STATE OF NORTH CAROLINA
DEPARTMENT OF TRANSPORTATION

PAT MCCRORY
GOVERNOR

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SECRETARY

April 03, 2013

MEMORANDUM TO: 2002 ROADWAY DESIGN MANUAL HOLDERS

FROM: ROBERT MCKEITHAN
PUBLICATIONS ENGINEER

SUBJECT: 2002 ROADWAY DESIGN MANUAL
REVISION No. 7

The following are The Revisions and New Guidelines to Part I of the Roadway Design Manual. Please insert these Revisions in your Manual in the appropriate place. The *2002 Roadway Design Manual* has been updated and is available on the web at:

https://inside.ncdot.gov/stage/connect/projects/Roadway/Pages/Roadway-Design-Manual.aspx

If you have any questions and comments about this revision or the Roadway Design Manual, Please contact Robert McKeithan (rmckeithan@ncdot.gov) or Edward Morrison (emmorrison@ncdot.gov) of the Transportation Program Management Unit.

REVISION NO. 7

Part I - Roadway Design Manual

1. Chapter 1 - Section 2A, Figure 1 Interstates, Freeways, Expressways, Other Four Lane Facilities

NOTE: Criteria for Roadway Typical Section and Slopes

2. Chapter 3 - Section 1E, Index of Sheets

NOTE: Changes to Trans. Management Plans and Pavement Marking Plans

3. Chapter 7 - Section 1, Sight Distances At railroads For Unsignalized Crossing

NOTE: 2001 Greenbook reference Changed to 2004

4. Chapter 7 - Section 1H, Sight Distances At railroads For Unsignalized Crossing

NOTE: Charts and Engineering drawings are being put back in the manual

5. Chapter 7 - Section 6, Typical Median Separated Island Detail

NOTE: Clarification of Minimum flat surface requirement

6. Chapter 8 - Sections 1, 6 & 15, Interchanges

NOTE: 2001 Greenbook reference Changed to 2004

7. Chapter 8 - Section 1, Figure 1 Design Widths of Pavements for Turning Roadways

NOTE: 2001 Greenbook reference Changed to 2004

8. Chapter 8 - Section 3, Guide Values for Ramp Design Speed

NOTE: Minimum Radius Charts Added

9. Chapter 8 - Section 8, Figure 1 Recommended Minimum Ramp Terminal Spacing

NOTE: Recommended Minimum Ramp Terminal Spacing Figure Added

10. Chapter 8 - Section 11, Acceleration and Deceleration Lanes

NOTE: Minimum Lengths for Entrance Exit Terminals Charts Added

RM

Attachments

PROJECT COST REDUCTION GUIDELINES (continued) 1-1E

b) Recheck need for detour structure. Is it possible to close road? Can a precast box culvert be used to allow closing of a road in a minimum amount of time? Can a Portable Detour Structure be used?

c) Consider submitting project for a Value Engineering study if the construction cost exceeds $2,000,000 and the design has not progressed past the right-of-way stage.

HOW TO SELECT A TYPICAL SECTION 1-2

For assistance in selecting a typical section, a brief explanation is provided for the major considerations that are directly or indirectly affected by the design criteria. Study each of these carefully before you begin to select a typical section.

The typical section should be based on sound engineering principles with primary emphasis being placed on the type of facility, traffic volumes, terrain, availability of right of way, grading, guardrail construction and economics.

On projects of major importance and where a significant savings can be realized, several design combinations should be considered. After the most feasible of the design combinations are chosen, an analysis should be made to select a typical section that will provide a safe and economical highway. An analysis in the early stages of design may determine that it is necessary to revise the typical section to:

1) Reduce right of way takings.
2) Improve grading operations.
3) Utilize waste material to flatten slopes which will provide greater roadside clearances and may sometimes eliminate the need for guardrail.
4) Reduce wetland taking in environmentally sensitive areas.

CRITERIA FOR ROADWAY TYPICAL SECTION AND SLOPES 1-2A

STANDARD METHOD OF CONSTRUCTING CUT AND FILL SLOPES

(A) Interstates, Freeways, Expressways and other four-lane facilities

See 1-2A, F-1 (A).

(B) Collectors and Locals (4000 ADT or less Design Year Traffic)

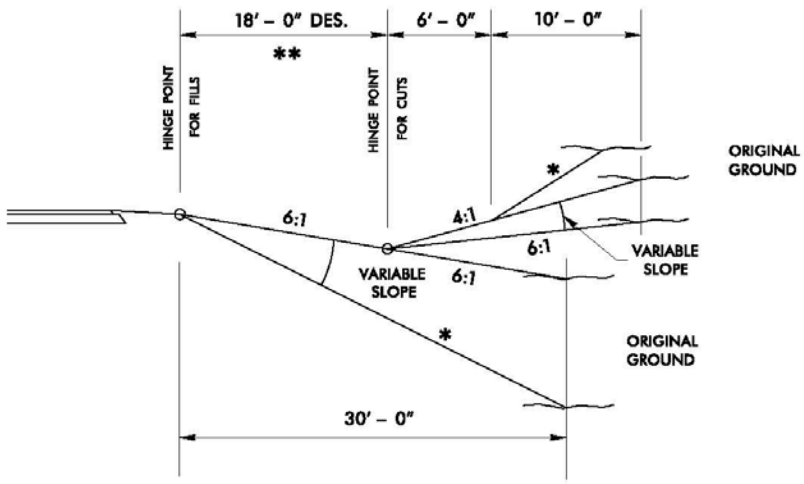
See 1-2A, F-1(B).

NOTE: These guidelines (A and B) apply to new construction, not 3-R Projects or subdivision roads.

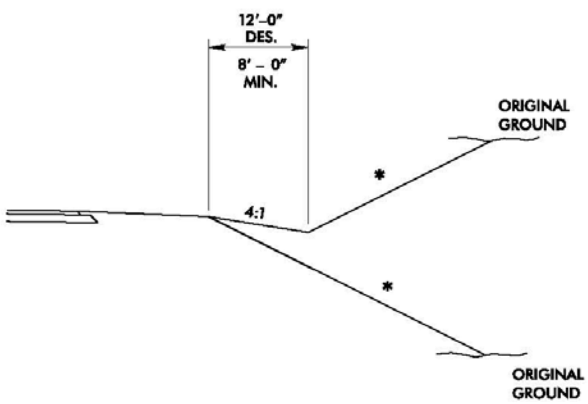
CRITERIA FOR ROADWAY TYPICAL SECTION AND SLOPES

1 – 2A
F – 1

(A) INTERSTATES, FREEWAYS, EXPRESSWAYS, OTHER FOUR LANE FACILITIES, ARTERIALS, COLLECTORS AND LOCALS (OVER 4000 ADT DESIGN YEAR TRAFFIC)



(B) COLLECTORS AND LOCALS (4000 ADT OR LESS DESIGN YEAR TRAFFIC)



* THE STEEPEST PRACTICAL SLOPES AS DETERMINED BY THE GEOTECHNICAL UNIT SHOULD BE UTILIZED
INTERSTATE SIDE SLOPES SHOULD NOT BE STEEPER THAN 2:1 EXCEPT IN ROCK EXCAVATION
FREEWAYS AND EXPRESSWAYS SHOULD NOT BE STEEPER THAN 1 1/2:1 TO 2:1.

** 12' – 0" MIN. ARTERIALS, COLLECTORS, LOCAL OVER 4000 ADT
15' – 0" MIN. INTERSTATE, FREEWAY, EXPRESSWAY, FOUR LANE

A GUARDRAIL STUDY WILL BE REQUIRED FOR FILL SLOPES STEEPER THAN 3:1
SEE HIGHWAY DESIGN MANUAL, PART I, CHAPTER 3

FOR SHOULDER WIDTHS, SEE HIGHWAY DESIGN MANUAL, PART I, CHAPTER 1–4B, F–1.

TWO FOOT MINIMUM DITCH DEPTH REQUIRED TO COVER DRIVEWAY PIPE

CHAPTER SEVEN

RAILROADS

SIGHT DISTANCES AT RAILROADS FOR UNSIGNALIZED CROSSING 7-1

The sight distance at railroad crossings is of utmost importance. When the exposure index does not merit grade separations or railroad signals, Section 7-1F, Figure 1, to provide safe stopping sight distances. If physical barriers exist and it is not economically feasible to provide the required distances, it shall be discussed with the Assistant State Roadway Design Engineer.

NOTE: Section 7-1F, Figure 1 is based on conditions of a 65′ truck crossing a single set of tracks at 90˚. This allows for a margin of safety for conditions using other design vehicles. If it is determined by the designer that a small number of trucks will be using the facility, consideration may be given to reducing the sight distance.

For Additional Information See:

A. Roadway Standard Drawings, Std. No. 1205.11 - Pavement Markings for Railroad Crossings

B. Chapter 18 of The Policy and Procedure Manual.

C. Railroad-Highway Grade Crossing Handbook "Report No. FHWA - TS - 86 - 215, September, 1986"

D. A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS (2004), Ch 9.

E. For Federal Highway-Rail Grade Crossing Relevant web links, go to: "http://safety.fhwa.dot.gov/fourthlevel/prof_res_hiwaygreadexing_links.htm".

$$d_H = AV_Vt + \frac{BV_V^2}{a} + D + d_e$$

$$d_T = \frac{V_T}{V_V}\left[(A)\,V_Vt + \frac{BV_V^2}{30f} + 2D + L + W\right]$$

d_H = sight-distance leg along the highway allows a vehicle proceeding to speed V_V to cross tracks even though a train is observed at a distance d_T from the crossing or to stop the vehicle without encroachment of the crossing area (ft)

d_T = sight-distance leg along the railroad tracks to permit the maneuvers described as for d_H(ft)

V_V = speed of the vehicle (mph)

V_T = speed of the train (mph)

t = perception/reaction time, which is assumed to be 2.5 s (This is the same value used in Chapter 3 to determine stopping sight distance.)

a = driver deceleration, which is assumed to be 11.2 ft/s^2. (This is the same value used in Chapter 3 to determine stopping sight distance.)

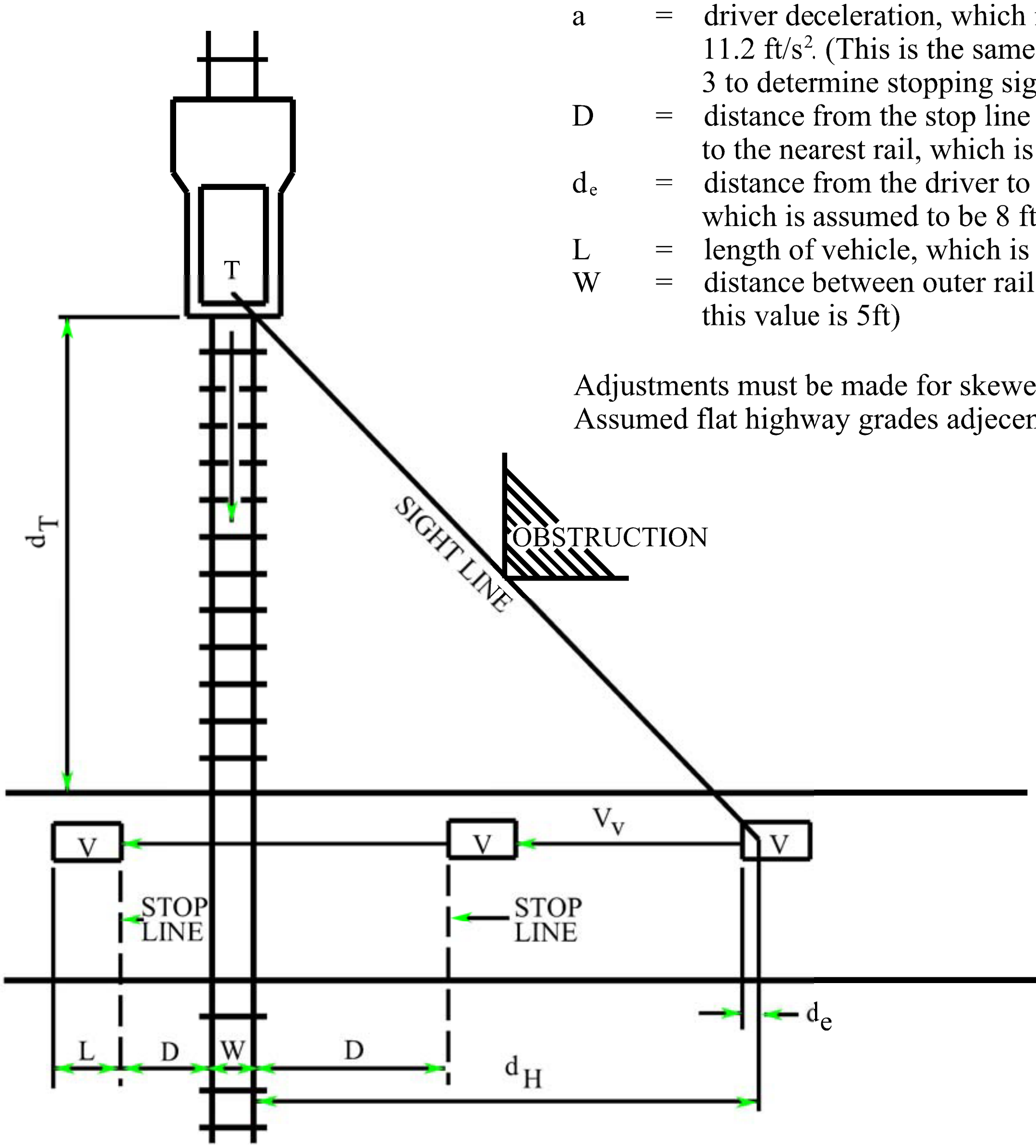
D = distance from the stop line or front of the vehicle to the nearest rail, which is assumed to be 15 ft

d_e = distance from the driver to the front of the vehicle, which is assumed to be 8 ft

L = length of vehicle, which is assumed to be 65 ft

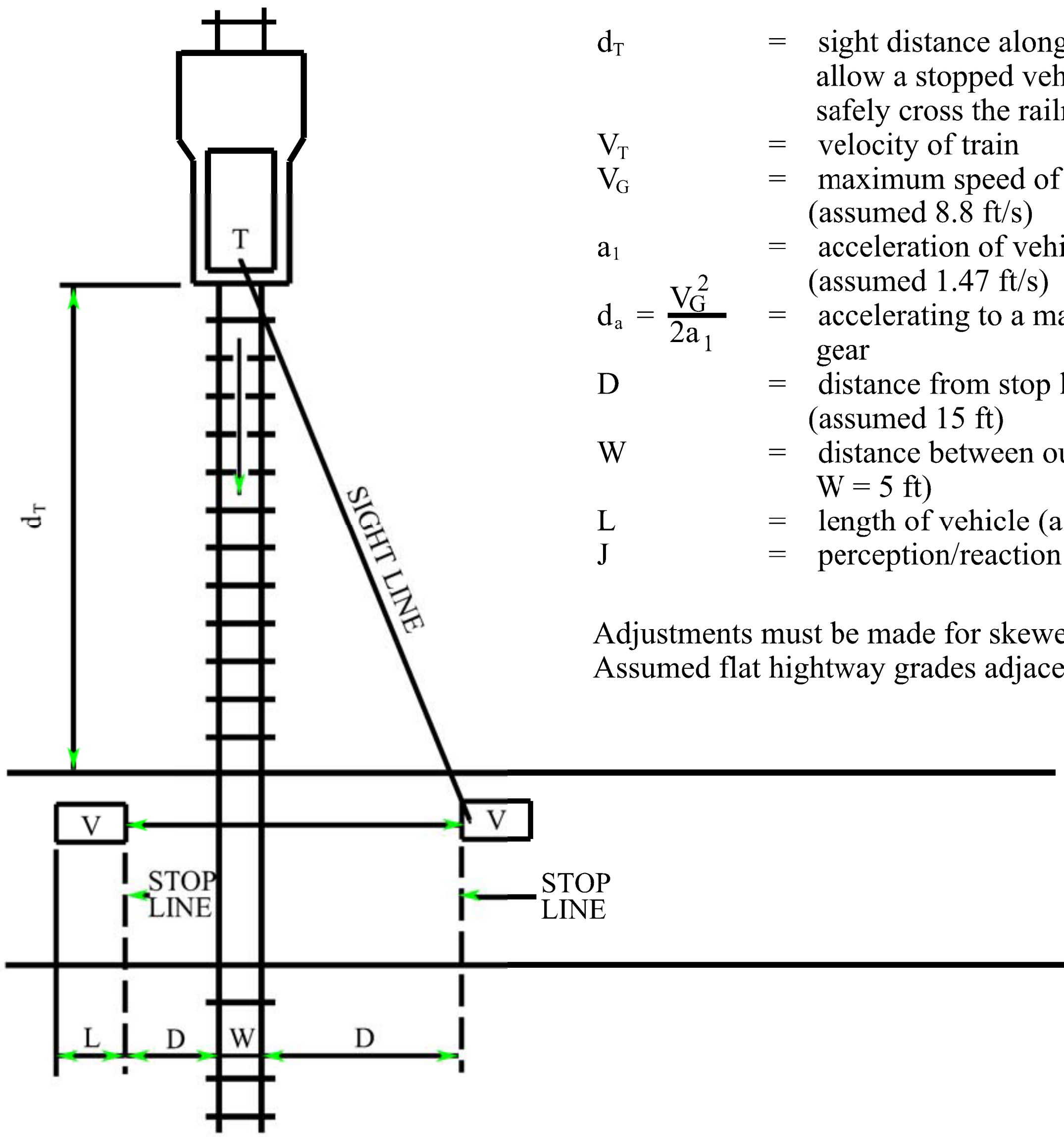
W = distance between outer rails (for a single track, this value is 5ft)

Adjustments must be made for skewed crossings.
Assumed flat highway grades adjecent to and at crossings.



CASE A: MOVING VEHICLE TO SAFELY CROSS OR STOP AT RAILROAD CROSSING

$$d = AV_T \left[\frac{V_G}{a_1} + \frac{L = 2D + W - d_a}{V_G} + J \right]$$



d_T	=	sight distance along railroad tracks to allow a stopped vehicle to depart and safely cross the railroad tracks
V_T	=	velocity of train
V_G	=	maximum speed of vehicle in first gear (assumed 8.8 ft/s)
a_1	=	acceleration of vehicle in first gear (assumed 1.47 ft/s)
$d_a = \frac{V_G^2}{2a_1}$	=	accelerating to a mazimum speed in first gear
D	=	distance from stop line to near rail (assumed 15 ft)
W	=	distance between outer rails (single track W = 5 ft)
L	=	length of vehicle (assumed 65 ft)
J	=	perception/reaction time (assumed 2.0 s)

Adjustments must be made for skewed crossings.
Assumed flat hightway grades adjacent to and at crossings.

CASE B: DEPARTURE OF VEHICLE FROM STOPPED POSITION TO CROSS SINGLE RAILROAD TRACK

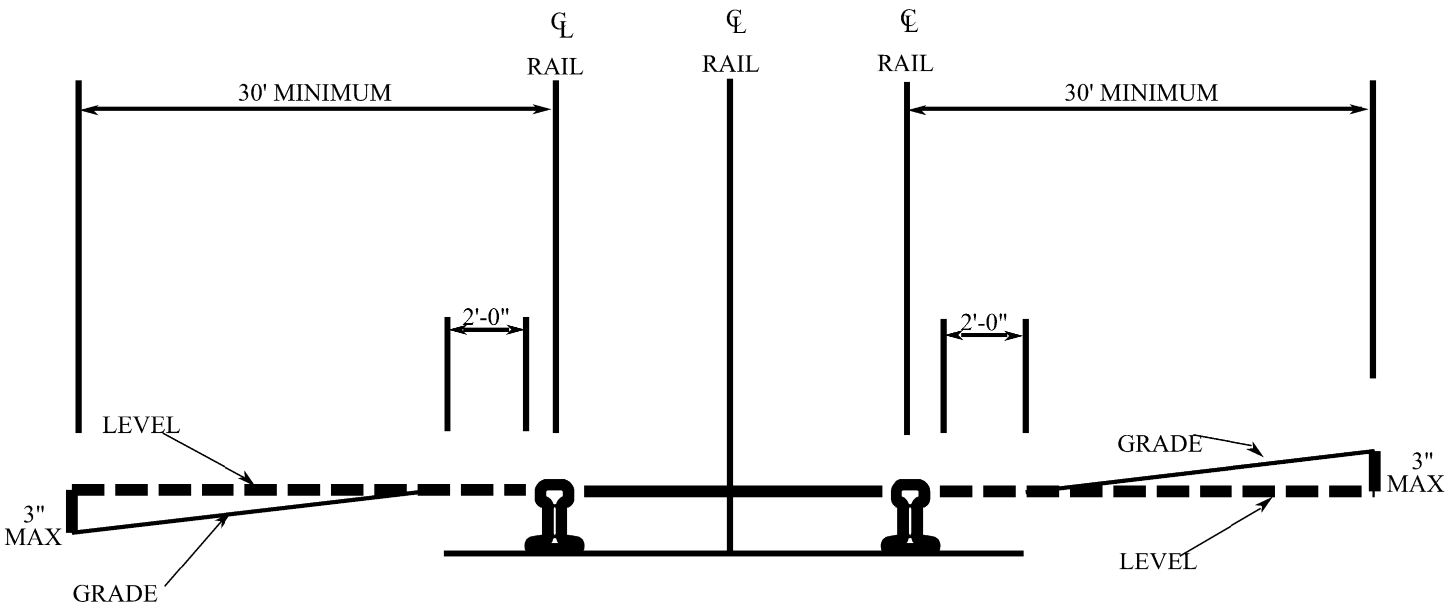
SIGHT DISTANCES AT RAILROADS
FOR UNSIGNALIZED CROSSING (Continued) 7-1H

US Customary									
Train Speed	Case B Departure from stop	Case A Moving Vehicle							
		Vehicle Speed (mph)							
(mph)	0	10	20	30	40	50	60	70	80
		Distance along railroad from crossing, d_T (ft)							
10	240	146	106	99	100	105	111	118	126
20	480	293	212	198	200	209	222	236	252
30	721	439	318	297	300	314	333	355	378
40	961	585	424	396	401	419	444	473	504
50	1201	732	530	494	501	524	555	591	630
60	1441	878	636	593	601	628	666	709	756
70	1681	1024	742	692	701	733	777	828	882
80	1921	1171	848	791	801	838	888	946	1008
90	2162	1317	954	890	901	943	999	1064	1134
		Distance along highway from crossing, d_H (ft)							
		69	135	220	324	447	589	751	931

Required design sight distance for combination of highway and train vehicle speeds; 65-ft truck crossing a single set of tracks at 90˚.

NOTE:
See Section 7-1F, Figure 1 for “Case A” – MOVING VEHICLE TO SAFETY CROSS OR STOP AT RAILROAD CROSSING and Section 7-1G, Figure 1 for “Case B” – DEPARTURE OF VEHICLE FROM STOPPED POSITION TO CROSS SINGLE RAILROAD TRACK.

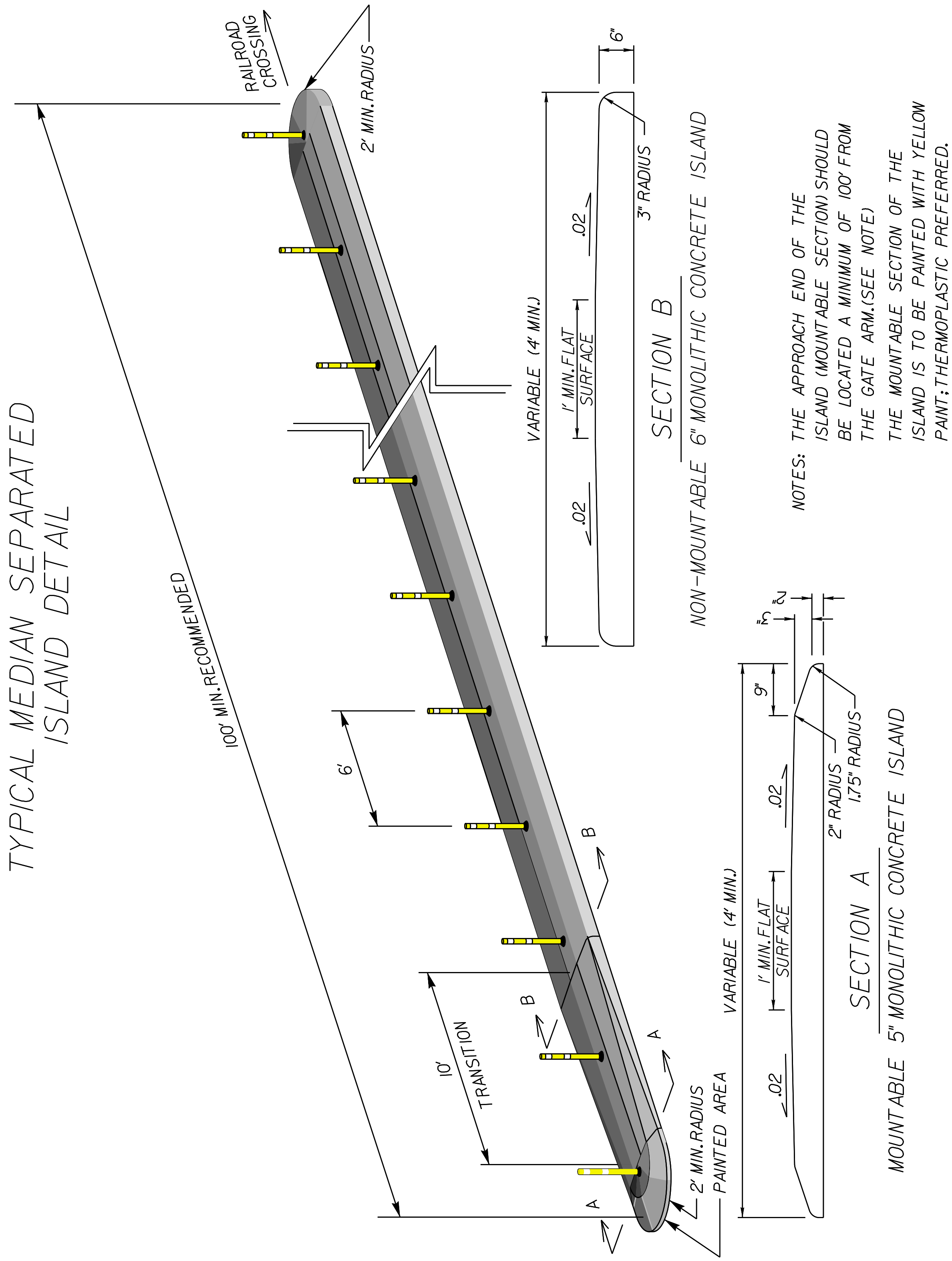
FOR ADDITIONAL INFORMATION, SEE A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS (2004), Ch 9.



RAILROAD - HIGHWAY GRADE CROSSING

NOTE: To prevent drivers of low-clearence vehicles from becoming caught on the tracks, the crossing surface should be at the same plane as the top of the rails for a distance of 2' outside the rails. The surface of the highway should not be more than 3" higher or lower than the top of nearest rail at a point 30' from the rail unless track superelevation makes a different level appropriate, as shown in the figure above.

FOR ADDITIONAL INFORMATION, SEE <u>A POLICY ON GEOMETRIC OF HIGHWAY AND STREETS (2004), Ch 9.</u>


TYPICAL MEDIAN SEPARATED
ISLAND DETAIL
100' MIN. RECOMMENDED
RAILROAD
CROSSING
2' MIN. RADIUS
6'
10'
TRANSITION
B
B
A
A
2' MIN. RADIUS
PAINTED AREA
6"
VARIABLE (4' MIN.)
1' MIN. FLAT
SURFACE
.02
.02
3" RADIUS
SECTION B
NON-MOUNTABLE 6" MONOLITHIC CONCRETE ISLAND
3"
2"
9"
VARIABLE (4' MIN.)
1' MIN. FLAT
SURFACE
.02
.02
2" RADIUS
1.75" RADIUS
SECTION A
MOUNTABLE 5" MONOLITHIC CONCRETE ISLAND
NOTES: THE APPROACH END OF THE
ISLAND (MOUNTABLE SECTION) SHOULD
BE LOCATED A MINIMUM OF 100' FROM
THE GATE ARM.(SEE NOTE)
THE MOUNTABLE SECTION OF THE
ISLAND IS TO BE PAINTED WITH YELLOW
PAINT; THERMOPLASTIC PREFERRED.

DESIGN WIDTHS OF PAVEMENTS FOR TURNING ROADWAYS 8 - 1
F - 1

Design Widths of Pavements for Turning Roadways

US Customary

Radius on inner edge of pavement	Pavement Width (ft)								
	Case I One-lane, one-way operation--no provision for passing a stalled vehicle			Case II One-lane, one-way operation--with provision for passing a stalled vehicle			Case III Two-lane operation--either one way or two way		
	Design traffic conditions								
R (ft)	A	B	C	A	B	C	A	B	C
50	18	18	23	20	26	30	31	36	45
75	16	17	20	19	23	27	29	33	38
100	15	16	18	18	22	25	28	31	35
150	14	15	17	18	21	23	26	29	32
200	13	15	16	17	20	22	26	28	30
300	13	15	15	17	20	22	25	28	29
400	13	15	15	17	19	21	25	27	28
500	12	15	15	17	19	21	25	27	28
Tangent	12	14	14	17	18	20	24	26	26

Width modification regarding edge treatment			
No stabilized shoulder	None	None	None
Sloping curb	None	None	None
Vertical curb: one side	Add 1 ft	None	Add 1 ft
Vertical curb: two sides	Add 2 ft	Add 1 ft	Add 2 ft
Stabilized shoulder, one or both sides	Lane width for conditions B & C on tangent may be reduced to 12 ft where shoulder is 4 ft or wider	Deduct shoulder width; minimum pavement width as under Case I	Deduct 2 ft where shoulder is 4 ft or wider

Note:
- A = predominantly P vehicles, but some condiseration for SU trucks.
- B = sufficent SU vehicles to govern design, but some consideration for semitrailer combination trucks
- C = sufficent bus and combination-trucks to govern design

RAMP DESIGN (continued) 8-2

Ramp design speeds should approximate the low volume running speed on the intersecting highways. This design speed is not always practicable and lower design speeds may be necessary.

GUIDE VALUES FOR RAMP DESIGN SPEED 8-3

GUIDE VALUES FOR
RAMP DESIGN SPEED AS RELATED TO HIGHWAY DESIGN SPEED

Highway design speed (mph)	30	35	40	45	50	55	60	65	70	75
Ramp design speed (mph)										
Upper range (85%)	25	30	35	40	45	48	50	55	60	65
Middle range (70%)	20	25	30	33	35	40	45	45	50	55
Lower range (50%)	15	18	20	23	25	28	30	30	35	40
Corresponding minimum radius (feet)	See charts 8-3 C-1 Thru C-5									

NOTE: Ramp design speeds above 30 mph seldom are applicable to loops. For highway design speeds of more than 50 mph, the loop design speed should not be less than 25 mph (150′ radius). For additional information, see A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS (2004), ch. 10.

Desirable curvatures for normal 50 mph design speeds in the vicinity of the gore areas are as follows:

Rural Exit	3 to 5 degrees
Rural Entrance	3 to 5 degrees
Urban Exit	4 to 6 degrees
Urban Entrance	3 to 6 degrees

CHART 1 **US CUSTOMARY** C-1

e (%)	V_d = 15 mph R (ft)	V_d = 20 mph R (ft)	V_d = 25 mph R (ft)	V_d = 30 mph R (ft)	V_d = 35 mph R (ft)	V_d = 40 mph R (ft)	V_d = 45 mph R (ft)	V_d = 50 mph R (ft)	V_d = 55 mph R (ft)	V_d = 60 mph R (ft)
1.5	796	1410	2050	2830	3730	4770	5930	7220	8650	10300
2.0	506	902	1340	1880	2490	3220	4040	4940	5950	7080
2.2	399	723	1110	1580	2120	2760	3480	4280	5180	6190
2.4	271	513	838	1270	1760	2340	2980	3690	4500	5410
2.6	201	388	650	1000	1420	1930	2490	3130	3870	4700
2.8	157	308	524	817	1170	1620	2100	2660	3310	4060
3.0	127	251	433	681	982	1370	1800	2290	2860	3530
3.2	105	209	363	576	835	1180	1550	1980	2490	3090
3.4	88	175	307	490	714	1010	1340	1720	2170	2700
3.6	73	147	259	416	610	865	1150	1480	1880	2350
3.8	61	122	215	348	512	730	970	1260	1600	2010
4.0	42	86	154	250	371	533	711	926	1190	1500

Note: Use of e_{max} = 4% should be limited to urban conditions.

Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 4%

CHART 2 **US CUSTOMARY** C-2

e (%)	V_d = 15 mph R (ft)	V_d = 20 mph R (ft)	V_d = 25 mph R (ft)	V_d = 30 mph R (ft)	V_d = 35 mph R (ft)	V_d = 40 mph R (ft)	V_d = 45 mph R (ft)	V_d = 50 mph R (ft)	V_d = 55 mph R (ft)	V_d = 60 mph R (ft)	V_d = 65 mph R (ft)	V_d = 70 mph R (ft)	V_d = 75 mph R (ft)	V_d = 80 mph R (ft)
1.5	868	1580	2290	3130	4100	5230	6480	7870	9410	11100	12600	14100	15700	17400
2.0	614	1120	1630	2240	2950	3770	4680	5700	6820	8060	9130	10300	11500	12900
2.2	543	991	1450	2000	2630	3370	4190	5100	6110	7230	8200	9240	10400	11600
2.4	482	884	1300	1790	2360	3030	3770	4600	5520	6540	7430	8380	9420	10600
2.6	430	791	1170	1610	2130	2740	3420	4170	5020	5950	6770	7660	8620	9670
2.8	384	709	1050	1460	1930	2490	3110	3800	4580	5440	6200	7030	7930	8910
3.0	341	635	944	1320	1760	2270	2840	3480	4200	4990	5710	6490	7330	8260
3.2	300	566	850	1200	1600	2080	2600	3200	3860	4600	5280	6010	6810	7680
3.4	256	498	761	1080	1460	1900	2390	2940	3560	4250	4890	5580	6340	7180
3.6	209	422	673	972	1320	1740	2190	2710	3290	3940	4540	5210	5930	6720
3.8	176	358	583	864	1190	1590	2010	2490	3040	3650	4230	4860	5560	6320
4.0	151	309	511	766	1070	1440	1840	2300	2810	3390	3950	4550	5220	5950
4.2	131	270	452	684	960	1310	1680	2110	2590	3140	3680	4270	4910	5620
4.4	116	238	402	615	868	1190	1540	1940	2400	2920	3440	4010	4630	5320
4.6	102	212	360	555	788	1090	1410	1780	2210	2710	3220	3770	4380	5040
4.8	91	189	324	502	718	995	1300	1640	2050	2510	3000	3550	4140	4790
5.0	82	169	292	456	654	911	1190	1510	1890	2330	2800	3330	3910	4550
5.2	73	152	264	413	595	833	1090	1390	1750	2160	2610	3120	3690	4320
5.4	65	136	237	373	540	759	995	1280	1610	1990	2420	2910	3460	4090
5.6	58	121	212	335	487	687	903	1160	1470	1830	2230	2700	3230	3840
5.8	51	106	186	296	431	611	806	1040	1320	1650	2020	2460	2970	3560
6.0	39	81	144	231	340	485	643	833	1060	1330	1660	2040	2500	3050

Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 6%

CHART 3 **US CUSTOMARY** C-3

e (%)	V_d = 15 mph R (ft)	V_d = 20 mph R (ft)	V_d = 25 mph R (ft)	V_d = 30 mph R (ft)	V_d = 35 mph R (ft)	V_d = 40 mph R (ft)	V_d = 45 mph R (ft)	V_d = 50 mph R (ft)	V_d = 55 mph R (ft)	V_d = 60 mph R (ft)	V_d = 65 mph R (ft)	V_d = 70 mph R (ft)	V_d = 75 mph R (ft)	V_d = 80 mph R (ft)
1.5	932	1640	2370	3240	4260	5410	6710	8150	9720	11500	12900	14500	16100	17800
2.0	676	1190	1720	2370	3120	3970	4930	5990	7150	8440	9510	10700	12000	13300
2.2	605	1070	1550	2130	2800	3570	4440	5400	6450	7620	8600	9660	10800	12000
2.4	546	959	1400	1930	2540	3240	4030	4910	5870	6930	7830	8810	9850	11000
2.6	496	872	1280	1760	2320	2960	3690	4490	5370	6350	7180	8090	9050	10100
2.8	453	796	1170	1610	2130	2720	3390	4130	4950	5850	6630	7470	8370	9340
3.0	415	730	1070	1480	1960	2510	3130	3820	4580	5420	6140	6930	7780	8700
3.2	382	672	985	1370	1820	2330	2900	3550	4250	5040	5720	6460	7260	8130
3.4	352	620	911	1270	1690	2170	2700	3300	3970	4700	5350	6050	6800	7620
3.6	324	572	845	1180	1570	2020	2520	3090	3710	4400	5010	5680	6400	7180
3.8	300	530	784	1100	1470	1890	2360	2890	3480	4140	4710	5350	6030	6780
4.0	277	490	729	1030	1370	1770	2220	2720	3270	3890	4450	5050	5710	6420
4.2	255	453	678	955	1280	1660	2080	2560	3080	3670	4200	4780	5410	6090
4.4	235	418	630	893	1200	1560	1960	2410	2910	3470	3980	4540	5140	5800
4.6	215	384	585	834	1130	1470	1850	2280	2750	3290	3770	4310	4890	5530
4.8	193	349	542	779	1060	1390	1750	2160	2610	3120	3590	4100	4670	5280
5.0	172	314	499	727	991	1310	1650	2040	2470	2960	3410	3910	4460	5050
5.2	154	284	457	676	929	1230	1560	1930	2350	2820	3250	3740	4260	4840
5.4	139	258	420	627	870	1160	1480	1830	2230	2680	3110	3570	4090	4640
5.6	126	236	387	582	813	1090	1390	1740	2120	2550	2970	3420	3920	4460
5.8	115	216	358	542	761	1030	1320	1650	2010	2430	2840	3280	3760	4290
6.0	105	199	332	506	713	965	1250	1560	1920	2320	2710	3150	3620	4140
6.2	97	184	308	472	669	909	1180	1480	1820	2210	2600	3020	3480	3990
6.4	89	170	287	442	628	857	1110	1400	1730	2110	2490	2910	3360	3850
6.6	82	157	267	413	590	808	1050	1330	1650	2010	2380	2790	3240	3720
6.8	76	146	248	386	553	761	990	1260	1560	1910	2280	2690	3120	3600
7.0	70	135	231	360	518	716	933	1190	1480	1820	2180	2580	3010	3480
7.2	64	125	214	336	485	672	878	1120	1400	1720	2070	2470	2900	3370
7.4	59	115	198	312	451	628	822	1060	1320	1630	1970	2350	2780	3250
7.6	54	105	182	287	417	583	765	980	1230	1530	1850	2230	2650	3120
7.8	48	94	164	261	380	533	701	901	1140	1410	1720	2090	2500	2970
8.0	38	76	134	214	314	444	587	758	960	1200	1480	1810	2210	2670

Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 8% (Continued)

CHART 4 **US CUSTOMARY** C-4

e (%)	V_d = 15 mph R (ft)	V_d = 20 mph R (ft)	V_d = 25 mph R (ft)	V_d = 30 mph R (ft)	V_d = 35 mph R (ft)	V_d = 40 mph R (ft)	V_d = 45 mph R (ft)	V_d = 50 mph R (ft)	V_d = 55 mph R (ft)	V_d = 60 mph R (ft)	V_d = 65 mph R (ft)	V_d = 70 mph R (ft)	V_d = 75 mph R (ft)	V_d = 80 mph R (ft)
1.5	947	1680	2420	3320	4350	5520	6830	8280	9890	11700	13100	14700	16300	18000
2.0	694	1230	1780	2440	3210	4080	5050	6130	7330	8630	9720	10900	12200	13500
2.2	625	1110	1600	2200	2900	3680	4570	5540	6630	7810	8800	9860	11000	12200
2.4	567	1010	1460	2000	2640	3350	4160	5050	6050	7130	8040	9010	10100	11200
2.6	517	916	1330	1840	2420	3080	3820	4640	5550	6550	7390	8290	9260	10300
2.8	475	841	1230	1690	2230	2840	3520	4280	5130	6050	6840	7680	8580	9550
3.0	438	777	1140	1570	2060	2630	3270	3970	4760	5620	6360	7140	7990	8900
3.2	406	720	1050	1450	1920	2450	3040	3700	4440	5250	5930	6680	7480	8330
3.4	377	670	978	1360	1790	2290	2850	3470	4160	4910	5560	6260	7020	7830
3.6	352	625	913	1270	1680	2150	2670	3250	3900	4620	5230	5900	6620	7390
3.8	329	584	856	1190	1580	2020	2510	3060	3680	4350	4940	5570	6260	6990
4.0	308	547	804	1120	1490	1900	2370	2890	3470	4110	4670	5270	5930	6630
4.2	289	514	756	1060	1400	1800	2240	2740	3290	3900	4430	5010	5630	6300
4.4	271	483	713	994	1330	1700	2120	2590	3120	3700	4210	4760	5370	6010
4.6	255	455	673	940	1260	1610	2020	2460	2970	3520	4010	4540	5120	5740
4.8	240	429	636	890	1190	1530	1920	2340	2830	3360	3830	4340	4900	5490
5.0	226	404	601	844	1130	1460	1830	2240	2700	3200	3660	4150	4690	5270
5.2	213	381	569	802	1080	1390	1740	2130	2580	3060	3500	3980	4500	5060
5.4	200	359	539	762	1030	1330	1660	2040	2460	2930	3360	3820	4320	4860
5.6	188	339	511	724	974	1270	1590	1950	2360	2810	3220	3670	4160	4680
5.8	176	319	484	689	929	1210	1520	1870	2260	2700	3090	3530	4000	4510
6.0	164	299	458	656	886	1160	1460	1790	2170	2590	2980	3400	3860	4360
6.2	152	280	433	624	846	1110	1400	1720	2090	2490	2870	3280	3730	4210
6.4	140	260	409	594	808	1060	1340	1650	2010	2400	2760	3160	3600	4070
6.6	130	242	386	564	772	1020	1290	1590	1930	2310	2670	3060	3480	3940
6.8	120	226	363	536	737	971	1230	1530	1860	2230	2570	2960	3370	3820
7.0	112	212	343	509	704	931	1190	1470	1790	2150	2490	2860	3270	3710
7.2	105	199	324	483	671	892	1140	1410	1730	2070	2410	2770	3170	3600
7.4	98	187	306	460	641	855	1100	1360	1670	2000	2330	2680	3070	3500
7.6	92	176	290	437	612	820	1050	1310	1610	1940	2250	2600	2990	3400
7.8	86	165	274	416	585	786	1010	1260	1550	1870	2180	2530	2900	3310
8.0	81	156	260	396	558	754	968	1220	1500	1810	2120	2450	2820	3220
8.2	76	147	246	377	533	722	930	1170	1440	1750	2050	2380	2750	3140
8.4	72	139	234	359	509	692	893	1130	1390	1690	1990	2320	2670	3060
8.6	68	131	221	341	486	662	856	1080	1340	1630	1930	2250	2600	2980
8.8	64	124	209	324	463	633	820	1040	1290	1570	1870	2190	2540	2910
9.0	60	116	198	307	440	604	784	992	1240	1520	1810	2130	2470	2840
9.2	56	109	186	291	418	574	748	948	1190	1460	1740	2060	2410	2770
9.4	52	102	175	274	395	545	710	903	1130	1390	1670	1990	2340	2710
9.6	48	95	163	256	370	513	671	854	1080	1320	1600	1910	2260	2640
9.8	44	87	150	236	343	477	625	798	1010	1250	1510	1820	2160	2550
10.0	36	72	126	200	292	410	540	694	877	1090	1340	1630	1970	2370

Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 10% (Continued)

CHART 5

US CUSTOMARY

C-5

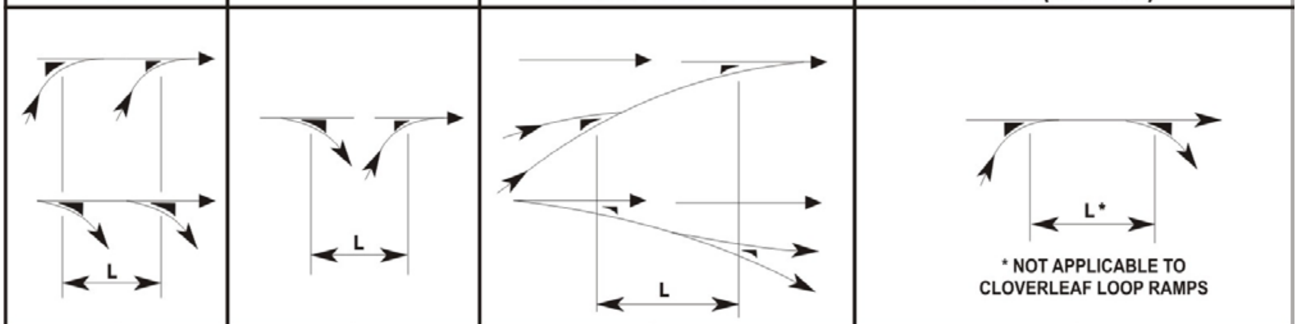
e (%)	V_d = 15 mph R (ft)	V_d = 20 mph R (ft)	V_d = 25 mph R (ft)	V_d = 30 mph R (ft)	V_d = 35 mph R (ft)	V_d = 40 mph R (ft)	V_d = 45 mph R (ft)	V_d = 50 mph R (ft)	V_d = 55 mph R (ft)	V_d = 60 mph R (ft)	V_d = 65 mph R (ft)	V_d = 70 mph R (ft)	V_d = 75 mph R (ft)	V_d = 80 mph R (ft)
1.5	950	1690	2460	3370	4390	5580	6910	8370	9990	11800	13200	14800	16400	18100
2.0	700	1250	1820	2490	3260	4140	5130	6220	7430	8740	9840	11000	12300	13600
2.2	631	1130	1640	2250	2950	3750	4640	5640	6730	7930	8920	9980	11200	12400
2.4	574	1030	1500	2060	2690	3420	4240	5150	6150	7240	8160	9130	10200	11300
2.6	526	936	1370	1890	2470	3140	3900	4730	5660	6670	7510	8420	9380	10500
2.8	484	863	1270	1740	2280	2910	3600	4380	5240	6170	6960	7800	8700	9660
3.0	448	799	1170	1620	2120	2700	3350	4070	4870	5740	6480	7270	8110	9010
3.2	417	743	1090	1510	1970	2520	3130	3800	4550	5370	6060	6800	7600	8440
3.4	389	693	1020	1410	1850	2360	2930	3560	4270	5030	5690	6390	7140	7940
3.6	364	649	953	1320	1730	2220	2750	3350	4020	4740	5360	6020	6740	7500
3.8	341	610	896	1250	1630	2090	2600	3160	3790	4470	5060	5700	6380	7100
4.0	321	574	845	1180	1540	1980	2460	2990	3590	4240	4800	5400	6050	6740
4.2	303	542	798	1110	1460	1870	2330	2840	3400	4020	4560	5130	5750	6420
4.4	286	512	756	1050	1390	1780	2210	2700	3240	3830	4340	4890	5490	6120
4.6	271	485	717	997	1320	1690	2110	2570	3080	3650	4140	4670	5240	5850
4.8	257	460	681	948	1260	1610	2010	2450	2940	3480	3960	4470	5020	5610
5.0	243	437	648	904	1200	1540	1920	2340	2810	3330	3790	4280	4810	5380
5.2	231	415	618	862	1140	1470	1840	2240	2700	3190	3630	4110	4620	5170
5.4	220	395	589	824	1090	1410	1760	2150	2590	3060	3490	3950	4440	4980
5.6	209	377	563	788	1050	1350	1690	2060	2480	2940	3360	3800	4280	4800
5.8	199	359	538	754	1000	1300	1620	1980	2390	2830	3230	3660	4130	4630
6.0	190	343	514	723	960	1250	1560	1910	2300	2730	3110	3530	3990	4470
6.2	181	327	492	694	922	1200	1500	1840	2210	2630	3010	3410	3850	4330
6.4	172	312	471	666	886	1150	1440	1770	2140	2540	2900	3300	3730	4190
6.6	164	298	452	639	852	1110	1390	1710	2060	2450	2810	3190	3610	4060
6.8	156	284	433	615	820	1070	1340	1650	1990	2370	2720	3090	3500	3940
7.0	148	271	415	591	790	1030	1300	1590	1930	2290	2630	3000	3400	3820
7.2	140	258	398	568	762	994	1250	1540	1860	2220	2550	2910	3300	3720
7.4	133	246	382	547	734	960	1210	1490	1810	2150	2470	2820	3200	3610
7.6	125	234	366	527	708	928	1170	1440	1750	2090	2400	2740	3120	3520
7.8	118	222	351	507	684	897	1130	1400	1700	2020	2330	2670	3030	3430
8.0	111	210	336	488	660	868	1100	1360	1650	1970	2270	2600	2950	3340
8.2	105	199	321	470	637	840	1070	1320	1600	1910	2210	2530	2880	3260
8.4	100	190	307	452	615	813	1030	1280	1550	1860	2150	2460	2800	3180
8.6	95	180	294	435	594	787	997	1240	1510	1810	2090	2400	2740	3100
8.8	90	172	281	418	574	762	967	1200	1470	1760	2040	2340	2670	3030
9.0	85	164	270	403	554	738	938	1170	1430	1710	1980	2280	2610	2960
9.2	81	156	259	388	535	715	910	1140	1390	1660	1940	2230	2550	2890
9.4	77	149	248	373	516	693	883	1100	1350	1620	1890	2180	2490	2830
9.6	74	142	238	359	499	671	857	1070	1310	1580	1840	2130	2440	2770
9.8	70	136	228	346	481	650	832	1040	1280	1540	1800	2080	2380	2710
10.0	67	130	219	333	465	629	806	1010	1250	1500	1760	2030	2330	2660
10.2	64	124	210	320	448	608	781	980	1210	1460	1720	1990	2280	2600
10.4	61	118	201	308	432	588	757	951	1180	1430	1680	1940	2240	2550
10.6	58	113	192	296	416	568	732	922	1140	1390	1640	1900	2190	2500
10.8	55	108	184	284	400	548	707	892	1110	1350	1600	1860	2150	2460
11.0	52	102	175	272	384	527	682	862	1070	1310	1560	1820	2110	2410
11.2	49	97	167	259	368	506	656	831	1040	1270	1510	1780	2070	2370
11.4	47	92	158	247	351	485	629	799	995	1220	1470	1730	2020	2320
11.6	44	86	149	233	333	461	600	763	953	1170	1410	1680	1970	2280
11.8	40	80	139	218	312	434	566	722	904	1120	1350	1620	1910	2230
12.0	34	68	119	188	272	381	500	641	807	1000	1220	1480	1790	2130

Exhibit 3-29. Minimum Radii for Design Superelevation Rates, Design Speeds, and e_{max} = 12% (Continued)

ARRANGEMENT FOR SUCCESSIVE RAMP TERMINALS 8-8

FIGURE 1 8 – 8

RECOMMENDED MINIMUM RAMP TERMINAL SPACING F - 1



EN-EN OR EX-EX		EX-EN		TURNING ROADWAYS		EX-EN (WEAVING)			
						SYSTEM TO SERVICE INTERCHANGE		SYSTEM TO SERVICE INTERCHANGE	
FULL FREEWAY	CDR OR FDR	FULL FREEWAY	CDR OR FDR	SYSTEM INTER-CHANGE	SERVICE INTER-CHANGE	FULL FWY.	CDR OR FDR	FULL FWY.	CDR OR FDR
MINIMUM LENGTHS MEASURED BETWEEN SUCCESSIVE RAMPS TERMINALS									
1000 ft	800 ft	500 ft	400 ft	800 ft	500 ft	2000 ft	1600 ft	1600 ft	1000 ft

ARRANGEMENT FOR SUCCESSIVE RAMP TERMINALS

NOTES: FDR – FREEWAY DISTRIBUTOR; CDR – COLLECTOR DISTRIBUTOR; EN - ENTRANCE; EX – EXIT

THE RECOMMENDATIONS ARE BASED ON OPERATIONAL EXPERIENCE AND NEED FOR FLEXIBILITY AND ADEQUATE SIGNING. THEY SHOULD BE CHECKED IN ACCORDANCE WITH THE PROCEDURE OUTLINED IN THE HIGHWAY CAPACITY MANUAL (4) AND THE LARGER OF THE VALUES IS SUGGESTED FOR USE. ALSO, A PROCEDURE FOR MEASURING THE LENGTH OF THE WEAVING SECTION IS GIVEN IN CHAPTER 24 OF THE 2000 HIGHAY CAPACITY MANUAL (4) THE "L" DISTANCES NOTED IN THE FIGURES ABOVE ARE BETWEEN LIKE POINTS, NOT NECESSARILY "PHYSICAL" GORES. A MINMUM DISTANCE OF 270 FT IS RECOMMENDED BETWEEN THE END OF THE TAPER FOR THE FIRST ON RAMP AND THE THEROETICAL GORE FOR THE SUCCEDING ON RAMP FOR THE EN-EN (SIMILAR FOR EX-EN)

FUTURE GUIDELINES 8-9

(This section has been reserved for future guidelines.)

MEDIAN DESIGNS IN INTERCHANGE AREAS 8-10

The median width of a facility should not be reduced through an interchange on either the mainline or the intersecting highway (-Y- Line), if the median is continuous. (See Chapter 1-6 in Part I of this manual.)

Traffic islands on -Y- Lines within the interchange should be provided for highways with four or more lanes. On facilities with three lanes, a 4 foot painted island should be provided. The justification of a left turn lane on the -Y- Line is discussed in 8-15 of this Chapter.

Typically on new facilities angular type exit and parallel type entrance ramps should be utilized. When adding or reconstructing an interchange on an existing facility, the designer should maintain the exit and entrance type if a definite pattern has been established on the freeway segment.

Parallel type entrance lanes should be used in locations where existing interchanges facilities are being up-graded and where right of way is at a premium. See Chapter 8-11, Figures 1-2 of this manual for sample deceleration and acceleration lanes. For additional information see Roadway Standard Drawings, Std. No. 225.03.

The designer should provide sufficient length to enable a driver to make the necessary change between the speed of operation on the highway and the speed on the turning roadway in a safe and comfortable manner. The following Figures and Tables show the appropriate method for obtaining the desirable length of a speed change lane, and how the AASHTO values should be applied to the standard entrance and exit types.

CHART 1 — MINIMUM ACCELERATION LENGTHS FOR ENTRANCE TERMINALS WITH FLAT GRADES OF TWO PERCENT OR LESS — C-1

US Customary

		Acceleration length, *L* (ft) for design speed of exit curve *V*N (mph)								
		Stop condition	15	20	25	30	35	40	45	50
		For average running speed on exit curve, *V'*a (mph)								
Highway design speed, *V* (mph)	Speed reached, *Va* (mph)	0	14	18	22	26	30	36	40	44
30	23	180	140	-	-	–	–	–	–	–
35	27	280	220	160	-	–	–	–	–	–
40	31	360	300	270	210	120	-	–	–	–
45	35	560	490	440	380	280	160	–	–	–
50	39	720	660	610	550	450	350	130	-	–
55	43	960	900	810	780	670	550	320	150	–
60	47	1200	1140	1100	1020	910	800	550	420	180
65	50	1410	1350	1310	1220	1120	1000	770	600	370
70	53	1620	1560	1520	1420	1350	1230	1000	820	580
75	55	1790	1730	1630	1580	1510	1420	1160	1040	780

V = design speed of highway (mph)

V_a = average running speed on highway (mph)

VN = design speed of exit curve (mph)

V'_a = average running speed on exit curve (mph)

NOTE: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.

CHART 2 MINIMUM DECELERATION LENGTHS FOR EXIT TERMINALS WITH FLAT GRADES OF TWO PERCENT OR LESS C-2

US Customary

Highway design speed, V (mph)	Speed reached, V_a (mph)	Deceleration length, L (ft) for design speed of exit curve VN (mph)								
		Stop condition	15	20	25	30	35	40	45	50
		For average running speed on exit curve, V'_a (mph)								
		0	14	18	22	26	30	36	40	44
30	28	235	200	170	140	–	–	–	–	–
35	32	280	250	210	185	–	–	–	–	–
40	36	320	295	265	235	185	155	–	–	–
45	40	385	350	325	295	250	220	–	–	–
50	44	435	405	385	355	315	2852	225	175	–
55	48	480	455	440	410	380	350	285	235	–
60	52	530	500	480	460	430	405	350	300	240
65	55	570	540	520	500	470	440	390	340	280
70	58	615	590	570	550	520	490	440	390	340
75	61	660	635	620	600	575	535	490	440	390

V = design speed of highway (mph)

V_a = average running speed on highway (mph)

VN = design speed of exit curve (mph)

V'_a = average running speed on exit curve (mph)

CHAPTER EIGHT

INTERCHANGES

The information contained in this chapter pertains to the design of ramp connections only. The designer should be familiar with Chapter 10 of the 2004 "A Policy on Geometric Design of Highways and Streets" before beginning the design of any interchange.

The configuration of an interchange should allow all movements to operate at an acceptable level of service as defined in the 1998 "Highway Capacity Manual". The Project Engineer should approve a preliminary design of the interchange before final surveys begin.

CONTROL OF ACCESS AT INTERCHANGES

Control of access along Y lines at interchanges is needed for a minimum of 1000′ beyond the ramp intersections. If for some reason this is not practical, we should provide full control of access for 350′ and then use a raised island to eliminate left turns for the remaining 650′.

LOOP DESIGN 8-1

TYPICAL SECTION:

2′-6″ curb and gutter is placed on the inside of all loops. Pavement widths should be designed to meet Design Widths of Pavements for Turning Roadways see 8-1 Figure 1. Case II (Provision for passing a stalled vehicle).

LOOP DESIGN (Continued) 8-1

SHOULDERS:

See Chapter 1-4D of this manual for width of usable shoulder on outside of loops.

ALIGNMENT:

Freeways - 150′ to 250′ radii unless conditions warrant otherwise. On interstate, loops should be designed for a 30 mph design speed where feasible. (230′ radii minimum for 30 mph design speed).

Expressways - A 150′ radius is acceptable on highways with a 50 mph or less design speed.

Appropriate deceleration and acceleration lanes should be provided for all loops. See Part 1, Section 8-7, Table 1 of this manual. For additional information, see A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS (2004), ch. 10, for acceleration and deceleration lane lengths.

RAMP DESIGN 8-2

TYPICAL SECTION:

Pavement width is normally 14 feet, but where traffic volumes or truck percentages are high, the designer should consider using a width of 16 feet. On the interstate system, the pavement width should be 16 feet.

SHOULDERS:

See Chapter 1-4E of this manual for width of usable shoulder. Paved shoulders are required on both sides.

ALIGNMENT:

Ramp alignments should be designed to provide room for future loop placement in the quadrants where loops could be placed to eliminate left turns from the Y line onto the ramp. Use a minimum of 170′ to 250′ radii for the future loop. Accommodate for the future loop lane under the bridge as well.

See the sight line and geometric measurements. For additional information, see A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS (2004), ch. 5. Detail of Measurement of Sight Distance at Ramp Terminals.

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With reduced handrail offsets specified in the Bridge Policy (see Chapter 6-1 of this Manual), horizontal sight distance has become a more critical element of interchange design. The more narrow bridge restricts the horizontal sight line, so that now the ramp terminal location, Y-line grade, and handrail offset must be considered in combination to attain the required sight distance across the bridge. Each interchange design must be individually studied to achieve the most cost effective combination of bridge width, ramp terminal location, and Y-line grade. A 6′ minimum handrail offset will be used on interchange bridges.

There are four basic options available to the designer for providing the required horizontal sight distances.

1. Design the Y-line grade to enable the driver to see over the bridge handrail and guardrail if present. (Chapter 8-7, Table 1 provides K values for Y-Line grades that will enable the ramp vehicle driver to see over the bridge handrail.)

2. Increase the bridge handrail offset and allow the horizontal sight line to fall inside the handrail. (Chapter 8-7, Table 1 provides K values for Y-Line grades that will allow a clear sight line inside the bridge handrail.)

3. Use the minimum handrail offset required by the Bridge Policy (see Chapter 6-1 of this Manual) and locate the ramp terminal a sufficient distance from the bridge end to provide the required sight distance. (The grade on Chapter 8-7, Table 2 shows the distance required from the end of bridge to ramp terminal that provides required horizontal sight distance with various bridge handrail offset distances. Conversely, this graph can show the available horizontal sight distance with set ramp terminals and handrail offset distances. This graph may also be use to derive combinations of handrail offsets and ramp terminal locations that may be necessary in an economic analysis of the interchange layout.)

4. Consider designing grades with the mainline carried over the Y-line.

This design may be cost effective with a narrow median on the mainline and a multilane Y-line. Earthwork costs are usually the critical cost elements in this option.

JUSTIFICATION OF LEFT TURN LANES ON TWO-LANE HIGHWAYS 8-15

The need for a left turn lane on an interchange -Y- line should be carefully evaluated by the designer, since it affects the width of the interchange bridge. The need for a left turn lane is determined by traffic volumes, speed, and safety benefits.

The method for determining the warrants for left turn lanes at unsignalized at-grade intersections (applicable to interchange ramp terminals) is addressed in the attached nomograph. The method utilizes a nomograph based on opposing volumes, left turn volumes, and through volumes. The time delays and queuing characteristics of the traffic volumes are the criteria utilized in establishing these nomographs.

The elements to be used in entering the appropriate nomograph are:

- Operating speed (see A POLICY ON GEOMETRIC DESIGN OF HIGHWAYS AND STREETS (2004), ch. 2.
- V/o, opposing traffic volume
- VL, left turning volume(VPH)
- Va, advancing traffic volume, including through, left turning, and right turning vehicles (design hour volume).
- VR, right turning volume(VPH)
- S, storage length required

If the intercept of V and Va falls right of the applicable S line, that is the amount of storage warranted.

*3-Series (cont'd)	Parcel Index Sheet (Applicable to projects with more than one plan sheet.)
4	The first plan sheet will always be Number 4. All other plan and profile sheets shall be numbered to fit the project conditions.
TMP-1, TMP-2, etc.	Transportation Management Plans
PMP-1, PMP-2, etc.	Pavement Marking Plans
E-1, E-2, etc.	Electrical Plan
EC-1, EC-2, etc.	Erosion Control Plans
L-1, L-2, etc.	Landscape Plans
SIGN-1, SIGN-2, etc.	Signing Plans
SIG-1, SIG-2, etc.	Signal Plans
UC-1, UC-2, etc.	Utility Construction Plans
UO-1, UO-2, etc	Utilities by others Plans
X-1A, X-1B, etc.	Cross-Section Summary Sheet
X-1, X-2, etc.	Cross-Sections
C-1, C-2, C-3, etc.	Culvert Plans
S-1, S-2, S-3, etc.	Structure Plans

Do not show total sheet numbers on the plans.