3.5	3.8	4.0
600	1.7		2.1		2.5		2.9	3.2	3.6	3.9
700	NC		1.9		2.3		2.7	3.0	3.4	3.7
800			1.7		2.1		2.5	2.8	3.2	3.5
900			1.6		2.0		2.4	2.7	3.0	3.4
1000			NC		1.9		2.2	2.5	2.8	3.2
1200					1.6		2.0	2.3	2.6	2.9
1300					NC		1.9	2.2	2.5	2.8
1400							1.8	2.1	2.4	2.7
1500							1.7	2.0	2.3	2.6
2000							NC	1.6	2.0	2.2
2500								NC	1.6	1.9
3000									NC	1.6
5000										NC
7000										

NC = NORMAL CROWN

NO SUPERELEVATION REQUIRED WHEN RADIUS IS GREATER THAN:

	40km/h	50km/h	60km/h	70km/h	80km/h	90km/h	100km/h
4% MAX:	679 m	950 m	1314 m	1737 m	2156 m	2636 m	3236 m
6% MAX:	47 m	84 m	141 m				

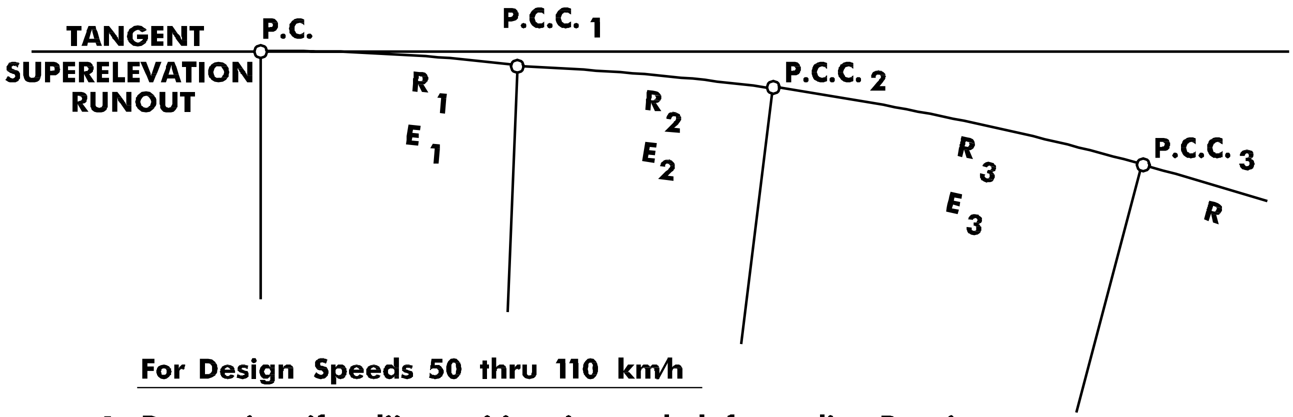
NOTE: SUPERELEVATION RATES LESS THAN 1.5% SHALL NOT BE USED.

TRANSITION CURVES

FIGURE: 4–D

BDC02MR–4

Note: All measurements are in meters unless otherwise noted.



For Design Speeds 50 thru 110 km/h

1. Determine if radii transition is needed for radius R using chart below.

   Transition curves not essential when radius is greater than:

Superelevation	50 km/h	60 km/h	70 km/h	80 km/h	90 km/h	100 km/h	110 km/h
6% Superelevation for rural hwys & rural or urban fwys	475	750	965	1150	1510	1860	2220
4% Superelevation for urban highways	300	430	630	860	1040	1220	NA

2. If required, use standard Transition Curves.
3. At P.C.C. 3 hold maximum E for radius R.
4. Using superelevation chart, determine if superelevation is needed for R_1.
5. If superelevation is needed for R_1 , use 2/3 maximum superelevation for R_1 at P.C.
6. Distribute superelevation evenly between P.C.C $_3$ and P.C.
7. Distribute superelevation at the same rate as in step 6 on tangent up to normal section. However, this superelevation transition may be reduced to 2% / sec. in certain locations, such as on short tangents between reverse curves or on a crest or sag vertical curve.

On Existing Roadways Or Where Radii Transitions Can Not Be Provided

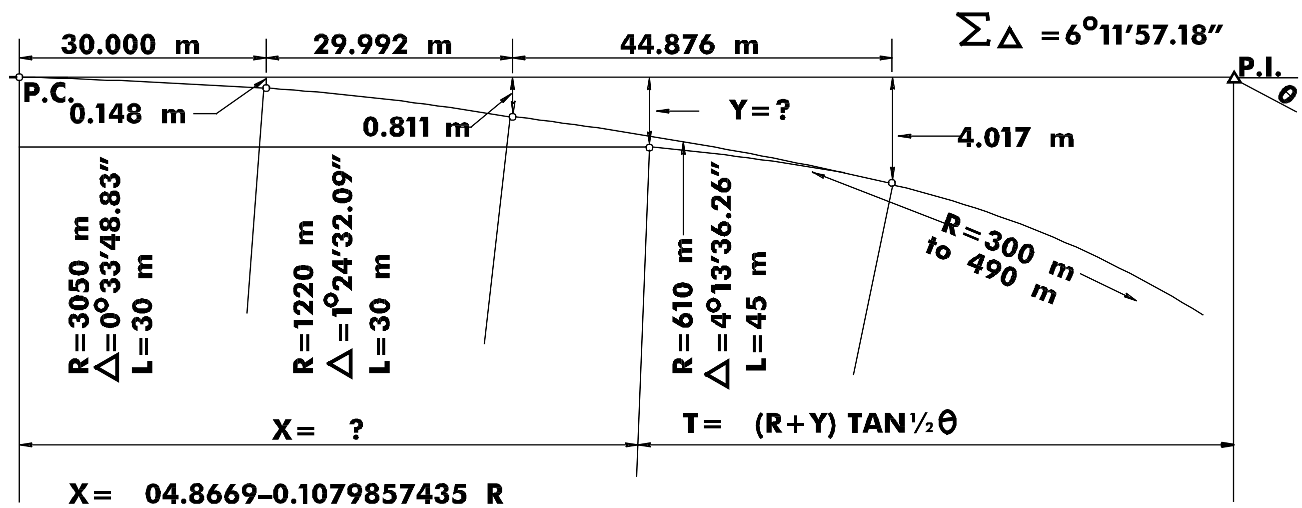
1. Determine maximum superelevation needed for radius (R).
2. See Section 4–03.2.2 for superelevation at P.C & P.T and superelevation transition.

TRANSITION CURVES

FIGURE: 4–E

BDC02MR–4

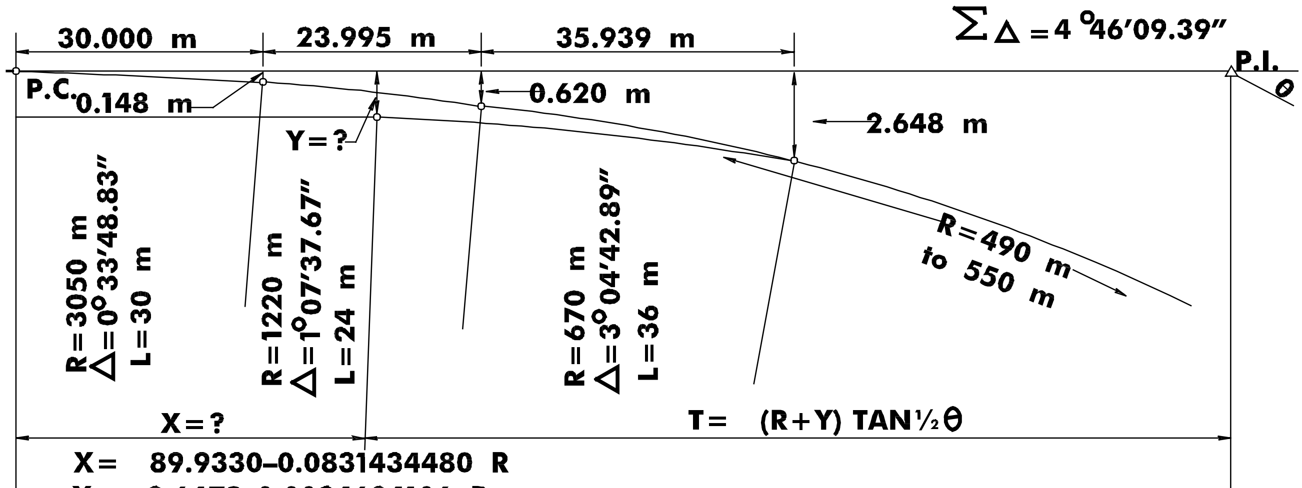
300 m TO 490 m RADIUS



X = 04.8669–0.1079857435 R

Y = 04.0170–0.0058475574 R

490 m TO 550 m RADIUS



X = 89.9330–0.0831434480 R

Y = 2.6478–0.0034624106 R

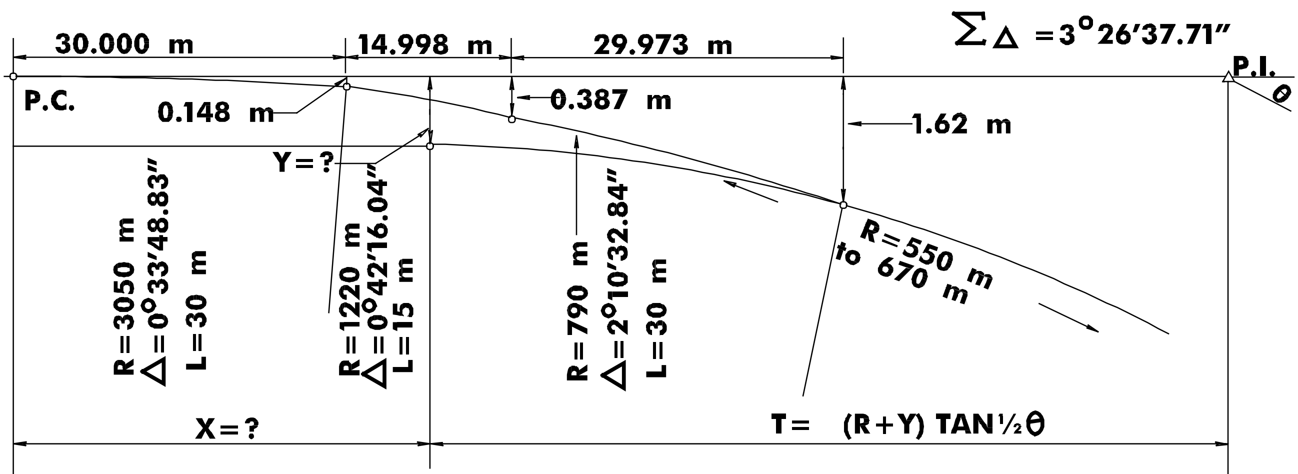
NOTE: TO LOCATE TRANSITION P.C.:
(1) FIND X AND Y FOR DESIRED RADIUS
(2) ADD RADIUS R TO Y DISTANCE
(3) FIND T FOR R&Y
(4) ADD T TO X FOR DISTANCE P.C. TO P.I.

TRANSITION CURVES

FIGURE: 4–F

BDC02MR–4

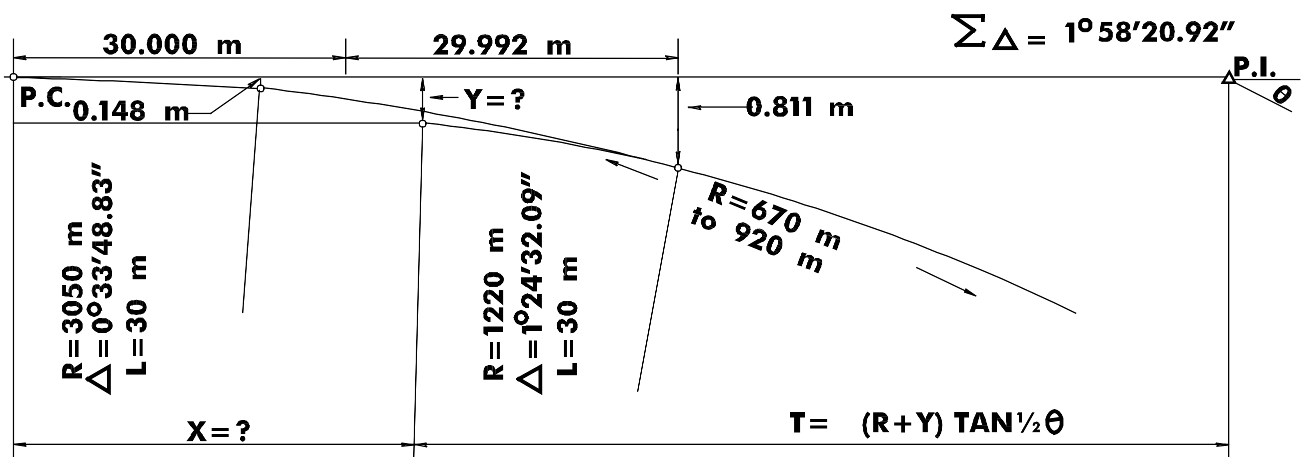
550 m TO 670 m RADIUS



X = 74.9703–0.0600696468 R

Y = 1.6204–0.0018058117 R

670 m TO 920 m RADIUS



X = 59.9914–0.0344194298 R

Y = 0.8114–0.0005925241 R

NOTE: TO LOCATE TRANSITION P.C.:

(1) FIND X AND Y FOR DESIRED RADIUS

(2) ADD RADIUS R TO Y DISTANCE

(3) FIND T FOR R&Y

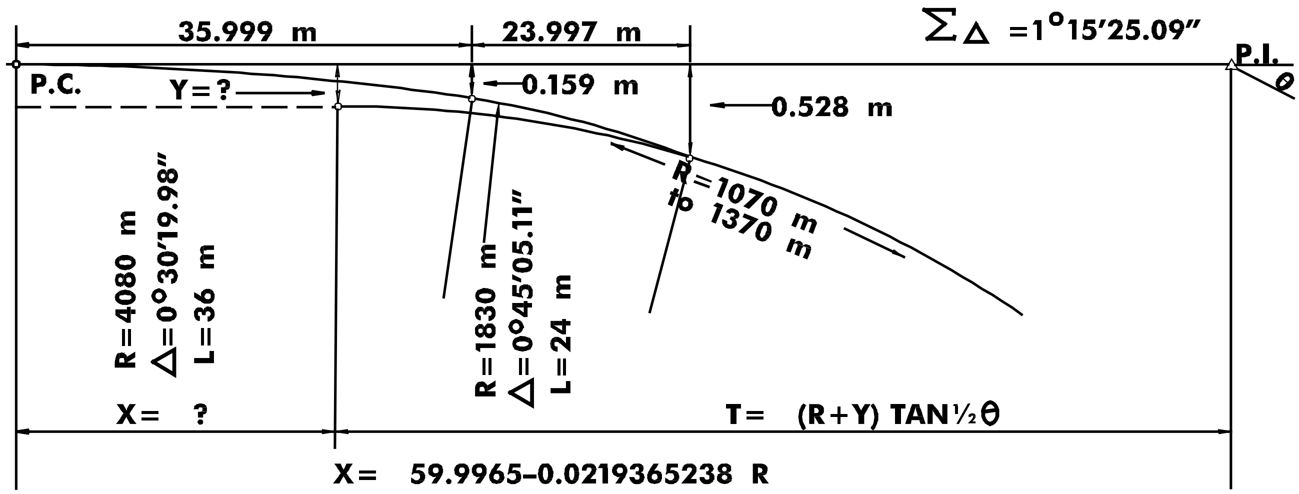
(4) ADD T TO X FOR DISTANCE P.C. TO P.I.

TRANSITION CURVES

FIGURE: 4–G

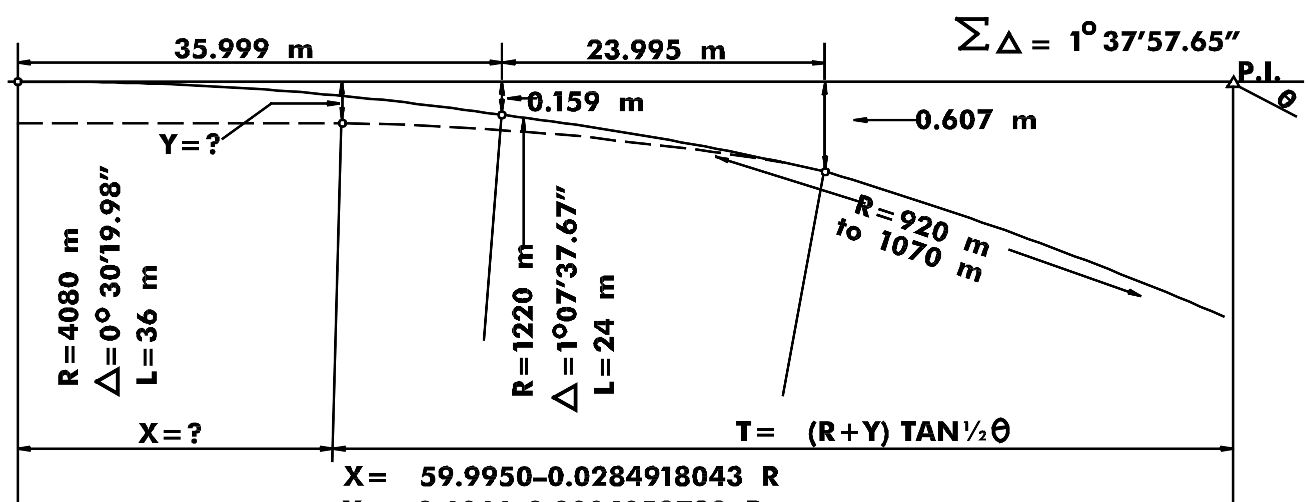
BDC02MR–4

1070 m TO 1370 m RADIUS



X= 59.9965–0.0219365238 R
Y= 0.5279–0.0002406345 R

920 m TO 1070 m RADIUS



X= 59.9950–0.0284918043 R
Y= 0.6066–0.0004059739 R

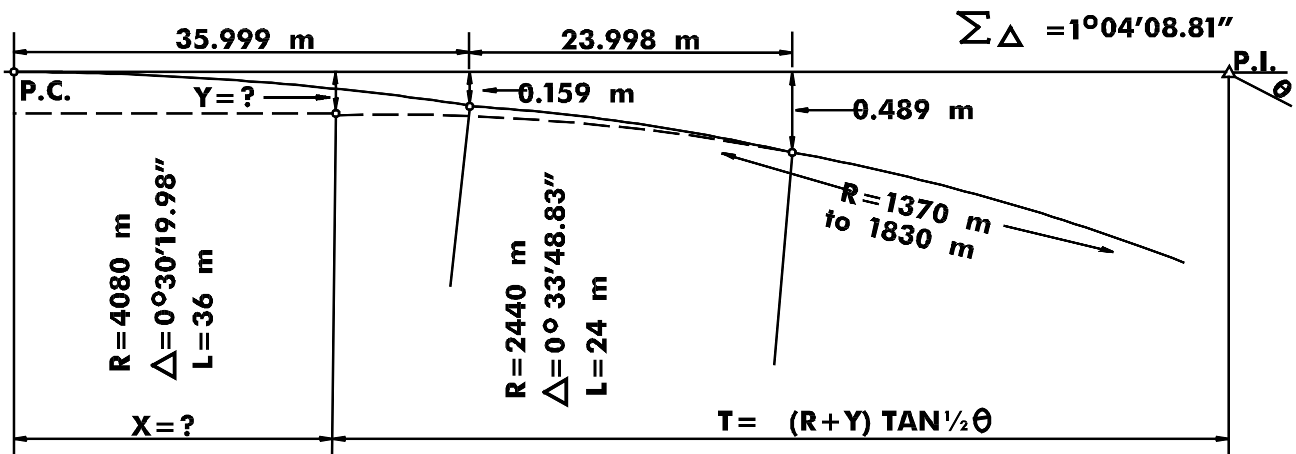
NOTE: TO LOCATE TRANSITION P.C.:
(1) FIND X AND Y FOR DESIRED RADIUS
(2) ADD RADIUS R TO Y DISTANCE
(3) FIND T FOR R&Y
(4) ADD T TO X FOR DISTANCE P.C. TO P.I.

TRANSITION CURVES

FIGURE: 4–H

BDC02MR–4

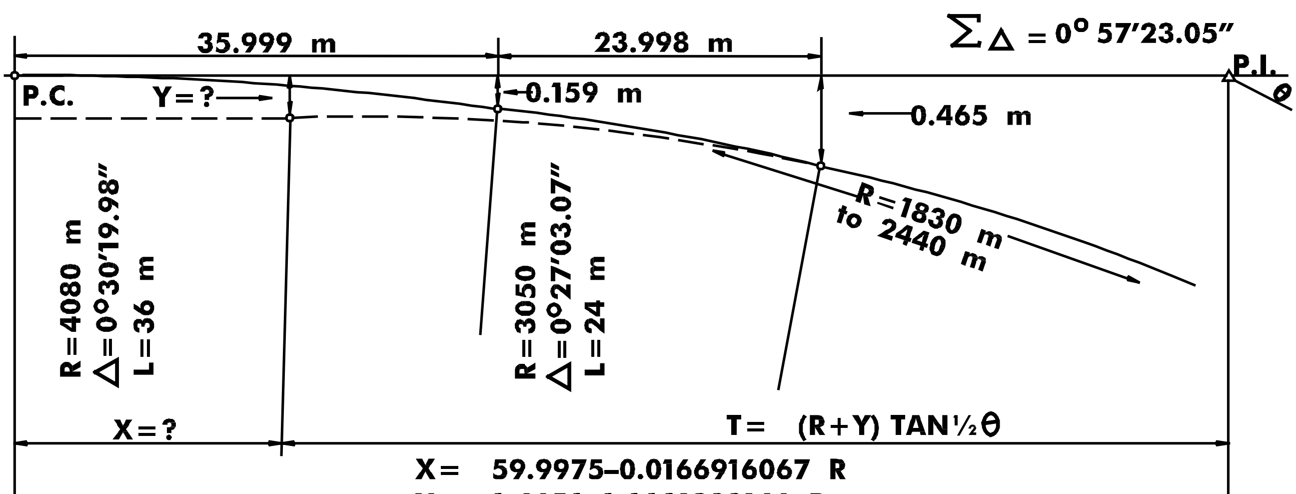
1370 m TO 1830 m RADIUS



X= 59.9972–0.0186585122 R

Y= 0.4886–0.0001740852 R

1830 m TO 2440 m RADIUS



X= 59.9975–0.0166916067 R

Y= 0.4650–0.0001393146 R

NOTE: TO LOCATE TRANSITION P.C.:

(1) FIND X AND Y FOR DESIRED RADIUS

(2) ADD RADIUS R TO Y DISTANCE

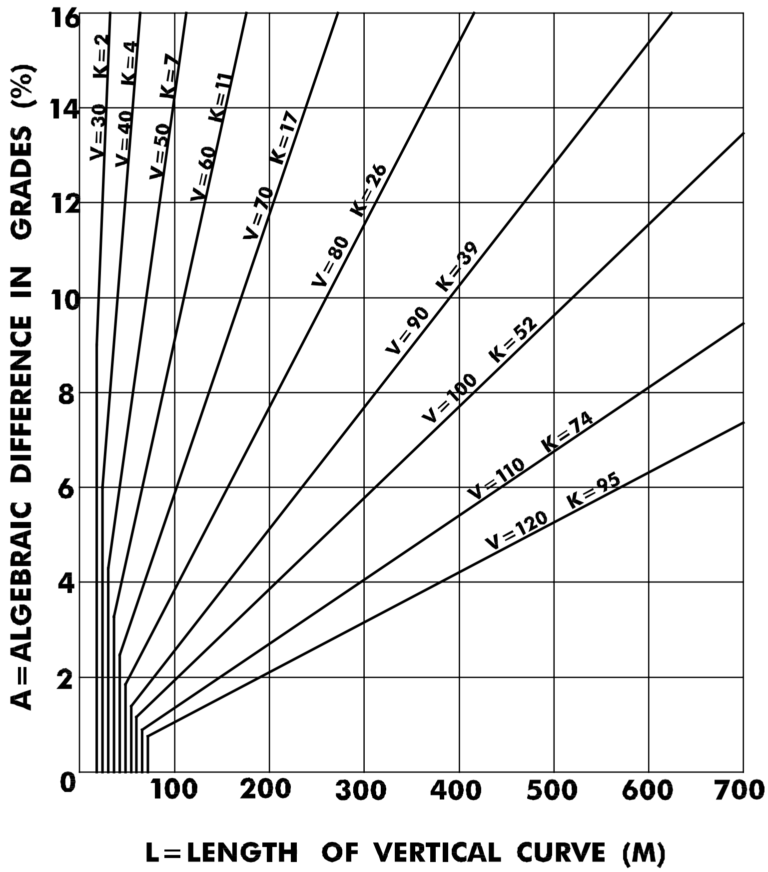
(3) FIND T FOR R&Y

(4) ADD T TO X FOR DISTANCE P.C. TO P.I.

DESIGN CONTROLS FOR CREST VERTICAL CURVES

FIGURE: 4–I

BDC02MR–4



NOTE: Drainage of the Roadway on CREST Vertical Curves must be more carefully designed when the Design Speed exceeds 90 KM / H

When S is greater than L, $L = 2S - \frac{658}{A}$

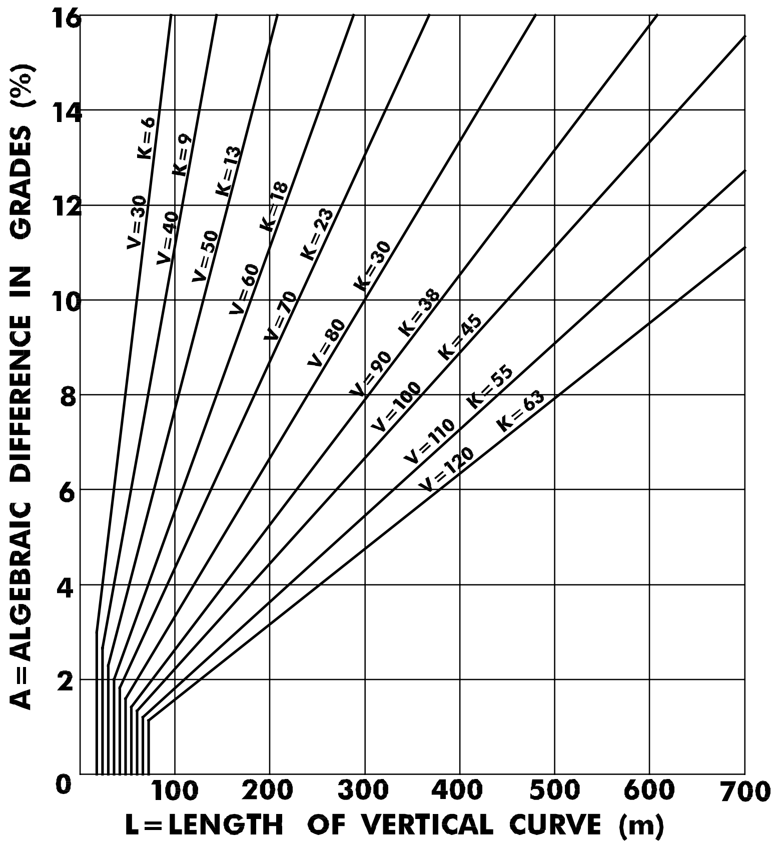
When S is less than L, $L = \frac{AS^2}{658}$

V = Design Speed, km/h
S = Stopping Sight Distance, m
A = Algebraic Difference In Tangent Grades, Percent
K = Horizontal Distance Required To Effect A Percent Change In Gradient, m
L = KA

DESIGN CONTROLS FOR SAG VERTICAL CURVES

FIGURE: 4–J

BDC02MR–4



NOTE: Drainage of the Roadway on SAG Vertical Curves must be more carefully designed when the Design Speed exceeds 100 KM / H

When S is greater than L, $L = 2S - \dfrac{120 + 3.5S}{A}$

When S is less than L, $L = \dfrac{AS^2}{120 + 3.5S}$

V = Design Speed km/h

S = Light Beam Distance, m

A = Algebraic Difference In Tangent Grades, Percent

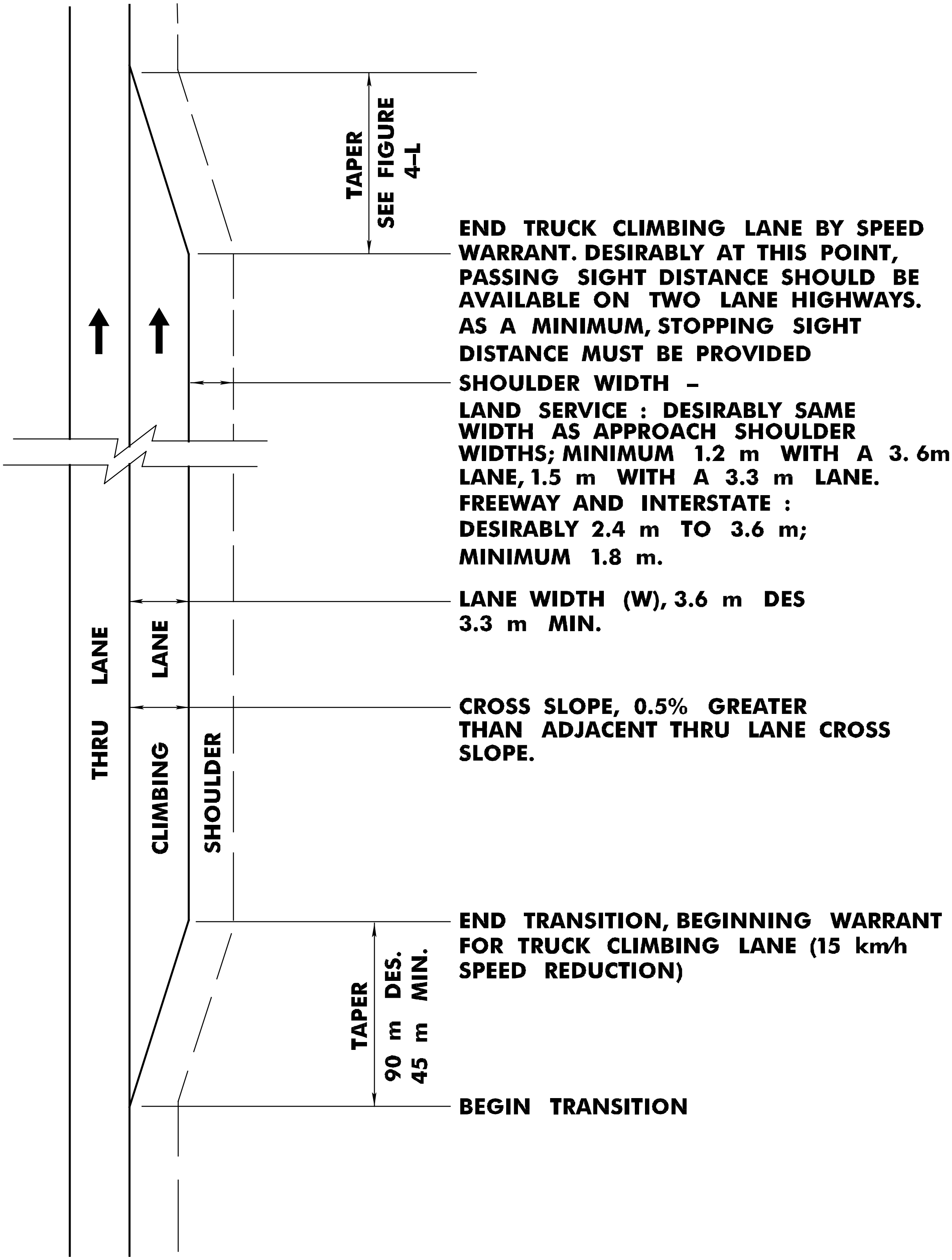
K = Horizontal Distance Required To Effect A Percent Change In Gradient, m

L = KA

CLIMBING LANE

FIGURE: 4–K

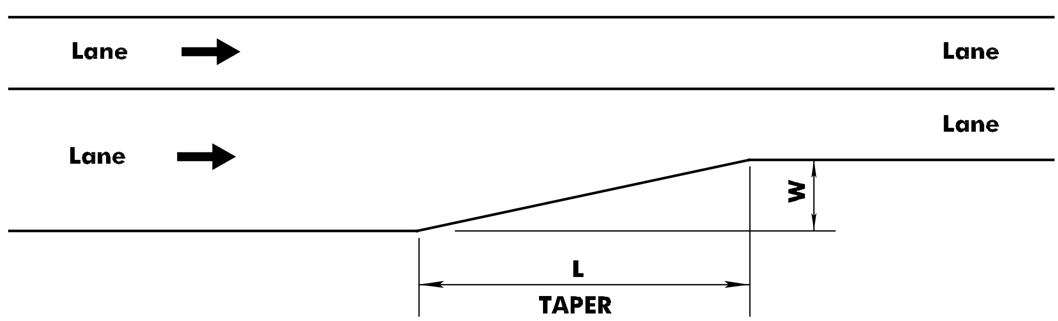
BDC02MR–4



LANE TRANSITION

FIGURE: 4–L

BDC02MR–4



FOR DESIGN SPEEDS GREATER THAN 60 km/h, $L = \frac{WV}{1.6}$

FOR DESIGN SPEEDS EQUAL TO OR LESS THAN 60 km/h, $L = \frac{WV^2}{150}$.

V = DESIGN SPEED (km/h)

W = LANE WIDTH REDUCTION (m)

L = TAPER LENGTH (m)

SIGHT DISTANCE AT INTERSECTIONS FOR LEFT, OR RIGHT TURNING & CROSSING VEHICLES WITH STOP CONTROL

FIGURE: 6–A

BDC02MR–4

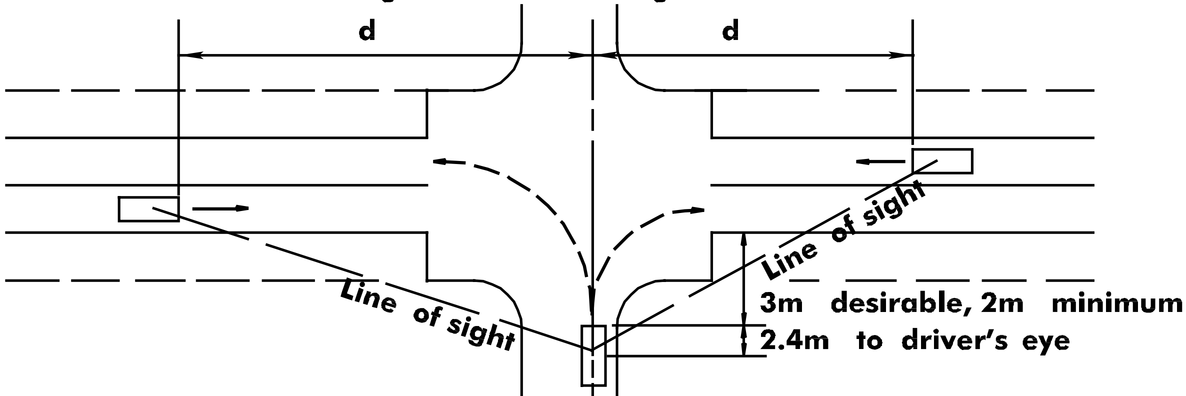
Intersection Sight Distance(d) Stop Control on Minor Road
Two Lane Highway

Design Speed	Left–Turn			Right–Turn or Cross		
	P	SU	WB	P	SU	WB
30	65	80	95	55	70	90
40	85	105	130	75	95	115
50	105	130	160	95	120	145
60	130	160	190	110	140	175
70	150	185	225	130	165	205
80	170	210	255	145	190	235
90	190	240	290	165	215	265
100	210	265	320	185	235	290
110	230	290	350	200	260	320
120	255	315	385	220	285	350

For highways with more than 2 lanes or when approach grade on minor road exceeds 3%, the distance (d) must be calculated using the formula: $d = 0.278 V t_g$

Design Vehicle	Time Gap, t_g Left–Turn	Time Gap, t_g Right–Turn & Cross
P	7.5 (See Notes)	6.5 (See Notes)
SU	9.5 (See Notes)	8.5 (See Notes)
WB	11.5 (See Notes)	10.5 (See Notes)

Notes:
1. For left turn or crossing add 0.5 sec. for P and 0. 7 sec. for SU & WB for each additional lane crossed.
2. For each percent the upgrade on minor road exceeds 3%, add 0.1 sec for right turn or crossing and 0.1 sec for left turn



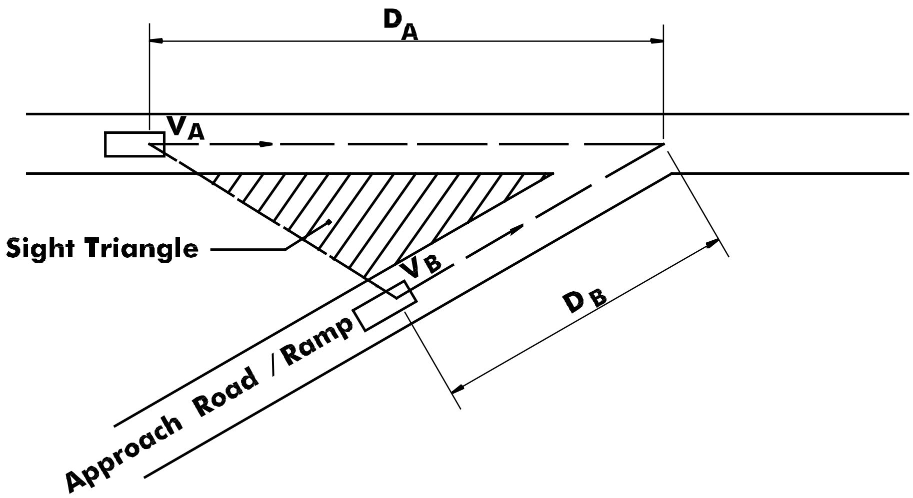
Source: A policy on Geometric Design of Highways and Streets, A.A.S.H.T.O., 2001

FIGURE INTENTIONALLY LEFT BLANK	FIGURE: 6–B
	BDC02MR–4

YIELD CONTROL

FIGURE: 6–C

BDC02MR–4



WITH ACCELERATION LANE

DESIGN AND/OR APPROACH SPEED (km/h) V_A OR V_B	DISTANCE (m) D_A OR D_B
30	25
40	35
50	40
60	50
70	60
80	65
90	75
100	85
110	90

WITHOUT ACCELERATION LANE

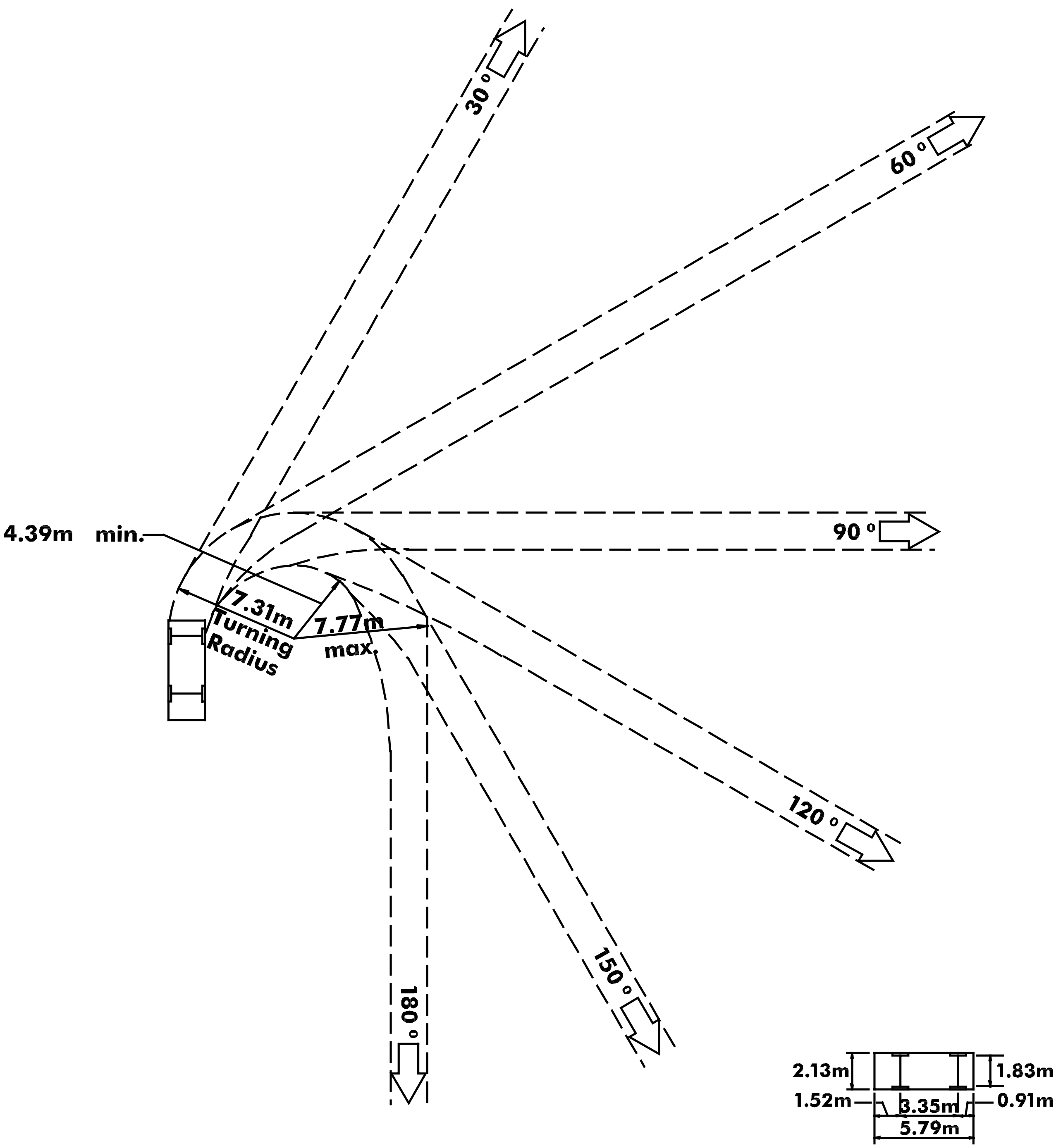
DESIGN AND/OR APPROACH SPEED (km/h) V_A OR V_B	DISTANCE (m) D_A	DISTANCE (m) D_B
30	70	35
40	90	50
50	115	65
60	135	85
70	160	105
80	180	130
90	205	160
100	225	185
110	245	220

Note: For ramps and minor roads D_B = 25m. For major roads use D_B from chart.

MINIMUM TURNING PATH FOR P DESIGN VEHICLE

FIGURE: 6–D

BDC02MR–4

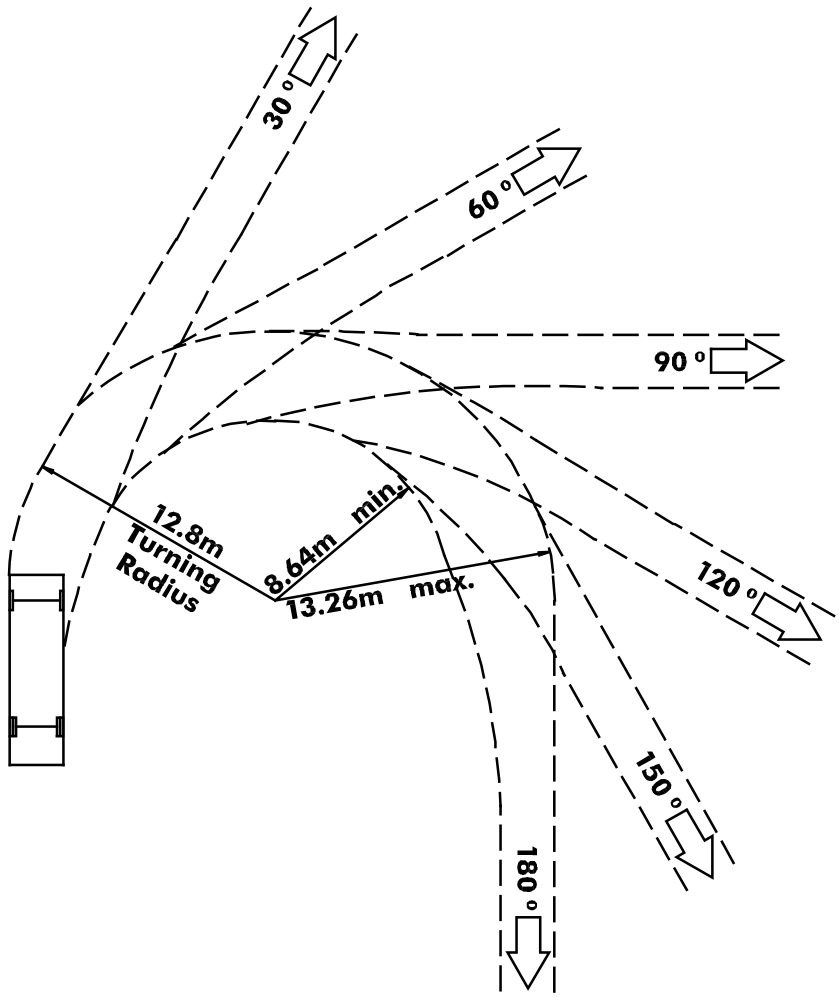


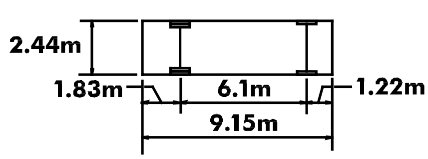
Source: A Policy on Geometric Design of Highways and Streets, A.A.S.H.T.O., 2001

MINIMUM TURNING PATH FOR SU DESIGN VEHICLE

FIGURE: 6–E

BDC02MR–4



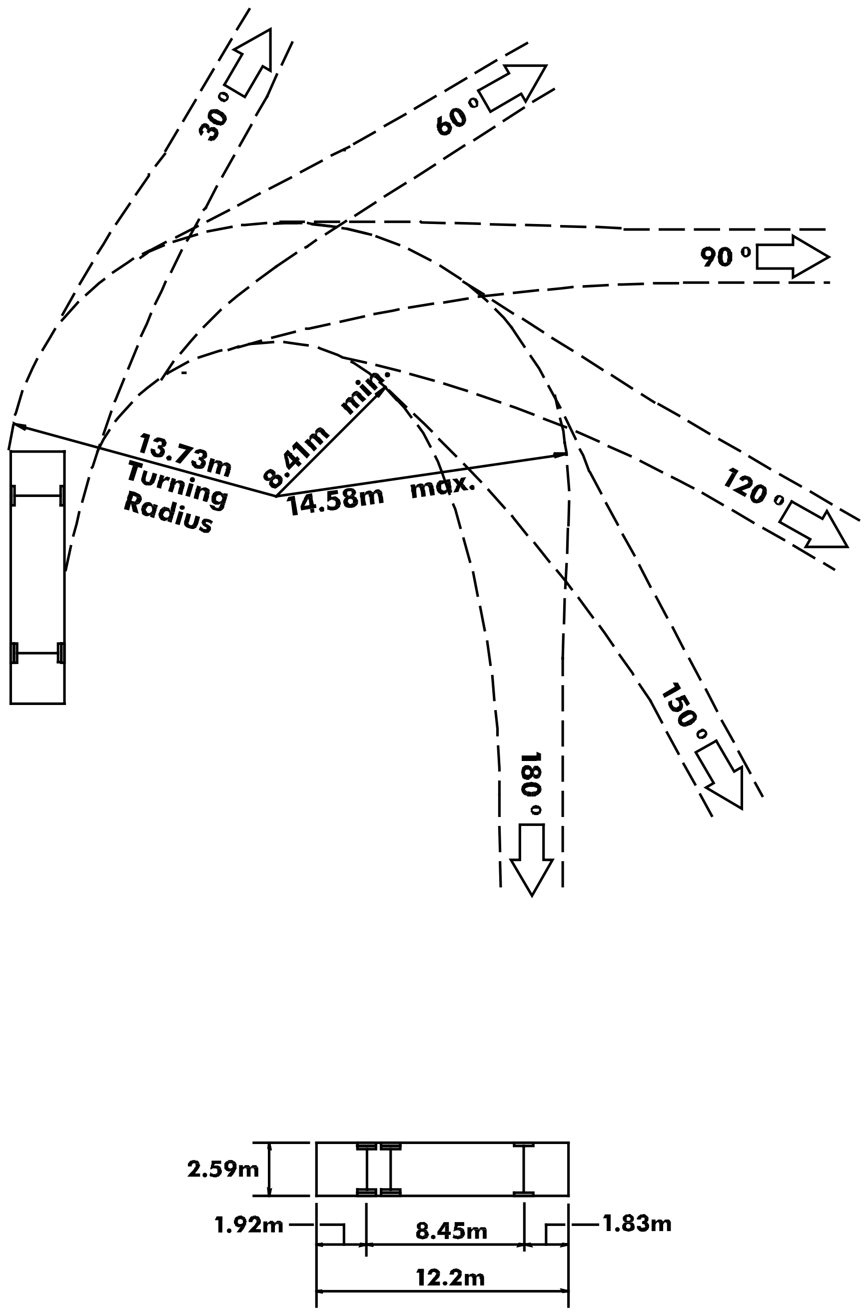


Source: A Policy on Geometric Design of Highways and Streets, A.A.S.H.T.O., 2001

MINIMUM TURNING PATH FOR BUS–12 DESIGN VEHICLE

FIGURE: 6–F

BDC02MR–4

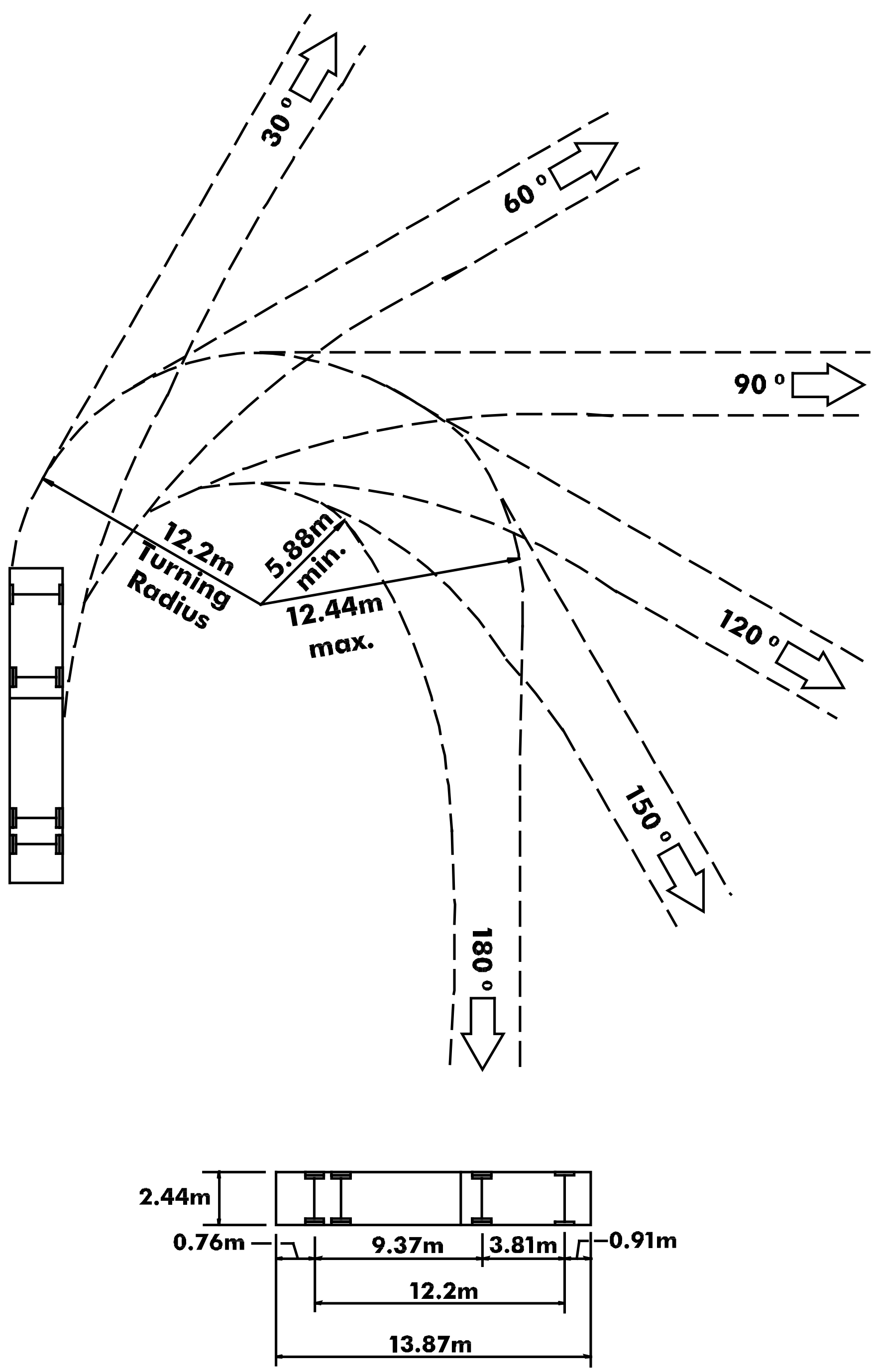


Source: A Policy on Geometric Design of Highways and Streets, A.A.S.H.T.O., 2001

MINIMUM TURNING PATH FOR WB–12 DESIGN VEHICLE

FIGURE: 6–G

BDC02MR–4

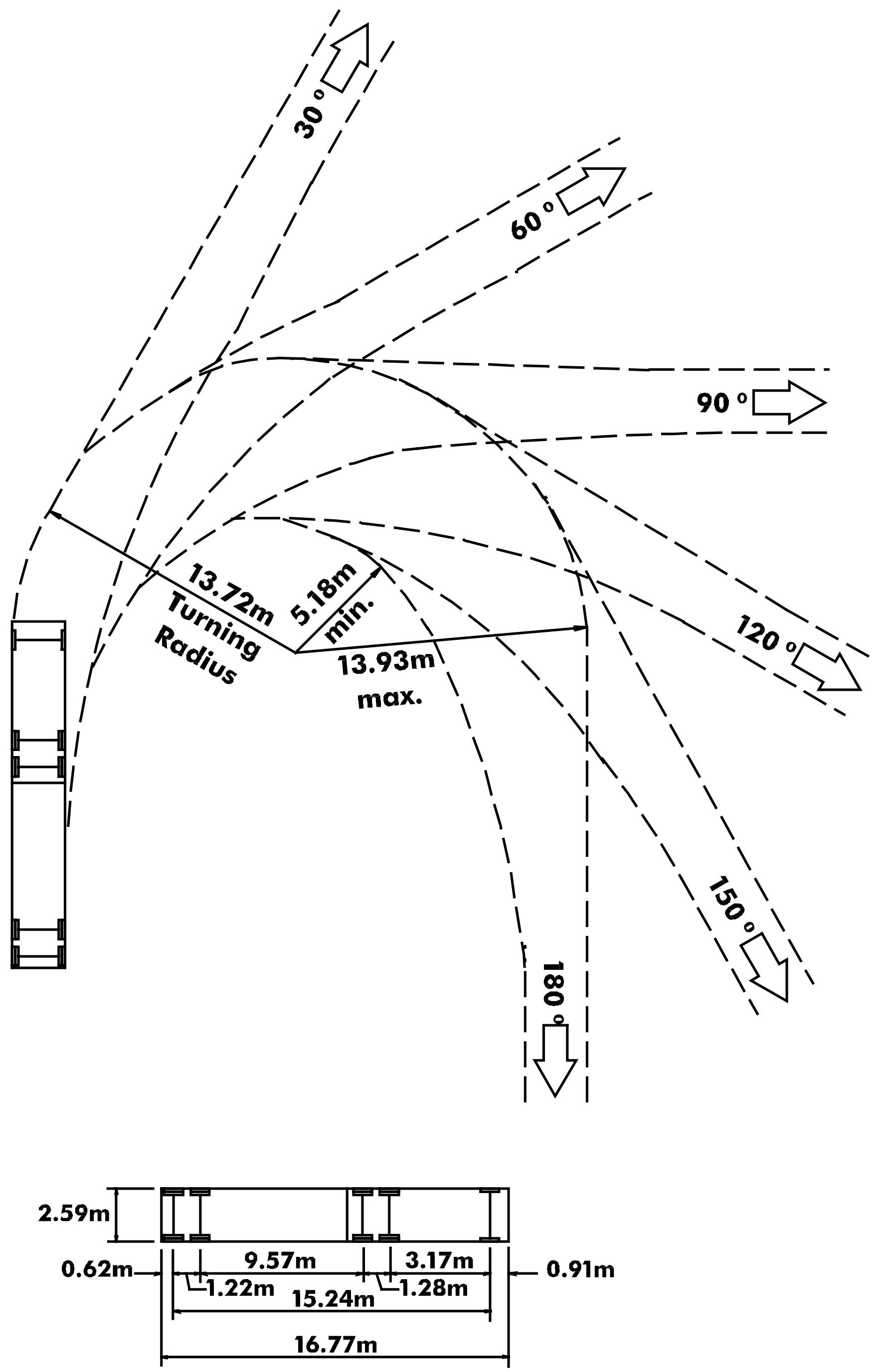


Source: A Policy on Geometric Design of Highways and Streets, A.A.S.H.T.O., 2001

MINIMUM TURNING PATH FOR WB–15 DESIGN VEHICLE

FIGURE: 6–H

BDC02MR–4

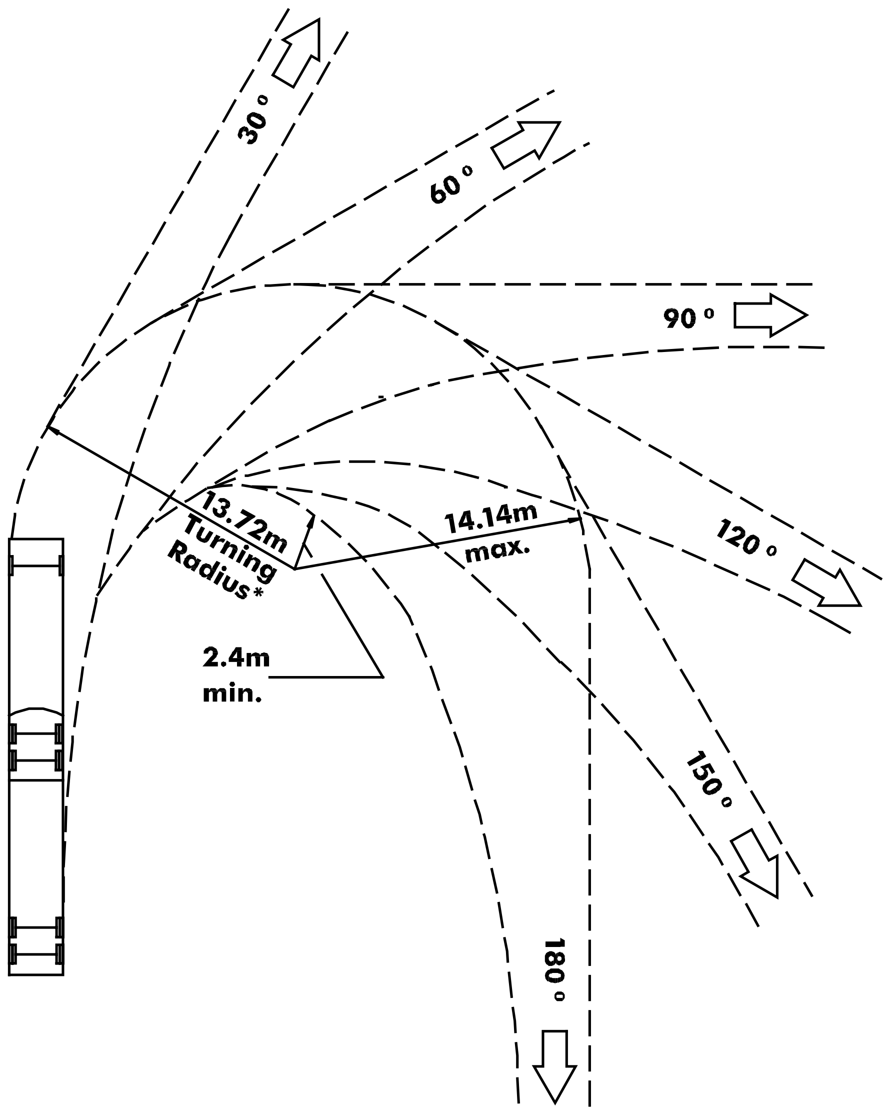


Source: A Policy on Geometric Design of Highways and Streets, A.A.S.H.T.O., 2001

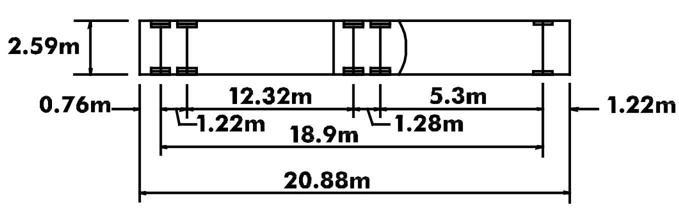
MINIMUM TURNING PATH FOR WB–19 DESIGN VEHICLE

FIGURE: 6–I

BDC02MR–4



Note: Caltrans 15.2m turning radius is approved for use also.

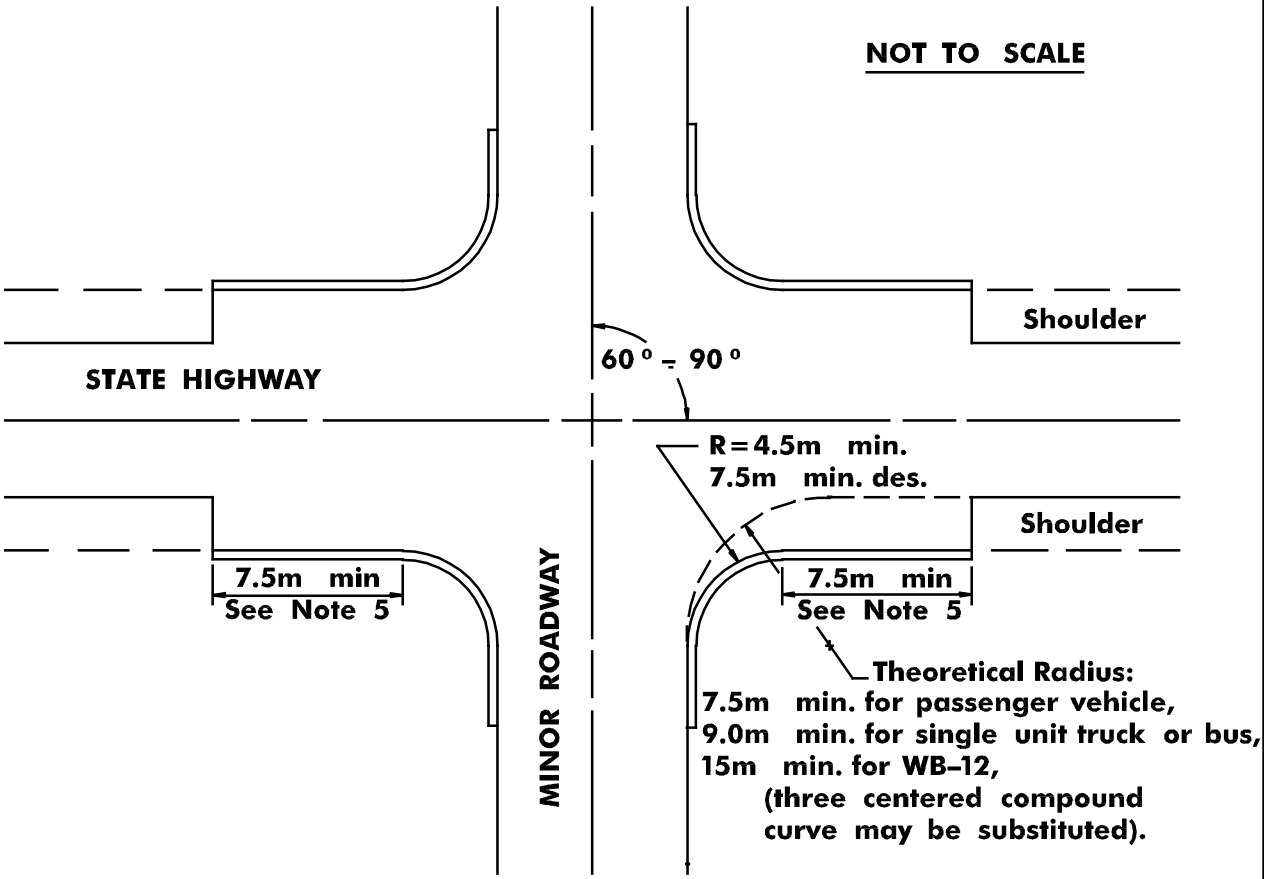


Source: A policy on Geometric Design of Highways and Streets, A.A.S.H.T.O., 2001

INTERSECTION TURNING RADII

FIGURE: 6–J

DATE: 12/18/95



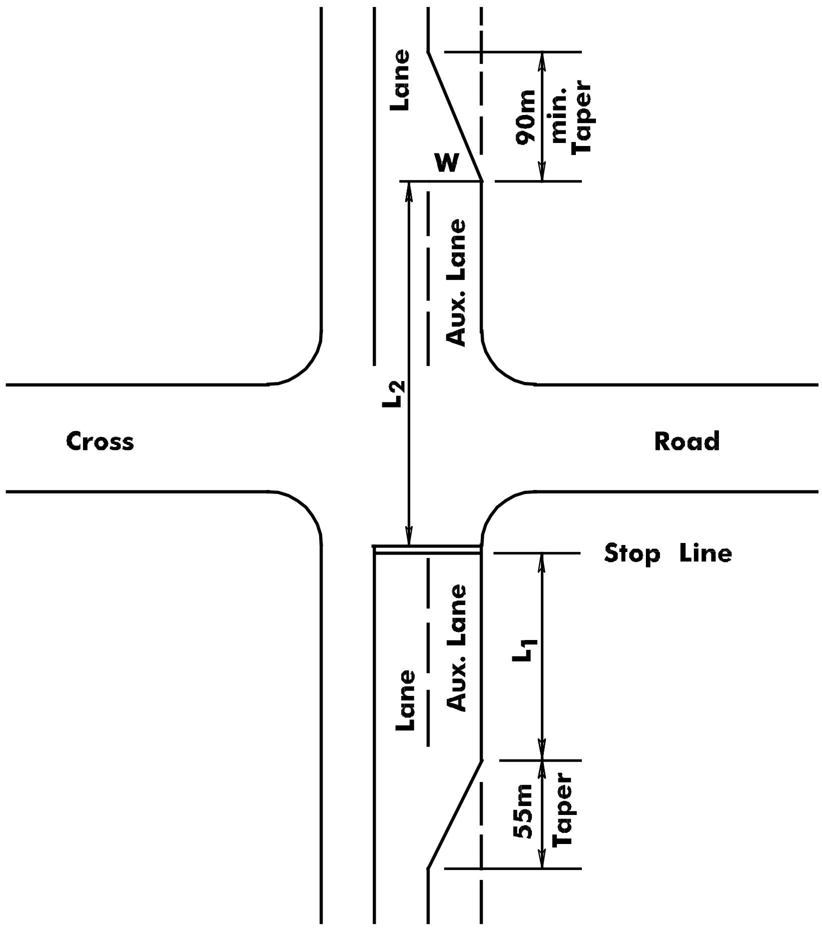
DESIGN GUIDELINES

1. Physical curb return should be clear of theoretical radius.
2. Truck volumes dictate the theoretical radius to be used. Where truck traffic is light, a SU truck radius should be used where possible.
3. A turning template for the appropriate design vehicle should be used to check the adequacy of radii returns, especially for WB–15 and WB–19 trucks.
4. For intersection skew angles less than 60 °, channelization should be provided.
5. Where turning volumes are high, auxilary lanes through the intersection may be warranted.
6. Check applicable sight distances.

AUXILIARY LANE ADDITIONS AT SIGNALIZED INTERSECTIONS

FIGURE: 6–N

BDC00MR–1



NOT TO SCALE

LENGTH OF ADDITIONAL WIDENING BEYOND INTERSECTION

L_2= Length of auxiliary lane equals the greater of:

DESIGN SPEED (km/h)	L_2 (m)
60 or less	100 min
70	145
80	195
90	275
100	370

OR

L_2 = 3 x minimum green time G (sec.) for approach signal
If G = 40, then L_2 = 3x40 sec = 120m

LENGTH OF ADDITIONAL WIDENING IN ADVANCE OF INTERSECTION

L_1= Length of auxiliary lane:

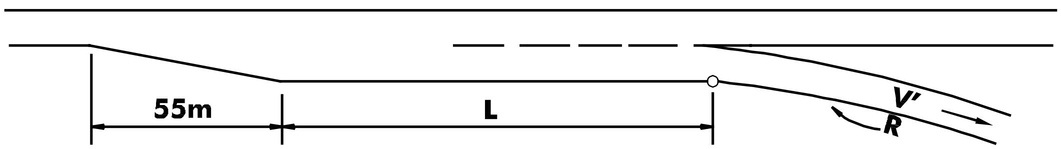
DESIGN SPEED (km/h)	L_1 (m)
60	95
70	110
80	130
90	145
100	170

LAND SERVICE HIGHWAYS ACCELERATION/DECELERATION LENGTHS

FIGURE: 6–O

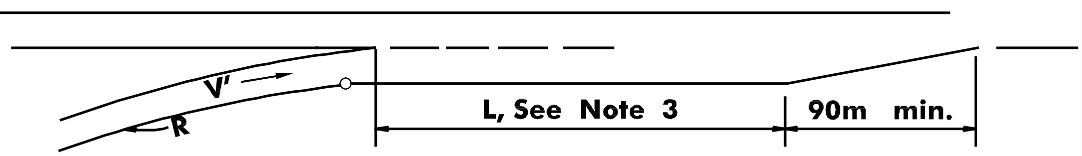
BDC02MR–4

LENGTH OF DECELERATION LANES



HIGHWAY DESIGN SPEED km/h (V)	L = LENGTH OF DECELERATION LANE – METERS							
	FOR DESIGN SPEED OF EXIT CURVE – km /h (V')							
	STOP CONDITION	20 10m R	30 25m R	40 45m R	50 70m R	60 150m R	70 195m R	80 250m R
	FOR AVERAGE RUNNING SPEED ON EXIT CURVE – km/h (V'a)							
	0	20	28	35	42	51	63	70
50	75	70	60	45	—	—	—	—
60	95	90	80	65	55	—	—	—
70	110	105	95	85	70	55	—	—
80	130	125	115	100	90	80	55	—
90	145	140	135	120	110	100	75	60
100	170	165	155	145	135	120	100	85
110	180	180	170	160	150	140	120	105

LENGTH OF ACCELERATION LANES



HIGHWAY DESIGN SPEED km/h (V)	L= LENGTH OF ACCELERATION LANE – METERS							
	FOR DESIGN SPEED OF ENTRANCE CURVE – km /h (V')							
	STOP CONDITION	20 10m R	30 25m R	40 45m R	50 70m R	60 150m R	70 195m R	80 250m R
	AND INITIAL SPEED – km/h (V'a)							
	0	20	28	35	42	51	63	70
50	60	50	30	—	—	—	—	—
60	95	80	65	45	—	—	—	—
70	150	130	110	90	65	—	—	—
80	200	180	165	145	115	65	—	—
90	260	245	225	205	175	125	35	—
100	345	325	305	285	255	205	110	40
110	430	410	390	370	340	290	200	125

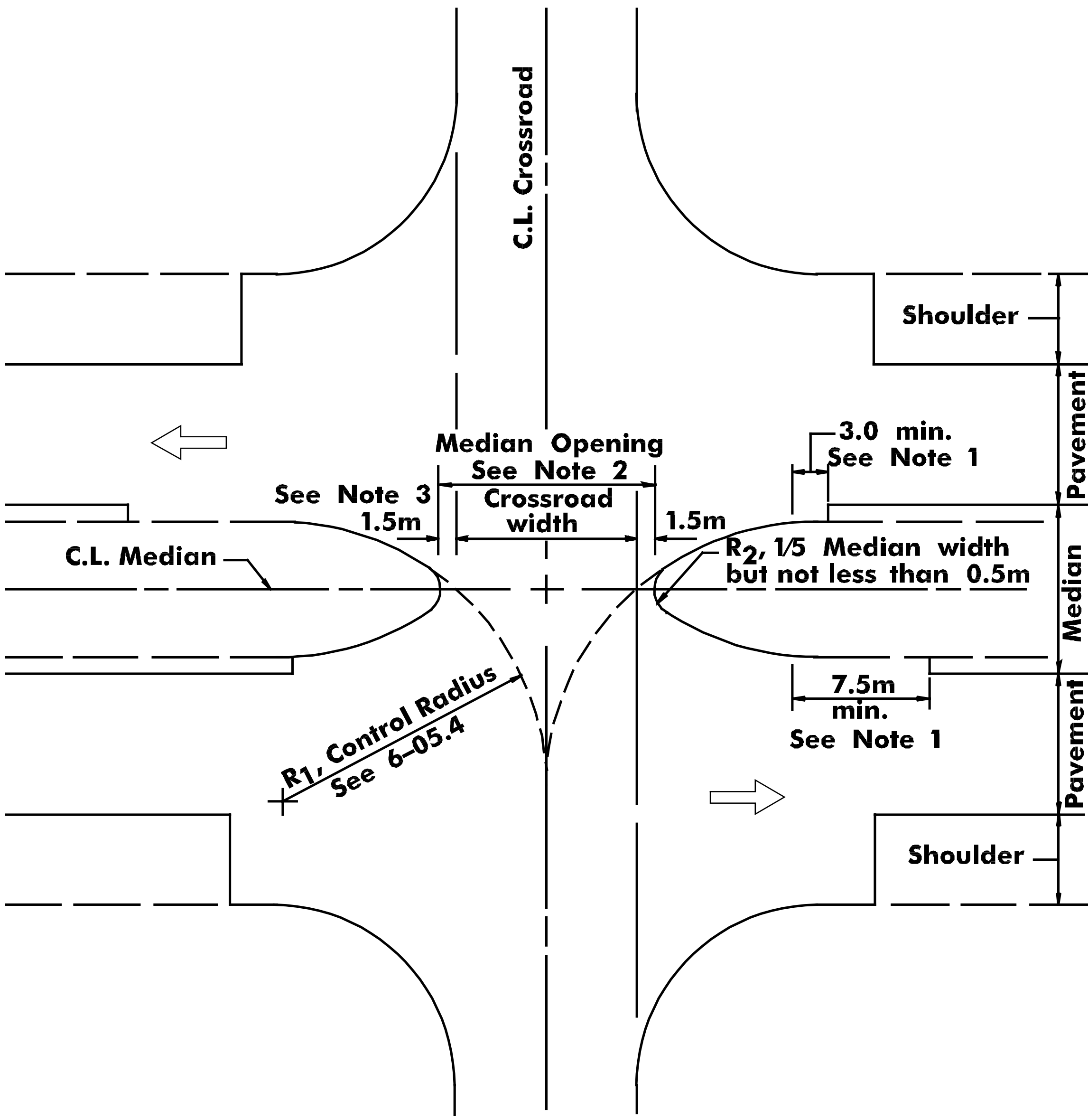
NOTES:
1. Minimum radii shown are for intersection curves. For design speeds of more than 60km/h, use values for open highway curves.
2. These tables apply to flat grades of 2% or less. For grades steeper than 2%, use the adjustments for grade in Exhibit 10–71 of the source shown below.
3. "L" may start back on the curvature of the ramp where the entrance radius is equal to or greater than 300m and the motorist on the ramp has an unobstructed view of traffic on the through lanes to his left.

Source: A Policy on Geometric Design of Highways and Streets, A.A.S.H.T.O., 2001

GRASS MEDIAN OPENING

FIGURE:6–P

BCD00MR–1



NOT TO SCALE

NOTES:

1. Where lane and shoulder pavement are different, use the mininmum offsets shown above.
2. In no case shall the length of the median opening be less than 12.0m.
3. Provide 3.0m where there is a marked or unmarked pedestrian crosswalk. As an alternative, provide median cut through for pedestrian traffic.

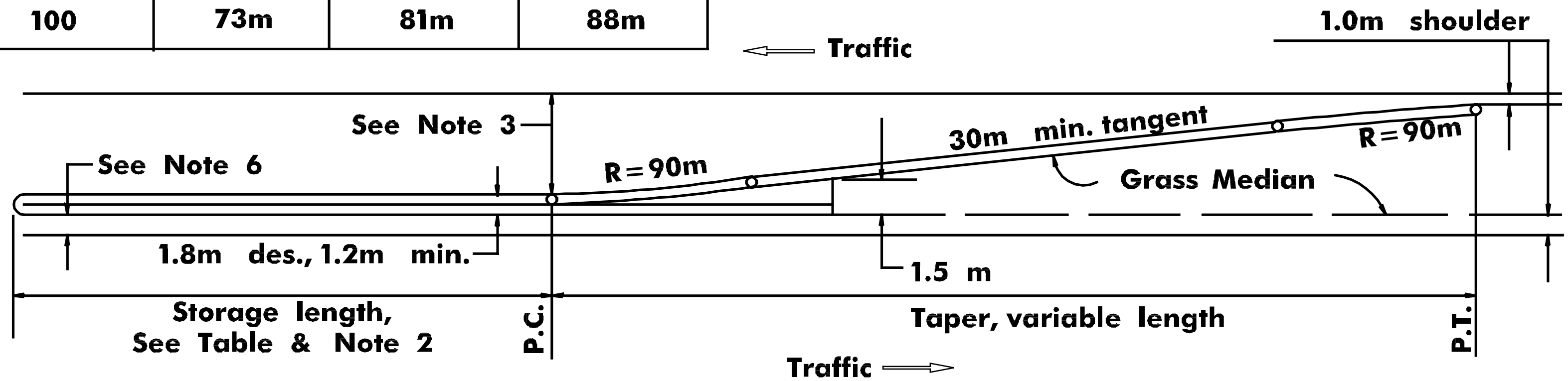
TYPICAL LEFT–TURN SLOT

FIGURE: 6–T

BDC00MR–1

Design Speed (km/h)	Storage Length	Taper Length	
		3.3m Lane	3.6m Lane
50	30m	40m	44m
60	38m	53m	58m
80	59m	67m	73m
100	73m	81m	88m

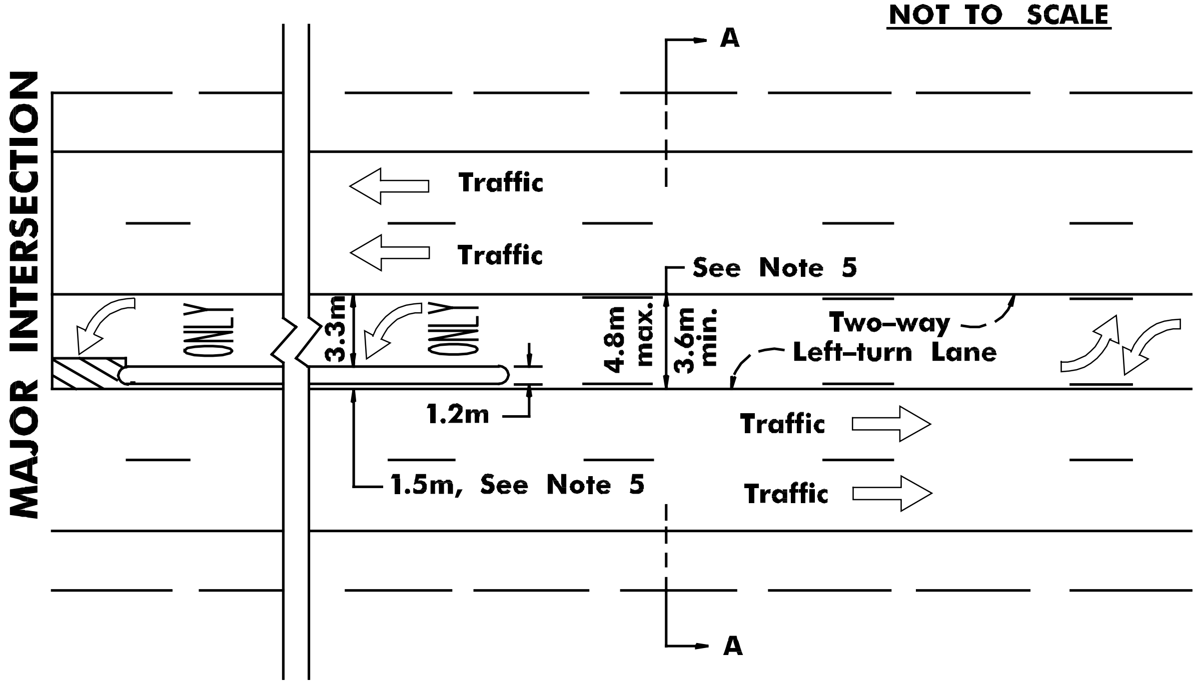
NOT TO SCALE

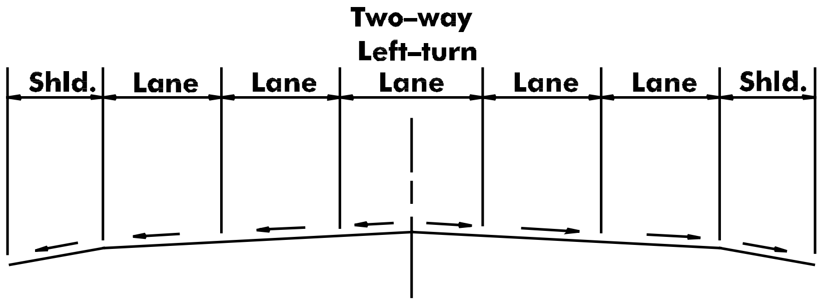


DESIGN GUIDELINES

1. Maximum vertical curb face: 100mm or use sloping curb.
2. Increase storage length, shown in table, 30m for each 100 DHV (turning vehicles) at unsignalized intersections. For signalized intersections check storage length with Traffic Engineering.
3. Left–turn lane width:
   Curbed: 4.2m desirable, 3.3m minimum.
   Painted: Desirably same width as adjacent traveled lane, minimum 3.3m. It may be reduced to 3m on projects where the posted speed is 60km/h or less.
4. For painted left–turn slots, a minimum 20m taper maybe provided.
5. Left–turn lane shall not be constructed adjacent to barrier curb.
6. Where there is a continuous median with shoulders, the offset shall be the same as the approach shoulder. Where there is no continuous median with shoulder, provide a 1.0m minimum offset.

TWO–WAY LEFT–TURN LANE	FIGURE: 6–U
	BDC02MR–4





SECTION A–A

DESIGN GUIDELINES

1. For desirable & minimum stopping sight distance requirements, see Table 6–1.
2. For proper signing and paint striping consult the "Manual On Uniform Traffic Control Devices", Sections 2B.22 and 3B.02.
3. Two–way left–turn lanes are not recommended where the number of thru lanes exceeds two lanes in each direction.
4. Divisional island used only when median width is 4.8m maximum.
5. Where the design speed is equal to or greater than 80km/h, the recommended paint line offset to divisional island is 0.6m, which would increase the two–way left–turn maximum lane width by 0.3m.

DESIGN WIDTHS OF PAVEMENT FOR TURNING ROADWAYS

FIGURE: 7-B

DATE: 12/18/95

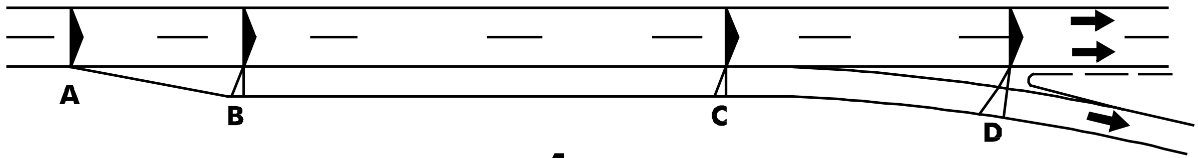
R RADIUS ON INNER EDGE OF PAVEMENT, METERS	PAVEMENT WIDTH (W) IN METERS FOR:		
	CASE I ENTRANCE TERMINAL WIDTH	CASE II RAMP PROPER WIDTH 1-LANE, ONE WAY OPERATION	CASE III RAMP PROPER WIDTH 2-LANE, ONE WAY OR TWO-WAY OPERATION
15	6	7.8	—
25	5.7	7.2	—
30	5.1	6.9	SEE NOTE 4
45	5.1	6.6	9.6
75	5.1	6.6	9.3
100	5.1	6.6	9
125	5.1	6.6	9
150	5.1	6.6	9
TANGENT	5.1	6.6	8.7

NOTES:
1. RAMP WIDTHS ARE APPLICABLE FOR RAMPS WITH OR WITHOUT CURB.
2. MINIMUM RAMP RADII WILL BE USED TO DETERMINE RAMP WIDTH. WIDTH WILL BE APPLIED THROUGH ENTIRE RAMP EXCEPT AT THE RAMP TERMINALS.
3. ON 2-LANE RAMPS WHERE SHOULDERS 1.2 m OR WIDER ARE PROVIDED, REDUCE RAMP PAVEMENT WIDTH BY 1.2 m.
4. 2-LANE OPERATION SHOULD NOT BE CONSIDERED ON RAMPS WITH RADII LESS THAN 45 m.
5. WHEN PERCENTAGE OF SEMITRAILER VEHICLES EXCEEDS 10%, INCREASE CASE I WIDTHS BY 0.3 m, CASE II AND CASE III WIDTHS BY 0.6 m.

DEVELOPMENT OF SUPERELEVATION AT FREE–FLOW RAMP TERMINALS

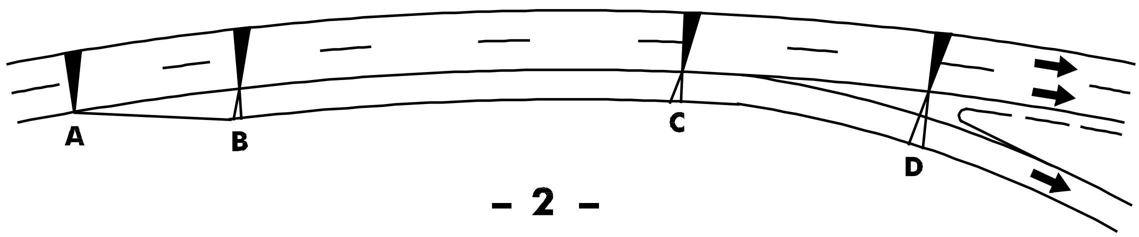
FIGURE: 7–H

DATE: 12/18/95



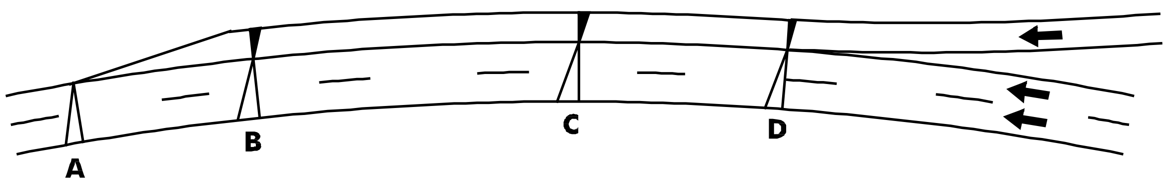
– 1 –

TANGENT DECELERATION LANE TYPE EXIT



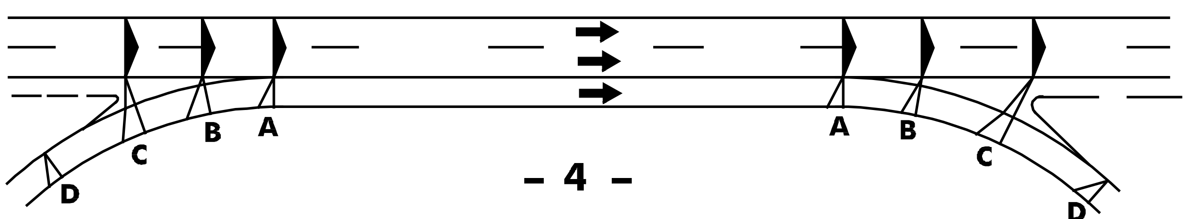
– 2 –

CURVED SECTION DECELERATION LANE TYPE EXIT



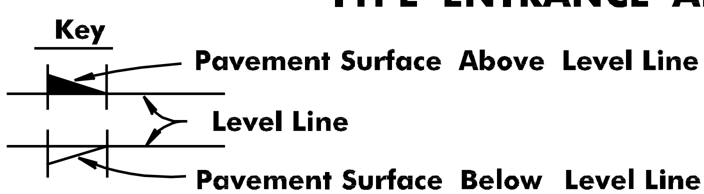
– 3 –

CURVED SECTION ACCELERATION LANE TYPE ENTRANCE ON HIGH SIDE OF SUPERELEVATION



– 4 –

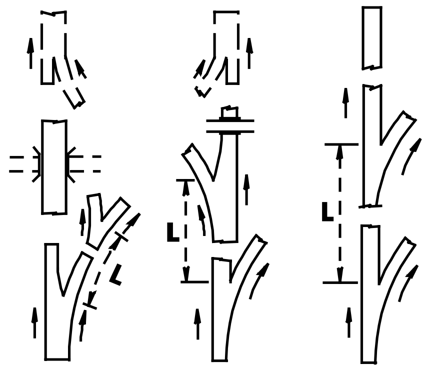
TANGENT SECTION CLOVERLEAF TYPE ENTRANCE AND EXIT



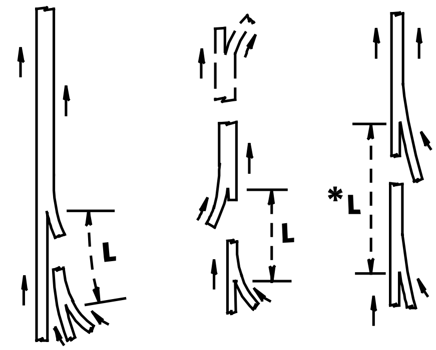
ARRANGEMENTS FOR SUCCESSIVE RAMP TERMINALS

FIGURE: 7–I

BDC02MR–4

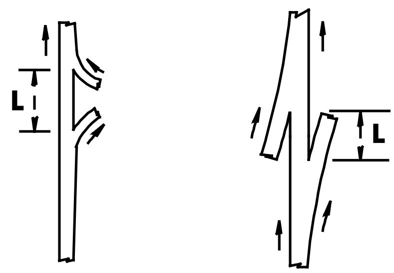


SUCCESSIVE EXIT TERMINALS



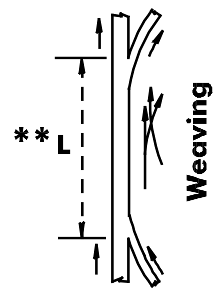
SUCCESSIVE ENTRANCE TERMINALS

L = 300m min. on Freeway or Interstate.
L = 240m min. on C–D Roads and other Arterials.



EXIT TERMINAL FOLLOWED BY ENTRANCE TERMINAL

L = 150m min. on Freeway or Interstate.
L = 120m min. on C–D Roads and other Arterials.



ENTRANCE TERMINAL FOLLOWED BY EXIT TERMINAL

L = 600m min. on Freeway or Interstate.
L = 480m min. on Rural Arterial or Collector.
L = 300m min. on Urban Arterial or Collector.

* L As noted but not less than length required for Accel. or Decel. Lanes.
** L Length shown is not applicable to distance between Loop Ramps or Cloverleaf interchanges.

All minimum lengths are measured from physical nose to physical nose. They should be checked in accordance with the procedure outlined in the Highway Capacity Manual and the larger of the values is suggested for use.

Adapted from: A policy on Geometric Design of Highways and Streets, A.A.S.H.T.O., 2001

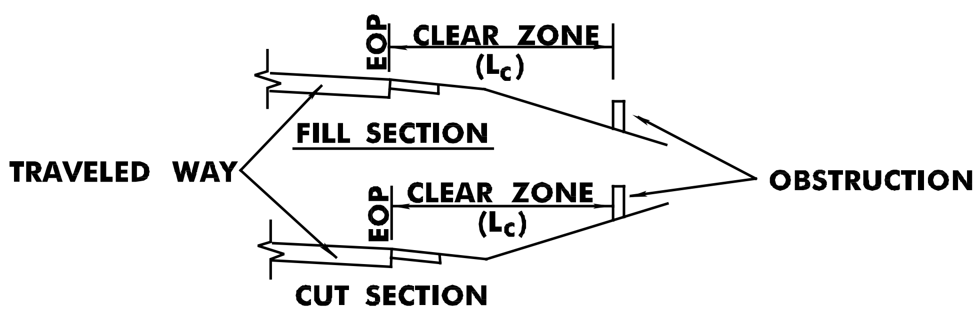
CLEAR ZONE (L_c)

FIGURE: 8–A

BDC02MR–4

THE FOLLOWING TABLE CONTAINS THE SUGGESTED RANGE OF CLEAR ZONE DISTANCES ON TANGENT SECTIONS OF ROADWAY BASED ON SELECTED TRAFFIC VOLUMES, SPEED AND ROADSIDE SLOPES:

DESIGN SPEED	DESIGN ADT	CLEAR ZONE DISTANCES (IN METERS FROM EDGE OF THROUGH LANE)				
		FILL SLOPES		CUT SLOPES		
		1:6 OR FLATTER	1:5 TO 1:4	1:3 OR STEEPER	1:4 TO 1:5	1:6 OR FLATTER
60 km/h OR LESS	UNDER 750	2.0–3.0	2.0–3.0	2.0–3.0	2.0–3.0	2.0–3.0
	750–1500	3.0–3.5	3.5–4.5	3.0–3.5	3.0–3.5	3.0–3.5
	1500–6000	3.5–4.5	4.5–5.0	3.5–4.5	3.5–4.5	3.5–4.5
	OVER 6000	4.5–5.0	5.0–5.5	4.5–5.0	4.5–5.0	4.5–5.0
70 – 80 km/h	UNDER 750	3.0–3.5	3.5–4.5	2.5–3.0	2.5–3.0	3.0–3.5
	750–1500	4.5–5.0	5.0–6.0	3.0–3.5	3.5–4.5	4.5–5.0
	1500–6000	5.0–5.5	6.0–8.0	3.5–4.5	4.5–5.0	5.0–5.5
	OVER 6000	6.0–6.5	7.5–8.5	4.5–5.0	5.5–6.0	6.0–6.5
90 km/h	UNDER 750	3.5–4.5	4.5–5.5	2.5–3.0	3.0–3.5	3.0–3.5
	750–1500	5.0–5.5	6.0–7.5	3.0–3.5	4.5–5.0	5.0–5.5
	1500–6000	6.0–6.5	7.5–9.0	4.5–5.0	5.0–5.5	6.0–6.5
	OVER 6000	6.5–7.5	8.0–10.0	5.0–5.5	6.0–6.5	6.5–7.5
100 km/h	UNDER 750	5.0–5.5	6.0–7.5	3.0–3.5	3.5–4.5	4.5–5.0
	750–1500	6.0–7.5	8.0–10.0	3.5–4.5	5.0–5.5	6.0–6.5
	1500–6000	8.0–9.0	10.0–12.0	4.5–5.5	5.5–6.5	7.5–8.0
	OVER 6000	9.0–10.0	11.0–13.5	6.0–6.5	7.5–8.0	8.0–8.5
110 km/h	UNDER 750	5.5–6.0	6.0–8.0	3.0–3.5	4.5–5.0	4.5–4.9
	750–1500	7.5–8.0	8.5–11.0	3.5–5.0	5.5–6.0	6.0–6.5
	1500–6000	8.5–10.0	10.5–13.0	5.0–6.0	6.5–7.5	8.0–8.5
	OVER 6000	9.0–10.5	11.5–14.0	6.5–7.5	8.0–9.0	8.5–9.0

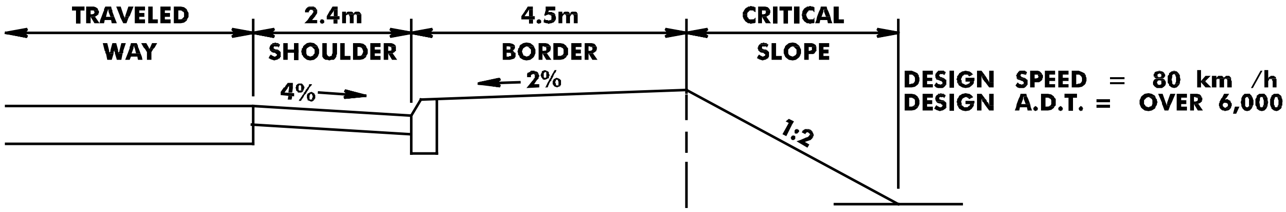


SOURCE: "CHAPTER 3: ROADSIDE TOPOGRAPHY AND DRAINAGE FEATURES." ROADSIDE DESIGN GUIDE. AASHTO, WASHINGTON D.C., 2002.

CLEAR ZONE EXAMPLES

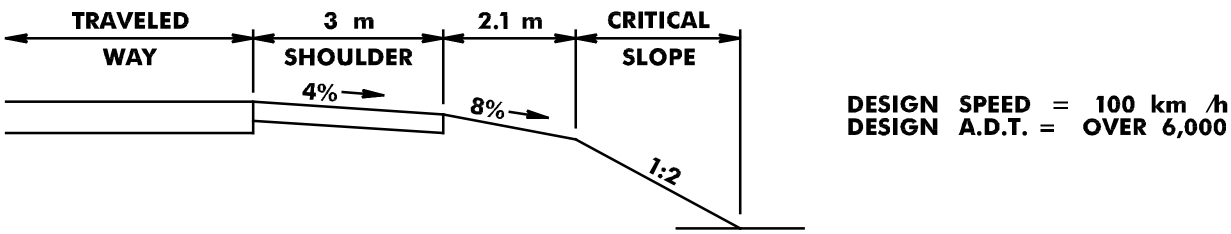
FIGURE: 8–B

BDC02MR–4



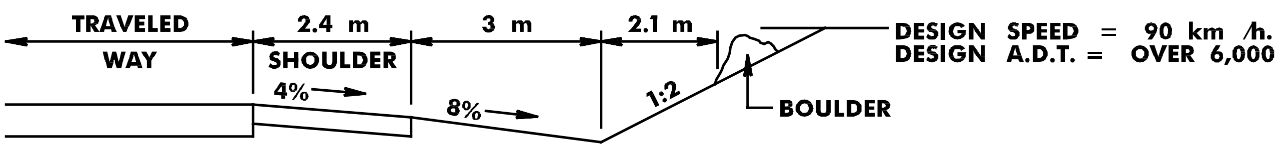
The suggested clear zone distance for the 2% slope (See Figure 8–A, Cut Slope, 1:6 or flatter) = 6 to 6.5 m.

The available 6.9 m is 0.4 to 0.9 m greater than the suggested recovery area, therefore, the critical slope (1:2) is outside the clear zone.

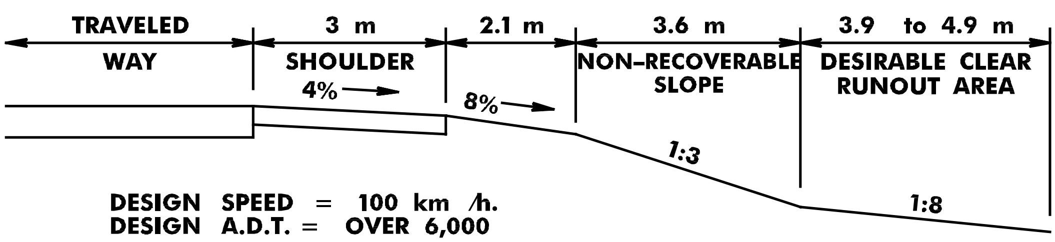


The suggested clear zone distance for the 8% slope (See Figure 8–A, Fill Slope, 1:6 or flatter) = 9 – 10 m.

The available 5.1 m is 3.9 to 4.9 m. less than the suggested recovery area, therefore, the critical slope (1:2) is inside the clear zone.



The suggested clear zone distance for the 8% slope (See Figure 8–A, Fill Slope, 1:6 or flatter) = 6.5 to 7.5 m. The available 5.4 m to the channel is 1.1 to 2.1 m less than the suggested recovery area for the fill slope. The channel is not within the preferred cross section area of Figure 8–R, but the boulder has 7.5 m available which is 0 to 1 m outside the clear zone for the fillslope. Since the channel bottom and backslope are free of obstructions within the clear zone, guide rail is not required.



The suggested clear zone distance for the 1:8 slope in the clear runout area (See Figure 8–A, Fill Slope 1:6 or flatter) = 9 – 10 m. The recovery distance before breakpoint of non–recoverable slope = 5.1 m. Therefore the desirable clear runout area is : 9 – 10 m minus 5.1m = 3.9 to 4.9 m.

HORIZONTAL CURVE ADJUSTMENTS FOR CLEAR ZONE

FIGURE: 8–C

BDC02MR–4

THE CLEAR ZONE WIDTHS OBTAINED FROM FIGURE 8–A SHOULD BE INCREASED ON THE OUTSIDE OF CURVES. THE AMOUNT OF INCREASE CAN BE DETERMINED BY THE FOLLOWING TABLE:

RADIUS (m)	Kcz (CURVE CORRECTION FACTOR)					
	DESIGN SPEED, km/h					
	60	70	80	90	100	110
900	1.1	1.1	1.1	1.2	1.2	1.2
700	1.1	1.1	1.2	1.2	1.2	1.3
600	1.1	1.2	1.2	1.2	1.3	1.4
500	1.1	1.2	1.2	1.3	1.3	1.4
450	1.2	1.2	1.3	1.3	1.4	1.5
400	1.2	1.2	1.3	1.3	1.4	
350	1.2	1.2	1.3	1.4	1.5	
300	1.2	1.3	1.4	1.5	1.5	
250	1.3	1.3	1.4	1.5		
200	1.3	1.4	1.5			
150	1.4	1.5				
100	1.5					

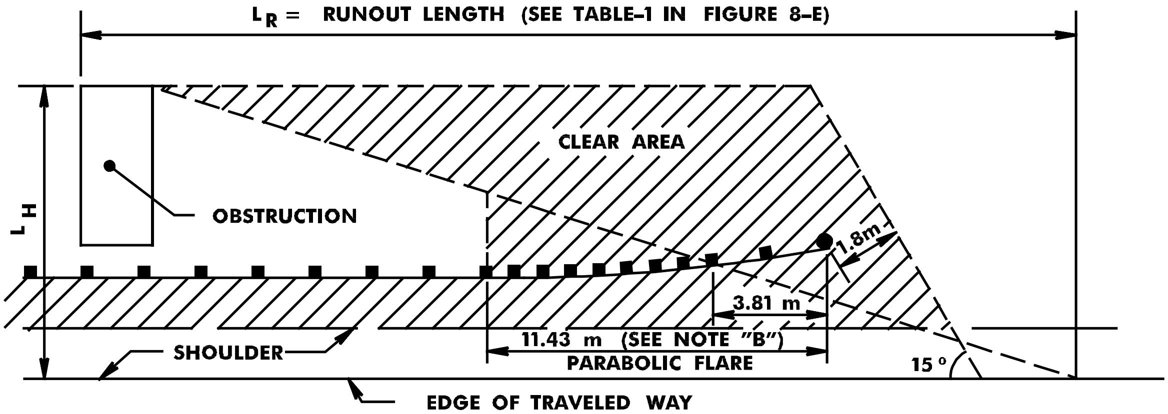
$$CZ_c = (L_c)(K_{cz})$$

CZ_c = CLEAR ZONE ON OUTSIDE OF HORIZONTAL CURVE, METER.
L_c = CLEAR ZONE DISTANCE FROM FIGURE 8–A, METER.
K_{cz} = CURVE CORRECTION FACTOR.

NOTE: CLEAR ZONE CORRECTION FACTOR IS APPLIED TO OUTSIDE OF HORIZONTAL CURVES ONLY. CURVES FLATTER THAN 900 m DO NOT REQUIRE AN ADJUSTED CLEAR ZONE. ALSO, ADJUSTMENTS ARE NOT NECESSARY FOR DESIGN SPEEDS LESS THAN 60 km/h.

SOURCE: "CHAPTER 3: ROADSIDE TOPOGRAPHY AND DRAINAGE FEATURES." ROADSIDE DESIGN GUIDE. AASHTO, WASHINGTON D.C., 2002.

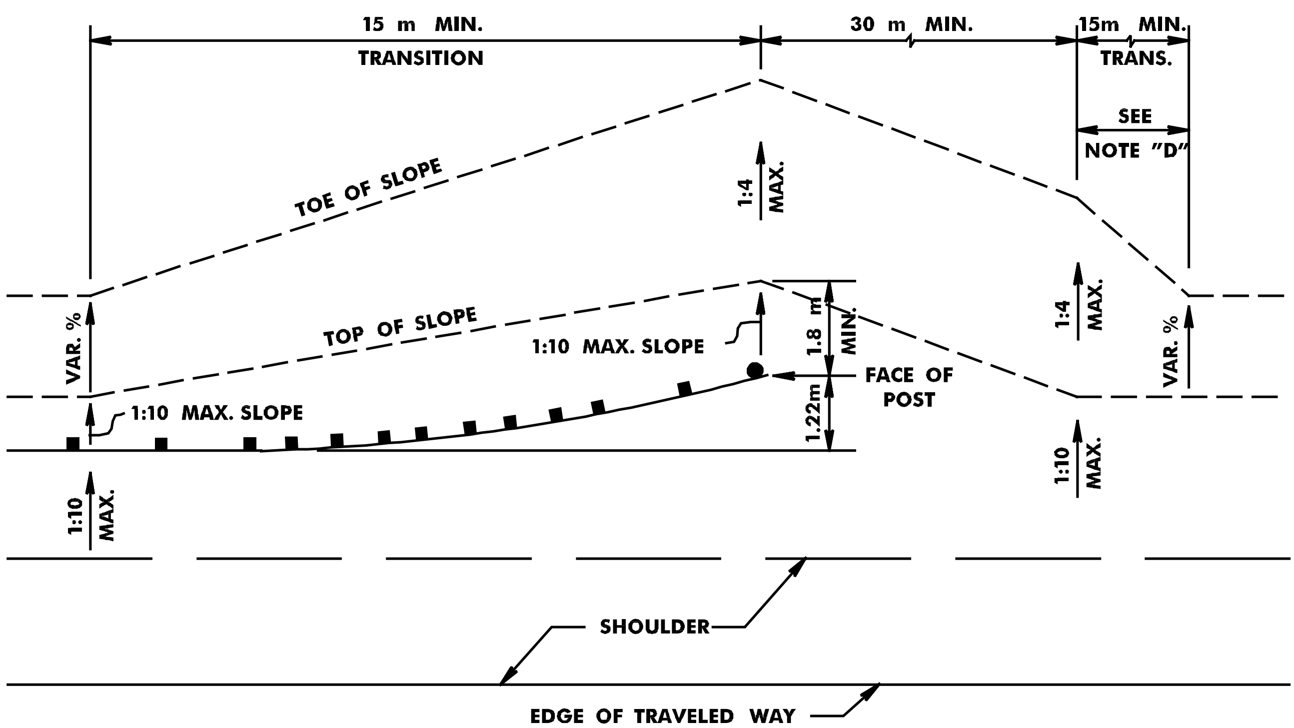
CLEAR AREA AT S.R.T. AND SLOPE TREATMENT AT S.R.T.	FIGURE: 8–D
	DATE:05/15/98



NOTE "A": NO FIXED OBJECTS SHOULD BE WITHIN THE CROSS–HATCHED AREA.

NOTE "B": RUB RAIL, REDUCED POST SPACING, AND DOUBLE RAIL ELEMENTS SHOULD NOT BE USED WITHIN THE 11.43 m PARABOLIC FLARE.

NOTE "C": IF L_H EXTENDS BEYOND R.O.W. LINE, THEN CLEAR AREA ONLY WITHIN R.O.W. LIMITS.

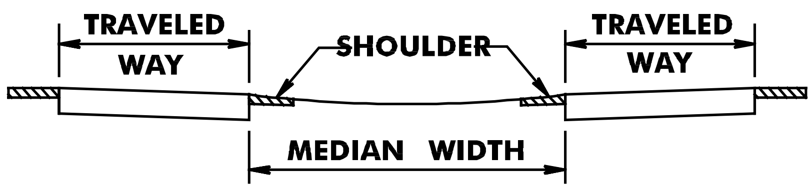


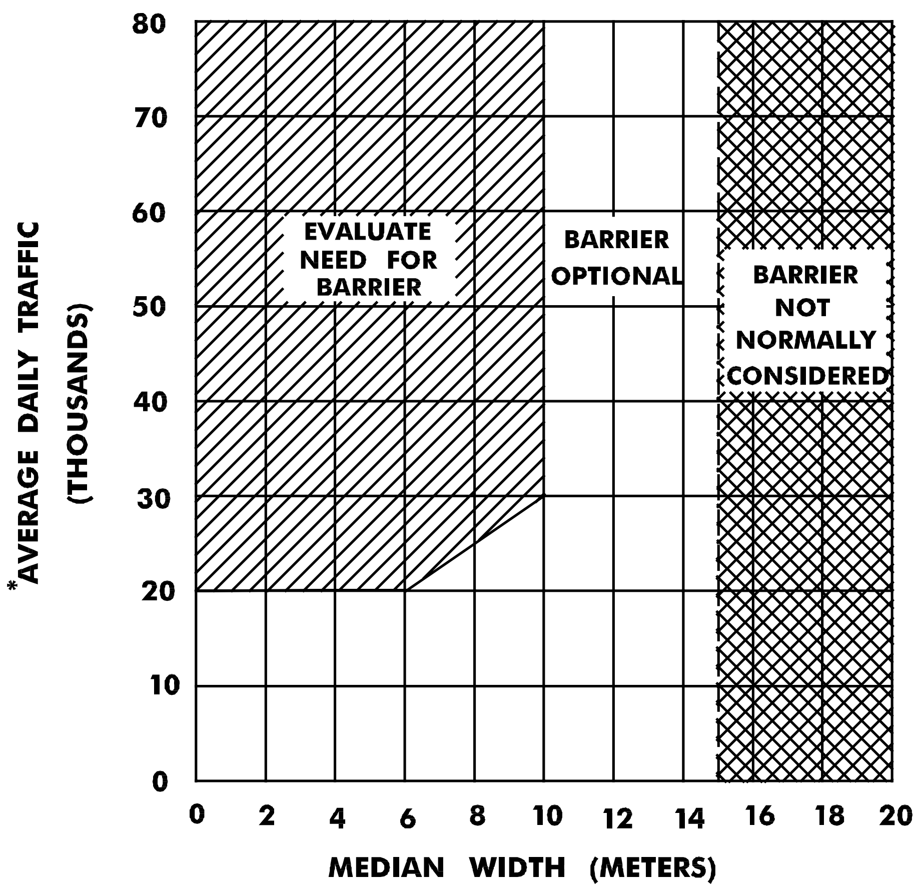
NOTE "D": THIS TRANSITION IS ONLY NEEDED IF THE APPROACHING SLOPE IS STEEPER THAN 1:4.

WARRANTS FOR MEDIAN BARRIER FOR FREEWAYS AND EXPRESSWAYS

FIGURE: 8–Q

BDC02MR–4

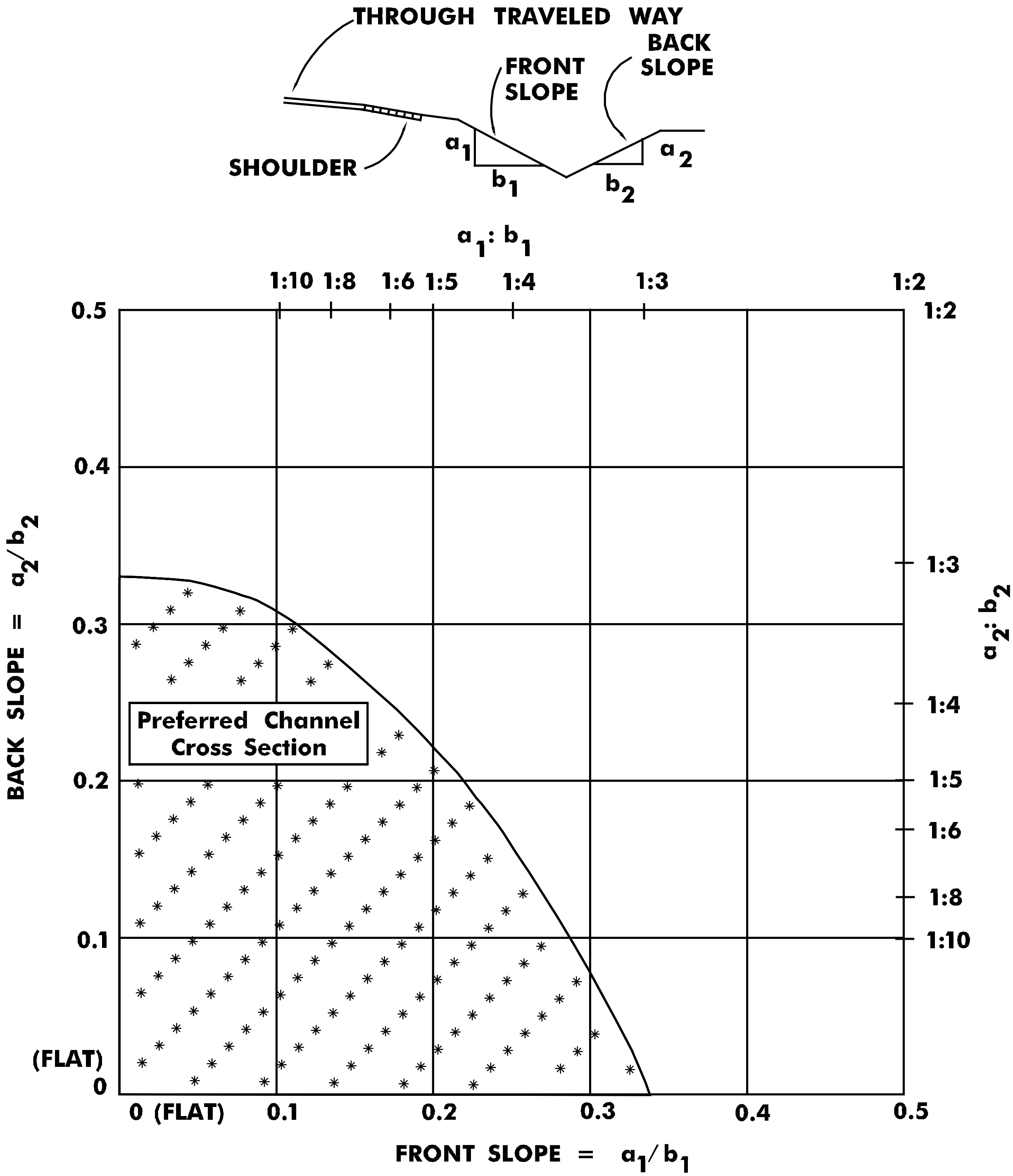




* BASED ON A 5–YEAR PROJECTION, TWO–WAY

SOURCE: ROADSIDE DESIGN GUIDE, AASHTO, 2002.

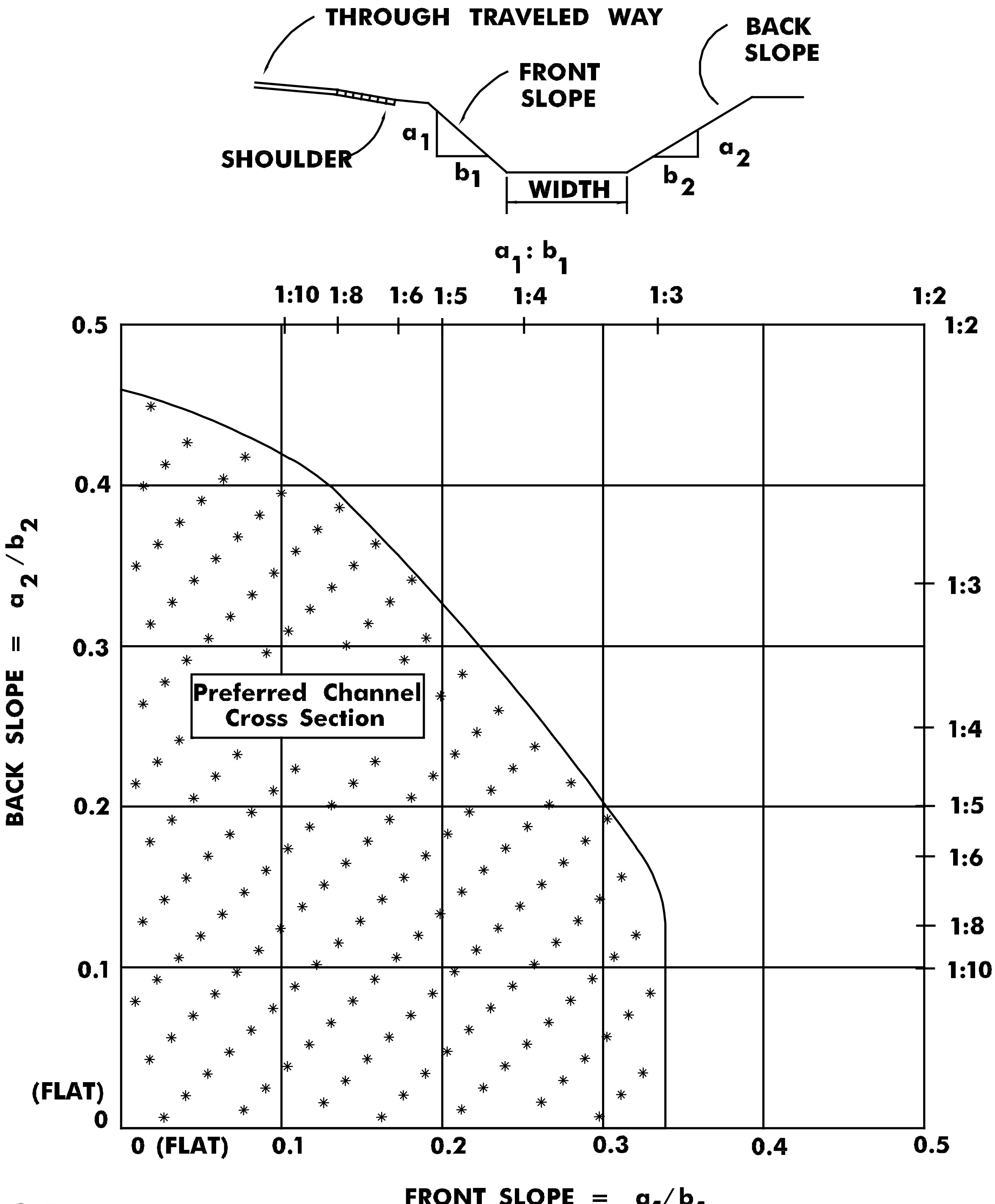
PREFERRED CROSS SECTIONS FOR CHANNELS WITH ABRUPT SLOPE CHANGES	FIGURE: 8–R BDC02MR–4



NOTE: THIS CHART IS APPLICABLE TO ALL VEE DITCHES, ROUNDED CHANNELS WITH A BOTTOM WIDTH LESS THAN 2.4 m, AND TRAPEZOIDAL CHANNELS WITH BOTTOM WIDTHS LESS THAN 1.2 m.

SOURCE: ROADSIDE DESIGN GUIDE, AASHTO, 2002.

PREFERRED CROSS SECTIONS FOR CHANNELS WITH GRADUAL SLOPE CHANGES	FIGURE: 8–S
	BDC02MR–4



NOTE:
THIS CHART IS APPLICABLE TO ROUNDED CHANNELS WITH BOTTOM WIDTHS OF 2.4 m OR MORE, AND TO TRAPEZOIDAL CHANNELS WITH BOTTOM WIDTHS EQUAL TO OR GREATER THAN 1.2 m.

SOURCE: ROADSIDE DESIGN GUIDE, AASHTO, 2002.

LENGTH OF NEED OF PRECAST CONCRETE CURB CONSTRUCTION BARRIER

FIGURE 14–A

BDC02MR–4

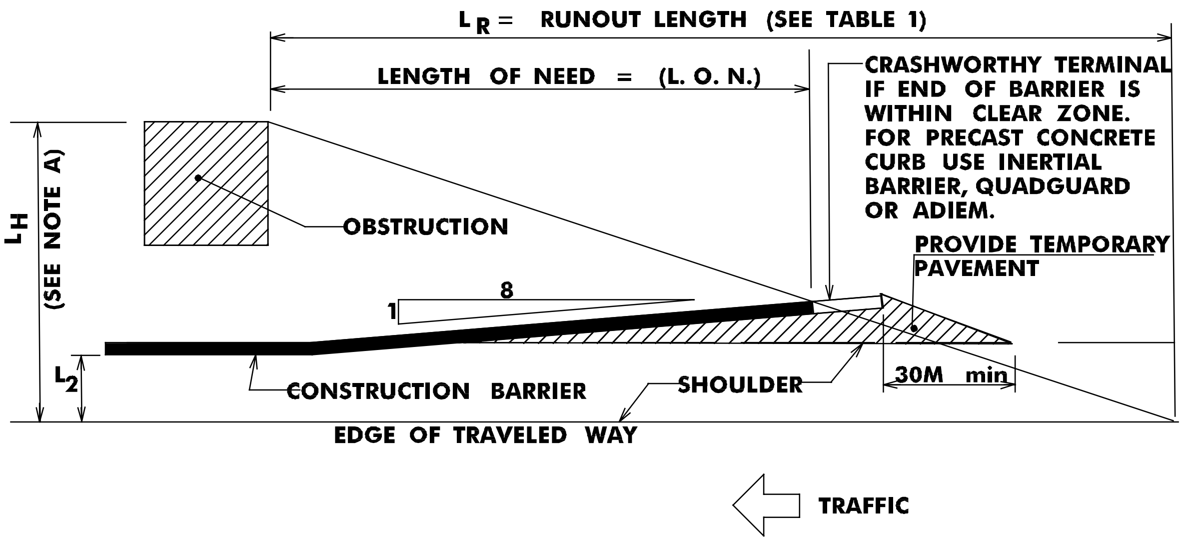


TABLE – 1

	TRAFFIC VOLUME (A.D.T.)			
	OVER 6000	2000–6000	800–2000	UNDER 800
DESIGN SPEED (km/hr)	RUNOUT LENGTH L_R (m)	RUNOUT LENGTH L_R (m)	RUNOUT LENGTH L_R (m)	RUNOUT LENGTH L_R (m)
110	145	135	120	110
100	130	120	105	100
90	110	105	95	85
80	100	90	80	75
70	80	75	65	60
60	70	60	55	50
50	50	50	45	40

NOTE A: IF OBSTRUCTION EXTENDS BEYOND CLEAR ZONE, MAKE L_H EQUAL TO CLEAR ZONE, EXCEPT IF OBSTRUCTION IS A CRITICAL SLOPE, SEE FIGURE 8–G.

NOTE B: IF ROADWAY IS CURVED, DRAW LAYOUT TO SCALE AND OBTAIN L.O.N. DIRECTLY BY SCALING FROM DRAWING.

NOTE C: IF BARRIER END IS PARALLEL TO ROADWAY (NO FLARE), THEN CHANGE "1/8" IN FORMULA TO "0".

EXAMPLE

$$L.O.N. = \frac{L_H - L_2}{\frac{1}{8} + \frac{L_H}{L_R}}$$

L_2 = 5 m

L_H = 7 m

L_R = 145 m

DESIGN SPEED = 110 km/hr

ADT = 26,000

$$L.O.N. = \frac{7 - 5}{\frac{1}{8} + \frac{7}{145}}$$

L.O.N. = 11.5 m

SOURCE: ROADSIDE DESIGN GUIDE, AASHTO., 2002