utilized beyond which the lateral friction is kept constant and superelevation is increased rapidly to maximum. This form of distribution is referred to as “Method 2” in AASHTO and is used in some urban areas. This method is particularly advantageous on low speed urban streets where, because of various constraints, superelevation frequently cannot be provided.

Rural & High Speed Urban Applications: Design Domain Quantitative Aids

Based on the described distribution method (Method 5), the recommended superelevation rate for various radii from 7000 m to the minimum radius, for each design speed and for superelevation rates of 0.04 m/m, 0.06 m/m, and 0.08 m/m have been developed. Also included are the spiral parameters applicable to rural and high speed urban roadways (spiral parameters will be addressed later in this Section). These values are illustrated in Tables 2.1.2.5 to 2.1.2.7 and are based on relationships presented in a 1974 publication[31].

Urban Roadways: Design Domain Technical Foundation

Tables 2.1.2.5 to 2.1.2.7 indicating the amount of superelevation for various radii are based on relatively low lateral friction values which give high superelevation rates. These superelevation rates and spiral parameters are appropriate for rural roadways and higher speed urban roadways where intersections are widely spaced and access is restricted. For other urban situations higher rates of superelevation are often not attainable due to constraints such as access requirements, elevations of the adjacent properties, drainage considerations, the profiles of intersecting streets and limiting slopes on boulevards and sidewalks.

An alternative method for selecting superelevation rates under low speed urban design conditions is to use higher lateral friction values, thereby reducing the superelevation requirements. These superelevation rates are illustrated in Figure 2.1.2.4 and generally apply to roadways through intersection areas where design constraints are numerous and limiting superelevation offers operational advantages for turning or crossing traffic. Drivers are generally accustomed to using more lateral friction when manoeuvring through intersections.

Urban Roadways: Design Domain Quantitative Aids

Figure 2.1.2.5 illustrates an alternative method for urban use for applying superelevation over a range of radii. This method, for urban conditions, is based on retaining normal crown until a lateral friction factor of 0.05 is required, at which point reverse crown is introduced. This friction factor is approximately equivalent to roadway icing conditions. An assumed operating speed of 10 km/h above design speed is used in the derivations to introduce the safety factor for overdriving or adverse pavement conditions. Figure 2.1.2.5 illustrates the derivation of these values for a design speed of 50 km/h, and maximum superelevation values of 0.04 m/m and 0.06 m/m.

For this alternative design method the minimum radius in reverse crown is determined by distributing the lateral friction coefficients in a rate proportional to the inverse of the radius. The linear distribution occurs between the lateral friction coefficient at the inverse of the minimum radius for normal crown, calculated as noted above, and the maximum lateral friction coefficient of the inverse of the minimum radius corresponding to full superelevation. Because a unique minimum radius corresponds to each of the two full superelevation rates, +0.04 m/m and +0.06 m/m, the proportional distribution produces different values for minimum radius at reverse crown.

This alternative method provides superelevation rates between the rural and urban high speed values, and the low speed urban values described earlier. Table 2.1.2.4, Table 2.1.2.8 and Table 2.1.2.9 reflect this alternative design method for selecting superelevations over a range of radii from 20 m to 7000 m. This provides the designer with a consistent means of selecting superelevation applicable to most urban conditions. Table 2.1.2.4 was discussed earlier under minimum radius, and Tables 2.1.2.8 and 2.1.2.9 provide superelevation rates for varying speeds and radii using the urban method described above. Table 2.1.2.8 is for

Table 2.1.2.5 Superelevation and Minimum Spiral Parameters, $e_{max} = 0.04$ m/m[1]

Design Speed (km/h)	40			50			60			70			80			90			100		
		A			A			A			A			A			A			A	
Radius (m)	e	2 lane	4 lane	e	2 lane	4 lane	e	2 lane	4 lane	e	2 lane	4 lane	e	2 lane	4 lane	e	2 lane	4 lane	e	2 lane	4 lane
7000	NC			NC			NC			NC			NC			NC			NC		
5000	NC			NC			NC			NC			NC			NC			NC		
4000	NC			NC			NC			NC			NC			NC			NC		
3000	NC			NC			NC			NC			NC			NC			RC	410	410
2000	NC			NC			NC			NC			RC	300	300	RC	315	315	RC	335	335
1500	NC			NC			NC			RC	240	240	RC	260	260	RC	275	275	0.022	290	290
1200	NC			NC			NC			RC	215	215	RC	230	230	0.021	245	245	0.026	260	260
1000	NC			NC			RC	180	180	RC	200	200	0.020	210	210	0.025	225	225	0.030	235	235
900	NC			NC			RC	175	175	RC	190	190	0.021	200	200	0.027	210	210	0.032	225	225
800	NC			NC			RC	165	165	RC	180	180	0.023	190	190	0.029	200	200	0.034	210	210
700	NC			RC	140	140	RC	150	150	0.020	165	165	0.026	175	175	0.032	185	185	0.036	200	200
600	NC			RC	130	130	RC	140	140	0.023	155	155	0.029	165	165	0.035	175	175	0.039	190	190
500	NC			RC	120	120	0.021	130	130	0.026	140	140	0.033	150	150	0.038	160	160	0.040	190	190
400	RC	95	95	RC	105	105	0.025	115	115	0.031	125	125	0.037	135	135	0.040	160	160	0.040	190	190
350	RC	90	90	0.020	100	100	0.027	110	110	0.033	120	120	0.039	135	135	0.040	160	160	min R = 490		
300	RC	80	80	0.023	90	90	0.031	100	100	0.036	110	110	0.040	135	135	min R = 380					
250	RC	75	75	0.026	85	85	0.034	90	90	0.038	110	110	0.040	135	135						
220	RC	70	70	0.029	80	80	0.036	90	90	0.039	110	110	min R = 280								
200	0.021	65	65	0.031	75	75	0.038	90	90	0.040	110	110									
180	0.023	65	65	0.033	70	70	0.039	90	90	0.040	110	110									
160	0.025	60	60	0.035	70	70	0.040	90	90	min R = 200											
140	0.028	55	55	0.037	70	70	0.040	90	90												
120	0.031	50	55	0.039	70	70	min R = 150														
100	0.034	50	50	0.040	70	70															
90	0.036	50	50	0.040	70	70															
80	0.038	50	50	min R = 100																	
70	0.040	50	50																		
60	0.040	50	50																		
	0.040	50	50																		
	min R = 60																				

$e_{max} = 0.04$

Notes:
- *e* is superelevation, m/m
- *A* is spiral parameter in metres
- NC is normal cross section
- RC is remove adverse crown and superelevate at normal rate
- Spiral length, $L = A^2$ / Radius
- Spiral parameters are minimum and higher values by be used
- For 6 lane pavement: above the dashed line use 4 lane values, below the dashed line, use 4 lane values x 1.15.
- A divided road having a median less than 3.0 m wide may be treated as a single pavement.

Table 2.1.2.6 Superelevation and Minimum Spiral Parameters, $e_{max} = 0.06$ m/m[1]

Design Speed (km/h)	40			50			60			70		
		A			A			A			A	
Radius (m)	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane
7000	NC			NC			NC			NC		
5000	NC			NC			NC			NC		
4000	NC			NC			NC			NC		
3000	NC			NC			NC			NC		
2000	NC			NC			NC			RC	275	275
1500	NC			NC			RC	225	225	RC	250	250
1200	NC			NC			RC	200	200	0.023	225	225
1000	NC			RC	170	170	0.021	175	175	0.027	200	200
900	NC			RC	150	150	0.023	175	175	0.029	180	180
800	NC			RC	150	150	0.025	160	160	0.031	175	175
700	NC			0.021	140	140	0.027	150	150	0.034	175	175
600	NC	120	120	0.024	125	125	0.030	140	140	0.037	150	150
500	RC	100	100	0.027	120	120	0.034	125	125	0.041	140	150
400	0.023	90	90	0.031	100	100	0.038	115	120	0.045	125	135
350	0.025	90	90	0.034	100	100	0.041	110	115	0.048	120	125
300	0.028	80	80	0.037	90	100	0.044	100	110	0.051	120	125
250	0.031	75	80	0.040	85	90	0.048	90	100	0.055	110	120
220	0.034	70	80	0.043	80	90	0.050	90	100	0.057	110	110
200	0.036	70	75	0.045	75	90	0.052	85	100	0.059	110	110
180	0.038	60	75	0.047	70	90	0.054	85	90	0.060	110	110
160	0.040	60	75	0.049	70	85	0.056	85	90	min R = 190		
140	0.043	60	70	0.052	65	80	0.059	85	90			
120	0.046	60	65	0.055	65	75	0.060					
100	0.049	50	65	0.058	65	70	min R = 130					
90	0.051	50	60	0.060	65	70						
80	0.054	50	60	0.060	65	70						
70	0.056	50	60	min R = 90								
60	0.059	50	60									
	0.059	50	60									
	min R = 55											

Design Speed (km/h)	80			90			100			110			120			130		
		A			A			A			A			A			A	
Radius (m)	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane
7000	NC			NC			NC			NC			RC			RC	710	710
5000	NC			NC			NC			RC	555	555	RC	580	580	RC	600	600
4000	NC			NC			RC	475	475	RC	495	495	RC	515	515	0.023	540	540
3000	NC			RC	390	390	RC	410	410	0.022	430	430	0.024	450	450	0.036	465	465
2000	RC	300	300	0.023	300	300	0.026	335	335	0.029	350	350	0.034	365	365	0.040	380	380
1500	0.024	250	250	0.029	270	270	0.032	290	290	0.036	305	305	0.042	315	315	0.049	330	335
1200	0.028	225	225	0.033	240	240	0.038	260	260	0.043	270	270	0.049	285	290	0.055	295	320
1000	0.032	200	200	0.037	225	225	0.043	235	235	0.048	245	255	0.054	260	280	0.058	280	300
900	0.034	200	200	0.039	200	200	0.045	225	225	0.051	235	250	0.057	250	270	0.060	280	300
800	0.036	175	175	0.042	200	200	0.048	210	215	0.054	220	240	0.059	250	260	min R = 950		
700	0.039	175	175	0.045	185	195	0.051	200	210	0.058	220	235	0.060	250	260			
600	0.042	175	175	0.048	175	185	0.054	190	200	0.060	220	220	min R = 750					
500	0.046	150	160	0.052	160	175	0.059	190	190	0.060	220	220						
400	0.051	135	150	0.057	160	165	0.060	190	190	min R = 600								
350	0.054	125	140	0.059	160	160	min R = 440											
300	0.057	125	135	0.060	160	160												
250	0.060	125	125	min R = 340														
220	0.060	125	125															
200	min R = 250																	

$e_{max} = 0.06$

Notes:
- *e* is superelevation m/m
- *A* is spiral parameter in metres
- NC is normal cross section
- RC is remove adverse crown and superelevate at normal rate
- Spiral length, $L = A^2$ / Radius
- Spiral parameters are minimum and higher values may be used
- For 6 lane pavement: above the dashed line use 4 lane values, below the dashed line, use 4 lane values x 1.15.
- A divided road having a median less than 3.0 m wide may be treated as a single pavement.

Table 2.1.2.7 Superelevation and Minimum Spiral Parameters, $e_{max} = 0.08$ m/m[1]

Design (Speed km/h)	40			50			60			70			80			90			100			110			120			130		
		A			A			A			A			A			A			A			A			A			A	
Radius (m)	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane	e	2 lane	3&4 lane
7000	NC			NC			NC			NC			NC			NC			NC			RC			RC			RC	710	710
5000	NC			NC			NC			NC			NC			NC			RC			RC	555	555	RC	580	580	0.021	600	600
4000	NC			NC			NC			NC			NC			NC			RC	480	480	RC	495	495	0.021	515	515	0.026	540	540
3000	NC			NC			NC			NC			NC			RC	390	400	RC	410	410	0.023	430	430	0.028	450	450	0.033	465	465
2000	NC			NC			NC			RC	270	270	0.021	300	300	0.026	300	300	0.026	335	335	0.033	350	350	0.040	365	365	0.047	380	380
1500	NC			NC			RC	225	225	0.021	255	255	0.027	250	250	0.032	270	275	0.033	290	290	0.042	305	305	0.050	315	330	0.058	330	365
1200	NC			NC			RC	200	200	0.026	220	220	0.032	225	225	0.038	240	240	0.040	260	260	0.050	270	285	0.059	285	320	0.067	295	350
1000	NC			RC	170	170	0.023	175	175	0.029	200	200	0.036	200	200	0.043	225	225	0.047	240	240	0.057	250	280	0.066	260	310	0.074	280	340
900	NC			RC	150	150	0.025	175	175	0.032	180	180	0.039	200	200	0.046	200	220	0.051	225	240	0.062	235	275	0.071	250	300	0.078	280	330
800	NC			0.020	150	150	0.027	160	160	0.035	175	175	0.042	175	195	0.049	200	220	0.055	210	235	0.066	220	270	0.075	250	295	0.080	280	330
700	NC			0.023	140	140	0.030	150	150	0.038	165	165	0.046	175	190	0.053	185	200	0.061	200	230	0.072	220	260	0.079	250	285	0.080	280	300
600	RC	120	120	0.026	125	125	0.034	140	140	0.042	150	160	0.050	165	185	0.058	175	200	0.067	190	225	0.77	220	250	0.080	250	285	min R = 830		
500	0.021	100	100	0.030	120	120	0.039	125	135	0.048	140	150	0.056	150	175	0.064	160	200	0.073	190	215	0.80	220	250	min R = 670					
400	0.025	90	90	0.035	100	110	0.045	115	125	0.054	125	150	0.063	135	165	0.071	160	185	0.080	190	200	min R = 530								
350	0.028	90	90	0.038	100	105	0.049	110	125	0.058	120	150	0.067	125	160	0.075	160	175	0.080	190	200									
300	0.031	80	90	0.042	90	100	0.053	100	120	0.063	120	140	0.072	125	150	0.080	160	175	min R = 390											
250	0.035	75	85	0.047	85	100	0.059	100	120	0.069	110	135	0.078	125	150	0.080	160	175												
220	0.039	70	80	0.051	80	100	0.062	95	110	0.073	110	125	0.080	125	150	min R = 300														
200	0.041	70	80	0.054	80	100	0.065	90	110	0.075	110	125	min R = 230																	
180	0.044	65	80	0.057	75	95	0.068	90	105	0.078	110	120																		
160	0.047	65	80	0.060	75	90	0.072	85	100	0.080	110	120																		
140	0.051	65	75	0.064	70	90	0.076	85	100	min R = 170																				
120	0.055	60	75	0.069	70	85	0.080	85	95																					
100	0.061	55	70	0.074	65	80	0.080	85	95																					
90	0.064	55	70	0.077	65	80	min R = 120																							
80	0.067	55	65	0.080	65	75																								
70	0.071	50	60	0.080	65	75																								
60	0.075	50	60	min R = 80																										
50	0.080	50	60																											
	0.080	50	60																											
	min R = 50																													

$e_{max} = 0.08$

Notes:
- *e* is superelevation, m/m
- *A* is spiral parameter in metres
- NC is normal cross section
- RC is remove adverse crown and superelevate at normal rate
- Spiral length, $L = A^2$ / Radius
- Spiral parameters are minimum and higher values by be used
- For 6 lane pavement: above the dashed line use 4 lane values, below the dashed line, use 4 lane values x 1.15.
- A divided road having a median less than 3.0 m wide may be treated as a single pavement.

Table 2.1.2.9 Superelevation Rate for Urban Design, e_{max} = 0.06 m/m[9]

Radius (m)	Design Speed (km/h) 30	40	50	60	70	80	90	100
7000	NC	NC	NC	NC	NC	NC	NC	NC
5000								
4000								NC
3000						NC	NC	RC
2000					NC	RC	RC	
1500				NC	RC			
1200				RC				
1000								
900			NC					
800			RC					RC
700		NC						0.025
600		RC					RC	0.035
500	NC					RC	0.030	0.048
400	RC					0.026	0.045	Min R=440
350					RC	0.035	0.056	
300					0.025	0.045	Min R=340	
250				RC	0.036	Min R=250		
200				0.024	0.053			
180				0.030	Min R=190			
160				0.037				
140			RC	0.046				
120			0.026	Min R=120				
100			0.036					
90			0.043					
80		RC	0.052					
70		0.024	Min R=75					
60		0.032						
50		0.044						
40	RC	Min R=40						
30	0.030							
20	0.056							
	Min R=20							
min. radius for normal crown	420	660	950	1290	1680	2130	2620	3180
min. radius for reverse crown	40	80	135	220	330	450	600	770

e_{max} = 0.06 m/m
NC = normal crown (-0.02 m/m)
RC = reverse crown (+0.02 m/m)

On tight horizontal curves, stopping sight distances may need to be increased to compensate for increased braking distances caused by lateral friction effects.

Design Domain Quantitative Aids: Stopping Sight Distance on Horizontal Curves

Table 2.1.2.10 shows the stopping sight distance while braking on a minimum radius curve. Values are provided for the higher and lower limits for each of the three values of maximum superelevation. Also included for comparison purposes are the normal stopping sight distances from Table 1.2.5.3.

The results of the calculations for stopping sight distance on minimum radius curves indicates that there is a large variance in the amount of increase for the various conditions.

1. The increase in stopping sight distance based on the lower range of assumed operating speeds (left columns in Table 2.1.2.10) vary from 0.3% to 4.2%.

2. The increase in stopping sight distance based on the upper range of assumed operating speeds (right columns in Table 2.1.2.10) vary from 3.2% to 8.8%.

Because of the variance, providing specific adjustments for radii other than minimum is difficult. Suggested approximate adjustments are:

- for the lower range of assumed operating speeds from 40 km/h to 80 km/h and for horizontal curves not exceeding 110% of the minimum radius curve, the stopping sight distance in Table 1.2.5.3 should be increased by 3.0%

- for the higher range of assumed operating speeds from 40 km/h to 120 km/h and for horizontal curves not exceeding 110% of the minimum radius curve, the stopping sight distance in Table 1.2.5.3 should be increased by 5.5%

If adjustments that are more precise than outlined above are required, the designer should use the formulae provided in this Section.

2.1.2.3 Spiral Curves

<u>Introduction</u>

A spiral curve is a curve with a constantly varying radius. The purpose of a spiral curve is to provide smooth transition and a natural driving path between a tangent and a circular curve.

Table 2.1.2.10 Calculated Stopping Sight Distance on Minimum Radius Curves

Design Speed (km/h)	Assumed Operating Speed (km/h)	Normal Stopping Sight Distance Table 1.2.5.3 (m)	Calculated Stopping Sign Distance on Minimum Radius Curves (m)		
			e = 0.04	e = 0.06	e = 0.08
40	40	45	46	46	46
50	47-50	60-65	60-66	60-66	60-66
60	55-60	75-85	77-90	77-90	77-90
70	63-70	95-110	98-120	97-120	97-120
80	70-80	115-140	117-152	116-152	116-151
90	77-90	130-170	135-180	134-180	134-181
100	85-100	160-210	161-219	160-218	160-219
110	91-110	180-250		181-259	181-259
120	98-120	200-290		205-302	204-301
130	105-130	230-330		229-338	229-338

Because spiral curves provide a natural path for the motorist to follow, centrifugal forces increase and decrease gradually as the vehicle enters and leaves the circular portion of the curve. This minimizes encroachment upon adjoining traffic lanes, promotes speed uniformity, and increases safety.

A spiral curve provides a convenient and desirable arrangement for developing superelevation runoff. A change from normal crown to a fully superelevated cross section is applied along the spiral curve length. Where the pavement section is widened around a circular curve, the spiral facilitates the transition width. Spiral curves also enhance the roadway appearance because there are no noticeable breaks in the alignment.

On rural roads, spiral curves should be utilized on new construction if the circular curves are superelevated. Some exceptions can be made with curves substantially flatter than the minimum required for the design speed. Also, when appropriate, simple curves may be used on existing paved roads to avoid the need to make minor realignments.

On urban roads, the incorporation of spiral curves into horizontal main line alignment design is normally limited to major arterials, expressways and freeways, where higher design speeds are used. Spiral curves are typically only applied to roadways with design speeds of 70 km/h and higher, and where superelevation of the circular curves is desirable. Spiral curves are also used in the design of interchange ramps. Where these roadway facilities have curbs, the use of spiral curves allows vehicles to negotiate the alignment at a constant offset from the curb, thus increasing driver comfort. For low speed urban design and retrofit situations, spiral curves are not typically used.

<u>Overview: Technical Foundation</u>

The spiral form most commonly used for road design is the clothoid, which, expressed mathematically, has the relationship where R varies with the reciprocal of L, where R is the radius of curve at a distance L from the beginning of spiral ($R \alpha 1/L$).

$$A^2 = RL \qquad (2.1.8)$$

Where A is a constant called the spiral parameter and has units of length.

All clothoid spirals are the same shape and vary only in their size. The spiral parameter is a measure of the flatness of the spiral, the larger the parameter the flatter the spiral.

A spiral curve in which one end of the spiral has a radius of infinity is referred to as a simple spiral and one in which the radii at both ends are less than infinity is referred to as a segmental spiral. An example of a segmental spiral is a spiral between two circular curves of different radii but in the same direction. A simple spiral is designated by its parameter and its end radius. A segmental spiral is designated by its parameter and its two end radii.

<u>The Spiral Parameter: Technical Foundation</u>

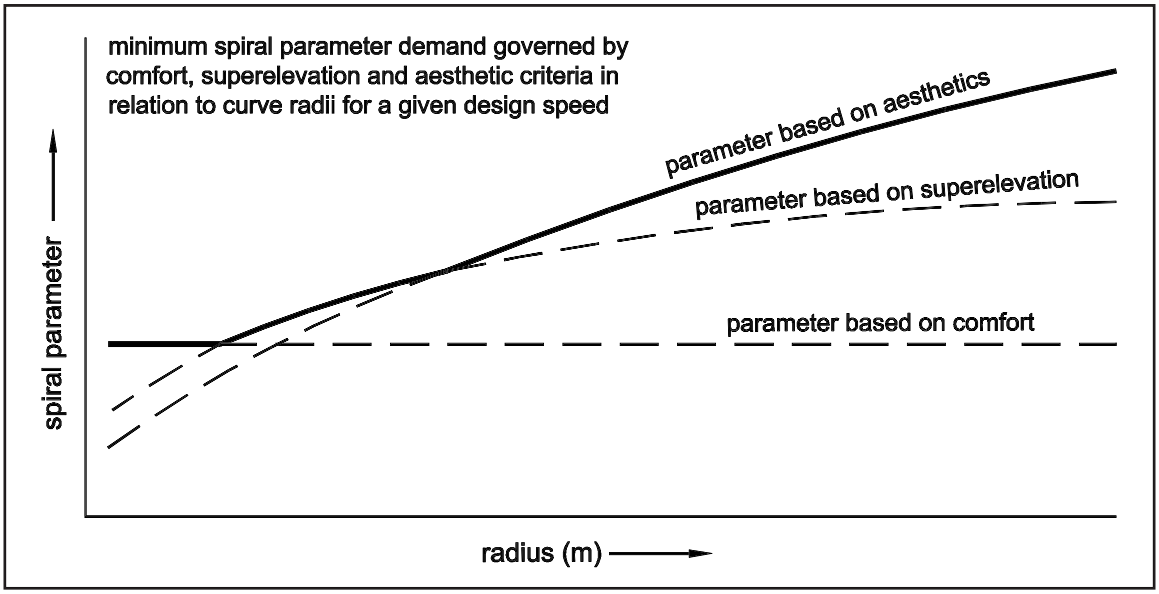
Spiral parameters can be developed based on three criteria:

- the maximum permissible rate of change of centripetal acceleration (which expresses comfort)
- superelevation runoff
- aesthetics

Figure 2.1.2.7 illustrates the conceptual results of these means of deriving the spiral parameter, and the basis for each of these methodologies is discussed in depth following this Figure.

The minimum spiral requirement for design is the highest of the three values and for the smaller radii the comfort criterion controls, for the next larger set of radii the relative slope criterion controls, and for the larger radii the aesthetic criterion controls.

Figure 2.1.2.7 Basis for Spiral Parameter Design Values[1]



Spiral Parameter Based on Comfort

A vehicle travelling along a circular curve experiences centripetal acceleration. As the vehicle is moving from a tangent direction to a circular curve path, the radius of curvature decreases from infinity to that of the circular curve. During this time the centripetal acceleration is increasing from zero to a constant. The rate of change of acceleration is high if the transition length is short and low if it is long. The rate of change of centripetal acceleration is a measure of discomfort to the vehicle occupants. If the transition length is short the vehicle occupants will experience a jerk. Lower values of acceleration and corresponding long transition are preferred.

One of the purposes of the spiral is to provide a length over which the driver can effect a change in curvature of the travelled path so as to position the vehicle centrally within the lane. For comfort therefore the spiral curve should provide sufficient length to allow a smooth increase in radial acceleration.

Centripetal acceleration is: $\frac{v^2}{R}$

Where R is the radius (m)
v is the speed (m/s)

Rate of change of centripetal acceleration is:

$$c = \frac{v^2}{Rt} \qquad (2.1.9)$$

Where t is travel time (s)

$$t = \frac{L}{v} \qquad (2.1.10)$$

Where L is length of curve (m)

$$c = \frac{v^3}{A^2} \qquad (2.1.11)$$

$$A = \frac{v^{1.5}}{c^{0.5}} \qquad (2.1.12)$$

If A is stated in metres, speed v in kilometres per hour and c in metres per second cubed, the expression becomes:

$$A = \frac{0.1464V^{1.5}}{c^{0.5}} \qquad (2.1.13)$$

Tolerable rate of change of centripetal acceleration varies between drivers. As a basis of design, the value used to provide minimum acceptable comfort is 0.6 m/s^3. The expression then becomes:

$$A = \frac{0.1464V^{1.5}}{0.6^{0.5}} \qquad (2.1.14)$$

$$A = 0.189V^{1.5} \qquad (2.1.15)$$

Using the above expression, the minimum spiral parameter based on comfort can be calculated for each design speed. It may be noted that the spiral parameter is independent of the radius. This is illustrated in Figure 2.1.2.7 by the comfort line parallel to the abscissa.

Spiral Parameter Based on Relative Slope

For superelevation runoff, the relative slope is defined as the slope of the outer edge of a pavement in relation to the profile control line. The maximum permissible value varies with design speed, and is shown in Table 2.1.2.11.

The minimum length of transition, l, is given by the equation

$$l = \frac{100\ we}{2s} \qquad (2.1.16)$$

Where w = the width of pavement in metres

e = the superelevation being developed in metres per metre

s = the superelevation runoff, percentage

l = measured in metres

Table 2.1.2.11 Maximum Relative Slope Between Outer Edge of Pavement and Centreline of Two-Lane Roadways

Design Speed (km/h)	Relative Slope (%)
40	0.70
50	0.65
60	0.60
70	0.55
80	0.51
90	0.47
100	0.44
110	0.41
120	0.38
130	0.36

For a given speed and radius, superelevation and relative slope are known and minimum lengths can be calculated. From minimum length and radius, the minimum spiral parameter can be calculated, using the expression:

$$A^2 = RL \qquad (2.1.17)$$

Spiral Parameter Based on Aesthetics

Short spiral transition curves are visually unpleasant. It is generally accepted that the length of a transition curve should be such that the driving time is at least 2 s. For a given radius and speed, therefore, the minimum length and minimum spiral parameter can be calculated using the expression:

$$A^2 = 0.56RV \qquad (2.1.18)$$

<u>Spiral Parameter: Design Domain Quantitative Aids</u>

Quantitative expressions of the design domain for the spiral parameter are given in Tables 2.1.2.5, 2.1.2.6 and 2.1.2.7 in Subsection 2.1.2.2 for a range of design speeds. Designers should note the following application heuristics in using these tables.

1. For any particular design speed and radius, the highest value of spiral parameter as determined by the methodologies discussed in the previous section are used for these calculations.

2. The design values in the tables are minimum and higher values should be used wherever possible in the interests of safety, comfort and aesthetics.

3. In design it is convenient to select values so that the radius is a standard value and A/R is a rational number as this permits spiral properties to be read directly from tables[33].

Spiral Parameter: Design Domain Application Heuristics

The application of spiral curves in horizontal alignment is a complex design problem that has many variations. A number of design domain application heuristics dealing with some of the more common instances are provided below.

1. A circular curve with simple spirals at both ends each having the same parameter value is referred to as a symmetrical curve. This condition represents the most common practice for spiraled curves.

2. Unsymmetrical curves are common at interchange ramps and loops and represent the case noted in #1 above, but with different parameter values for the spirals at each end.

3. Successive circular curves with different radii but in the same direction are best joined by a spiral curve. Where this occurs, the “joining spiral” is referred to as a segmental spiral. The minimum spiral parameter to be used is found by referring to Tables 2.1.2.5, 2.1.2.6 and 2.1.2.7 and using the smaller of the two radii.

4. In some instances, successive circular curves with different radii may connect directly to one another if the difference in radii is not too large. See Subsection 2.1.2.6 for additional guidance in this regard.

5. Circular, non-successive curves in the same direction joined by a short length of tangent should normally be joined instead by a spiral. The use of tangents to achieve such connections results in what is commonly referred to as a “broken back” curve. It is only justified when some other consideration, for example, property constraints or construction cost, outweighs the visual disadvantages. The term “short tangent” is very subjective and there are several definitions for this term. One suggests that a short tangent length may be regarded as one which allows a driver on the first curve to see at least some part of the following curve. Another definition suggests a broken back curve occurs when the tangent length (m) is less than four times the design speed. If possible a more desirable solution to a broken back curve is to eliminate the short tangent and insert an appropriate circular curve or better yet a segmental spiral curve.

6. A change of direction from one tangent to another may be accomplished by successive spiral curves without a length of circular curve between them. Such a transition curve is referred to as symmetrical where the two spiral parameters are the same, and unsymmetrical where they are different. The minimum permissible spiral parameter to be used in such a situation is the minimum for the design as shown in Tables 2.1.2.5, 2.1.2.6 and 2.1.2.7.

7. A reversal in curvature direction may be accomplished through successive simple spiral curves without a length of tangent between them. The spiral parameters should be at least the minimum for the design speed. However, the alignment will have an improved appearance if the minimum spiral parameter values are exceeded or if a length of tangent is inserted between the two spirals. Superelevation is applied as

To illustrate the use of equations 2.1.20 and 2.1.21, consider a curve with R = 0.35 km that is between tangents that are 0.9 km long and which intersect at a deflection angle of 120° (2.09 radians). The roadway is 11.2 m wide and has spiral transition curves. What would be the safety consequences of increasing the radius to 0.437 km^2.

In equation 2.1.20, the parameter 0.96 applies to collisions/million vehicle km on a straight section. Therefore, on the two tangents we expect 2 x 0.9 x 0.96 = 1.73 collisions/million vehicles. The length of the curve is 0.35 x 2.09 = 0.732 km. On the curve one may expect (0.96 x 0.732 + 0.0245/0.35 - 0.012) x 0.978^{37-30} = 0.65 collisions/million vehicles. The correction factor for tangent length is $e^{-(0.62 - 1.2x0.35) \times (1.2 - 0.9)}$ = 0.94. Thus, with a 0.35 km radius curve one should expect 1.73 + 0.65 x 0.94 = 2.34 collisions/ million vehicles.

Suppose now that a curve with R = 0.437 km is considered for the same conditions. The larger radius implies a longer curve and shorter tangents. Each tangent length is reduced by (0.437-0.35) x tan (2.09/2) = 0.15 km. Thus, on the two tangents one expects 2 x (0.9 - 0.15) x 0.96 = 1.44 collisions/million vehicles. The length of the curve is now 0.437 x 2.09 = 0.913 km and with long tangents one may expect on it (0.96 x 0.913 + 0.0245/0.437 - 0.012) x 0.978^{37-30} = 0.8 collisions/million vehicles. The correction factor now is $e^{-(0.62 - 1.2 \times 0.437) \times (1.2 - 0.75)}$ = 0.96. Thus, with a R=0.437 km curve one should expect 1.44 + 0.8 x 0.96 = 2.21 collisions/million vehicles. If AADT = 5000, the collision reduction is (2.34 - 2.21) x 5000 x 365 x 10^{-6} = 0.24 collisions/ year. Additional safety benefits may accrue to the curves connected to the other ends of the tangents.

There is a simple approximate way of determining the collision reduction which is due to increasing the radius. Disregarding the safety benefit due to shorter tangents, when $R_1 < R_2$:

expected collision reduction per unit of time = (collisions on 1 km of straight road per unit of time) x (reduction in path length in km) + 0.0245 x million vehicles per unit of time x $(1/R_2 - 1/R_1)$

The “reduction in path length in km” is given by

$$(R_1 - R_2) \times [2\tan(D/2) - D] \qquad (2.1.22)$$

in which D is the deflection angle in radians.

Numerical Example:

As in the previous numerical example, let R_1 = 0.437 km, R_2 = 0.35 km, D = 120° (2.09 radians) and Volume = 5000 x 365 x 10^{-6} = 1.825 million vehicles/year. The reduction in path length is (0.437-0.35) x [2tan (2.09/2) - 2.09] = 0.12 km. If there are 0.96 collisions/million vehicle km on a straight road, then, with 1.825 million vehicles per year, one expects 0.96 x 1.825 = 1.75 collisions/year for a kilometre of road. Thus, the expected collision reduction is 1.75 x 0.12 + 0.0245 x 1.825 x (1/0.35-1/0.437) = 0.21 + 0.03 = 0.24 collisions/year.

In summary, when a horizontal curve is to be fitted between tangents with a given deflection angle, the greater the radius of the curve the fewer collisions will occur on the roadway.

3. Where possible, gradients lower than the maximum values shown should be used.

4. Maximum values should only be exceeded after a careful assessment of safety, cost, property and environmental implications.

5. The choice of maximum gradient may have a bearing on related design features; for example, whether or not a truck climbing lane or escape lane is required.

6. While Table 2.1.3.1 provides general guidance, the designer should be aware that the factors that should be considered in establishing the maximum grade for a section of roadway include:

   - road classification
   - traffic operation
   - terrain
   - climatic conditions
   - length of grade
   - costs
   - property
   - environmental considerations
   - in urban areas, adjacent land use

7. Maximum grades of 3 to 5% are considered appropriate for design speeds of 100 km/h and higher. This may have to be modified in regions with severe topography such as mountainous terrain, deep river valleys, and large rock outcrops.

8. Maximum grades of 7 to 12% are appropriate for design speeds of 50 km/h and lower. If only the more important roadways are considered, 7% or 8% would be a representative maximum grade for a design speed of 50 km/h.

9. Control grades for other speeds between 50 km/h and 100 km/h are intermediate between the above extremes.

<u>Minimum Grades: Design Domain Application Heuristics</u>

Rural Roadways

1. On uncurbed roadways, level grades are generally acceptable provided the roadway is adequately crowned, snow does not interfere with surface drainage, and ditches have positive drainage. Roadway crown is discussed in Section 2.1.5. Refer to Chapter 2.2 for guidelines for the design of roadside open ditches and to relevant drainage publications.

Curbed Roadways (generally in urban areas)

1. To ensure adequate drainage, curbed roadways typically have a minimum longitudinal grade of 0.50% or 0.60%, depending on local policy.

2. In certain rare design cases, when no other alternative is feasible, a grade of 0.30% may be used as an absolute minimum preferably in combination with highly stable soils and rigid pavements.

3. For retrofit projects, longitudinal grades below the normal minimum of 0.50% or 0.60% may be considered where flatter grades allow the retention, rather than the removal, of existing pavements.

4. The minimum gradients outlined are suitable for normal conditions of rainfall and drainage outlet spacing. Where less than the normal minimum gradient is utilized, the lengths of such grades should be limited to short distances, and their location and frequency become important considerations. In special cases, hydraulic analysis is required to determine the extent of water spread on the adjacent travel lane. False grading, (where the pavement grade is not parallel to the top of curb), to ensure adequate drainage is an effective means of maintaining minimum grades in flat, highly constrained areas. False grading is addressed in Section 2.1.5, Cross-Slope.

5. At intersections, minimum gradients are particularly important to avoid operational safety concerns related to ponding or icing conditions.

6. Ensuring positive drainage is a key element in the grading designs of curb returns and the large paved surfaces of intersection areas. Suggested guidelines include a minimum gradient of 0.6% along curb returns, and a minimum of 1.0% combined crossfall and longitudinal gradient within the limits of an intersection. Further information on intersection drainage considerations is contained in Chapter 2.3, Intersections.

7. For gravelled roadways or public lanes, a longitudinal grade of 0.8% or more is desirable to ensure adequate surface drainage, unless parallel ditches are provided. A grade of 0.5% may be used as an absolute minimum.

Drainage around Curves

1. Avoid areas of level grades around curves if there is possibility of raised medians being installed in the future. The installation of such a median as part of a road rehabilitation or widening project may preclude the possibility of providing drainage in the median. Superelevated sections of pavement that drain towards a depressed median, and utilize longitudinal grade in the median, may require longitudinal grade on the pavement for drainage, if the median is subsequently raised.

2.1.3.3 Vertical Curves

Introduction

The function of a vertical curve is to provide a smooth transition between adjacent grades.

The form of curve used for vertical curves is a skewed parabola, positioned so that basic measurements can be made horizontally and vertically. Curves are described as crest or sag depending on their orientation.

One of the properties of the parabola is that the rate of change of grade with respect to length is constant. For this reason sight distance available to a driver travelling on a crest curve is constant throughout the length of the curve. This is one of the reasons for the use of the parabola for vertical curves. A second advantage of the parabolic curve is that its calculation is much simpler than a circular or other curve that might be considered.

Since the rate of change of grade is constant with respect to length, this property is used to designate the size of the curve. The length of a section of curve measured horizontally over which there is a change of grade of 1% is a constant for the curve and is referred to as the K value. For example, a K value of 90 means a horizontal distance of 90 m is required for every one percent gradient change. K is a measure of the flatness of a curve, the larger the K value the flatter the curve, in the same way that radius is a measure of the flatness of a circular curve. For crest curves K is negative, and for sag curves K is positive.

$$K = L / A \qquad (2.1.23)$$

Where:

L = Horizontal length of vertical curve (m)

A = Algebraic difference of grade lines (%)

K = Parameter which has a constant value in a given vertical curve.

In certain situations, because of critical clearance or other controls, the use of unsymmetrical vertical curves may be required. Because their use is infrequent, the derivation and use of the appropriate formulae have not been included in the following section. For use in such limited instances, refer to unsymmetrical curve data found in a number of highway engineering texts.

Table 2.1.3.4 K Factors to Provide Minimum Stopping Sight Distance on Sag Vertical Curves[1]

Design Speed (km/h)	Assumed Operating Speed (km/h)	Stopping Sight Distance (m)	Rate of Sag Vertical Curvature (K)			
			Headlight Control		Comfort Control	
			Calculated	Rounded	Calculated	Rounded
30	30	29.6	3.9	4	2.3	2
40	40	44.4	7.1	7	4.1	4
50	47-50	57.4-62.8	10.2-11.5	11-12	5.6-6.3	5-6
60	55-60	74.3-84.6	14.5-17.1	15-18	7.7-9.1	8-9
70	63-70	99.1-110.8	19.6-24.1	20-25	10.0-12.4	10-12
80	70-80	112.8-139.4	24.6-31.9	25-32	12.4-16.2	12-16
90	77-90	131.2-168.7	29.6-40.1	30-40	15.0-20.5	15-20
100	85-100	157.0-205.0	36.7-50.1	37-50	18.3-25.3	18-25
110	91-110	179.5-246.4	43.0-61.7	43-62	21.0-30.6	21-30
120	98-120	202.9-285.6	49.5-72.7	50-73	24.3-36.4	24-36
130	105-130	227.9-327.9	56.7-85.0	57-85	27.9-42.8	28-43

Values for sag curvature based on the comfort criterion are shown in Table 2.1.3.4.

These K values for sag curves are useful in urban situations such as underpasses where it is often necessary for property and access reasons to depart from original ground elevations for as short a distance as possible. Minimum values are normally exceeded where feasible, in consideration of possible power failures and other malfunctions to the street lighting systems. Designing sag vertical curves along curved roadways for decision sight distance is normally not feasible due to the inherent flat grades and resultant surface drainage problems.

2.1.3.4 Vertical Alignment: Design Domain Additional Application Heuristics

<u>Vertical Alignment Principles: Application Heuristics</u>

The following principles generally apply to both rural and urban roads. A differentiation between rural and urban is made in several instances where necessary for clarity.

1. On rural and high speed urban roads a smooth grade line with gradual changes, consistent with the class of road and the character of the terrain, is preferable to an alignment with numerous breaks and short lengths of grade. On lower speed curbed urban roadways drainage design often controls the grade design.

2. Vertical curves applied to small changes of gradient require K values significantly greater than the minimum as shown in Tables 2.1.3.2 and 2.1.3.4. The minimum length in metres should desirably not be less than the design speed in kilometres per hour. For example, if the design speed is 100 km/h, the vertical curve length is at least 100 m.

3. Vertical alignment, having a series of successive relatively sharp crest and sag curves creating a “roller coaster” or “hidden dip” type of profile is not recommended. Hidden dips can be a safety concern, particularly at night. Such profiles generally occur on relatively straight horizontal alignment where the roadway profile closely follows a rolling natural ground line. Such roadways are unpleasant aesthetically and more difficult to drive. This type of profile is avoided by the use of horizontal curves or by more gradual grades.

4. A broken back grade line (two vertical curves in the same direction separated by a short section of tangent grade) is not

desirable, particularly in sags where a full view of the profile is possible. This effect is very noticeable on divided roadways with open median sections.

5. Curves of different K values adjacent to each other (either in the same direction or opposite directions) with no tangent between them are acceptable provided the required sight distances are met.

6. An at-grade intersection occurring on a roadway with moderate to steep grades, should desirably have reduced gradient through the intersection, desirably less than 3%. Such a profile change is beneficial for vehicles making turns and stops, and serves to reduce potential hazards.

7. In sections with curbs the minimum longitudinal grade is 0.5%. Within superelevated transition areas, it might sometimes be virtually impossible to provide this minimum grade. In such cases, the longitudinal grade length below 0.5% should be kept as short as possible. Additional information on minimum grades and drainage is provided in Subsection 2.1.3.2.

8. A superelevation transition occurring on a vertical curve requires special attention in order to ensure that the required minimum curvature is maintained across the entire width of pavement. The lane edge profile on the opposite side of the roadway from the control line may have sharper curvature due to the change in superelevation rate required by the superelevation transition. It is, therefore, necessary to check both edge profiles and to adjust the desired minimum vertical curvature.

9. Undulating grade lines, with substantial lengths of down grade, require careful review of operations. Such profiles permit heavy trucks to operate at higher overall speeds than is possible when an upgrade is not preceded by a down grade. However, this could encourage excessive speed of trucks with attendant conflicts with other traffic.

10. On long grades it may be preferable to place steepest grade at the bottom and decrease the grades near the top of the ascent or to break the sustained grade by short intervals of flatter grade instead of a uniform sustained grade that might be only slightly below the allowable maximum. This is particularly applicable to low design speed roads and streets[2].

11. To ensure a smooth grade line on high speed routes a minimum spacing of 300 m between vertical points of intersection is desirable.

12. The design of vertical alignment should not be carried out in isolation but should have a proper relationship with the horizontal alignment. This is discussed in Section 2.1.4.

<u>Drainage: Application Heuristics</u>

1. Where uncurbed sections are used and drainage is effected by side ditches, there is no limiting minimum value for gradient or limiting upper value for vertical curves.

2. On curbed sections where storm water drains longitudinally in gutters and is collected by catch basins, vertical alignment is affected by drainage requirements. Minimum gradients are discussed in Subsection 2.1.3.2.

3. The profile of existing or planned stormwater piping is an important consideration in setting urban roadway grades. Storm sewer pipes typically have minimum depths to prevent freezing. These requirements are considered in setting catchbasin elevations.

4. Where the storm sewer system is not sufficiently deep to drain the streets by gravity flow, lift stations are an alternative. However, lift stations are generally considered undesirable due to the high costs associated with installation, operation and maintenance. Malfunctions at the lift station during a rain storm can also have a major detrimental impact on the street system and the adjacent developments.

Railways

1. Minimum vertical clearance over railways is 6.858 m (22.5 feet) measured from base of rail.

2. The minimum vertical clearance where ballast lifts are contemplated is 7.163 m (23.5 feet) measured from base of rail elevation to the underside of the overpass structure.

3. In all cases, it is good practice to confirm the specific clearance requirements with the pertinent railway company, as well as with the appropriate Federal and Provincial agencies, before designs are finalized.

Overhead Utilities

1. The vertical clearance requirements for roadways crossing beneath overhead utilities vary with the different agencies. In the case of overhead power lines, the clearance varies with the voltage of the conductors. The clearance requirements in each case should be confirmed with the controlling agency.

Pedestrian Overpasses

1. Normally, the minimum vertical clearance for a pedestrian overpass structure is set at 5.3 m or 0.3 m greater than the clearance of any existing vehicular overpass structure along that same route. This lessens the chances of it being struck by a high load - an important consideration - since a pedestrian overpass, being a relatively light structure, is generally unable to absorb severe impact and is more likely to collapse in such an event. The increased vertical clearance reduces the probability of damage to the structure and improves the level of safety for pedestrians using the structure.

Bikeways and Sidewalks

1. For bikeways, the minimum vertical clearance provided is 2.5 m.

2. It is desirable to allow up to 3.6 m of vertical clearance in order to provide an enhanced design and permit access for typical service vehicles.

3. Similar vertical clearances are normally provided for sidewalks since cyclists may occasionally use the sidewalk, even where not legally permitted.

4. If it can be clearly determined that cyclists will not use the pedestrian sidewalk, minimum clearances in accordance with the National or Provincial Building Codes could be employed.

5. Further information on sidewalks and bikeways is provided in Chapters 2.2, 3.3 and 3.4.

Waterways

1. Over non-navigable waterways, bridges and open footing culverts, the vertical clearance between the lowest point of the soffit and the design high water level shall be sufficient to prevent damage to the structure by the action of flow water, ice flows, ice jams or debris.

2. For navigable waterways, navigational clearance is dependent on the type of vessel using the waterway and should be determined individually.

3. Clearances should also conform to the requirements of the Navigable Waters Protection Act of Canada.

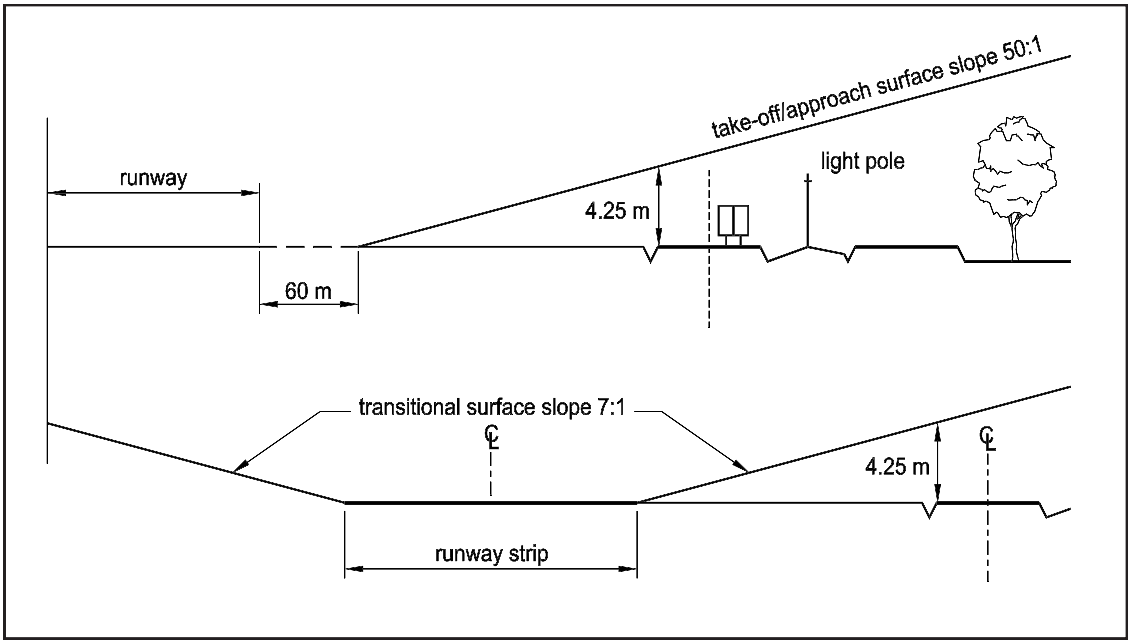
Airways

1. Vertical clearance to airways is as indicated in Figure 2.1.3.3.

2. Lighting poles should be contained within the clearance envelope.

3. The dimensions are for preliminary design. Specific dimensions should be approved by the designated Transport Canada representative.

Figure 2.1.3.3 Airway Clearance[1]



2.1.3.5 Explicit Evaluation of Safety

General

Vertical alignment design has a significant impact on safety in areas where vehicles are required to frequently stop and start. Excessive grades in intersection areas and at driveways can contribute significantly to collision frequencies during wet or icy conditions. Efforts are normally made to provide as flat a grade as practicable in these critical areas, while meeting the minimum slopes needed for adequate surface drainage.

Collision Frequency on Vertical Alignment

The following information on collision frequency is derived from a 1998 research paper[29].

The vertical profile of a road is likely to affect safety by various mechanisms. First, vehicles tend to slow down going up the grade and speed up going down the grade. Speed is known to affect collision severity. Thus on the up grade collisions tend to be less severe than on the down grade. Since down grade collisions tend to be more severe, a larger proportion of collisions tend to get reported. Thus, the severity and frequency of reporting collisions are affected by the grade. Second, road grades affect the diversity of speeds. This is thought by some to affect collision frequency. Third, road profile affects the available sight distance and gradient affects braking distance. All of these factors may affect collision frequency and severity. Finally, grade determines the rate at which water drains from the pavement surface and this too may affect safety. The traditional belief was that safety-related design attention should focus on crest curves and sag curves. It turns out that while the vertical profile is an important determinant of the future safety of a road, sight distance at crest or sag curves is not as important as it seemed.

At present the quantitative understanding of how grade affects safety is imprecise. All studies using data from divided roads concluded that collision frequency increases with gradient on down grades. Some studies concluded that the same is true for up grades, while others concluded to the contrary. Estimates of the joint effect of grade on both directions of travel vary. It is suggested that the conservative Collision Modification Factor of 1.08 be used for all roads. That is, if the gradient of a road section is

2.1.4 COORDINATION AND AESTHETICS

2.1.4.1 Introduction

The visual aspect of the road as viewed by the driver and passenger is considered to be an important feature of geometric design. The provision of visual comfort helps make driving a more relaxing experience resulting in better and safer traffic operation. Features which are aesthetically disturbing to the motorist are to be avoided. An unsightly road is a blight on the landscape whereas an aesthetically pleasing facility can become an asset, enhancing the area through which it passes.

To produce an aesthetically pleasing facility the designer requires an appreciation of the relationship between the road and its surroundings. Some specific principles to be considered include:

- blending of the road with the surrounding topography
- developing independent alignments for each roadway of a divided facility when right of way permits
- continuous curvilinear design rather than long-tangent, short-curve design
- integration of horizontal and vertical alignment
- implementation of designs with visually pleasing structures, retaining walls and landscaping

In many cases the above principles can be achieved at an acceptable extra cost. In cases where additional cost is a factor, the benefits are assessed against expenditure. In considering the costs and benefits of design trade-offs of this sort, the designer should ensure that safety-related factors are considered explicitly. In addition, many of the basic elements of design coordination contribute to the design consistency aspects of road design, and should thus be considered in that context as well. The issue of design consistency is discussed in Chapter 1.4 of this Guide.

Examples of good and poor application of the above principles are illustrated in Figures 2.1.4.1 to 2.1.4.12. Each photograph or perspective sketch has a brief comment describing the significant visual qualities.

2.1.4.2 Alignment Coordination: Technical Foundation

The principal guides for horizontal and vertical alignment are set out in Sections 2.1.2 and 2.1.3. A section of road might be designed to meet these guides, yet the end result could be a facility exhibiting numerous unsatisfactory or displeasing characteristics. Horizontal and vertical alignments are permanent design elements for which thorough study is warranted. It is extremely difficult and costly to correct alignment deficiencies after the road is constructed. On freeways there are numerous controls such as multilevel structures and costly right of way. On most arterial streets heavy development takes place along the property lines, which makes it impractical to change the alignment in the future. Thus, compromises in alignment designs must be weighed carefully. Any initial savings may be more than offset by the economic loss to the public in the form of collisions and delays.

It is difficult to discuss the combination of horizontal alignment and profile without reference to the broader subject of location. The subjects are mutually interrelated and what may be said about one generally is applicable to the other. It is assumed here that the general location has been fixed and that the problem remaining is the specific design and harmonizing of the vertical and horizontal lines, such that the finished road or street will be an economical, pleasant, and collision-free facility on which to travel. The physical controls or influences that act singularly or in combination to determine the type of alignment are the character of road justified by the traffic, topography, and subsurface conditions, existing cultural development, likely future developments, and location of the terminals. An initial design speed is established when determining the general location, but as design proceeds to more detailed alignment and profile it assumes greater importance, and the speed chosen for design acts to keep all elements of design in balance. The final design speed chosen may be different from the initial design speed. Design speed determines limiting values for many elements such as curvature and sight distance and influences many other elements such as width, clearance, and maximum gradient; all are discussed in the preceding parts of this chapter.

Horizontal and vertical alignments should not be designed independently. They complement each other, and poorly designed combinations can spoil the good points and aggravate the deficiencies of each. Horizontal and vertical alignments are among the more important of the permanent design elements of the road. Excellence in their design and in the design of their combination increase usefulness and safety, encourage uniform speed, and improve appearance.

During the location stage and the design phase of a facility, the finished roadway is viewed in three dimensions and the consequences of various combinations of horizontal and vertical alignment on the utility, safety and appearance of the completed project are considered.

2.1.4.3 Alignment Coordination: Design Domain Application Heuristics

A number of application heuristics which can assist the designer in preparing well-coordinated and aesthetic plans are offered below.

1. Curvature and grades should be in proper balance. Tangent alignment or flat curvature at the expense of steep or long grades and excessive curvature with flat grades are both poor design. A logical design that offers the most in safety, capacity, ease and uniformity of operation, and pleasing appearance within the practical limits of terrain and area traversed is a compromise between the two extremes.

2. Vertical curvature superimposed on horizontal curvature, or vice versa, generally results in a more pleasing facility, but it should be analyzed for effect on traffic. Successive changes in a profile not in combination with horizontal curvature may result in a series of humps visible to the driver for some distance, an undesirable condition as previously discussed. The use of horizontal and vertical alignments in combination, however, may also result in certain undesirable arrangements, as discussed later in this section.

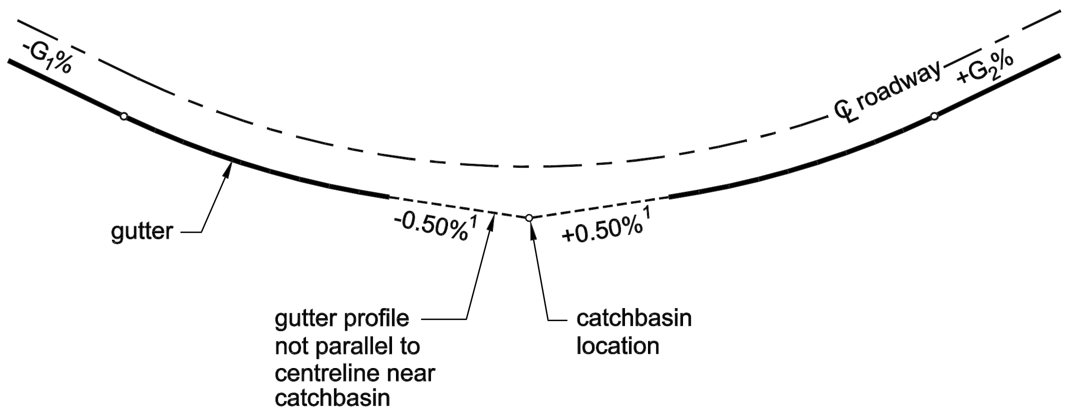
3. Sharp horizontal curvature should not be introduced at or near the top of a pronounced crest vertical curve. This condition is undesirable in that the driver cannot perceive the horizontal change in alignment, especially at night when the headlight beams go straight ahead into space. The difficulty of this arrangement is avoided if the horizontal curvature leads the vertical curvature, i.e., the horizontal curve is made longer than the vertical curve. Suitable design can also be made by using design values well above the minimums for the design speed.

4. Somewhat allied to the above, sharp horizontal curvature should not be introduced at or near the low point of a pronounced sag vertical curve. Because the road ahead is foreshortened, any significant horizontal curvature assumes an undesirable distorted appearance. Further, vehicular speeds, particularly of trucks, often are high at the bottom of grades, and erratic operation may result, especially at night.

5. On two-lane roads and streets the need for safe passing sections at frequent intervals and for an appreciable percentage of the length of the road often supersedes the general desirability for combination of horizontal and vertical alignment. In these cases it is necessary to work toward long tangent sections to secure sufficient passing sight distance in design.

6. Horizontal and vertical alignments should be made as flat as feasible at intersections where sight distance along both roads or streets is important and vehicles may have to slow or stop.

Figure 2.1.5.1 False Grading and Cross-Slopes



Note: Minimum longitudinal gutter grades of 0.30% are acceptable.

2.1.5.4 Cross-Slope Arrangements: Application Heuristics

The direction of the cross-slopes, or the cross-slope arrangements for various classes of roads, are described below and illustrated in Figure 2.1.5.2. The rate of cross-slope in each case depends on the type of surface as noted earlier, as well as, if relevant, on the width of roadways and drainage considerations.

On tangent sections of roadway, cross-slope is normally applied to drain storm water to the side of the roadway. On two-lane roads the pavement is normally crowned at the centreline and the pavement slopes down to each edge.

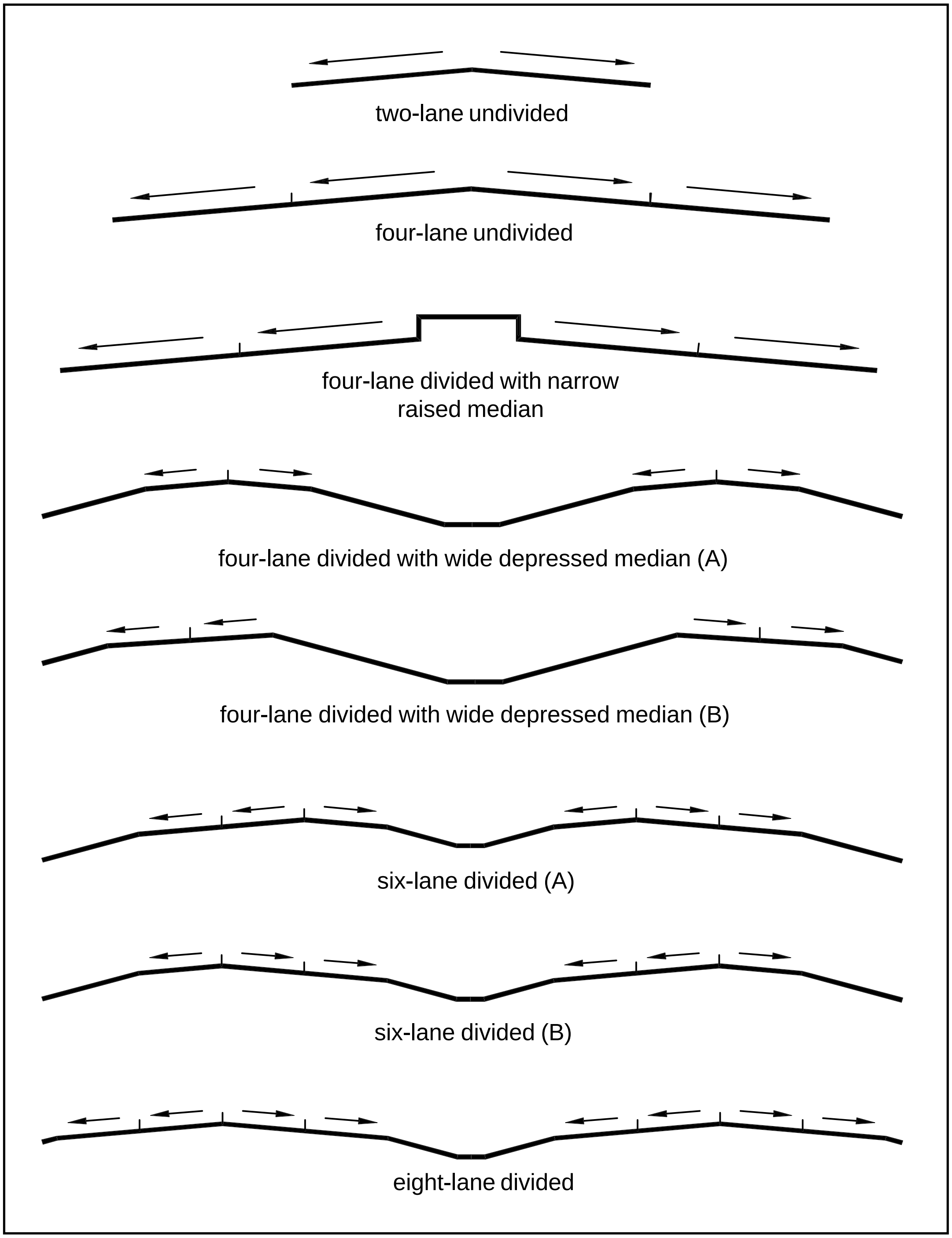
On four-lane undivided roads and four-lane divided roads with a flush median, the crown is normally placed in the centre of the pavement or median, and cross-slope to each pavement edge is 0.02 m/m.

On a four-lane divided road with a depressed median, a crown may be placed at the centre of each roadway with a cross-slope of 0.02 m/m to each edge, or both lanes may drain away from the median. These alternates are illustrated in Figure 2.1.5.2 (four lane divided, alternates A and B). The advantages of the crown are storm water drains to both sides of the roadway and it facilitates the treatment of the roadway with de-icing chemicals which are spread in a narrow strip about the crown line, allowing the action of traffic and cross-slope to further spread the chemicals across the entire pavement. If the road eventually requires expansion to six lanes by adding two lanes in the median, the additional lanes will slope toward the median. The advantages of both lanes draining away from the median is the reduction in median drainage provision.

If a four-lane divided road is to be expanded to six lanes within a short period of time of initial construction, it is normally designed for six lanes and built without the median lanes initially. In this case both lanes of each roadway slope toward the outer edge.

For six-lane divided roads, the crown for each roadway is applied to either edge of the centre lane, in which case one or two lanes drain toward the median. With two lanes draining

Figure 2.1.5.2 Application of Cross-slope on Various Types of Roads[7]



toward the median, at locations where an auxiliary lane is added, two lanes are draining in each direction. This location of the crown is also convenient for an initial stage of four-lane divided. If the six-lane cross section is to be a stage of an eight-lane cross section, a crown located at the common edge of the median and centre lane is preferred to avoid three lanes draining toward the median. The above is illustrated in Figure 2.1.5.2 (six-lane divided, alternates A and B).

Cross-slope on auxiliary lanes is the same as that of the adjacent through lane.

At intersections where two roads on tangent intersect, normal cross-slope is maintained on the major road, and cross-slope on the minor road is run out on the approaches to the intersection to match the profile of the major road. This treatment is typical of intersections controlled by a stop sign on the minor road. In the case of an intersection where the two roads are of equal importance, or where the intersection is signalized, the normal cross-slope is run out on all four approaches so that the cross-slope on each road matches the profile of the crossing road. Simply put, the pavements are warped to maintain smooth profiles for traffic on both roads. This topic is dealt with in more detail in Chapter 2.3.

For roadways on structures, the cross-slope is a minimum of 0.02 m/m.

For resurfacing, design guidelines and acceptable limits are provided in Table 2.1.5.1 for pavement cross-slope related to design speed.

The cross-slope for the other types of lanes addressed later in the chapter, including climbing lanes, passing lanes and service roads generally adhere to the following criteria.

For truck climbing lanes the cross-slope is usually handled the same as the addition of a lane to a multilane road. Two common practices are:

1. The continuation of the cross-slope of the through lanes.

2. A small increase in cross-slope compared to the through lanes. On superelevated sections the cross-slope is usually a continuation of the through lanes unless truck speeds are extremely slow and icy conditions prevail in which case an adjustment in the truck climbing lane cross-slope may be desirable.

For passing lanes the practice is similar to that for truck climbing lanes except that slow speeds are not an issue.

The cross-slope for service roads, express collector systems, and weaving lanes basically follow the same principles as would be applied to a similar class of through lane.

Table 2.1.5.1 Pavement Cross-Slope for Resurfacing[1]

	Cross-Slope (m/m)		
Design Speed	**Design Guidelines**	**Acceptable Limits**	**Maximum Algebraic Difference (driving lanes) (m/m)**
60 km/h	0.02	0.015-0.04	0.08
80 km/h	0.02	0.015-0.035	0.07
100 km/h	0.02	0.015-0.03	0.06
120 km/h	0.02	0.015-0.025	0.05

2.1.5.5 Cross-Slope Changes: Application Heuristics

Sometimes it is necessary to change the cross-slope on tangents for reasons other than superelevation. As in superelevation it is important that the change is gradual enough to provide visual driving comfort. This is achieved by having an acceptable relative slope which is a slope or profile of the outer edge of pavement in relation to the profile of the centreline. It is dependent on the rate of cross-slope being developed, the length over which it is developed, and the width of the pavement. It is therefore an expression of rate of change of cross-slope. The maximum relative slope normally applied varies with design speed. Table 2.1.5.2 provides values for maximum relative slope for two-lane roadways. For four-lane and six-lane roadways, the lengths are increased by 1.5 and 2.0 times that for two-lane roadways, respectively.

Design values for rates of change of cross-slope are shown on Table 2.1.5.3. These values are suitable for single-lane ramps. For two-lane ramps lower values are appropriate. Theoretically, the maximum rate of change for two-lane roadways should be 50% of that for single-lane roadways, however, this may generate transition lengths which cannot be achieved at an acceptable cost and values of 75% are acceptable.

The minimum transition length is given by the equation:

$$l = \frac{100we}{2s} \tag{2.1.29}$$

Where: w = the width of pavement (m)

e = the change in super-elevation developed (m/m)

s = the relative slope (%)

Table 2.1.5.3 Design Values for Rate of Change of Cross-Slope for Single-Lane Turning Roads[1]

Design Speed (km/h)	Rate of Change of Cross-Slope (m/m/m length)
25 and 30	0.0025
40	0.0023
50	0.0020
55 and more	0.0016

The phenomenon of adjacent traffic lanes having different rates of cross-slope or superelevation gives rise to a ridge at the common edge, referred to as algebraic difference or roll-over.

Too great a difference in cross-slope may cause vehicles travelling between lanes to sway, giving

Table 2.1.5.2 Maximum Relative Slope Between Outer Edge of Pavement and Centreline of Two-Lane Roadway[1]

Design Speed (km/h)	Relative Slope (%)
40	0.70
50	0.65
60	0.60
70	0.55
80	0.51
90	0.47
100	0.44
110	0.41
120	0.38
130	0.36

2.1.7 LANE AND ROUTE CONTINUITY AND WEAVING

2.1.7.1 Lane Continuity

Technical Foundation

In designing the lane arrangement of a freeway, design volume, maintenance of basic lanes and lane balance are all taken into account. A further consideration is that of lane continuity.

A driver needs to recognize which lanes are basic or through, to avoid being inadvertently led by the lane markings to an undesired ramp lane. If good lane balance is applied and basic lanes are maintained, together with all exits and entrances having a single lane on the right, lane continuity will naturally follow. Where entering and exiting ramps have two or more lanes, and at transfer lanes on collector roadways on express-collector systems, lane continuity can be lost.

Best Practices

Figure 2.1.7.1 illustrates examples of lane continuity. Included is an example of non-compliance of lane continuity followed by an illustration of a rearrangement of lanes and ramps in order to achieve lane continuity. In illustration (i) three basic lanes are maintained, all ramps are single lane on the right and the principles of lane balance are observed. Lane continuity is maintained. In illustration (ii) there are three basic lanes, all ramps are on the right and have two lanes, lane balance is preserved and lane continuity is maintained.

In illustration (iii) there are two single-lane entrances on the right, a two-lane exit on the right, a two-lane (transfer) entrance on the left and a two-lane (transfer) exit on the left. Although three basic lanes are maintained through the section and the principles of lane balance are observed, only one of the basic lanes is continuous. This confuses the driver, causes turbulence and unnecessary lane changing in the traffic operation, and is potentially hazardous.

Illustration (iv) shows how the deficiencies of illustration (iii) can be resolved. The number of lanes on each ramp and transfer roadway are the same, proper lane balance is observed and all three basic lanes are continuous. This is accomplished by use of auxiliary lanes on both the left and right-hand sides of the roadway.

2.1.7.2 Route Continuity[1]

Technical Foundation

Route continuity refers to the provision of a directional path along and throughout the length of a designated route. Continuity of route designation, either by name or number, is important to ensure operational uniformity and to reassure the driver that he is on the intended course. The principle of route continuity simplifies the driving task in that it reduces lane changes, simplifies signing, delineates the through route, and reduces the driver’s search for directional signing.

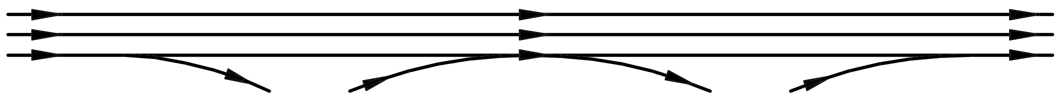
Desirably, the through driver, especially the unfamiliar driver, should be provided a continuous through route on which it is not necessary to change lanes and through traffic vehicular operation occurs on the left of all other traffic. In maintaining route continuity through cities and bypasses, interchange configurations need not always favour the heavy movement but rather the through route. To accomplish this, heavy movements can be designed on flat curves with reasonably direct connections and auxiliary lanes, equivalent operationally to through movements.

Best Practices

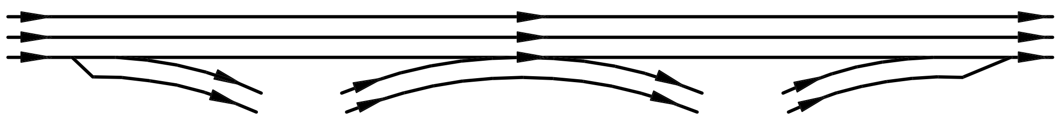
Adherence to the above route continuity principles influences the configuration of interchanges. In Figure 2.1.7.2, two continuity arrangements are illustrated.

In illustration (i) Highway 1 is a north-south route and Highway 2 is an east-west route, in which case a conventional four level fully-directional interchange is appropriate and the designated through routes are consistent with the route numbers. In illustration (ii), Highway A is an east-south route and Highway B is a north-west

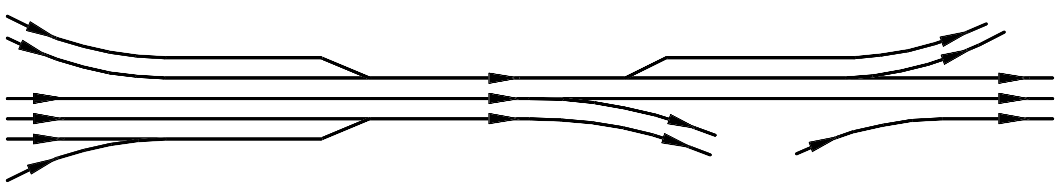
Figure 2.1.7.1 Examples of Lane Continuity[1]



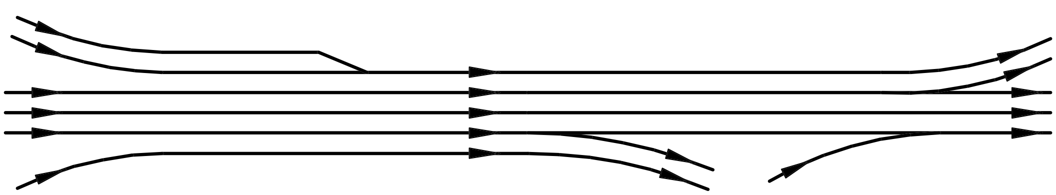
i) three basic lanes, single lane ramps on the right
proper lane balance, lane continuity maintained



ii) three basic lanes, two lane ramps on the right
proper lane balance, lane continuity maintained

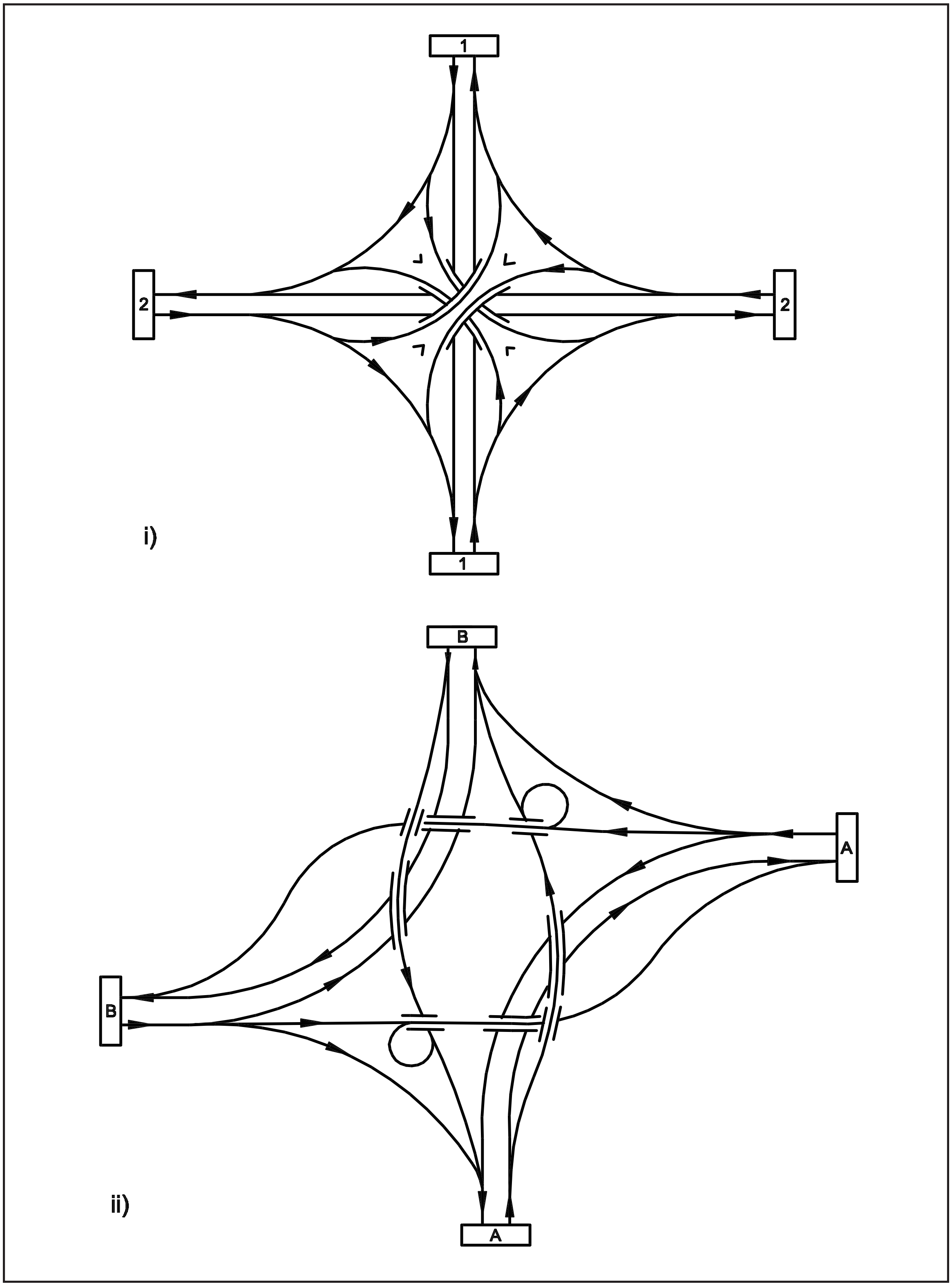


iii) three basic lanes, proper lane balance but only
one through lane is continuous, lane continuity lost.



iv) three basic lanes, proper lane balance, basic
(through) lanes are continuous, lane continuity is restored

Figure 2.1.7.2 Illustration of Route Continuity[1]



route. The through routes, and the route names and numbers, are maintained and the ramps carry traffic between route numbers.

If the conventional configuration of illustration (i) were applied to the Highway A/B interchange, the through route numbers would be carried on ramps. This would confuse a driver who expects to exit on a ramp (on the right) only when departing from the through route number of another route. The designated through route name or number, therefore, influences the selection of the configuration of the interchange.

2.1.7.3 Weaving

Technical Foundation

Weaving sections are roadway segments where the pattern of traffic entering and leaving at contiguous points of access result in vehicle paths crossing each other. Weaving sections may occur within an interchange, between entrance ramps followed by exit ramps of successive interchanges, and on segments of overlapping roadways. The weave section operations are an important consideration in the location of ramp terminals.

If the frequency of lane changes in a weaving section is similar to that on an open road, the section is said to be “out of the realm of weaving”, but where they exceed the normal frequency the condition is described as “weaving”.

There are three primary types of weaving sections which are determined by the operational features such as number of entry lanes, number of exit lanes, and their impact on how lane changing must take place. The three types of weaving sections are: Type A, Type B, and Type C. These three types of weaving sections are illustrated in Figure 2.1.7.3.

Type A weaving section requires that each weaving vehicle make one lane change in order to execute the desirable movement. Type A weaving section is also broken into two distinct weaving types. The first is a one sided weave, and the second is called a major weave with crown line.

Type B weave section is classified as a major weaving section because it involves multilane entry and/or exit lanes. Two critical characteristics that distinguish Type B weaving areas are: 1) one weaving movement may be accomplished without making any lane changes, and 2) the other weaving movement requires at most one lane change. Type B weaving sections are extremely efficient in carrying large weaving volumes, primarily because of provisions of a through lane for one of the weaving movements.

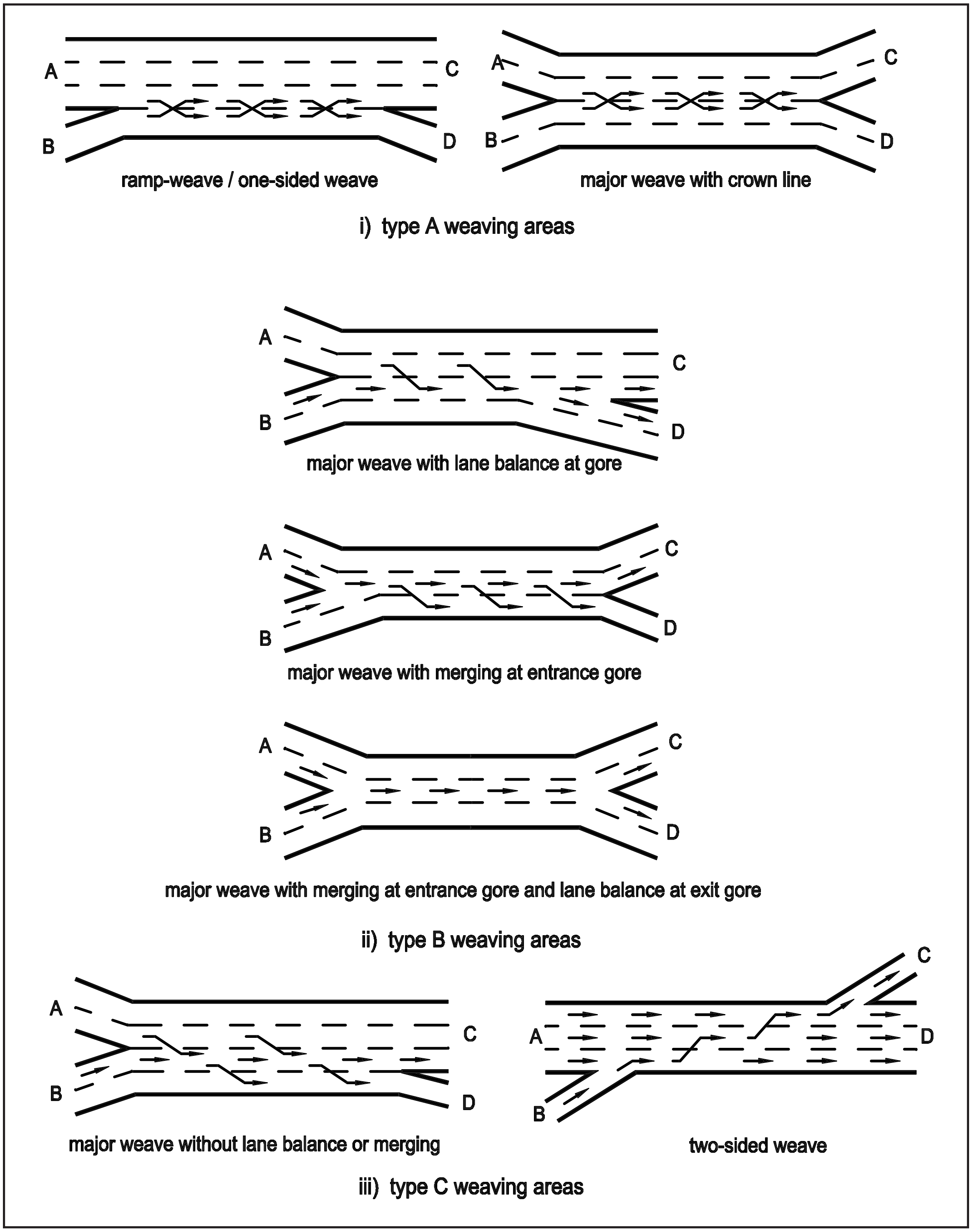
Type C weaving sections are similar to Type B weaving sections in that one or more through lanes are provided for one of the weaving movements. The distinguishing differences between Type B and Type C weave sections is the number of lane changes required for the other weaving movement.

Best Practices

The conflict between entering and exiting traffic tends to interrupt the operation of normal through traffic, precipitates turbulence in traffic flow and has the effect of reducing service volumes and capacity. Undesirable weaving conditions may be alleviated by increasing the number of lanes in the weaving section or increasing the length between successive entrance and exit gores. Weaving sections may be eliminated from the main facility by the selection of interchange forms that do not have weaving, or by incorporation of collector distributor roadways. Although interchanges that do not involve weaving operate better than those that do, interchanges with weaving areas nearly always are less costly than those without.

If the above measures are not effective or not feasible for resolving weaving concerns, it may be necessary to eliminate certain turning movements or relocate them elsewhere, in the interest of maintaining the operational integrity of the freeway. Alternatively, a weaving condition may be eliminated by separating the conflicting traffic movements vertically by introducing a

Figure 2.1.7.3 Types of Weaving Sections[8]



grade separation of basket weave configuration. These solutions are illustrated in Figure 2.1.7.4.

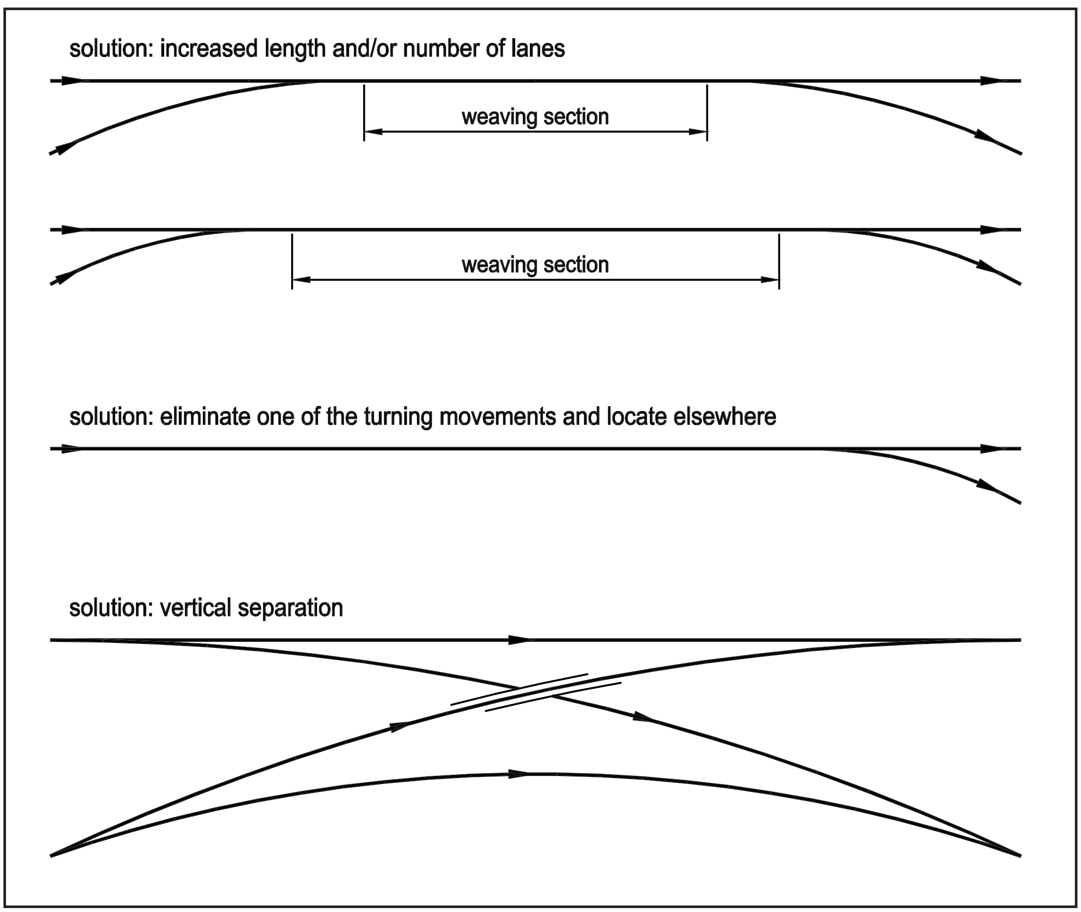
The minimum length, number of lanes, and capacity of the weaving sections are determined using procedures in the Highway Capacity Manual Special Report 209, 3rd Edition, Transportation Research Board, Washington, D.C. (Revised) 1997.

For efficient operation on freeways, weaving length between a freeway interchange and an arterial interchange normally should be in the range of 800 m to 1000 m and between arterial interchanges in the range of 550 m and 700 m. It is recognized that in many cases shorter weaving lengths may be imposed by other constraints such as the location (spacing) of existing arterial roads. Such shorter weaving lengths operate with varying levels of quality and safety depending on local conditions and features such as traffic volumes, sight distance, visibility, horizontal and vertical alignment, and cross section elements.

Weaving sections longer than 1000 m will frequently be out of the realm of weaving.

Weaving length is measured from the point where lane edges at the merge are 0.5 m apart to where lane edges at the diverge are 3.7 m, illustrated in Figure 2.1.7.5.

Figure 2.1.7.4 Solutions for Undesirable Weaving[1]



In spite of these factors, the following general conclusions can provide design assistance:

1. A number of studies indicate that increased shoulder width is more beneficial to safety at higher traffic volumes than lower volumes.

2. There are some indications in the research that roads with wider shoulders may tend to have collisions of greater severity. This phenomenon may be due to the higher running speeds that such wider shoulders may encourage in some instances.

3. Shoulders wider than about 2.0 m to 2.5 m increase the number of injury collisions in some circumstances. Once again, this phenomenon may be due to the higher running speeds that wider shoulders may encourage. It is thus logical that, in situations where wider shoulders are to be used, particular attention should be paid to providing an appropriate forgiving roadside design.

4. The safety effect of wide shoulders on level and straight roads is less than on sharp horizontal curves and on roads with steep grades.

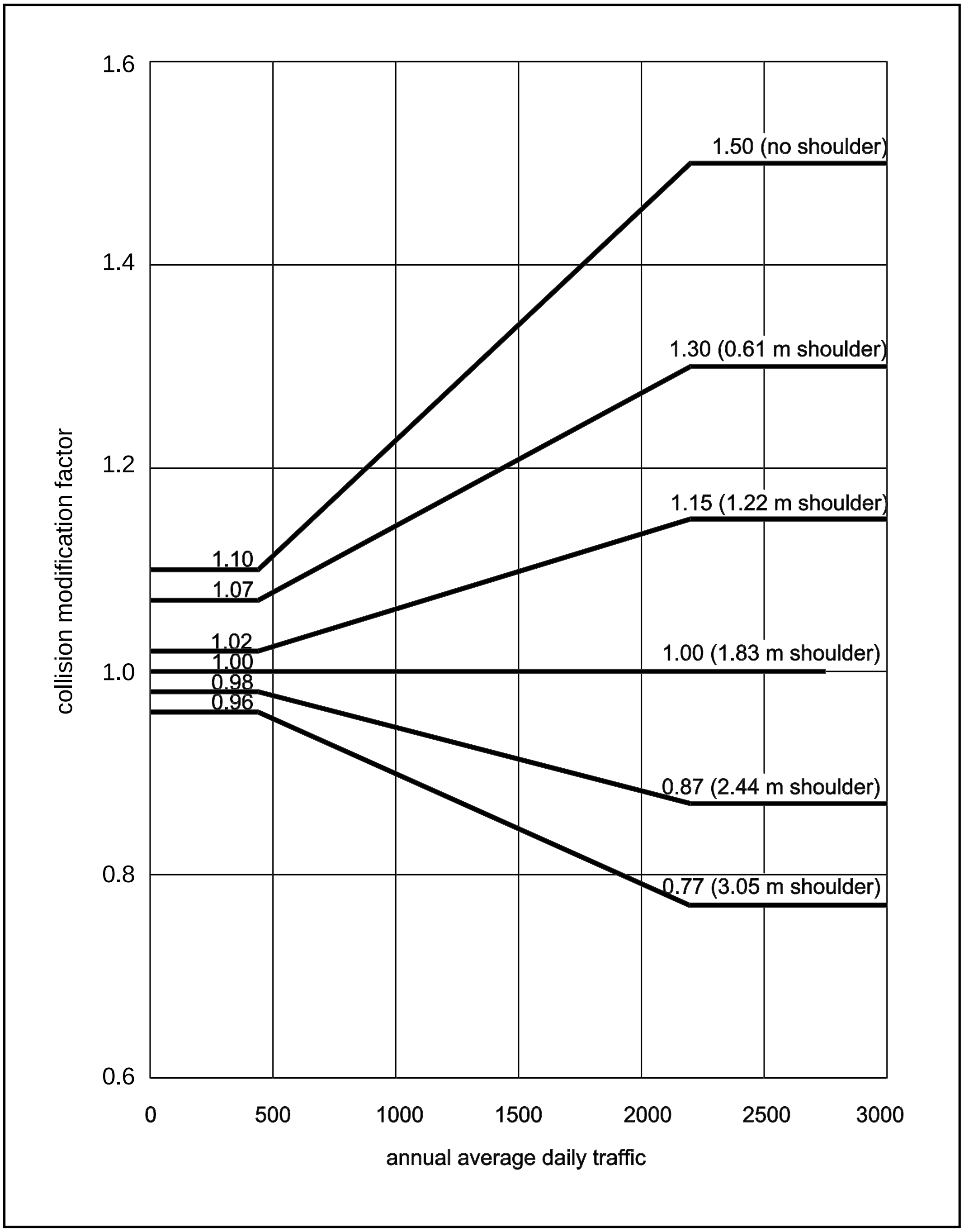
5. Wider shoulders tend to have fewer run-off-road and opposite direction collisions. However, they may in some instances be associated with greater levels of 'other' types of collisions.

6. Shoulders fulfil an important function as a refuge area for broken-down vehicles. Making adequate provision for the refuge of such vehicles is particularly important on high-volume facilities such as rural and urban freeways. In consideration of this fact, and of the consequent need to allow drivers and passengers room to move around the vehicle for maintenance and/or repair purposes, the design domain for shoulder width allows for widths up to 3.0 m. In instances where the lower limits of the design domain are used, particular attention should be paid to providing a forgiving roadside.

7. The Collision Modification Factor (CMF) related to shoulder width and annual average daily traffic (AADT) is shown on Figure 2.2.4.6. This Figure is based on run-off-road and opposite direction collisions. If the information shown in Figure 2.2.4.6 is to be applied to total collisions, an appropriate correction needs to be applied. For example, if half of the collisions on a given road are of the run-off-road and opposite direction type, the CMF needs to be applied only to half of the total number of collisions. If the "other" collision types are affected by shoulder width, the CMF needs to be reduced further.

8. Provision of full shoulders instead of only curb and gutter on multilane suburban highways is associated with a 10% lower collision rate.[18]

Figure 2.2.4.6 Collision Modification Factor for Various Shoulder Widths versus Annual Average Daily Traffic [10]



2.2.5 MEDIANS AND OUTER SEPARATIONS

2.2.5.1 Technical Foundation

A median may be defined as that portion of a road which physically separates the travel lanes of traffic in opposing directions. Median width is the lateral dimension measured between the inner (left) edges of the travel lanes and includes the left shoulder, the gutter or offset widths, as shown in Figures 2.2.1.1 and 2.2.1.2.

A median is a safety device which provides some measure of freedom from interference of opposing traffic. Medians provide a recovery area for errant vehicles, storage area for emergencies, speed-change lanes for left-turn and U-turn traffic, and reduce headlight glare. Medians add to a sense of open space and freedom, particularly in urban areas.

Medians should be visible day and night and should be in definite contrast to adjacent travel lanes. Medians may be flush with, raised above, or depressed below adjacent travel lanes.

Median widths may be as narrow as 1.0 m and as wide as 30 m. Widths above 3.0 m are usually associated with independent alignments, in which case the roadways are designed separately, and the area between is largely left in its natural state. Median widths for rural typical sections are normally multiples of 1.0 m and for urban typical sections multiples of 0.2 m.

Medians may serve as escape routes and provide a clear zone for vehicles that are avoiding possible collisions with vehicles in their own lanes.[11] The major uses of a median separation are to eliminate the risk of head-on collisions and to control access. Increasing median width reduces the frequency of cross-median collisions. Collision frequencies generally decrease with increasing median widths; however, current research is insufficient to allow quantification of the collision rates versus median width.[11] For that reason, medians should be as wide as economically possible. In any case, the median width should be in balance with the other elements of the cross section and the character of the area.

An outer separation is that portion of an arterial street, road, expressway or freeway which physically separates the outside travel lanes of a roadway from an adjacent frontage/service road or collector road. The width of an outer separation is measured from the outer (right) edge of the travel lanes to the closest edge of the parallel frontage/service road or collector road, and includes the shoulder and gutter or offset widths, as shown in Figure 2.2.1.1.

A discussion regarding median barriers is presented in Chapter 3.1.

2.2.5.2 Freeway and Expressway Medians: Application Heuristics

1. Rural freeways usually have depressed medians of sufficient width to allow the road bed to drain into the median and to eliminate the need for median barriers.

2. Median side slopes are kept flat so that a vehicle leaving the travelled lanes has an opportunity to recover control minimizing occupant injury and vehicle damage. Overturning crashes are more frequent for deeply depressed medians with slopes of 4:1 or steeper for median widths of 6.0 to 12.0 m.[12] Slopes steeper than 4:1 should be avoided and flatter slopes are desirable where feasible in terms of cost, drainage and property. See Chapter 3.1 for additional discussion of slopes.

3. Wide medians promote safety by reducing the possibility of collision by vehicles travelling in opposite directions; and they promote a sense of well-being for the travelling public. Rural freeway medians in the order of 20 m are common and may be as much as 30 m. Wider medians constitute separate alignment and independent design.

4. In metropolitan fringe areas it may be appropriate to build a rural freeway with a

depressed median, recognizing that the character of the area will become urban and that future lanes will be required together with a flush or raised median. The ultimate cross section is designed and then elements removed to determine the depressed median width for the first stage.

5. In developing the ultimate cross section, the designer should be aware that the installation of median barriers increases the collision rate but reduces crash severity because of the reduction or elimination of head-on collisions.[12]

6. Consideration should be given to providing sufficient median width to preclude median barriers in the ultimate stage when the future lanes are added.

7. Medians for urban freeways normally are either flush or raised with a median barrier. Median dimensions depend on shoulder widths, barrier type, and the need for provision of structure piers. See Chapter 3.1 for a discussion of median barriers.

8. The normal width of a left shoulder for an urban freeway ranges between 1.5 m and 2.5 m. Therefore the median width should be at least 3.0 m plus the width of the selected barrier plus allowances for such factors as barrier deflection on impact if median barrier systems are used, illumination poles, overhead sign footings and bridge piers. Further discussion on barriers is contained in Chapter 3.1.

9. Typical freeway medians are shown in Figure 2.2.5.1.

2.2.5.3 Arterial Road Medians: Application Heuristics

1. A flush median without barrier may be appropriate for rural highways with low to medium volumes and operating speeds. This median is normally slightly crowned to effect drainage, and is normally paved, often in the same surface material as the adjacent lanes. It is advantageous, however, to surface the median in a contrasting texture and/or colour to alert the errant driver travelling in the median. Widths of flush highway medians without median barriers can vary between 1.0 m and 4.0 m.

2. Wider flush medians with barriers normally apply to high speed rural arterial roads.

3. Medians in urban areas may be either flush or raised.

4. A median in an urban area normally does not have a barrier because it has to be terminated at intersections and some entrances, in which case the safety benefits of the median are offset by the hazard of the barrier ends. Where a barrier is applied, it is usually a concrete barrier in a flush median. Shoulders are not normally justified in urban areas, in which case the concrete barrier is offset 0.5 m from the edge of travelled lane. Additional median width might be required to accommodate illumination plant, bridge piers or traffic control devices.

5. Where provision for a left-turn lane is required, the median may require widening to provide the appropriate lane width, rounded up to a multiple of 0.2 m.

6. Additional width may be required for bridge piers regardless of whether barrier protection is provided. Figure 2.2.5.2 gives suitable dimensions for raised median treatment at bridge piers for arterial roads.

7. Additional median width may be required if heavy volumes of multi-trailer trucks are anticipated at unsignalized intersections.

8. Medians on a divided urban street serve a variety of important purposes related to safety, traffic operations, access control, and aesthetics including:

   - physical separation of opposing traffic flows
   - storage area for left-turning vehicles out of the path of the through traffic stream

2.2.7 CURBS AND GUTTERS

2.2.7.1 Technical Foundation

Curbs are raised or vertical elements, located adjacent to a travelled lane, parking lane or shoulder. They may be employed with all types of urban streets for any or all of the following reasons:

- drainage control
- delineation of the pavement edge or pedestrian walkways to improve safety
- right-of-way reduction with the elimination of open ditch drainage
- reduction in maintenance operations
- access control or provision
- aesthetics

Curbs are used only to a limited extent on rural roadways where drainage is usually controlled by means of drainage channels. Concrete gutters are typically used to facilitate longitudinal drainage along urban roadways. They are often cast integrally with curbs, but may also have a "vee" shape when used adjacent to a concrete traffic barrier.

2.2.7.2 Curbs: Best Practices

There are three general types of curb: barrier, semi-mountable and mountable (see Figure 2.2.7.1). Each type may be designed as a separate unit or integrated with a gutter to form a combination curb and gutter section.

1. Barrier type curb is vertical or near vertical, with a typical height of 150 mm, and is intended primarily to control drainage and access, as well as to inhibit low speed vehicles from leaving the roadway.

2. When struck at high speeds, barrier curbs can result in loss of vehicle control and - in spite of its name - is inadequate to prevent a vehicle from leaving the roadway.

3. In addition, the barrier type curb can contribute to a high speed errant vehicle vaulting over a semi-rigid traffic barrier under certain conditions. For this reason, barrier curb is generally not used on urban freeways and is considered undesirable on expressways and arterials with design speeds in excess of 70 km/h. Barrier curb is never used in combination with rigid concrete barrier systems.

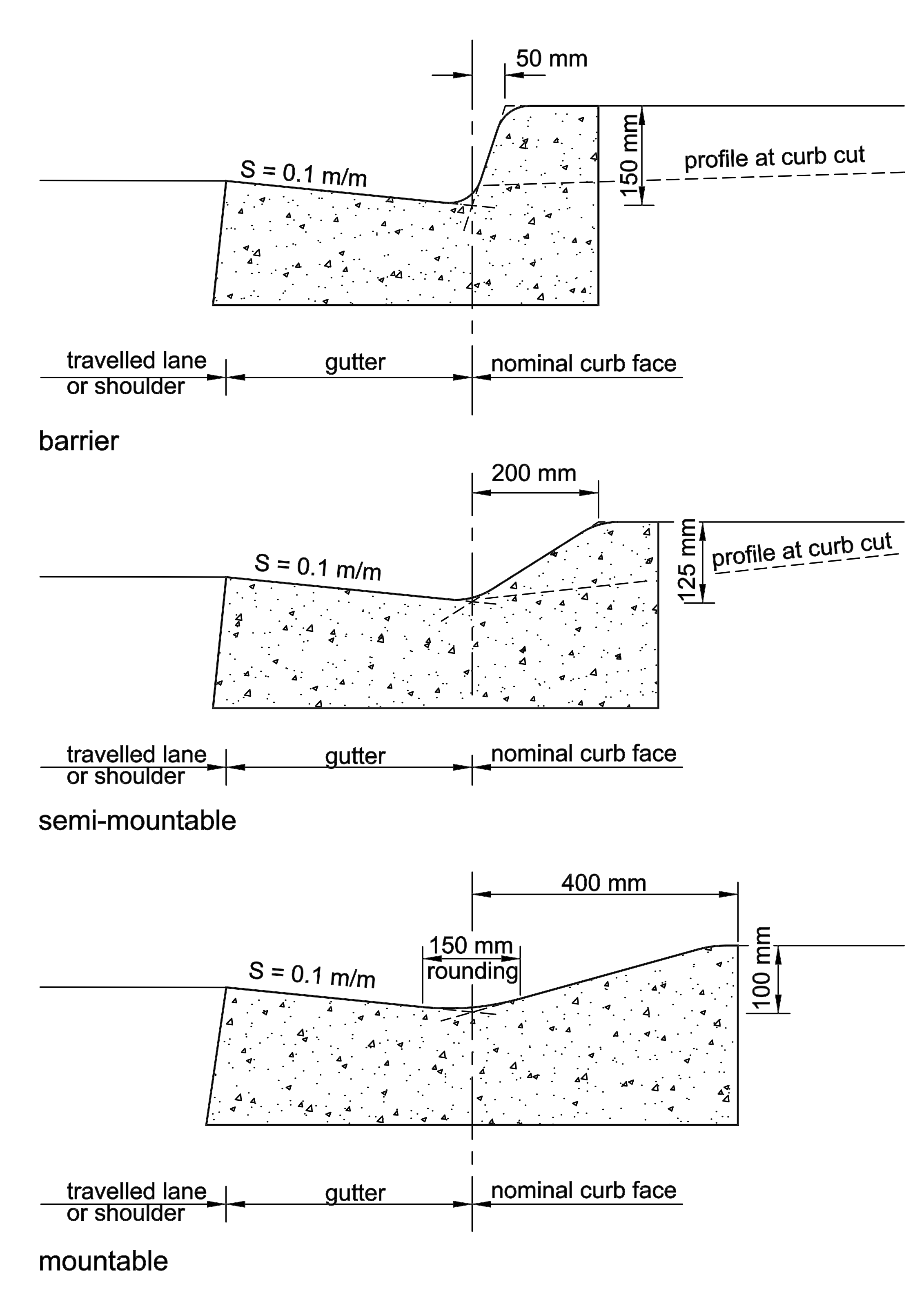
4. Semi-mountable curb is considered to be mountable under emergency conditions. Its face slope ranges from 0.250 m/m to 0.625 m/m with a maximum vertical height of 125 mm. Semi-mountable curbs are used on urban freeways, expressways, and on high speed arterials (design speed over 70 km/h) as a trade-off between drainage requirements and, when required, the functional needs of semi-rigid traffic barrier systems.

5. Mountable curb contains a relatively flat sloping face (0.10 m/m to 0.25 m/m) to permit vehicles to cross over it easily. While mountable curb may be used in conjunction with either semi-rigid or rigid traffic barrier systems, it is preferable not to use a curb in combination with either of these traffic barrier systems.

6. The cross section dimensions of concrete curbs and curbs with gutters vary between municipal jurisdictions. Standardization of curb and gutter dimensions within a given jurisdiction is desirable for economy and uniformity in construction and maintenance practice.

7. When introducing a curb at the transition between typical rural and urban road cross sections, the curb on the urban section is normally flared out to match the edge of shoulder on the rural section. Flare rates of 24:1 for a design speed of 80 km/h and 15:1 for 50 km/h are considered appropriate. The end of the curb is normally tapered down to be flush with the shoulder surface to prevent blunt impacts between the curb and vehicle tires or snow clearing equipment.

Figure 2.2.7.1 Curb and Gutter Types



preferably by using a quantitative benefit cost analysis technique and documented by the road designer in the process of reaching such a decision.

3. Guidelines for minimum horizontal clearances on bridges on urban local and collector roads are provided in Table 2.2.10.1.

4. Guidelines for minimum horizontal clearance at bridges on rural roads are shown in Table 2.2.10.2.

5. Guidelines for minimum horizontal clearances at bridges on urban arterial roads and urban freeways are shown on Figures 2.2.10.1 and 2.2.10.2.

6. Horizontal clearances to a rigid barrier on overpass bridges on arterial roads are shown on Figure 2.2.10.3 while horizontal clearances to a rigid barrier on overpass bridges on freeways are provided on Figure 2.2.10.4. The rigid barrier assists in keeping errant vehicles on the bridge structure and is preferred where pedestrians are accommodated on the overpass.

7. Appropriate barrier transitions should be used on the approaches to the structure.[1] Refer to Chapter 3.1 for additional information.

2.2.10.4 Vertical Clearances: Design Domain and Application Heuristics

Refer to Chapter 2.1.

Table 2.2.10.2 Horizontal Clearance at Bridges on Rural Roads

	Design Speed (km/h)	Short Overpass (<50 m) Left	Short Overpass (<50 m) Right: No Sidewalk	Short Overpass (<50 m) Right: Sidewalk	Long Overpass (>50 m) Left	Long Overpass (>50 m) Right: No Sidewalk	Long Overpass (>50 m) Right: Sidewalk
Undivided Local	50		1.2	0.5		1.0	1.0
	60		1.2	0.5		1.0	1.0
	70		1.2	0.5		1.0	1.0
	80		1.2	0.5		1.2	1.0
	90		1.2	0.5		1.2	
	100		1.2	0.5		1.4	
Undivided Collector	60		1.5	1.0		1.2	1.0
	70		1.5	1.2		1.2	1.0
	80		2.0	1.2		1.0	1.0
	90		2.0	1.5		1.2	
	100		2.5	1.5		1.4	
Divided Collector	70	1.2	1.5	1.2	1.0	1.2	1.0
	80	1.2	2.0	1.2	1.0	1.2	1.0
	90	1.2	2.0	1.5	1.0	1.2	
	100	1.2	2.5	1.5	1.0	1.4	
Undivided Arterial	80		2.5	1.5		1.5	
	90		2.7	1.5		1.5	
	100		3.0	2.0		1.6	
	110		3.0	2.5		1.7	
	120		3.0	2.5		1.8	
	130		3.0	2.5		1.8	
Divided Arterial	80	1.5	2.5		1.0	1.5	
	90	1.5	2.7		1.0	1.5	
	100	2.0	3.0		1.0	1.6	
	110	2.0	3.0		1.0	1.7	
	120	2.0	3.0		1.0	1.8	
	130	2.0	3.0		1.0	1.8	
Freeway	100	2.5	3.0		1.5	2.0	
	110	2.5	3.0		1.5	2.0	
	120	2.5	3.0		1.5	2.0	
	130	2.5	3.0		1.5	2.0	

Notes:
1. For short overpasses (<50 m) shoulder widths should be carried across bridge.
2. All clearances should meet requirements for sight distance.

Figure 2.2.11.2 Nomograph for Predicting Utility Pole Collision Rate[14]

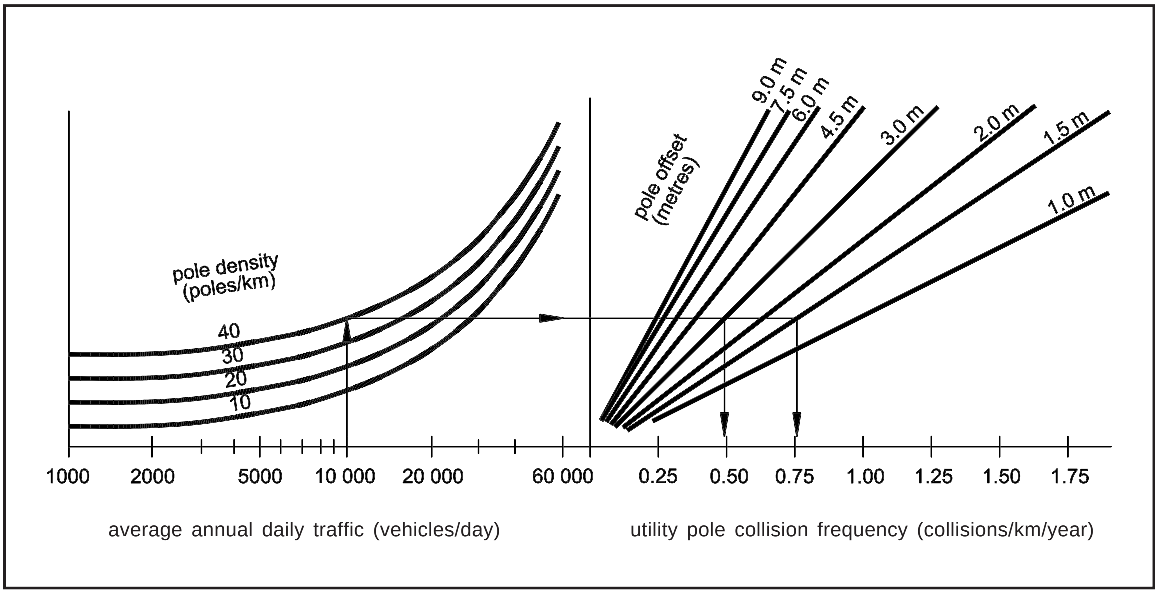
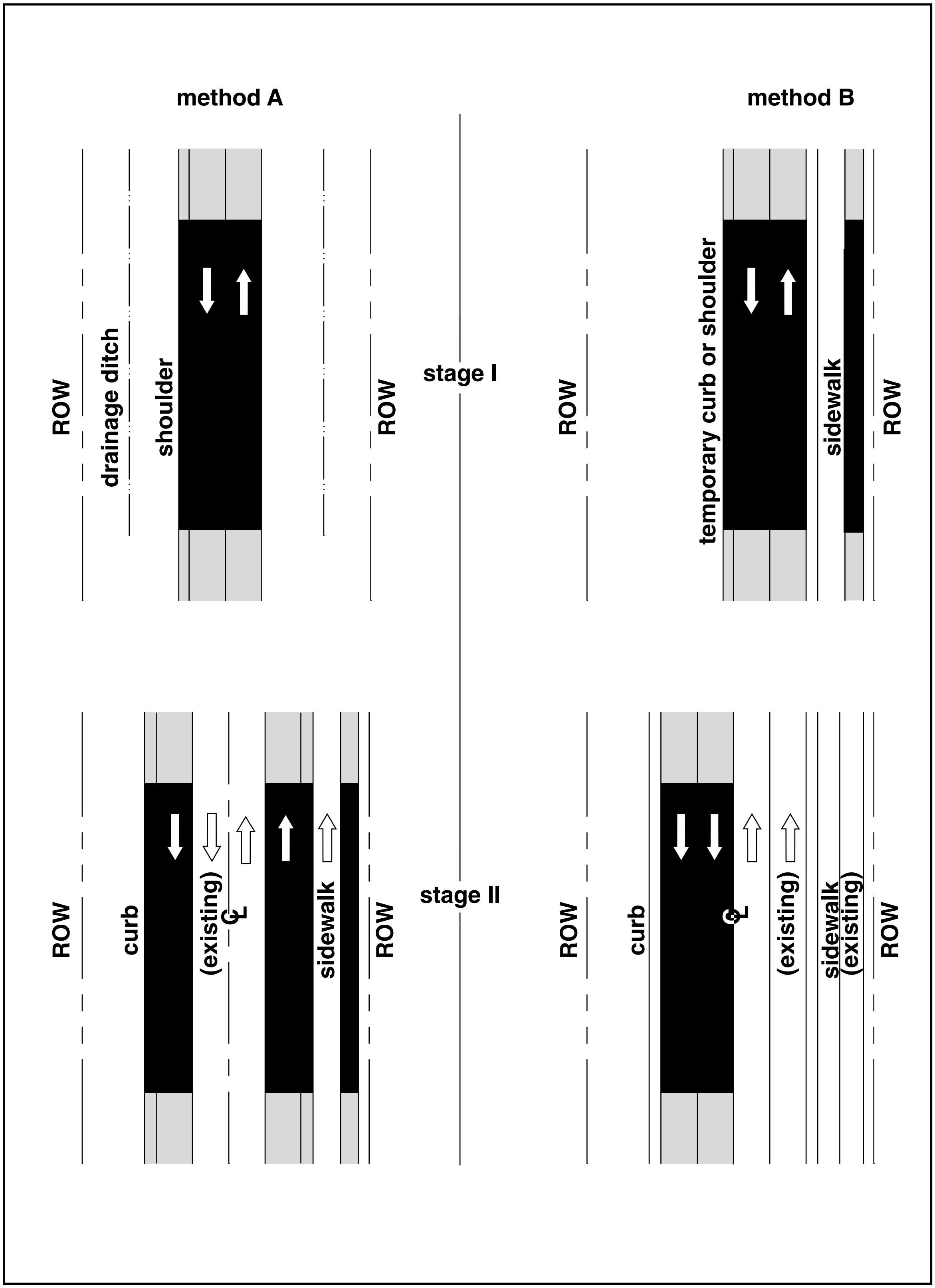


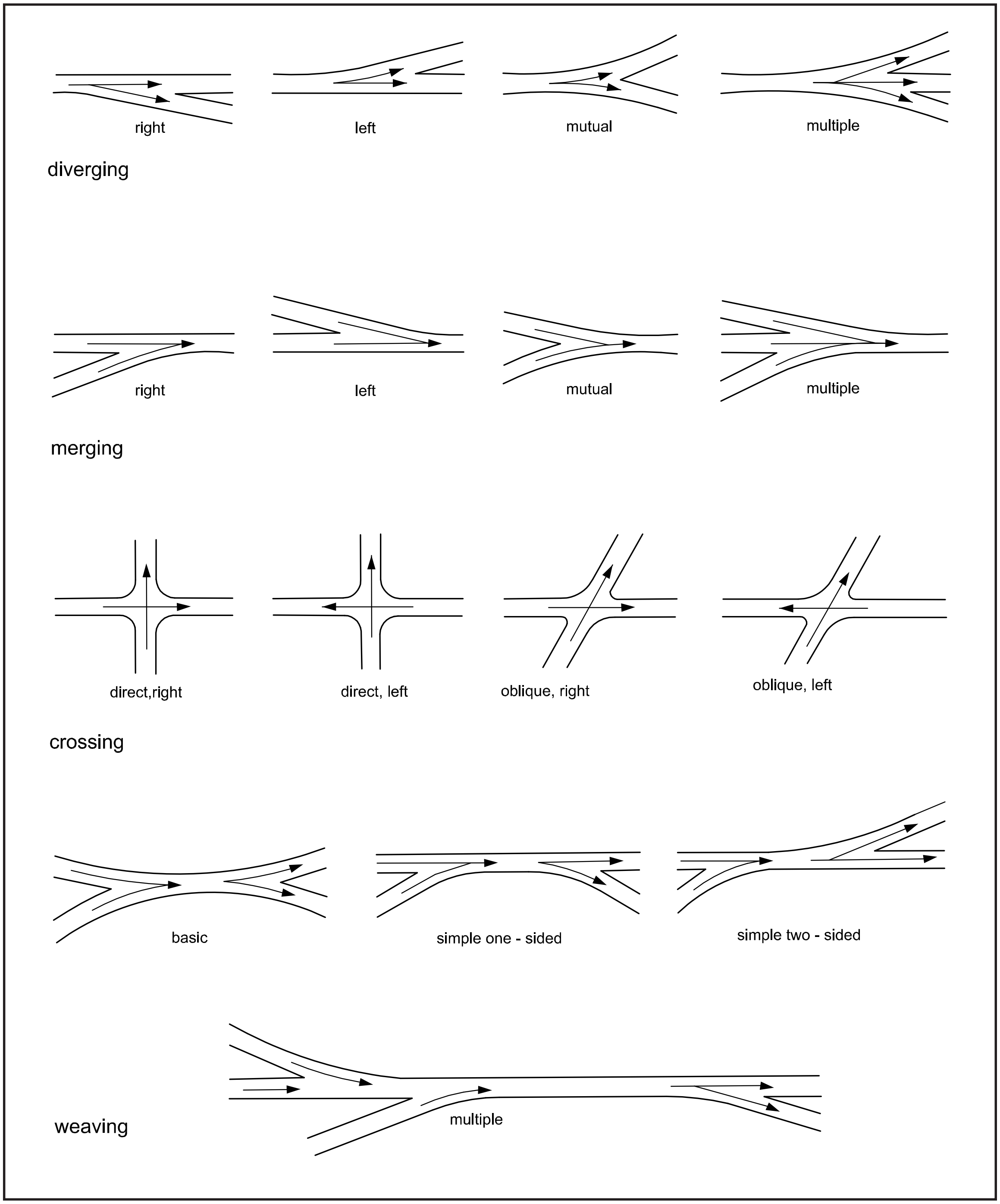
Figure 2.2.12.2 Staging of a New Four-Lane Undivided Arterial Street



Tables

Figures

Figure 2.3.1.2 Typical Traffic Movements Within an Intersection and its Approach



Diverging and merging may be to the right, to the left, mutual or multiple.

Crossings are termed “direct” if the angle of intersection is between 70° and 110° (right-angled intersection) or “oblique” if the intersection angle is less than 70° or greater than 110° (oblique intersection).

Weaving consists of the crossing of traffic streams moving in the same direction. It is accomplished by a merging manoeuvre followed by a diverging manoeuvre. Weaving sections may be considered to be simple or multiple with a further subdivision into one-sided or two-sided weaving.

Conflicts

Every rural and urban at-grade intersection has conflict areas. One of the main objectives of intersection design is to minimize the severity of potential conflicts between all intersection manoeuvres.

A traffic conflict occurs whenever the paths followed by vehicles diverge, merge or cross.

The number of traffic conflicts at intersections depends on:

- the number of one-way or two-way approaches to the intersection
- the number of lanes at each approach
- signal control
- traffic volumes
- the percentage of right or left turns

As an example, Figure 2.3.1.3[1,6] shows conflict points for a T-intersection (three-legged) and a cross-intersection (four-legged). With the addition of a single intersection leg, the number of conflict points increases from 9 to 32. The difference in collision rates at three- and four-legged intersections is also illustrated on Figure 2.3.1.3. It is shown that as traffic volume increases, the role the increased number of conflict points at a four-legged intersection plays in collision rate, becomes more significant.[6] The designer should be cautioned, however, that the number of conflict points for offset, or split T-intersection arrangements, as shown in Figures 2.3.1.1 and 2.3.2.1, would not be 18 (2 x 9); instead the number of conflict points for this type of intersection configuration (two T-intersections) would likely be larger than at a single cross-intersection with 32 conflict points.

The conflict areas are divided into two categories:

- major conflict areas where head-on, right-angle or rear-end collisions may occur
- minor conflict areas where sideswipe collisions may take place

Illustrations of traffic conflict areas are shown in Figure 2.3.1.4.[1] It should be noted that the 90° T- and cross-intersections have the smallest conflict areas in comparison to the skewed cross-intersection and the multi-legged intersection which have the largest.

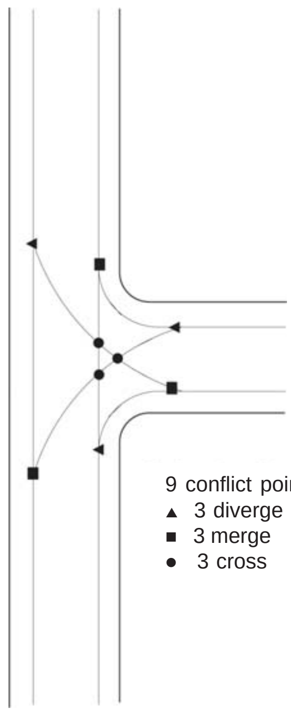
Channelized intersections with auxiliary lanes further reduce the conflict area size and the number of vehicles passing through the same intersection point by separating traffic movements into definite paths of travel using pavement markings and islands. For further information on channelized intersections, see Section 2.3.6.

In urban environments especially, conflicts can also occur between vehicles and pedestrians, and vehicles and bicyclists. Vehicles typically conflict with pedestrian crossing manoeuvres. Vehicles can conflict with any bicycle manoeuvre. The 90° T- and cross-intersections are the most straightforward intersections for pedestrian and bicycle manoeuvres, channelization may increase vehicle/pedestrian conflicts as pedestrians attempt to cross the turning roadway.

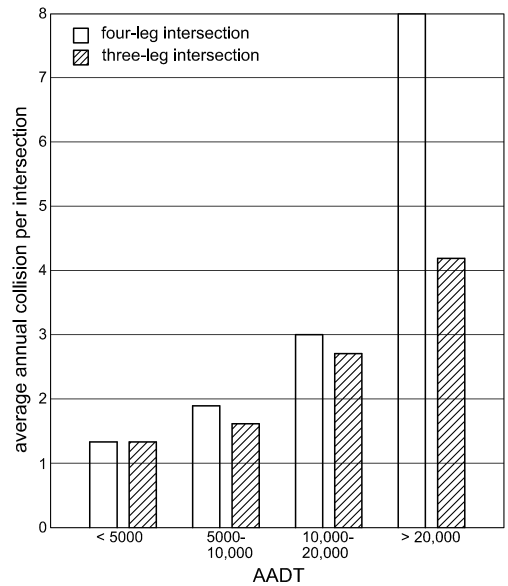
Prohibited Turns

Prohibited turns can be discouraged by designing tight or extended curb returns which make it difficult to achieve these turns. Channelization is also used to restrict or prevent prohibited, undesirable or wrong-way movements. In

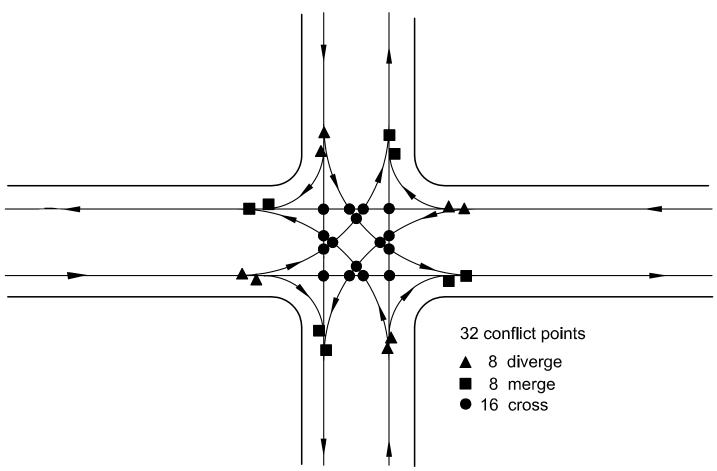
Figure 2.3.1.3 Conflict Points and Collision Rates of Three- and Four-Legged Intersections [1,6]



three-legged intersection[1]

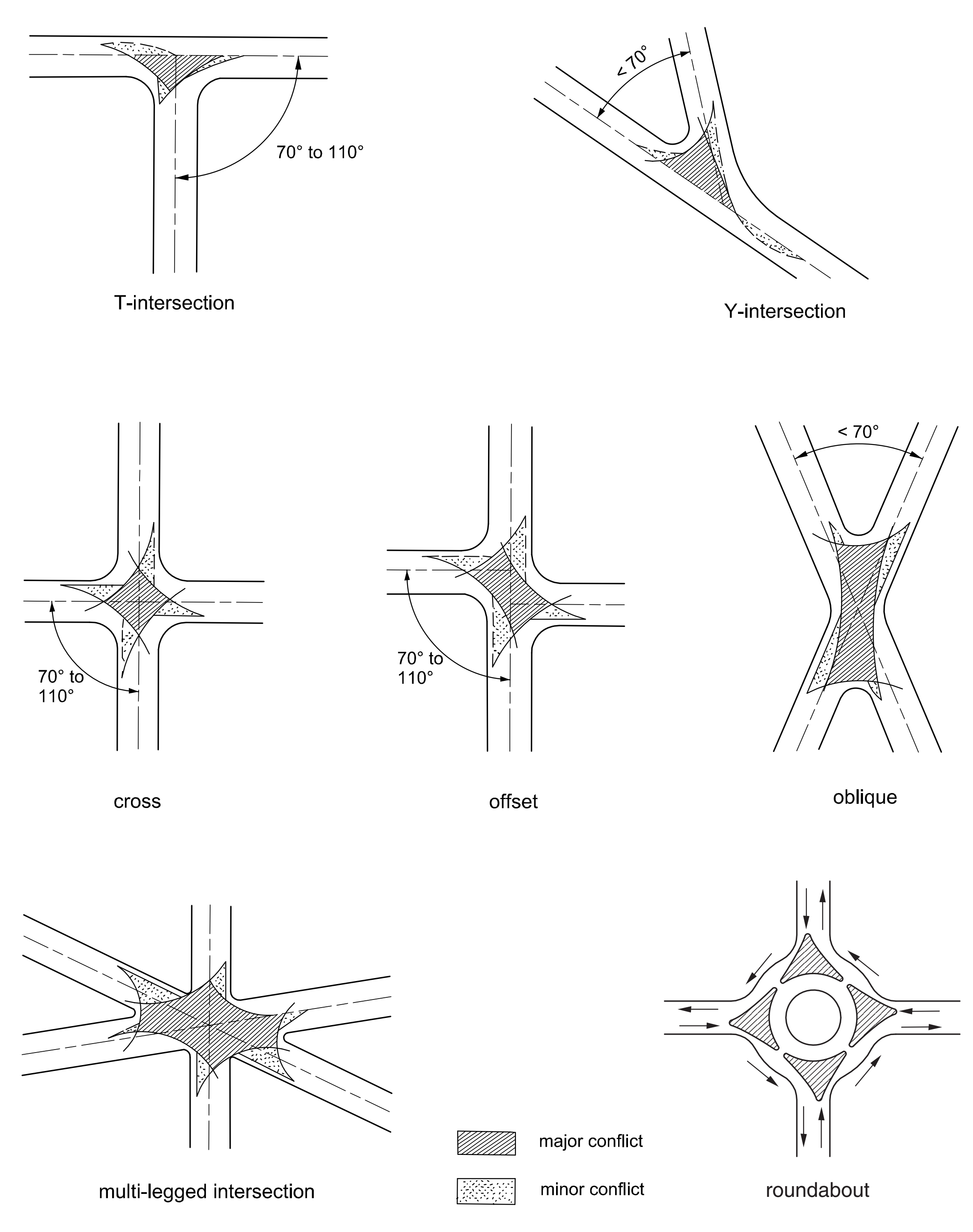


collision rates at three-legged and four-legged intersections[6]



four-legged intersection[1]

Figure 2.3.1.4 Conflict Areas at Intersections [1]



2.3.2 ALIGNMENT

2.3.2.1 Design Speed

The following is a discussion of design speed as it pertains to intersections. Chapter 1.2 presents discussion on roadway design speed.

Rural

In a rural environment, the design speed of the major roadway is used for the main intersection approaches to determine taper lengths, deceleration and acceleration lengths, and other geometric features specific to traffic on the major roadway.

Design speed is typically not reduced at rural intersections where drivers are accustomed to long periods of uninterrupted travel. Inattentive drivers should be alerted to the fact that an intersection is ahead and should have enough time to react accordingly by providing adequate deceleration and acceleration lengths, etc. for the design speed.

Urban

In general, it is desirable to maintain the design speed of a roadway as it passes through an intersection, particularly for a roadway where the traffic has or may have the right of way through the intersection. Examples of this situation are:

- an intersection controlled by traffic signals or which may be controlled by signals in the future
- a major road crossing a minor road where the minor roadway has a stop or yield control, and the major roadway is not controlled
- an uncontrolled intersection

For an urban roadway controlled by a yield sign at an intersection, approach speeds in the order of 25 km/h are common. A suitable design speed for such an approach roadway within the zone of the intersection would be 35 km/h.

Where traffic on a minor roadway is, and will likely always be, controlled by a stop sign at an intersection, the design speed of the minor roadway can be reduced through the intersection area. As a basic requirement, it is important to provide sufficient sight distance for the design vehicle to safely depart from the stopped position and make the desired manoeuvre through the intersection.

If a design speed equal to or greater than the existing posted speed cannot be achieved through an intersection, changes to the posted speed, the implementation of speed advisory signing or similar treatment should be considered. Sound judgement is called for in selecting the design elements that meet the expectations of the driver.

2.3.2.2 Horizontal Alignment

Intersections are ideally located on tangent sections. Location of intersections on curves is not desirable due to decreased visibility, increased conflict potential for vehicles crossing the major roadway, and complications with roadway superelevation and pavement widening on curves. Intersections on curves are discussed further in Subsection 2.3.2.5.

It is desirable that intersecting roads meet at, or nearly at, right angles.

The benefits of a 90° angle of an intersection are:

- reduced size of conflict area (see Figure 2.3.1.4)
- improved driver visibility
- more favourable condition for drivers to judge the relative position and relative speed of an approaching vehicle and to decide when to enter or cross the major road
- reduced length of time of a crossing manoeuvre
- general decrease in severity of collisions (collisions occurring at an impact angle of

90° are generally less severe than those occurring at angles of greater than 90°)[1]

While crossing at 90° is preferable in most cases, it is occasionally necessary and even advantageous to skew the crossing (for example, to favour a heavier turning movement). However, angles less than 70°and greater than 110° are typically not desirable. For example, at a skewed T-intersection with an angle less than 70° certain undesirable conditions exist because of the flat angle of entry. Vehicles which do stop are standing in a position that affords poor visibility for the driver to judge the speed and the distance of approaching vehicles on the major roadway. Also, for a skew right, vehicles leaving the major roadway to enter the minor roadway with a right turn are encouraged to do so at high speeds and for a skew left, drivers tend to cut the corner at higher speeds, thereby travelling in the opposing lane for a considerable distance and creating a safety concern.[1]

Particular consideration should be given to maintaining an angle of skew within 10° of right angle (i.e. between 80° and 100°), when any of the following conditions exist:

- two minor roadways with design hour volume (DHV) greater than 200 v/h (on both roadways) intersect
- minor roadway with DHV greater than 200 v/h intersecting with a major road
- two major roads intersect
- either of the intersecting roadways has more than two basic lanes
- sight distance is at a minimum
- design speed on either intersecting roadway for through traffic is greater than 80 km/h[1]

In the case of existing roads that intersect between 70° and 80° (or 110° and 100°) with no collision or performance concerns, a realignment to 80° (or 100°) may not be cost effective.

The practice of realigning roads intersecting at acute angles in the manner shown in Figure 2.3.2.1 (A and B) is beneficial. Ideally, the curves used to realign the roads would avoid a decrease in operating speed along the realigned roadway. The practice of constructing short radii horizontal curves on minor roadway approaches to achieve right-angle intersections may be acceptable but not necessarily desirable in the urban and rural settings. These curves result in increased lane infringements because motorists tend to drive flatter curves by encroaching on a portion of the opposite lane. Also, the traffic control devices at the intersection may be obscured resulting in the need for the installation of advanced warning signing.[11]

It should be noted that although examples C and D on Figure 2.3.2.1 provide poor network continuity, both examples may be acceptable alternatives. If implemented, suitable physical barriers or other obstructions should be placed across the former right of way of the minor road. These visual obstructions are desirable to alert the driver on the minor road that the road is realigned and is no longer on continuous tangent through the intersection. Assuming a four-lane undivided arterial road, the split T-intersection arrangement, example C (offset-right), introduces back-to-back left turns on the major roadway, which are generally undesirable unless left-turn auxiliary lanes can be provided. This layout, however, has the advantage of requiring the driver, wishing to cross the major road, to select a gap in only the traffic approaching from the left, and then make a conventional right turn followed by a left-hand merge manoeuvre to reach the left-turn auxiliary lane. However, if no left-turn lane is provided, vehicles travelling along the minor roadway may hold up traffic while waiting for a gap to turn left. With example D (offset-left), the turns introduced on the major roadway by the minor roadway crossing manoeuvres are right turns only, which minimize the impact on through traffic on the major roadway. However, the driver attempting to cross on the minor roadway is required to select coincidental gaps in the traffic streams from both directions on the major roadway. Moreover, the driver is required to make right-hand merge manoeuvres on the major operating conditions. If such combinations are encountered, the intersection is normally relocated or improved through realignment of one or both of the intersecting roads, to improve safety. If the combined horizontal and vertical alignment of the intersection creates poor visibility of the intersection, advance warning of upcoming intersections should be provided through appropriate signing.

The bar charts shown on Figure 2.3.2.8[6] illustrate the differences in the relative collision rates between tangent roads both with and without intersections and curved roads with intersections and with intersections on grade.

In all cases, the provision of appropriate sight distances is important to promote collision free operation. The combination of horizontal and vertical geometry at an intersection should produce traffic lanes that are visible to the driver at all times and provide a clear definition of the desired path of any permitted turn or direction of travel.

2.3.2.5 Reduced Superelevation Through Intersections

The general controls and considerations that determine the maximum rates of superelevation for through roadways discussed in Chapter 2.1 also apply to roadway sections with intersections. However, within an intersection, drivers anticipate and accept operation with higher lateral friction than they do midblock, especially in an urban environment. As such, lower rates of superelevation can be used on curves through intersections.

Typical maximum superelevation rates (e_{max}) used at intersections are:

- rural areas: e_{max} = 0.04 m/m to 0.06 m/m
- urban areas: e_{max} = 0.02 m/m to 0.08 m/m

The application of e_{max} is discussed in detail in Chapter 2.1.

Figure 2.3.2.9 illustrates a suggested interrelationship between speed, lateral friction and superelevation for roadways through intersections. The superelevation values are based on the lateral friction factors from Chapter 2.1. Each speed has a constant friction factor; the variables are radius and superelevation.

The following illustrates the use of Figure 2.3.2.9:

1. An intersection is located on a curved roadway with a 200 m radius curve and a design speed of 70 km/h. A superelevation rate is required to allow proper operation of the major through roadway.

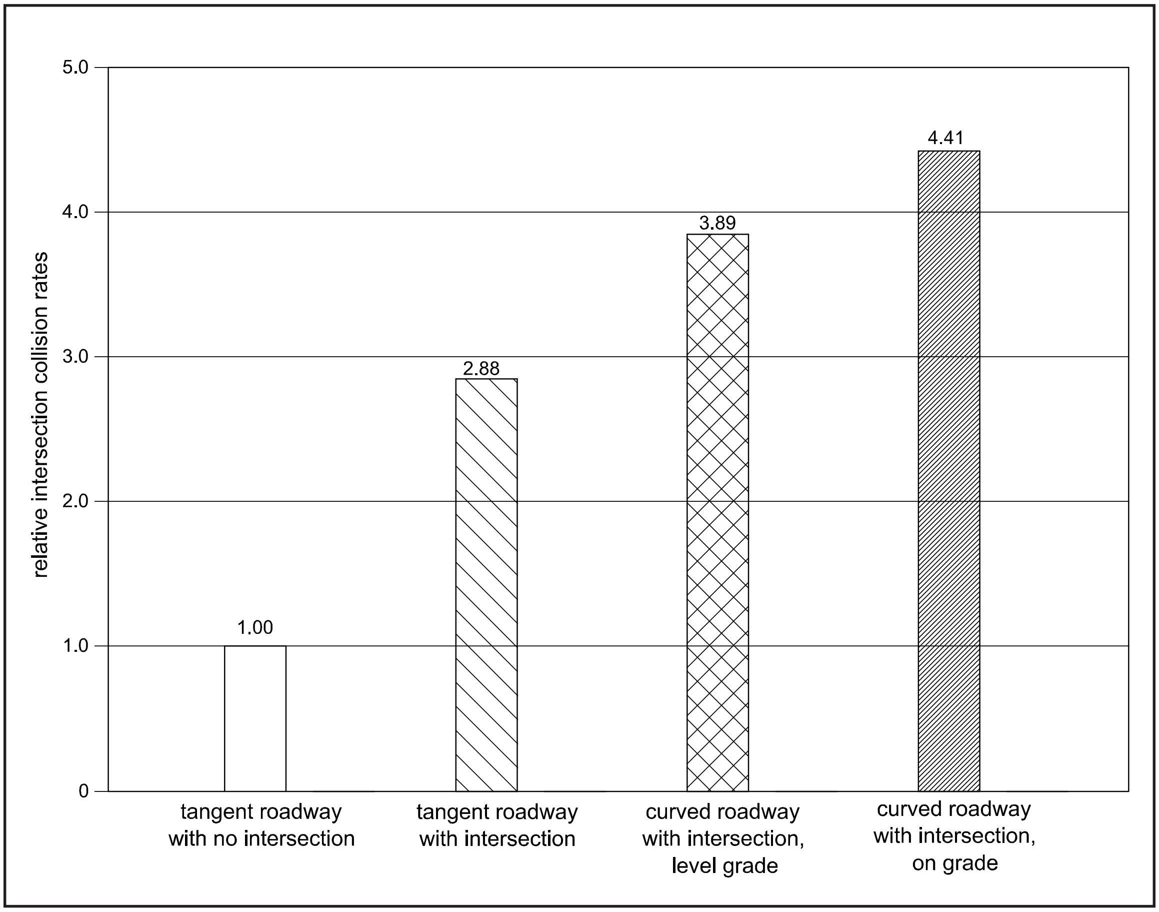
2. Chapter 2.1 shows 0.059 m/m superelevation required for e_{max} = 0.06 m/m. This would be inappropriate for an intersection. Figure 2.3.2.9 shows that the normal rate of superelevation through the intersection can be safely reduced to a minimum 0.023 m/m. Given this situation, the occupants of the vehicles will feel some decrease in comfort due to increased centrifugal force.

The principles of superelevation runoff discussed in Chapter 2.1 for open roadway conditions apply generally to intersections on curves. The controls for the rate of change of cross-slope are primarily those of comfort and appearance, and vary with the design speed. As the design speed is reduced the length over which a change in superelevation can be made is also reduced. Design values for rates of change of cross-slope are shown in Table 2.3.2.1.

Superelevation commensurate with curvature and speed is seldom practical through intersections and at the terminals of turning roadways. In some cases, the following solutions can be used:

- the through curving roadway cross section is widened at the intersection (see Chapter 2.1)
- the normal cross-slope of the through pavement is retained through the intersection

Figure 2.3.2.8 Effect of Geometry on Intersection Collision Rates[6]



Two-Centred Compound Circular Curve

The two-centred compound curve is the preferred design for all types of large trucks and usually fits the minimum inside sweep of a design tractor trailer combination adequately. Although a three-centred curve better fits the inside sweep of a tractor trailer combination, a number of benefits to using a two-centred curve over a three-centred curve have been identified:

- less pavement area for two-centred curves than for three-centred curves
- intersecting road vehicles are forced to proceed slowly with two-centred curves
- stop sign can be placed closer to the intersecting road centreline (more visible) with two-centred curves
- two-centred curve design tends to be more economical

In addition, a two-centred curve may be used to lay out the right edge of pavement for vehicles making a right turn from the minor roadway and a three-centred curve could be used for the right shoulder for vehicles making a right turn from the major roadway.

Figure 2.3.4.5[1] illustrates the application, symbol and nomenclature of the two-centred compound circular curve elements.

For large tractor trailer combinations the recommended radii combination should be checked with the appropriate template and adjusted if necessary. The clearance between the inner rear wheel and the edge of pavement should be 0.5 m preferably and not less than 0.25 m. When applying the template the vehicle should be properly positioned within the traffic lane at the beginning and end of the turn and the inner rear wheel path should clear the curve with the indicated minimum clearance. The application of the design vehicle template is described in Chapter 1.2.

When facilitating large trucks in non-industrial areas, consideration should be given to channelization to avoid large paved areas that may be confusing to a driver and difficult to control in terms of orderly movement of vehicles.

When pedestrians are a consideration at a signalized wide open-throat intersection the “walk” and clearance times may be affected, hence provision of adequate service and protection for pedestrians may be required.[1]

Three-Centred Compound Circular Curve

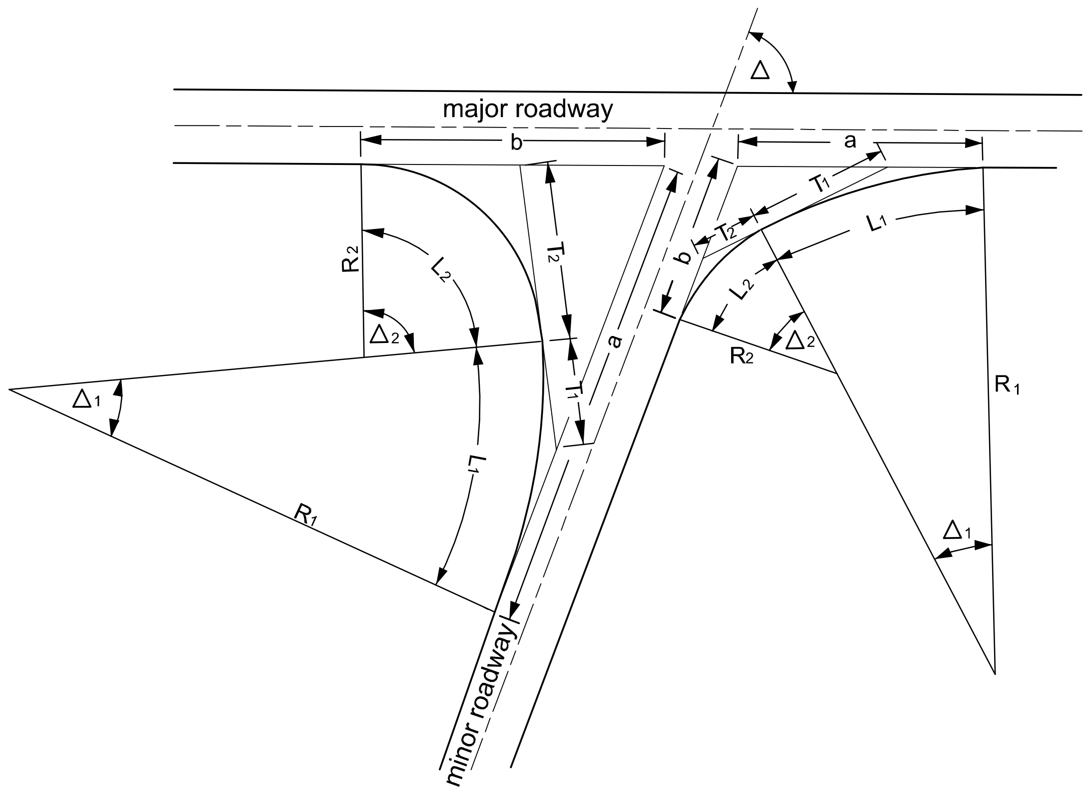
To fit the edge of pavement closely to the minimum inside sweep of a tractor trailer combination, the application of a symmetrical arrangement of three-centred curves has proven advantageous. This design is the practical equivalent to a curve transition for most or all of its length. A three-centred curve is typically used at a major intersection with exclusive left- or right-turn lanes. In an operational sense, it is superior to the minimum circular arc design because it better fits the inner rear wheel turning path of a tractor trailer, while providing some margin for driver error and requiring less pavement.

Three-centred curve design for angles of turn more than 90° may result in unnecessarily large paved intersections, portions of which are often unused. This situation may lead to confusion among drivers and present a hazard to pedestrians. These conditions may be alleviated to a considerable extent by the use of asymmetrical three-centred compound curves, or by using large radii, coupled with corner islands. In Figure 2.3.4.6[10] the elements of three-centred symmetric/asymmetric compound curves are illustrated.

The two-centred curve provides for tractor trailer off-tracking, however, there is not as much room for driver error as there is on the three-centred curve.

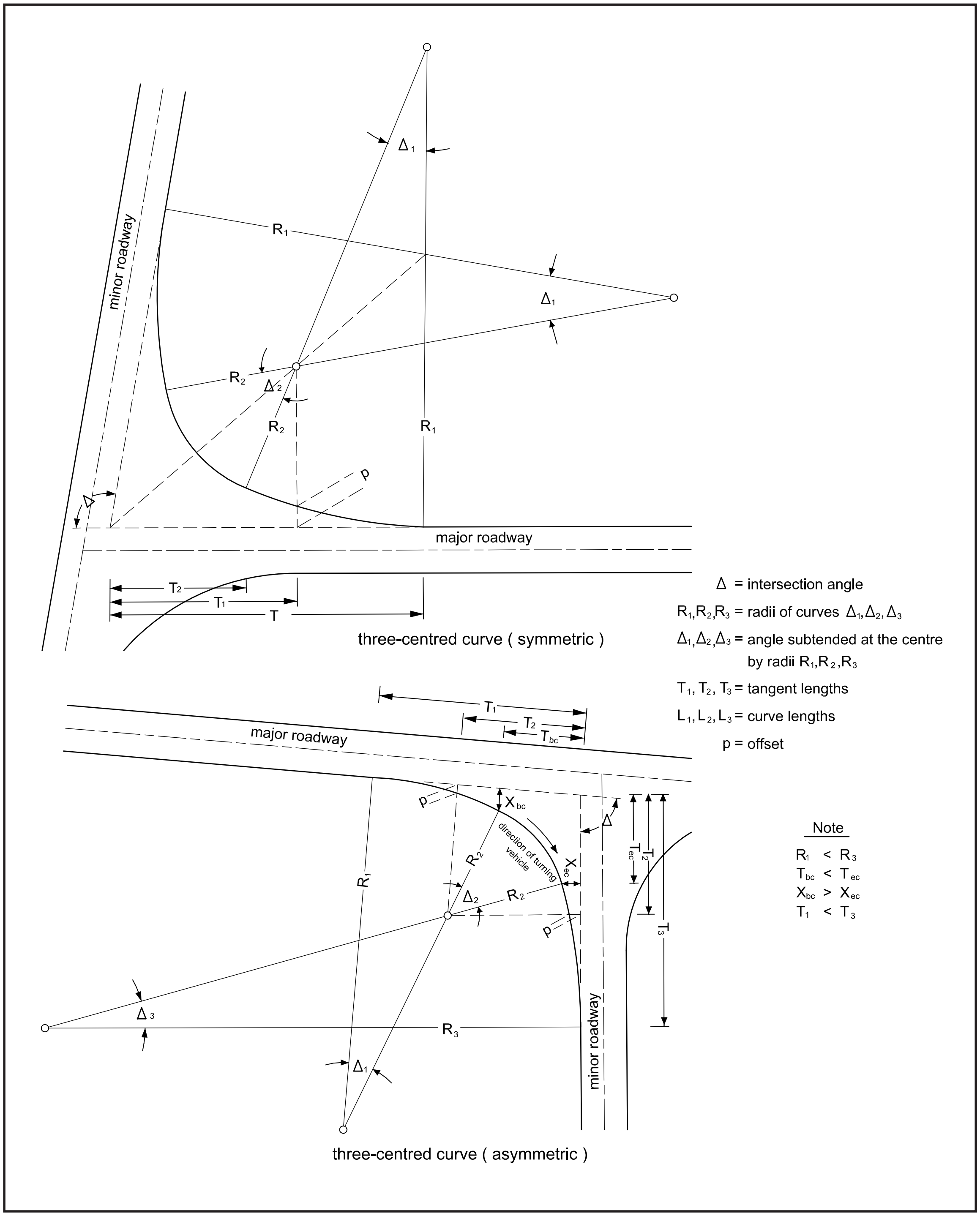
The use of a two-centred curve is permitted in situations where a three-centred curve would normally be used, but cost to purchase extra right of way is extreme or where surrounding roadway geometrics do not allow for the application of a three-centred compound curve.[10]

Figure 2.3.4.5 Edge of Pavement Design – Two-Centred Compound Curve[1]



Δ = intersection angle of the two roads or angle of turn
R_1 = radius of curve Δ_1
R_2 = radius of curve Δ_2
Δ_1 = angle subtended at the centre by radius R_1
Δ_2 = angle subtended at the centre by radius R_2
T_1, T_2 = length of tangent
L_1, L_2 = length of curve
a = long tangent of the compound circular curve
b = short tangent of the compound circular curve

Figure 2.3.4.6 Edge of Pavement Design - Three-Centred Compound Curve[10]



2.3.4.4 Shoulders at Simple Intersections

An urban cross section with barrier curb does not typically include shoulders. However, where shoulders are provided, they are paved and mountable curb and gutter is used.

The rural roadway at intersections includes shoulders or equivalent lateral clearance outside the edges of pavement. A shoulder at intersections is provided for the same reasons as that for the open roadways. It is an area adjacent to the driving lane where a driver can make a stop in case of an emergency. It can also provide width for the occasional oversized vehicle and may be used as a bypass lane for emergency vehicles.

Due to improper vehicle operations, shoulders at intersections are subject to deterioration at a faster rate than along open roadways. Edge of pavement drop-off and gravel strewn onto the pavement are main concerns which require frequent inspection and maintenance. This section deals with shoulder treatment at intersections designed to minimize these concerns.

At intersections the required shoulder width varies from a minimum of 0.5 m to that equal to the open roadway shoulder width. Where two roads of different operational characteristics and functions intersect, the shoulder width at the intersection normally varies and serves as a transition from a wide shoulder on the main roadway to a narrow shoulder on the minor roadways.

Where the major roadway is designed with auxiliary lanes, the shoulder width from the major roadway to the minor roadway is transitioned along the arc length of the edge of pavement curve.

For the far side of the major roadway intersection, the shoulder width of the minor roadway is extended along the arc length of the edge of pavement and transitioned to the shoulder width of the main roadway within the 30 m recovery taper length, as shown in Figure 2.3.4.7.[1]

A uniform intersection shoulder width is designed where the intersecting roadways are of similar importance.

There are three types of shoulder treatments at simple rural intersections:

- gravel shoulders (rural)
- paved shoulders (rural)
- concrete curb and gutter (rural and urban)

Each type of shoulder treatment is applied at intersections with or without tapers or deceleration lanes.

Edge of pavement delineators may be used in conjunction with either gravel or paved shoulder treatment at intersections. Generally, the application of delineators is discouraged as they are often damaged or destroyed by turning vehicles and their effectiveness is greatly reduced. They also cause maintenance problems during snow removal operations. However, delineators may provide a guidance to the drivers exiting and entering the roadway at locations with restricted visibility and during poor weather conditions.

Shoulder treatment at intersections should be evaluated and designed for each location based on existing and anticipated future traffic volumes and operational considerations.

<u>Gravel Shoulders</u>

Shoulder treatment at intersections is usually achieved by surfacing the shoulder with gravel, see Figure 2.3.4.8[1]. However, unstabilized shoulders generally undergo consolidation with time and the elevation of the shoulder at the pavement edge tends to become somewhat lower resulting in pavement drop-off. Also turning manoeuvres contribute to gravel tracking onto the pavement area. As such, gravel shoulders in intersection areas require regular maintenance.

<u>Paved Shoulders</u>

At intersections, drivers of large trucks sometimes cut across the shoulder when

Figure 2.3.5.1 Typical Right-Turn Taper Lane Design at T-Intersections[1]

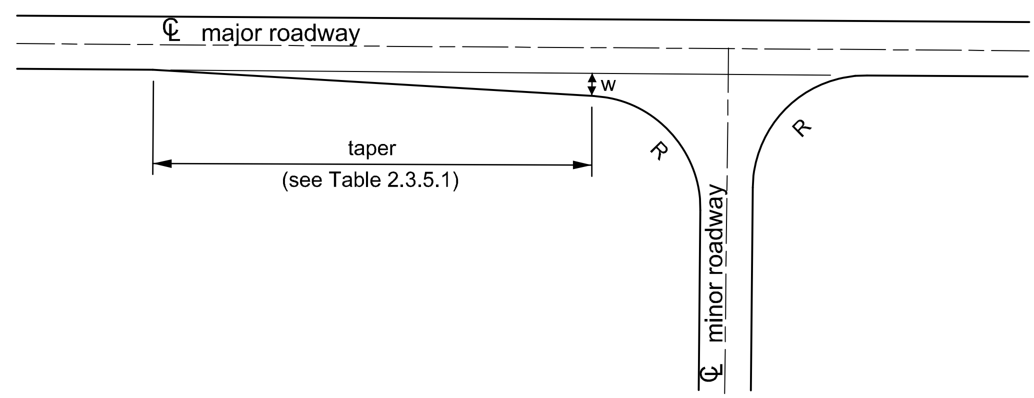
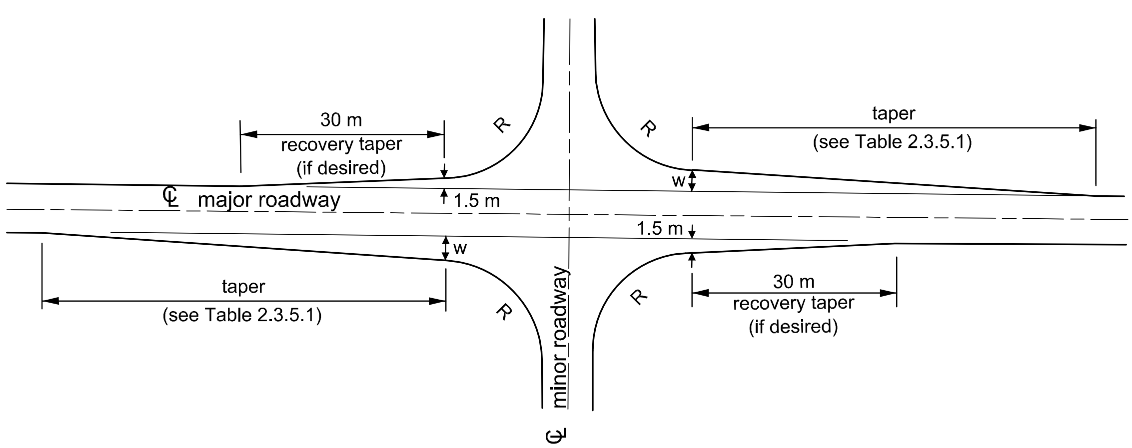


Figure 2.3.5.2 Typical Right-Turn Taper Lane Design at Cross-Intersections[1]



Note : R is based on design vehicle (see Chapter 1.2).

Table 2.3.5.1 Right-Turn Tapers Without Auxiliary Lanes

Design Speed (km/h) (through roadway)	Taper Ratio	Taper Length for w = 3.5 m (m)	Horizontal Curve[(a)], (R) (m)
50	15:1	53	500
60	18:1	63	750
70	21:1	74	1000
80	24:1	84	1200

Note: a) Flat radii as indicated can be used rather than tangent alignments, for right-turn tapers.

2.3.5.4 Design Elements for Right-Turn Tapers with Auxiliary Lanes

The length of an auxiliary lane is based on deceleration and storage requirements.

Deceleration should occur exclusively within the auxiliary lane, although in an urban environment deceleration (up to 15 km/h) over the bay taper is normally tolerable (especially in a peak-hour condition).

Suggested taper and parallel lengths are shown on Table 2.3.5.2 and illustrated on Figure 2.3.5.3. Adjustments for intersections on curves are discussed in Subsection 2.3.8.8.

Auxiliary lanes can be developed using reverse curves or straight line tapers; reverse curves are typically used in an urban environment with curb and gutter.

On high-speed roads the taper length to the auxiliary lane should generally conform to that discussed in Section 2.4.6 - Interchange Ramps.

Where auxiliary lanes are used for the storage of turning vehicles at unsignalized intersections, the length of the lane in addition to deceleration length and exclusive of taper, is usually based on the number of vehicles that are likely to accumulate in two minutes (2 min). The storage length required is calculated by the following formula and can be used for right- or left-turning vehicles:

$$S = \frac{NL}{30} \qquad (2.3.3)$$

Where: S = storage length (m)

N = design volume of turning vehicles (v/h)

L = length (m) occupied by each vehicle (see Chapter 1.2)

At signalized intersections, the storage lane length should accommodate about 1.5 times the average number of vehicles to be stored per cycle for roadways with design speeds of 60 km/h or less, and about twice the average number of vehicles for design speeds greater than 60 km/h.

The storage length calculated above should be checked against capacity analysis to ensure an acceptable level of service.[22]

The required storage for two-lane operation is one half that for a single-lane operation.

Where there is a possibility that an auxiliary lane may be used for either storage or deceleration, the length is determined for both conditions and the total is used in design. For urban and suburban roads, the left-turn lane length tends to be used mainly for storage during peak hours (typically slower peak hour speeds require less length for deceleration) and mainly for speed change at off-peak hours (the queue length tends to be smaller but the speed in off-peak tends to be greater). For auxiliary lane widths, refer to Chapter 2.2.

2.3.6.3 Guidelines for the Application of Channelization

Channelization may also be implemented for any of the functions described in Subsection 2.3.6.1.

Guidance in determining the need for channelization is also provided in other publications.[22]

2.3.6.4 Right-Turn Designs

The right-turn channelization volume warrant presently in use is approximately 60 v/h. Most of the research in this area has been limited and consequently difficulties in developing applicable criteria exist.[1]

Figure 2.3.6.1 illustrates the typical layout and dimensions for four types of right-turn designs: stop, yield, merge and added lane. The form of traffic control should be selected to suit the design and to minimize conflicts between right-turning vehicles, and left-turning and through vehicles.

<u>Stop Design</u>

The right-turn design for a stop condition at an intersection normally consists of a simple radius and does not require channelization.

Information on right-turn designs with simple radii is provided in Section 2.3.4 and 2.3.5 of this Guide.

<u>Yield Design</u>

At major intersections, such as arterial/arterial or collector/arterial intersections, or within industrial areas, the right-turn design for a yield condition is typically a three-centred curve with sufficient radii to provide a small island. In industrial areas with minimal pedestrian activity, the raised island is normally omitted to provide more manoeuvring area for large turning trucks. The three-centred curve may be preceded on the approach leg by either a right-turn auxiliary lane or a taper. Turning roadways are addressed in Section 2.3.7. The auxiliary lane design is more common in urban areas and is typically applied in consideration of capacity and storage requirements.

At the intersection of two local roads or a local and a collector road, particularly in residential areas, the right-turn design for a yield condition could be a simple radius without an island.

<u>Merge Design</u>

Merge right-turn designs are applicable for conditions where a turning speed of greater than 40 km/h is desired at the intersection for capacity or operational reasons. Designs of this type are normally used at freeway and expressway ramp terminals, and for connections onto high-speed arterials. The application of the merge design is also a function of volume. If the merging volumes are too high, the result can be congestion, in which case an added lane design is preferred.

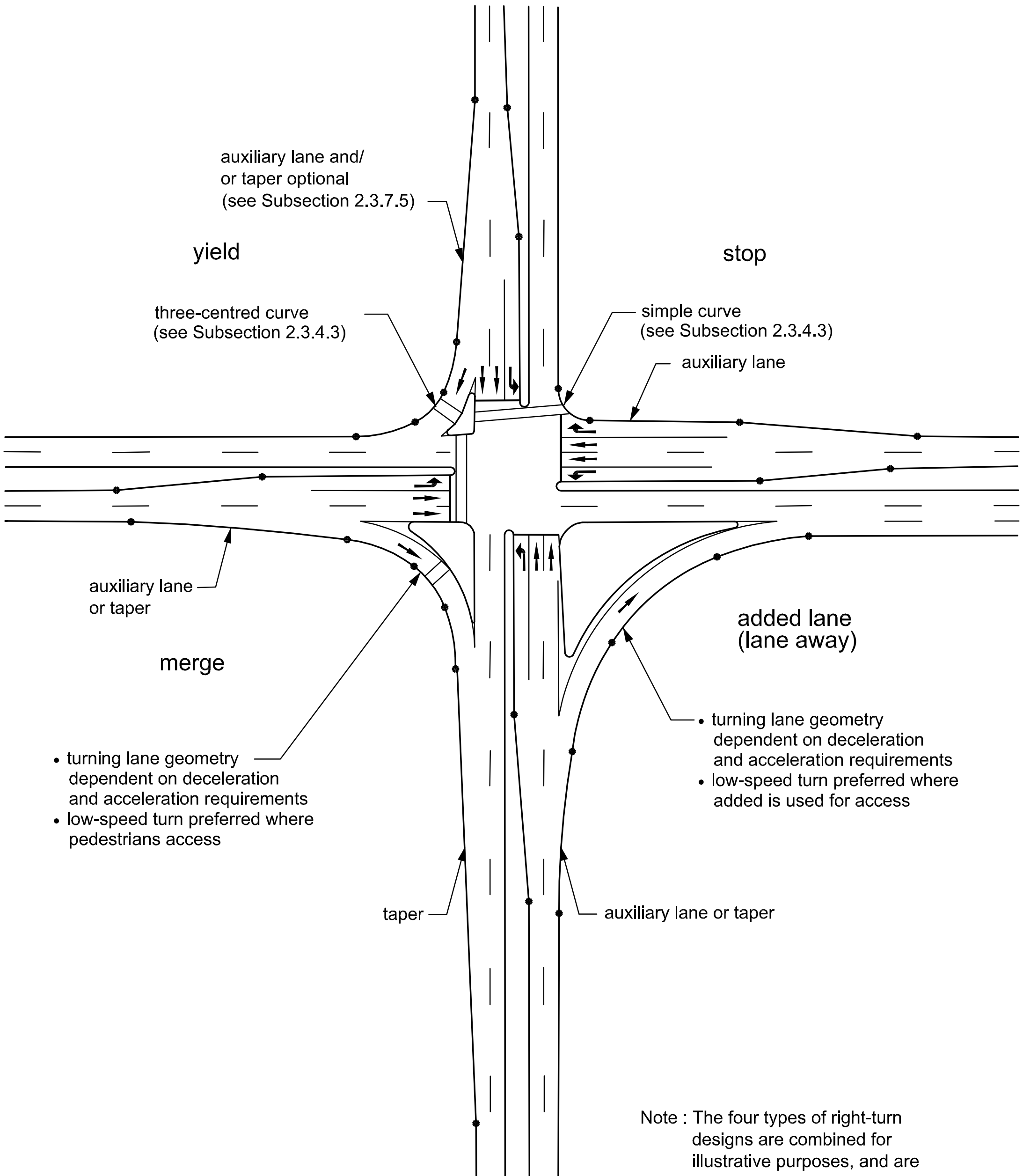
The turning roadway (see Section 2.3.7) is introduced by a right-turn auxiliary lane and/or a tapered approach, which provides the necessary deceleration characteristics.

<u>Added Lane (Lane Away) Design</u>

The added lane (lane away) right-turn design is normally for high volume right-turning movements. This design is also appropriate at an intersection where an auxiliary lane is introduced for access purposes, as discussed in Chapter 3.2.

If the added lane is an auxiliary lane used for access purposes, radii providing lower turning speeds, 40 km/h or less, are suitable. Where the added lane is an additional through lane on a high-speed road, the right-turn design can vary substantially. Where no right of way, physical constraints or intersection spacing limitations are present and there are no pedestrian crossing considerations, the right turn is often designed to minimize the speed differential between the vehicles on the adjacent through lane and the turning vehicles on the added lane at the convergence point.

Figure 2.3.6.1 Typical Right-Turn Designs



2.3.6.5 Traffic Islands

General

An island is a defined area between traffic lanes for control of vehicle movements in intersection areas or for pedestrian refuge. Islands may be raised areas or may be painted. In rural areas the two most desirable and commonly used treatments are the raised island with mountable curbs and the painted island. In urban areas barrier curbs are used to protect pedestrians and to reduce the risk of vehicles striking poles, etc.

Delineation and approach end treatment is critical to good channelization design. Island delineation can be divided into the following types:

Curbed Islands

This type can be applied universally and provides the most positive traffic delineation. Mountable curbs should be used in most cases. In rural areas where curbs are not common, this treatment is often limited to islands of small to intermediate size. Pedestrian refuge islands are usually protected with barrier curb.

Painted Islands

This type of island is generally designed in urban or suburban areas where speeds are low and space is limited. Application of this type of island may be considered in rural areas in advance of raised median island, where maintenance and snow removal make curbs undesirable, and where high approach speeds (urban or rural) make a curb a potential hazard. However, snow accumulation can obliterate pavement markings.

Non-Paved Areas Formed by Pavement Edges

This type of island is usually used for larger islands at rural intersections where there is sufficient space and/or where added expense of curbs may not be warranted or may pose a traffic hazard. This island type may be supplemented by delineators on posts, other guide posts, a mounded earth treatment or appropriate landscaping.

Temporary Island Installations

This type is usually constructed of asphalt curbing, precast bumper curbing or sand bags[1]. Such islands would typically be used in construction work zones.

Islands are grouped into three functional classes which are illustrated on Figures 2.3.6.2[1], 2.3.6.3[1] and 2.3.6.4[1] and are described below:

Directional

Directional islands control and direct traffic movements. They guide the driver into the proper channel for the intended route. Directional islands are of many shapes and sizes, depending upon conditions and dimensions. A common form is one of triangular shape to separate right-turning traffic from through traffic.

Divisional

Divisional islands, also called raised median islands, are introduced at intersections, usually on approach legs, to separate streams of traffic travelling in the same or opposite direction. These islands are particularly advantageous in controlling left turns at skewed intersections and at locations where separate channels are provided for right-turning traffic.

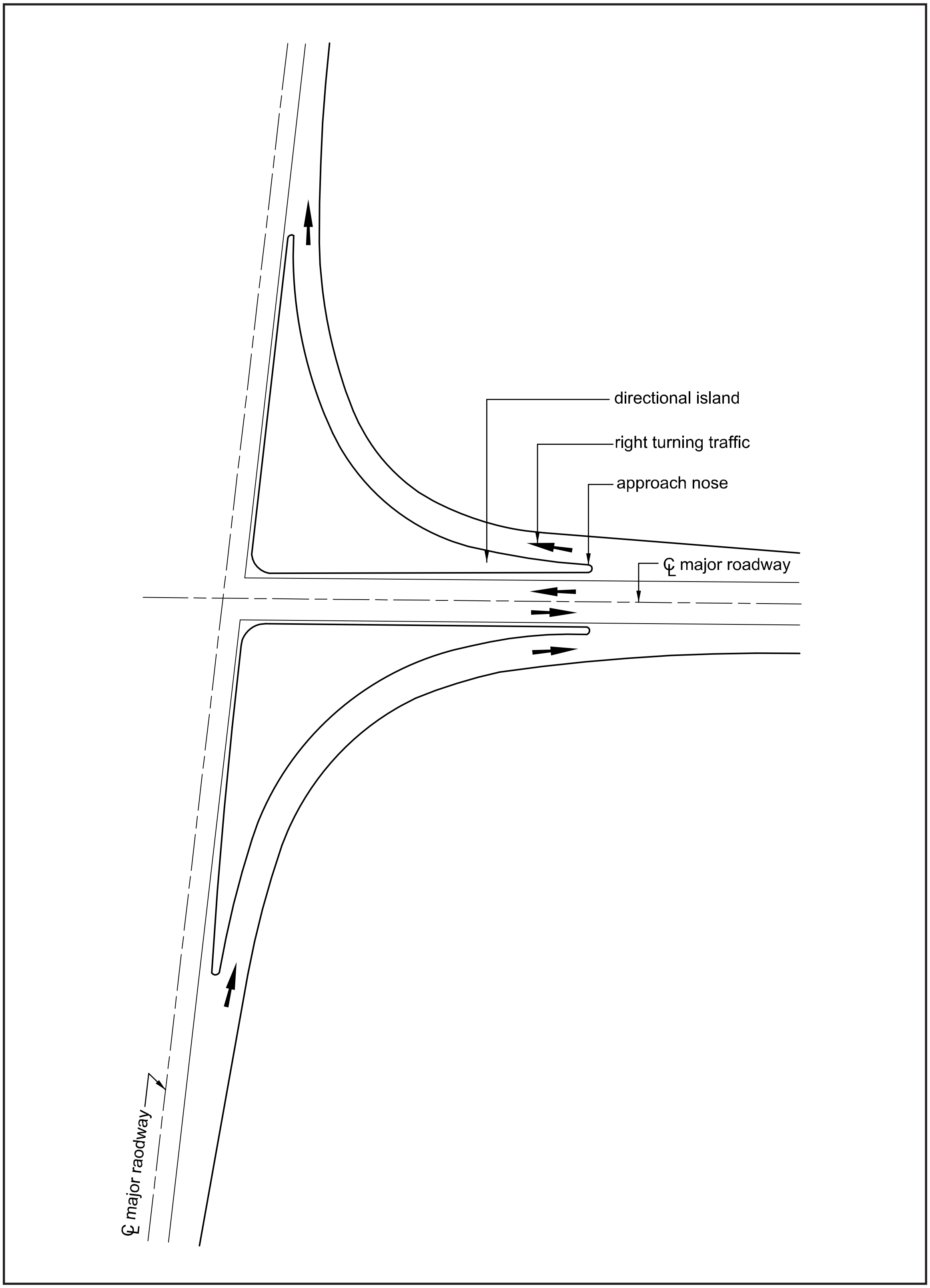
Two types of divisional islands are commonly used:

- opposing divisional islands (for T-intersections)
- offset divisional islands (for cross-intersections)

These islands are shown on Figures 2.3.6.5[1] and 2.3.6.6.[1]

Where the roadway is on a tangent, reverse curve alignment is necessary to introduce dividing islands. In rural areas where speeds are high, reversals in alignment should have radii of at least 2000 m. A median on an approach leg may

Figure 2.3.6.2 Directional Islands[1]



lane. The lane, median and gutter widths shown are typical and vary in accordance with cross section requirements.

The raised median, protecting the left-turn area, is effective in clearly defining the through vehicle paths and the left-turn storage area in all weather conditions. Also, if accesses exist in close proximity to the intersection, the raised median reduces the type and number of turning-vehicle conflicts within the zone of the intersection. However, in instances where the length available for the left-turn auxiliary lane may not be sufficient to store all the left-turn vehicles during peak periods, it is advantageous to use a painted rather than a raised median area in advance of the left-turn lane. In this case, the painted median area can be used to provide additional storage during occasional peak traffic periods, reducing the problem of left-turning vehicles blocking the through lanes.

The approach and departure taper designs are a function of the design speed of the roadway. For high-speed roads (design speeds >70 km/h), the importance of using a gradual taper cannot be over emphasized. Refer to Table 2.3.8.1 for approach and departure taper geometry with design speed.

The characteristics of each of the three methods of introducing a median, as shown on Figure 2.3.8.7, are described in the following paragraphs.

Method "A" illustrates the geometry for a median introduced totally to the left of the roadway centreline. A lateral shift is not required for the traffic approaching the intersection. For this condition to occur on both approaches to a single intersection, the centrelines of the approach roadways must be offset from each other. Although this is a desirable means of introducing a median, it is a rare case, occurring only where excess right of way is available, where the roadways are not centred within the right of way, or where the rights of way are offset appropriately across the intersection. In this method, only the lanes leaving the intersection are required to taper back to the normal undivided roadway cross section. The departure taper typically commences at the beginning of the parallel lane portion of the left-turn lane, to minimize the median length.

Method "B" shows the centreline continuous through the intersection and the roadway widened symmetrically. In this method, the departure taper is continued beyond the approach taper, enabling the nose of the introduced median to be on the left side of the roadway centreline on the approach. The geometry results in a longer median length than that created by Methods "A" or "C".

Method "C" is similar to Method "B" in that the roadway is widened symmetrically about the centreline. The departure taper commences near the beginning of the parallel lane portion of the left-turn lane to reduce the median length. The approach nose to the median is centred on the roadway centreline.

Divided Roadway

Figure 2.3.8.8 illustrates a typical layout of a left-turn lane and a right-turn lane along a divided roadway. The right-turn lane layout is also applicable to undivided roadways.

Left-Turn Slip-Around Treatment at T-Intersections

A left-turn slip-around can be introduced on a two-lane roadway at T-intersections under the following conditions:

- where the left-turning volumes do not warrant a full left-turn lane but are sufficient to potentially affect through traffic
- where through vehicles bypassing occasional left-turning vehicles throw gravel from the shoulder onto the roadway

The slip-around design is comprised of an auxiliary lane and tapers at each end, as shown in Figure 2.3.8.9[1]. See Subsection 2.3.8.3 for taper lengths.

Usually the slip-around design is not applied on four-lane undivided roadways; however, where the left-turn lane is not warranted and turning vehicles impede the through traffic, the slip-around has its merit.

Figure 2.3.8.8 Turning Lane Design, Raised Median

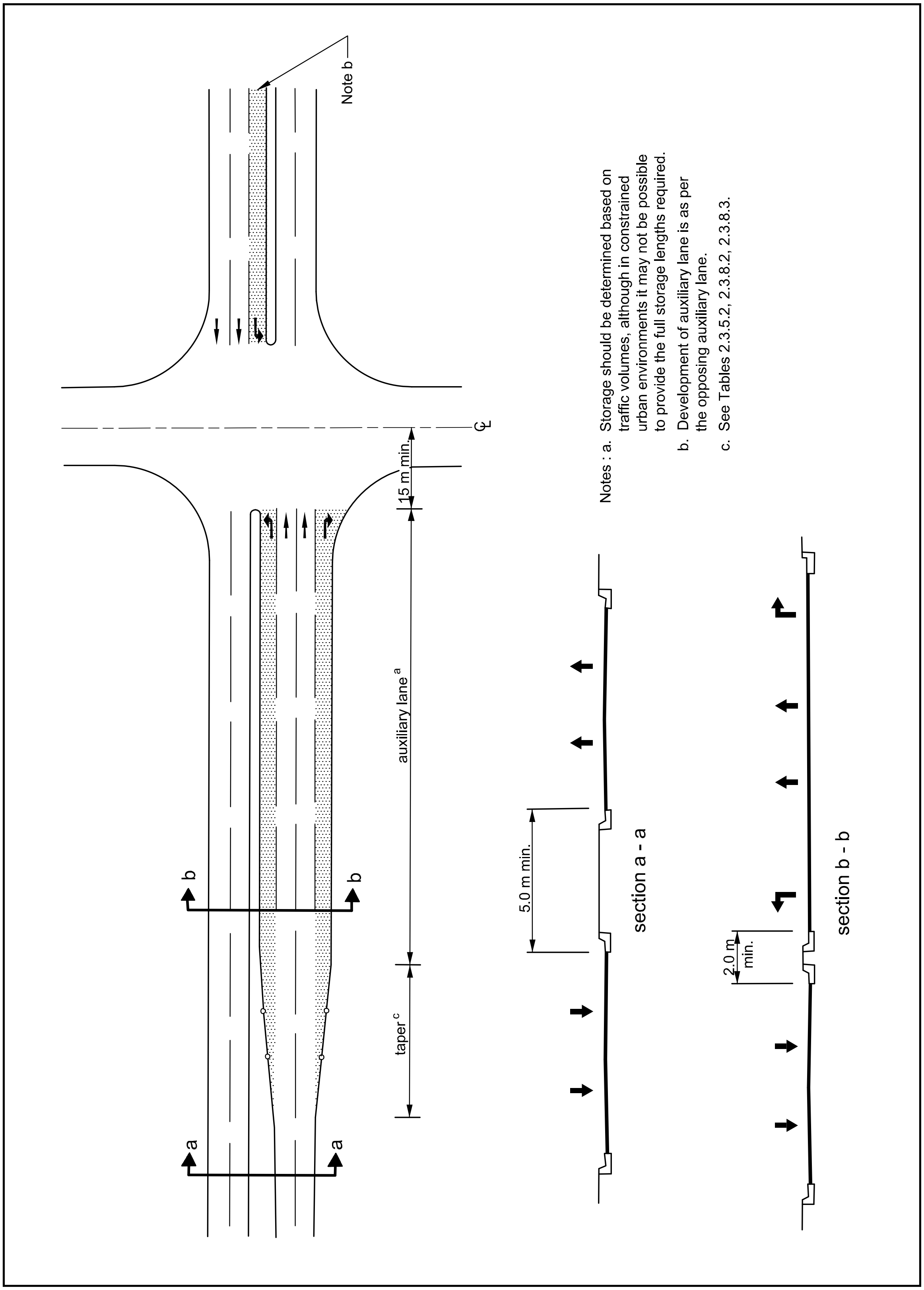
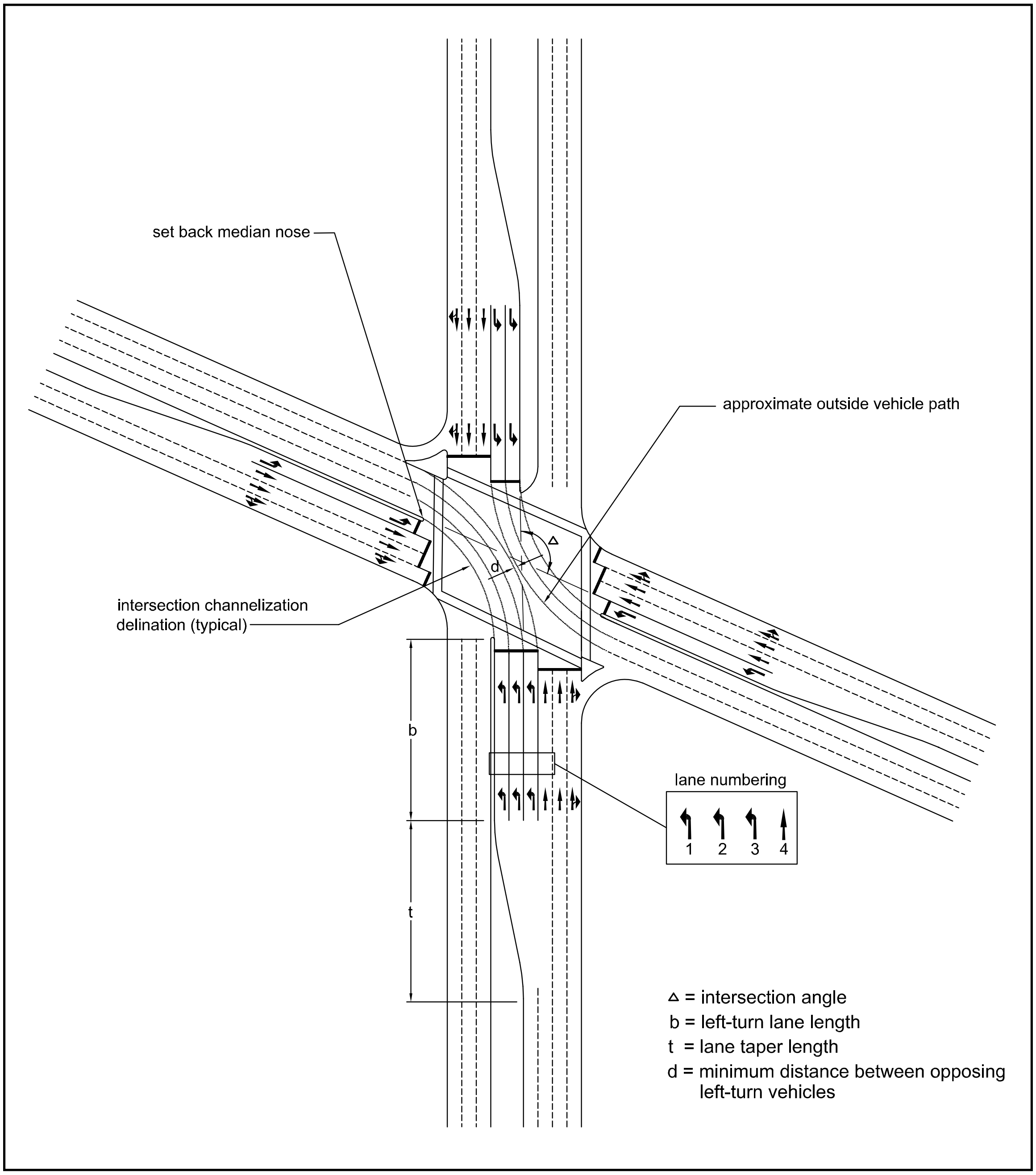


Figure 2.3.8.15 Triple Left-Turns[19]



e) Approach and Departure Lane Widths

Left-turn approach lane widths used at a triple left-turn have been at least 3.3 m. Similarly, downstream departure lane widths have been designed to an absolute minimum of 3.5 m with a desirable width of 3.7 m. A key factor controlling the geometry of the downstream receiving throat width is the tracking path of the design vehicle as it transitions from a circular to a tangential motion. The tracking path approximates a spiral as the design vehicle completes the left-turn movement. Therefore, the width of the clear portion of the intersection may need to be widened based on the design vehicle turning characteristics. The turning geometry may be accommodated by setting the median island nose on the receiving crossing roadway a significant distance back from the intersection. A 0.6 m offset from the vehicle turning path in Lane 1 has been used in locating the median island nose. The receiving roadway width at the intersection may also be widened by increasing the curb return radius of the opposite intersection quadrant. These geometric adjustments have to be carefully evaluated for the intersection angle and roadway widths. Due to the high volumes of vehicle traffic, raised median islands of at least 0.6 m width (1.5 m desirable) should be used on the approach and departure legs of an intersection with two-way traffic. Wider roadway median islands provide the intersection with larger radius curves thus improving the intersection's left-turning geometry. A raised median island has been found to provide a driver in Lane 1 with a visual point of reference to guide a vehicle through the left-turn manoeuvre. A raised median island also provides delineation for the stop bar location on the receiving roadway. This is especially important when the left-turn lane stop bar is offset from the through movement to accommodate triple left-turn lane geometry.

f) Auxiliary Lane and Taper Length

These are determined as per single and double left-turn lanes. The total storage capacity of all lanes should be considered as well as deceleration lengths.

g) Roadway Delineation and Signage Considerations

Even though intersection geometry may be adequate to accommodate three-abreast left-turn movements, roadway delineation and signage are equally important to the safe operation of the facility. Advance overhead signage of the triple left-turn lane configuration is a critical element to inform motorists of the intersection and lane options. Turn lanes and paths should also be clearly delineated to avoid driver confusion.

h) Summary

As with all geometric designs, site-specific conditions must be reviewed and good engineering judgement applied to each design of triple left-turn lanes. As more facilities are constructed and used by the motoring public, additional research should be conducted to include topics such as:

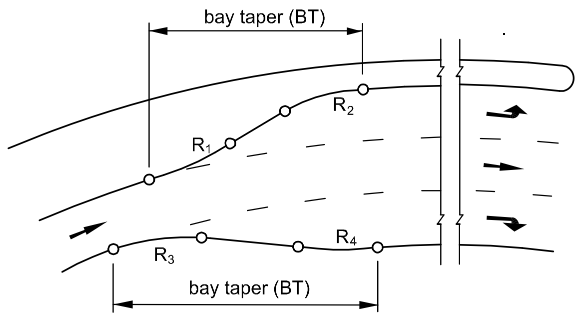
- collision rate comparisons between dual and triple left-turn lane installations
- lane utilization
- determination of saturation flow rates for triple left-turn lanes
- a comparison of left-turn capacity among single, double and triple turn lanes
- the effects of downstream weaving on the uniform loading of triple left-turn storage bays and intersection left-turn capacity[19]

2.3.8.7 Slot Left-Turn Lanes

General

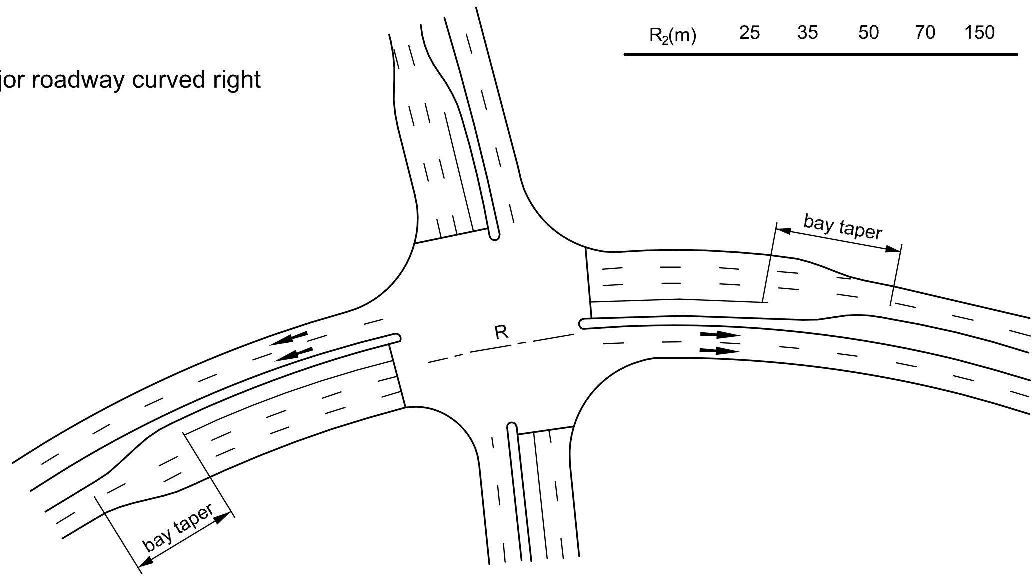
Slot left-turn lanes may be provided at intersections along major arterial roads or expressways wherever a median of about 10.8 m or more in width is available. This width is needed to accommodate a divisional island between the left-turn lane and the adjacent through lane. Typical designs are shown on Figure 2.3.8.16. The major advantages are:

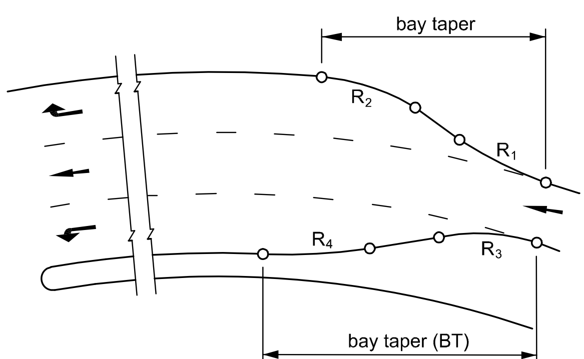
Figure 2.3.8.18 Intersections on Curve



turn lane controls (for 60 km/h)					
R	100	250	350	600	⩾800
BT(m)	20	25	30	35	50
R_1(m)	30	55	80	105	150
R_2(m)	25	35	50	70	150

major roadway curved right





turn lane controls (for 60 km/h)					
R	100	250	350	600	⩾800
BT(m)	20	25	30	35	50
R_3(m)	25	35	50	70	150
R_4(m)	30	55	80	105	150

major roadway curved left

Note : Bay taper length and radii are based on a design speed of 60 km/h example.

Figure 2.3.8.18 depicts shorter tapers with asymmetrical smoothing curves. The same criteria for adjusting taper lengths for curvilinear tapers can be applied to straight-line taper designs for turn lanes on the outside of main line curves. With the straight-line taper design, however, shortening the taper on the inside of the main line curve is not necessary. The start of the auxiliary lane on the inside of the curve remains distinctive due to the absence of the smoothing curve.

Alternatively, some jurisdictions lengthen straight-line tapers along the inside of main line curves to provide the same rate of lateral shift from the main line as that provided in a normal tangent section.

Superelevation

The maximum superelevation suggested through an urban intersection area is 0.04 m/m. This allows reasonably smooth operation for turning vehicles, especially those turning against the superelevation. The superelevation rate on an auxiliary lane at an intersection curve is normally the same as that of the through lanes. In restricted areas, it may be advantageous to reduce the superelevation along the auxiliary lane (as compared to the through lane) reflecting the lower running speeds of the turning vehicles on the auxiliary lane.

At intersections controlled with traffic signals, reduced superelevation rates may be considered along the curved roadway to improve the profile for the through vehicles on the cross roadway. For reduced superelevation rates, as related to radius of curve and design speed, based on maximum lateral friction factors see Chapter 2.1. It provides an alternate method of selecting superelevation rates based on lower maximum lateral friction factors, which provides greater protection against skidding during slippery pavement conditions.

2.3.9 TRANSITION BETWEEN FOUR-LANE ROADWAY AND TWO-LANE ROADWAY AT INTERSECTIONS

2.3.9.1 Undivided Roadways

The lane arrangement for the transition from a four-lane to two-lane roadway, and conversely from two-lane to four-lane roadway, is illustrated in Figure 2.3.9.1[1]. The typical taper lengths for diverging and merging values are shown in Table 2.3.9.1[1], as well as the design domain for parallel lane length ‘A’ beyond the intersection.

Special consideration is given to the merging operation by providing increased taper lengths, since it is recognized that merging is more critical when drivers, missing the warning signs, may be surprised by the sudden lane drop. Length ‘A’ is needed for signing purposes.

2.3.9.2 Divided Roadways

Principles similar to those used for undivided roadways are employed in the initial design stages of a divided control access roadway, see Figure 2.3.9.2[1].

Figure 2.3.9.1 Transition Between Undivided Four-Lane Roadway and Two-Lane Roadway at an Intersection[1]

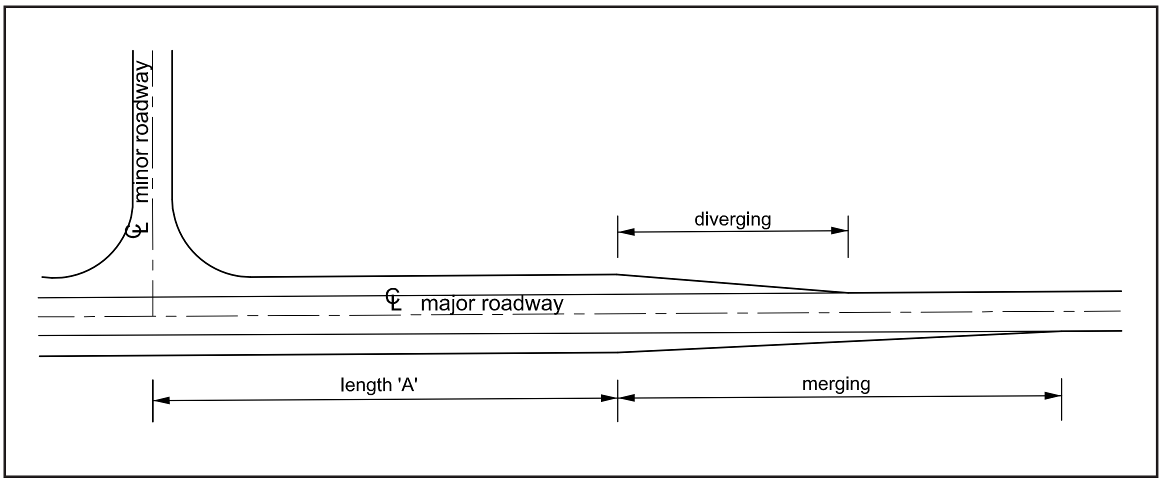


Table 2.3.9.1 Parallel Lane and Taper Lengths for Transition between Undivided Four-Lane Roadway and Two-Lane Roadway[1]

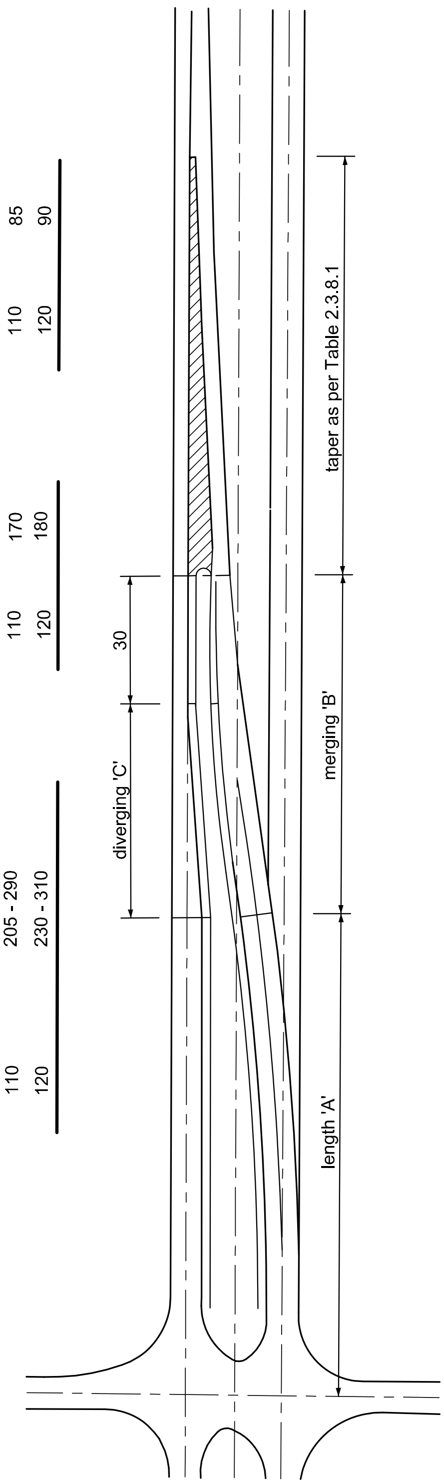
Design Speed (km/h)	Length ‘A’ Design Domain (m)	Merging Taper (m)	Diverging Taper (m)
50	80 – 150	85	40
60	100 – 175	100	50
70	120 – 195	115	60
80	140 – 215	130	70
90	160 – 240	145	75
100	180 – 265	160	80
110	205 – 290	170	85
120	230 - 310	180	90

Figure 2.3.9.2 Transition between Four-Lane Divided and Two-Lane Roadway Merge[1]

length 'A'	
design speed (km/h)	design domain (m)
50	80 - 150
60	100 - 175
70	120 - 195
80	140 - 215
90	160 - 240
100	180 - 265
110	205 - 290
120	230 - 310

merging 'B'	
design speed (km/h)	taper (m)
50	85
60	100
70	115
80	130
90	145
100	160
110	170
120	180

diverging 'C'	
design speed (km/h)	taper (m)
50	40
60	50
70	60
80	70
90	75
100	80
110	85
120	90



2.3.11 MEDIAN OPENINGS

2.3.11.1 Use and Function

Openings in medians perform one of the following functions:

- accommodate cross traffic and left-turn movements at intersections
- permit left turns for access to adjacent development at locations other than intersections (such development normally has significant traffic generation or other special characteristics)
- permit U-turns on divided roadways at locations other than intersections

The use and function of median openings are related to road classification.

Median openings are generally not provided along freeways except for accommodation of U-turns for emergency and maintenance vehicles. Along major divided arterials with fully controlled access, median openings are normally provided only at the intersections. For divided arterials where the provision of access to adjacent developments is only partially controlled, median openings may be provided at entrances to major developments, such as shopping centres, which generate significant traffic volumes (refer to Chapter 3.2). The provision of median openings may also be considered for other developments, such as shipping terminals, which generate significant volumes of large trucks. Such vehicles are typically discouraged from making U-turn movements within the road network and median openings may be a preferred means of providing access where other alternatives are not possible. The spacing of the median openings normally conforms to the intersection spacing guidelines provided in Section 2.3.1.

Where intersections along divided arterials are widely spaced and access to adjacent development is permitted, median openings that safely accommodate only U-turn movements may be desirable to improve access.

In certain circumstances, median openings or crossings may be provided to allow emergency vehicles to exit from adjacent fire, police and ambulance facilities, or to make turns that are otherwise prohibited at intersections. On divided arterial roads with widely spaced intersections and controlled access, median openings to permit only emergency vehicles to make U-turn movements may also be desirable.

2.3.11.2 Elements of Design

The design of a median opening and median ends should be based on traffic volumes, adjacent land use and the type of turning vehicles, as discussed in Chapter 1.2. Crossing and turning traffic should operate in conjunction with the through traffic on the divided roadway. As such, it is necessary to know the volume and composition of all movements occurring simultaneously during the design hours.[12] The design of a median opening becomes a matter of considering what traffic is to be accommodated, choosing the design vehicle to use for layout controls for each through and turning movement, investigating whether larger vehicles can turn without undue encroachment on adjacent lanes, checking the intersection for capacity, and evaluating the potential for operational problems related to undesirable driving behaviour. If the capacity is exceeded by the traffic load, the design should be expanded, possibly by widening or otherwise adjusting widths for certain movements.

The length of the median opening, measured nose to nose, is normally a function of the median width, the turning paths of the design vehicles making the left turn from and to the crossing roadway, and the location of the pedestrian crossings. If U-turns are permitted at the median opening, additional length may be required to avoid conflicts. The bullet nose design is advantageous in reducing the length of the median opening, thereby bringing the raised median end in close proximity to the pedestrian crossing and making it available for refuge.

The minimum lengths of median openings needed to accommodate varying turning radii and median widths should be determined based on the design vehicle (see Chapter 1.2).

The minimum length of the median opening, in all cases, should be 12.0 m, or the width of the crossing roadway measured between the outside edges of pavement plus 3.0 m, whichever is greater.

See Chapter 3.1 for discussion and further references on median barriers, end treatments and transitions.

2.3.11.3 U-Turns

Median openings designed to accommodate vehicles making U-turns are needed on some divided roadways in addition to openings provided for cross- and left-turning movements. The locations for separate U-turn median openings are as follows:

- beyond an intersection for accommodating minor turning movements not otherwise provided for in the intersection
- ahead of an intersection where through and other turning movements would be interfered with by U-turn movements at the intersection
- at regularly spaced openings to accommodate maintenance and emergency vehicle operations

Unless the median is wide, U-turning vehicles interfere with through traffic by encroaching on part of the through traffic lanes. U-turns are made at low speeds and the required speed change normally is made on the through traffic lanes. Moreover, U-turns often require weaving to and from the other lanes of the divided roadway. Allowing U-turns across narrow medians where through traffic flow may be impeded is undesirable and may create safety concerns.

The provision of median openings specifically for U-turns is effective along divided roadways where intersections are widely spaced and access to adjacent developments is permitted. This provision may be particularly important where U-turns are selectively restricted or totally prohibited by law at the signalized intersections. Median openings for U-turns reduce circulation on the adjacent local road system and improve access to adjacent developments by simplifying the manoeuvres necessary to reverse direction.

Normally, protected left-turn lanes are provided in advance of the median opening for the U-turn manoeuvre. In areas with low traffic speeds and low U-turning volumes, the U-turn movement may be permitted from the through lane. In these limited cases, the median is normally of sufficient width to allow a single vehicle to stop in the median opening without encroaching into the paths of the through traffic.

Table 2.3.11.1 provides the minimum median widths required for various design vehicles to make three types of U-turn manoeuvres on a divided road: auxiliary lane to inner lane; auxiliary lane to outer lane, four-lane divided roadway; auxiliary lane to outer lane, six-lane divided roadway. The minimum median widths for the latter two cases are based on through lane widths of 3.7 m. As demonstrated by the table, the median widths needed to accommodate the turning paths of trucks are beyond the typical widths available in urban areas. In special cases, where truck drivers must be able to reverse their direction, other alternatives such as jug handle left turns may be considered.

2.3.11.4 Emergency and Maintenance Vehicle Crossings

There are some locations where it is desirable to allow only emergency and maintenance vehicles to cross the median, including:

- at fire, police or ambulance facilities along divided roadways
- along divided roadways with fully controlled access and widely spaced intersections, and where median openings for all vehicular traffic are not desirable
- at intersections where it is desirable, from a traffic operations perspective, to restrict

intersection), although complete reliance should not be placed on the roundabout alone to act as an indicator to drivers

- emphasize the transition from a rural to an urban or suburban environment
- accommodate very sharp changes in route direction which could not be achieved by curves, even of undesirable radii
- provide a greater measure of safety at sites with high rates of right-angle, head-on, left/through, and U-turn collisions
- replace existing all-way stop control
- accommodate locations with low or medium traffic volumes, instead of signals

Roundabouts should be sited on level ground preferably, or in sags rather than at or near the crests of hills because it is difficult for drivers to appreciate the layout when approaching on an up gradient. However, there is no evidence that roundabouts on hill tops are intrinsically dangerous if correctly signed and where the visibility standards have been provided on the approach to the yield line. Roundabouts should not normally be sited immediately at the bottom of long descents where the down grade is significant for large vehicles and loss of control could occur.

Roundabouts may not be effective when the flow of heavy vehicles is great or long delays on one approach exists.

2.3.12.4 Geometry/Road Capacity

As noted above, roundabouts can improve road safety and increase capacity. Table 2.3.12.1 provides a summary of the relationship between geometric parameters and capacity.

Capacity is very sensitive to increases in the approach width V. This is normally the half width of the approach roadway and can only be increased if sufficient roadway width allows the centreline to be offset.

The entry geometry is defined by the entry width E and the flare length l^l. Capacity is extremely sensitive to increase in either, and considerable scope exists for increasing capacity by various combinations.

Increasing the entry radius R above 20 m only improves capacity very slightly. However, as values drop below 15 m capacity reduces at an increasing rate.

The entry angle '∅'is fixed by the alignment of the approach roadways and there is, therefore, little scope for varying '∅' sufficiently to have a significant effect on capacity.

When designing a roundabout the approach width is a known fixed value. The capacity is thereafter almost totally determined by the entry width and the flare length, as typical values of the other geometric parameters have only a minor influence.

Reducing the inscribed circle diameter reduces capacity. If, however, by reducing the inscribed circle radius an increase in the entry geometry can be achieved, then a large net increase in capacity is produced; mini roundabouts (diameter less than 4 m) are the limiting case. As the entry width increases, the entry deflection is reduced and consideration should be given to safety.

Increasing the number of entry lanes or increasing the width of these lanes has the potential for increased traffic conflict. Widening entry lanes is a concern for the safety of cyclists.

Table 2.3.12.1 Geometry/Capacity Relationships

Increase Parameter		Capacity Change
the approach width	V	rises rapidly
the entry width	E	rises rapidly
the flare length	l^l	rises slowly
the entry angle	∅	drops slowly
the inscribed circle diameter	D	rises slowly
the entry radius	R	rises slightly

2.3.12.5 Safety Analysis

Recent research in Europe has shown that collision rates can be decreased by replacing conventional intersections with roundabouts. The Netherlands achieved a 95% reduction in injuries to vehicle occupants at locations where roundabouts were installed.[20] On inter-urban roads in France, the average number of collisions resulting in injuries was 4 per 100 million vehicles entering roundabouts, compared with 12 per 100 million vehicles entering intersections with stop or yield signs. The safety of roundabouts, installed mostly in France's urban and suburban areas, including residential areas, was generally superior to that of signalized intersections.[20] Researchers noted that large roundabouts with wide entries and heavy bicycle traffic appeared to be less safe than other roundabouts. In Germany the number of collisions was 1.24 per 1 million vehicles entering small roundabouts, compared with 3.35 for signalized intersections, and 6.58 for old traffic circles.[20] In Norway an extensive collision analysis also revealed that roundabouts are safer than signalized intersections. The number of collisions resulting in injuries was 3 per 100 million vehicles entering three-legged roundabouts and 5 per 100 million vehicles entering signalized three-legged (T-) intersections; it was 5 for four-legged roundabouts and 10 for four-legged (cross-) intersections (with and without signals).[20]

In the United States, a recent study confirms the safety benefits of roundabouts. An investigation of six sites in Florida, Maryland and Nevada revealed that the conversion of T- and cross-intersections (stop controlled and signalized) to roundabouts decreased collision rates.[20] According to the study, which was sponsored by the Federal Highway Administration, the reduction was statistically significant.

Given that roundabouts have only recently begun to appear in North America, roadway agencies have had little opportunity to gather empirical data on the safety benefits of the structures. Fortunately, similarities between collision-prediction models developed in the United Kingdom for roundabouts and those developed in the United States for cross-intersections allow agencies to compare in theory the safety of both types of intersections. Both the U.K. and the U.S. models yield estimates of collisions resulting in nonproperty damage.[20] In addition, both models use state-of-the-art regression analysis (Poisson and negative binomial) and samples of sufficiently large size to relate collisions to particular roadway characteristics. On the basis of these similarities, one could draw the conclusion that roundabouts in the United States have the potential to increase safety when compared with conventional intersections, just as they are projected to do in the United Kingdom.

Nevertheless, notwithstanding their good record, great care should be taken in layout design to secure the essential safety aspects. The most common problem affecting safety is excessive speed, both at entry or within the roundabout. The most significant factors contributing to high entry and circulating speeds are:

- inadequate entry deflection
- a very acute entry angle which encourages fast merging manoeuvres with circulating traffic
- poor visibility to the yield line
- poorly designed or positioned warning and advance direction signing
- "Reduce Speed Now" signs, where provided, being incorrectly sited
- more than four entries leading to a large configuration

Additionally, safety aspects to be considered in designing a layout include the following:

1. Angle between legs: The collision potential of an entry depends on both the angle counter clockwise between its approach leg and the next approach leg, and the traffic flows. A high-flow entry should have a large angle to the next entry, and a low-flow entry

2.4.3 INTERCHANGE LOCATION AND SPACING

Rural freeways passing close to or through communities require interchanges suitably located to serve the needs of the community. It is sufficient to provide one interchange for small communities; larger communities require more. The precise location depends on the particular needs of the community; however, as a general guide, interchanges should be located at arterial roads recognized as major components in the road system, having good continuity and a capability for expansion if required. Interchanges should be located in the proximity of major development areas; for example, central business areas and areas of existing or future concentrations of commercial or industrial development. As a general guide in rural areas, interchanges are normally spaced at between 3 km and 8 km.

On urban freeways, traffic conditions and driver behaviour and expectations are different from those of rural freeways, and this influences interchange spacing. Operating speeds tend to be lower, trip lengths shorter, traffic volumes higher, and drivers are accustomed to, and anticipate the need for taking a variety of alternative actions in rapid succession. Interchanges spaced at more than 3 km over a length of urban freeway normally cannot provide adequate service to urban development, and closer interchange spacing is called for. If successive interchanges on urban freeways are too close, however, the operation of the freeway might become impaired and the freeway loses its capacity to collect and deliver traffic from the crossing arterial roads.

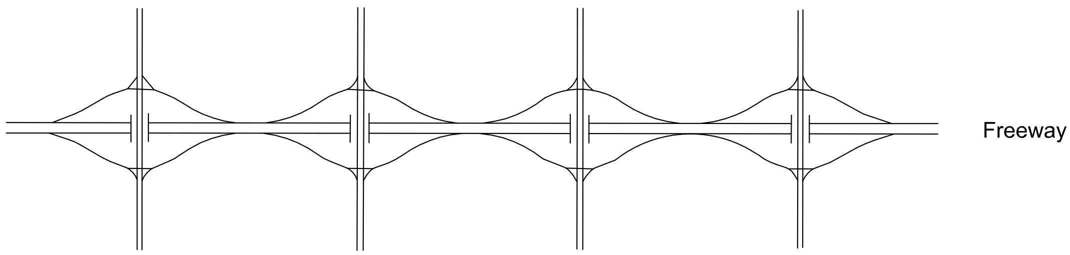
Interchange spacing in urban areas generally ranges from 2 km to 3 km. Interchanges should be located at major arterial roads, forming part of the arterial system of roads for the urban area and providing, or having the potential to provide, capacity to deliver to and collect from the interchanges.

Minimum spacing of interchanges is determined by the distance required for weaving (see Chapter 2.1), speed change lanes, and the appropriate placement of directional signs.

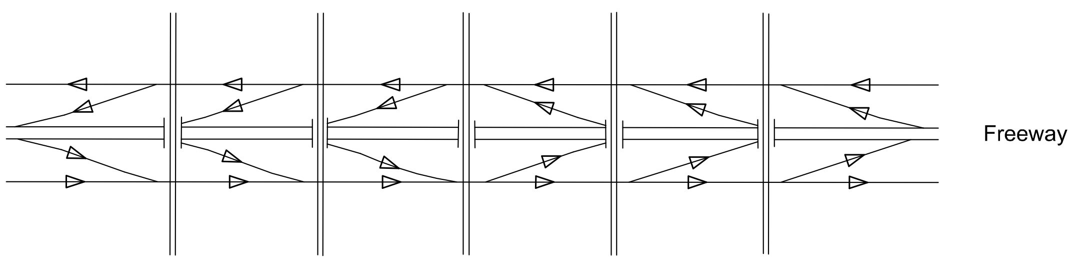
If the arterial roads are spaced closer than 2 km, it is usually necessary either to omit some of the interchanges in favour of grade separations alone or adopt some alternative means of combining interchanges to serve closely located arterial roads. Figure 2.4.3.1 illustrates how this might be done. In the upper diagram the arterial roads are spaced at 2 km to 3 km, allowing each arterial to be served by its own interchange. In the lower three diagrams the arterial roads are at less than 2 km, calling for some form of combined ramp / service road system to provide the overall interchange ramp capacity to serve the arterial system. The most suitable configurations of ramps are very much site specific, and the design is dependent on the layout of the arterial network and the particular needs of the community it serves.

Freeway collision rates tend to increase as interchange spacing decreases in urban areas.[7] This effect on collision rates is an important consideration in urban area interchange spacing.

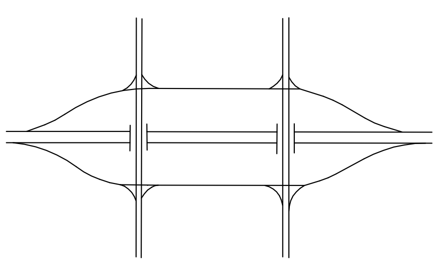
Figure 2.4.3.1 Interchange Spacing on Urban Freeways



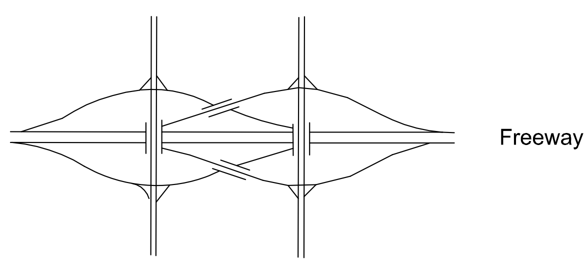
arterial roads at 2 km to 3 km spacing, each served by an interchange



arterial roads at less than 2 km spacing, served as a group by ramp nesting



arterial roads at less than 2 km spacing,
served by a combined interchange



arterial roads at less than 2 km spacing,
served by graded separated ramps

2.4.5 INTERCHANGE TYPES

2.4.5.1 General

There is a wide variety of interchange types available to the designer. The classification of the intersecting roads is a prime determinant in the selection of the most suitable interchange type for any particular application. Subsection 2.4.5.2 deals with interchanges between roads classified as freeways, either four-leg or three-leg. Subsection 2.4.5.3 discusses interchanges between freeways and other roads, which are normally arterial roads but in some cases are collector roads. Subsection 2.4.5.4 discusses suitable types for interchanges between roads, neither of which is a freeway or an expressway. Such applications are normally between two arterial roads or an arterial road and a collector road. In rare instances, there is an application for an interchange between an arterial road and a local road.

Most interchanges provide for all movements between intersecting roads. Interchanges that provide a limited number of movements are referred to as partial interchanges. For any movement provided for in a partial interchange, the corresponding return movement desirably should be available, since the driver expects to be able to retrace his route in the return direction on any particular trip. The absence of the return movement may even create a safety concern if a frustrated motorist attempts a “wrong way” movement to regain access to a freeway.

The selection of the most suitable interchange for any particular application, and the details of its design, depend on a number of controls and other considerations, among the most important of which are:

- safety
- functional and design classification of intersecting roadways
- adjacent land use
- design speed
- traffic volume and traffic mix
- number of interchange legs
- traffic control devices
- topography
- right of way and property requirements
- service to adjacent communities
- systems considerations and design consistency
- environmental considerations
- economics

The relative importance of these controls and considerations varies between interchanges. For any particular site, each control is examined and its relative importance assessed. Alternative types and configurations are then studied to determine the most suitable in terms of the more important controls. While the selection of the best interchange type may vary between sites, it is important to provide regional consistency, where possible, in order to reinforce driver experience. This would in turn improve driver expectancy and hence safety.

2.4.5.2 Interchanges Between Freeways

Interchanges between two freeways are normally the most costly in terms of construction cost and property requirements.

Since freeways are fully-controlled access facilities, at-grade intersections within the interchange configuration are inappropriate and it is mandatory that they be avoided.

Fully directional interchanges provide for right and left turns through large radius ramps having design speeds in the order of 70% to 80% of freeway design speeds and having overall deflection angles in the order of 90^{o}. Partially directional interchanges provide for some left-turn movements by means of loop ramps, which have lower design speeds. Partially directional interchanges have applications where there are property limitations, or where some left-turn volumes are low.

Four-Leg Interchanges

Figure 2.4.5.1 illustrates fully-directional and partially-directional four-leg interchanges between freeways. The fully-directional type shown in illustration (i) provides single exits from all four directions and directional ramps for all eight turning movements. The through roads and ramps are separated vertically on four levels.

Partially-directional interchanges allow the number of levels to be reduced as the number of loop ramps is increased. The single-loop arrangement, illustration (ii), and two-loop arrangement in (iii) and (iv), require three levels. A configuration in which loop ramps are carrying the lighter volumes of left-turning traffic is preferable, and levels should be arranged so that exit loop ramps are on upgrades encouraging deceleration, thus increasing safety. Illustration (v) is a full cloverleaf with collector lanes, incorporating loops for four left turning movements and two levels separating the through roads vertically. This type of interchange introduces undesirable weaving sections, and is only suitable where left-turn volumes are low and property is readily available. The collector roads are added to this type of interchange so that the weaving manoeuvres inherent in this interchange type occur on the collectors rather than on the main line.

Three-Leg Interchanges

Figure 2.4.5.2 illustrates a variety of fully-directional and partially-directional three-leg interchanges between freeways.

Illustrations (i) and (ii) are fully-directional interchanges requiring three levels of roadways. Illustration (iii) is fully-directional, requiring only two levels. Illustrations (iv) and (v) are partially-directional referred to as trumpet interchanges and each incorporates one loop ramp for a left-turn movement. The choice between the two depends on the availability of property, but desirably the loop carries the smaller volume of the two left-turn movements.

2.4.5.3 Interchanges Between Freeways and Other Roads

In interchange types for this application, all ramps diverging and merging with the freeway have acceleration and deceleration lanes so that traffic can enter and exit freeway lanes at, or close to, freeway speeds. Where the ramps connect with the crossing roads, intersections occur and are designed with suitable traffic control devices.

Left-turn movements often are accommodated on loop ramps, and loop ramps carrying traffic entering the freeway are preferable to loop ramps carrying traffic exiting the freeway. Where other considerations permit, it is desirable to arrange the levels of the through roads so that ramp traffic entering the freeway is on a downgrade and ramp traffic exiting the freeway is on an upgrade, to assist in acceleration and deceleration respectively.

The choice of carrying the freeway over or under the crossing road depends on topography, cost and other environmental considerations. There are a number of operational advantages in carrying the freeway under the crossing road as discussed in Subsection 2.4.1.2.

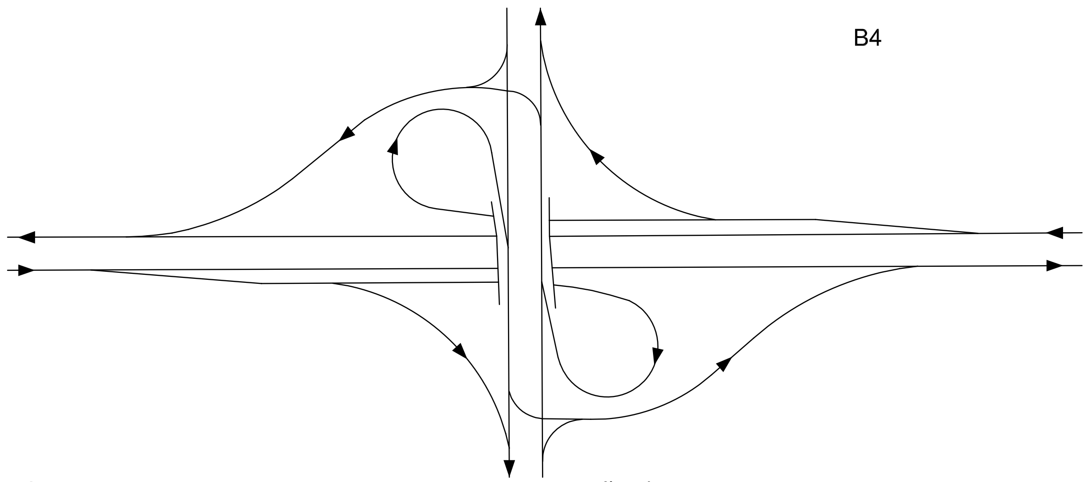
Interchanges between freeways and other categories of crossing road normally provide for all turning movements. In some cases it may be necessary to eliminate some turning movements; however, in such cases, for any movement that is provided the corresponding return movement desirably should be available.

Figures 2.4.5.3 to 2.4.5.6 illustrate interchanges for application between freeways and arterial roads. They are shown diagrammatically with the arterial road crossing over the freeway and with single exit ramps from the freeway.

Diamond Interchanges

Figure 2.4.5.3 illustrates a Simple Diamond interchange. The ramps intersect with the crossing road at at-grade intersections controlled by traffic signals or, where volumes

Figure 2.4.5.5 Parclo B Interchanges

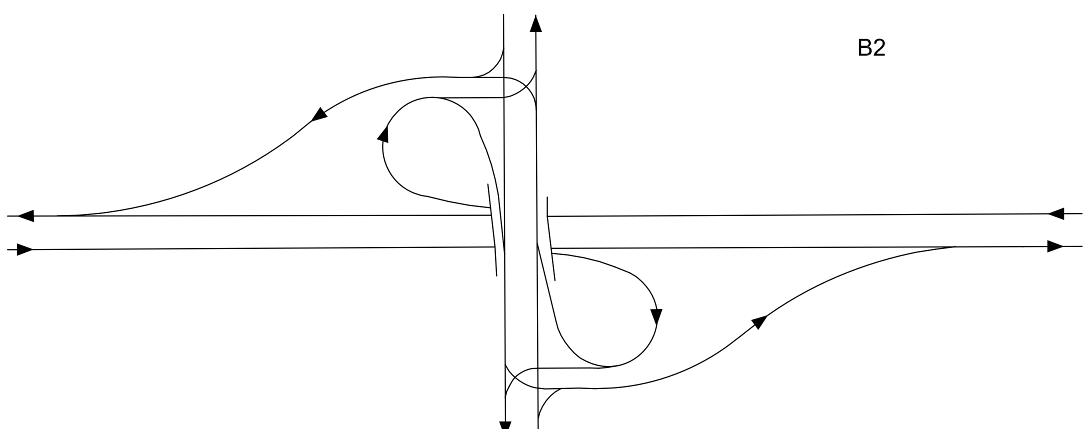


advantages

- single exits in advance of structure (if collectors provided)
- single exit feature simplifies signing of freeway (if collectors provided)
- not conducive to wrong way movements
- high capacity
- exiting traffic does not have to stop

disadvantages

- high speed traffic from the freeway must exit on loop ramps
- loop ramp hidden on the far side of structure



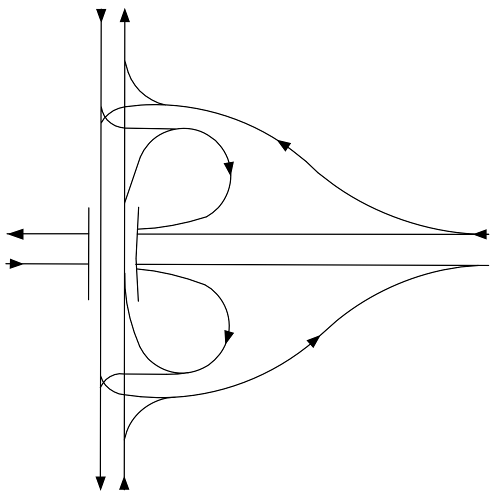
advantages
(compared with parclo B4)

- reduced property requirement
- lower construction cost
- can be used as initial stage of B4

disadvantages
(compared with parclo B4)

- reduced capacity
- right turn traffic from the freeway must exit on loop ramp

Figure 2.4.5.6 Parclo AB, Trumpet and Rotary Interchanges



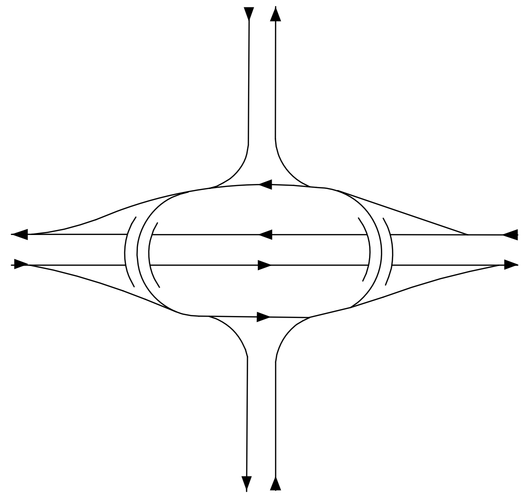
parclo AB

advantages

- requires property on one side of the crossing road only

disadvantages

- short weaving section on crossing road
- limited capacity



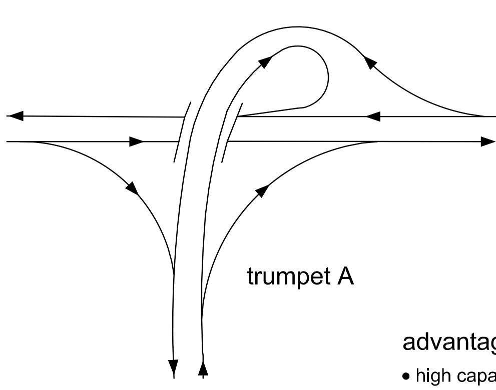
rotary

advantages

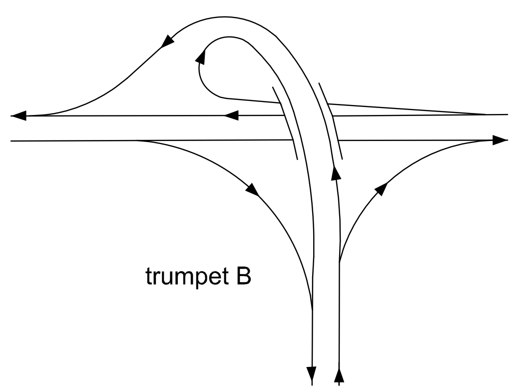
- no intersections

disadvantages

- the weaving sections limit the speed and capacity
- through crossing road traffic interrupted
- complex signing
- not suitable for future expansion
- requires two wide structures
- not in common use, hence low driver familiarity



trumpet A



trumpet B

advantages

- high capacity
- no intersections

disadvantages

- loop ramp traffic off high speed road (trumpet B)

Exit Terminal Length

Table 2.4.6.2 shows design domain for lengths of deceleration lanes in single-lane exits. For parallel lane form, the length of deceleration lane, L_d is measured from the end of the taper (L_t). For the direct taper form, the length L_d is measured from the point at which the auxiliary lane is 3.5 m wide.

The length of deceleration, L_d, is measured to the beginning of the ramp controlling curve. If the ramp is relatively straight and ends at a stop condition, as in a diamond interchange, the length L_t may be measured to the intersection. Measurement of L_t and L_d are illustrated in Figure 2.4.6.1.

The length of an exit terminal is essentially determined by the distance required for deceleration after the vehicle has left the through lanes. It is based on three factors in combination as discussed in Subsection 2.4.6.2. They are:

- the running speed on the through lanes
- the control speed of the ramp proper
- the manner of deceleration

The design domain in Table 2.4.6.2 is based on the different assumptions applied to these three factors:

1. The running speed on the through lanes is assumed to be in the range of "operating speeds" used in Chapter 1.2.

2. The control speed of the ramp proper at the downstream of the exit terminal is included in the range of ramp design speeds in Table 2.4.6.1. The speed change lane lengths corresponding to the ramp control speeds, which are governed by the design speed of the turning roadway curve, are shown in Table 2.4.6.2.

3. The manner of deceleration includes a domain of two different concepts. For both concepts, it is assumed that drivers travel at the beginning of the speed change lane (start of taper) at the operating speed and the speed is maintained until the end of the taper in the case of the parallel lane form or until the speed change lane has widened to 3.5 m in taper form. The first concept, which provides the basis of the lower values of the design domain, assumes braking begins at the start of the parallel section (parallel lane form) or at the 3.5 m wide point (taper form) to decelerate to the control speed of the ramp. The second concept, which provides the basis of the higher values of the design domain, assumes that the vehicle travels for 2.0 seconds to 4.0 seconds in gear without braking followed by leisurely braking to the control speed of the ramp. This latter condition is considered very generous, particularly in an urban situation. Two seconds in gear is assumed for roadway design speeds up to 110 km/h; while 4.0 seconds in gear is assumed for roadway design speeds of 120 km/h and higher.

For two-lane freeway exits, the operation is different from the single-lane exit. This operation may require a longer speed change lane than that for the single lane exit. The length is based on a different concept whereby the exit curve to the bullnose is considered to govern the manoeuvre of the vehicle exiting to the left-hand lane of the two lanes. A total speed change length (L_d) in the order of 400 m to 450 m for mainline roadway speeds of 100 km/h or above is recommended by both AASHTO and the Geometric Design Standards for Ontario Highways for two-lane exits.

Where deceleration lanes are on grades equal or steeper than 3%, the length shown in Table 2.4.6.2 should be adjusted by the appropriate grade factor shown in Table 2.4.6.3.

Exit Ramp Transition Curve Criteria

Interchange ramp alignments with their relatively severe geometrics are particularly appropriate for the use of spirals. A spiral curve beginning in the vicinity of the bullnose is normally inserted between the exit curve or taper ending at the bullnose, and the controlling curve on the exit ramp as shown in Figure 2.4.6.2.

Table 2.4.6.3 Grade Factors for Speed Change Lanes

DECELERATION LANES		
Grade	**Design Speed of Roadway (km/h)**	**Grade Factor* For All Turning Roadway Design Speeds**
6% to 5% up		0.8
3% to 5% up		0.9
less than 3%	all	1.0
3% to 5% down		1.2
5% to 6% down		1.4

ACCELERATION LANES						
Grade	**Design Speed of Roadway (km/h)**	**Grade Factor* For Turning Roadway Design Speeds**				
		40	**50**	**60**	**70**	**80**
6% to 5% up	60	1.5	1.5			
	70	1.5	1.6	1.8		
	80	1.6	1.7	1.9	2.0	
	90	1.7	1.8	2.0	2.2	
	100	1.9	2.0	2.2	2.4	2.6
	110	2.0	2.2	2.4	2.7	2.9
3% to 5% up	60	1.3	1.3			
	70	1.3	1.3	1.4		
	80	1.4	1.4	1.4	1.5	
	90	1.4	1.5	1.5	1.5	
	100	1.5	1.5	1.6	1.6	1.6
	110	1.5	1.6	1.7	1.7	1.8
		Grade Factor* For All Turning Roadway Design Speeds				
less than 3% (up or down)	all	1.0				
3% to 5% down	60	0.7				
	70	0.7				
	80	0.65				
	90	0.6				
	100	0.6				
	110	0.6				
5% to 6% down	60	0.6				
	70	0.6				
	80	0.55				
	90	0.5				
	100	0.5				
	110	0.5				

Note: * grade factor = ratio of length on grade tolength on level (as shown in Tables 2.4.6.2 and 2.4.6.5) ramp curve to the point at which the auxiliary lane is 3.5 m wide. Acceleration lengths, L_a, are measured from the end of the ramp controlling curve. Measurement of L_t and L_a is illustrated in Figure 2.4.6.1.

The length of an entrance terminal is based on three factors in combination as discussed in Subsection 2.4.6.2. They are:

- merging with the through traffic
- control speed of the ramp proper
- manner of acceleration

The merging speed with the through traffic is assumed to be the range of operating speeds used in Chapter 1.2.

The control speed of the ramp proper at the upstream of the entrance terminal is included in the range of ramp design speeds in Table 2.4.6.1. The speed change lane lengths corresponding to the ramp control speeds, which are governed by the design speed of the turning roadway curve, are shown in Table 2.4.6.5.

The manner of acceleration assumes an increase of speed based on the acceleration of passenger cars tested in the U.S.[3,14]

Where acceleration lanes are on grades steeper than 3%, the length shown in Table 2.4.6.5 should be adjusted by the appropriate grade factor in Table 2.4.6.3.

The length of an entrance terminal also depends on the relative volumes of through and entering traffic. Longer entrance terminals (i.e. the higher values of the design domain in Table 2.4.6.5) are desirable on higher volume roads to enable entering traffic to merge with through traffic safely and conveniently.

Trucks and buses require longer acceleration lanes than passenger cars. Where a substantial number of large vehicles entering the road is expected, longer acceleration lanes are appropriate.

Research has shown that safety can be increased on acceleration lanes with increased length, especially for high speed facilities.

Where the entrance terminals occur on a crest curve, sight distances to the lane drop may be affected, longer acceleration lanes will then be required as discussed later in the 'Sight Distance' Subsection.

<u>Entrance Ramp Transition Curve Criteria</u>

In entrance ramp design, a spiral is introduced in the vicinity of the bullnose between the ramp controlling curve and the entrance curve or taper to effect a smooth transition. Acceleration starts at the beginning of the spiral, and usually continues beyond the spiral on the ramp terminal section. The radius of the spiral increases so as to accommodate the increasing speed of the vehicle accelerating. The spiral parameter needs to be small enough to provide a sufficiently rapid rate of increase in radius to match the acceleration of the vehicle. On the other hand the spiral parameter needs to be sufficiently large to ensure that the comfort, superelevation and aesthetic criteria are met. The spiral parameter, can be selected from the range given in Table 2.4.6.6.

Table 2.4.6.6 Spiral Parameter for Entrance Ramp Transition Curves

Ramp Controlling Curve Speed (km/h)	Design Domain of Spiral Parameter (m)
40	50-80
50	65-130
60	85-140
70	110-280
80	125-360

<u>Sight Distance at Entrance Terminals</u>

At entrance terminals, the driver is looking for a gap in the traffic in the adjacent lanes in order to effect a lane change and merge. A driver therefore has to look back to find an appropriate gap. This view is best provided by maintaining the vertical alignment of the ramp in the vicinity of the nose at elevations similar to, or above, those of the through road. If the ramp is significantly lower, the driver might have some difficulty effecting a safe merge. If the ramp is higher, the driver normally has good visibility unless the view is obscured by a traffic barrier or other visual obstruction.

A driver begins accelerating from the ramp controlling circular curve some distance before the nose, usually in the vicinity of the beginning of the spiral curve. At this point, the driver looks for a gap in the stream of traffic in the adjacent lane. The line of sight is taken to be at 120^{o} from the direction of travel and the object to be seen is taken to be in the centre of the adjacent lane, 1.0 m above the pavement surface. This is illustrated in Figure 2.4.6.4.

To allow the driver to make a merging manoeuvre safely, ideally he requires a view of the entire speed change lane at the nose as illustrated on Figure 2.4.6.4. The driver may not have this view if the speed change lane occurs on a crest curve, in which case the vertical alignment should be adjusted so as to shift the crest curve away from the speed change lane. If this is not feasible, either the speed change lane should be lengthened or the crest curve should be flattened to provide, preferably, decision sight distance to the end of the taper as discussed in Chapter 1.2.

2.4.6.5 Ramp Terminal Spacing

Successive ramp terminals on freeways/ expressways or within an interchange are spaced to allow drivers to make decisions in sufficient time to make safe manoeuvres.

In the case of successive exits, the distance is based on the provision of adequate signing. In the case of successive entrances, the length is based on the merging manoeuvre length required for the first entrance.

An entrance followed by an exit terminal creates a weaving condition, and is discussed in Chapter 2.1.

The distance between an exit followed by an entrance needs to be sufficient to allow a vehicle on a through lane to prepare for the merge ahead after passing the exit nose.

Figure 2.4.6.5 shows the minimum values for ramp terminal spacing based on design speeds. Additional distances may be required to ensure signing requirements are met.

2.4.6.6 Safety and Design Overview

The fundamental principle of an interchange is the movement of vehicles through the interchange in the safest, most efficient manner possible. The ability of an interchange to accommodate drivers in this manner is closely related to the efficiency with which the information is provided to the driver and with the degree to which driver expectancy is met at the interchange.

Interchanges present the motorist with a complex set of decisions that require quick evaluation and action. Designers can reduce drivers' stress at interchanges by keeping the alignment simple and direct, maintaining design consistency, providing sight distances greater than the minimum stopping sight distances, and using above minimum design criteria for other geometric elements.

Collisions on ramps and connecting roads generally increase with traffic volume and with decreasing curve radius.[7] It also appears that upgrade exit ramps have lower collision rates and thus it is preferable, from a safety view point, for the connecting road to pass over the freeway or higher speed road. The use of collector lanes for high volume interchanges enhance safety, especially where loop ramps are used[7]. The use of a collector introduces an intermediate-speed facility between the freeway and the off-ramp thereby encouraging speed slow-down prior to entering the off-ramp.

One of the key issues related to interchange design involves heavy truck incidents at interchanges. In general, tight radius curves on ramps and short speed change lanes cause problems with heavy trucks. Truck incidents on interchange ramps generally involve loss of control leading to rollover or jack-knife. Recent

2.4.7 OTHER INTERCHANGE DESIGN FEATURES

2.4.7.1 Operational Analysis

A series of proposed interchanges or a proposed single ramp exit/entrance can be subjected to an operational analysis, including capacity and design features, after the planning or preliminary design stage. The operational analysis is basically a test for ease of operation and for route continuity from a driver's point of view, both of which are affected by the location, proximity, sequence of exits and entrances, merging and diverging movements, necessary weaving, practicability of signing, visibility of the target destination, and clarity of paths to be followed. The operational analysis can be considered as an application of the 'driver workload' approach to the evaluation of design consistency.

A route may be tested by isolating a single path of travel and examining it only with regard to other features of the layout that will affect a driver on the path being tested. The test can be made using an overall plan on which the number of traffic lanes, the peak hour volumes, the expected running speeds, the visibilities of downstream features, and the signing are shown.

The operational analysis indicates whether or not confusion is likely due to the close proximity of exits and entrances, or traffic conflict is likely because of weaving movements. It should illustrate also the clarity of the path and the feasibility of signing. The test may show that the path is easy to travel, direct in character, or it may show that the path is sufficiently complex and confronted with disturbing elements that require adjustments in design.

The operational analysis can also include a check of the design adequacy, e.g. deceleration length and ramp radius, in conjunction with the test for ease of operation. This is a particularly useful approach in checking the overall design of interchanges that have non-typical configurations.

2.4.7.2 Ramp Metering

Ramp metering consists of traffic signals installed on entrance ramps in advance of the entrance terminal to control the timing and number of vehicles entering the freeway. The traffic signal may be pre-timed or traffic-actuated to release the entering vehicles individually or in platoons.

The purpose of ramp metering is to reduce congestion or improve merge operations on urban freeways. This strategy has been proven effective in many applications throughout North America. Ramp metering can optimize freeway vehicle flows and improve merge area safety through uniform spacing of entering vehicles.

The ramp metering system can include the following:

- stop bar marking upstream of the ramp bullnose
- passage loop downstream of the stop bar to monitor vehicles released from the queue
- ramp control signs
- demand detector loops upstream of stop bar
- upstream queue detector on the ramp immediately downstream of the arterial street bullnose
- freeway detector loop on the upstream approach to determine available gaps

Provisions for ramp metering bypass lanes may be required for potential implementation of high occupancy vehicle priority features. The suitability of ramp configurations to accommodate bypass lanes hinges on the availability of required length along the ramp.

2.4.7.3 Bus Interface

Accommodating transit on a freeway can lead to a high level of transit service in larger cities. An interchange can provide a transfer point between freeway bus service and local bus feeder service and, potentially, commuter parking facilities.

Design considerations of bus interface at interchange includes the following:

- safety
- capital costs
- alignment and geometrics
- intended operation with respect to interface location, platform facilities and local bus facilities
- impact on crossing and bus road access /egress locations

Sufficient sight distance on the bus route is important for maintaining safe and efficient operation of the buses. Interfaces at interchanges should be located as closely as practicable to crossing roads of suitable geometrics and ridership potential. Pedestrian walk distances should be minimized between local and freeway buses.

On diamond-type ramps, the bus interface may consists of a widened shoulder area adjacent to the ramp roadway or may be on a separate road. Generally, bus interface provided on entrance ramp is preferred. Bus interfaces are more difficult to provide effectively within cloverleaf or directional-type interchanges. Guidelines[4] have been developed for use in preliminary design of Parclo A interchanges where bus interfaces are to be protected. Typical bus interfaces at Diamond interchange and at Parclo A interchange are shown in Figure 2.4.7.1.

2.4.7.4 Pedestrians

The accommodation of pedestrians through an urban interchange should be considered during the development of interchange configuration.

In addition to allowing the driver to detect the presence of pedestrians, adequate sight distance for the pedestrians must be provided where pedestrians are expected to cross an interchange ramp. Pedestrians must be able to perceive gaps in the traffic flow. At ramp crossings where there are insufficient gaps in the traffic flow to allow pedestrians to cross, pedestrian-actuated signals or a pedestrian overpass/underpass can be considered. In general, it is desirable to provide pedestrian crossings in the shortest distance feasible.

The accommodation of bicycles at interchanges is discussed in Chapter 3.4.

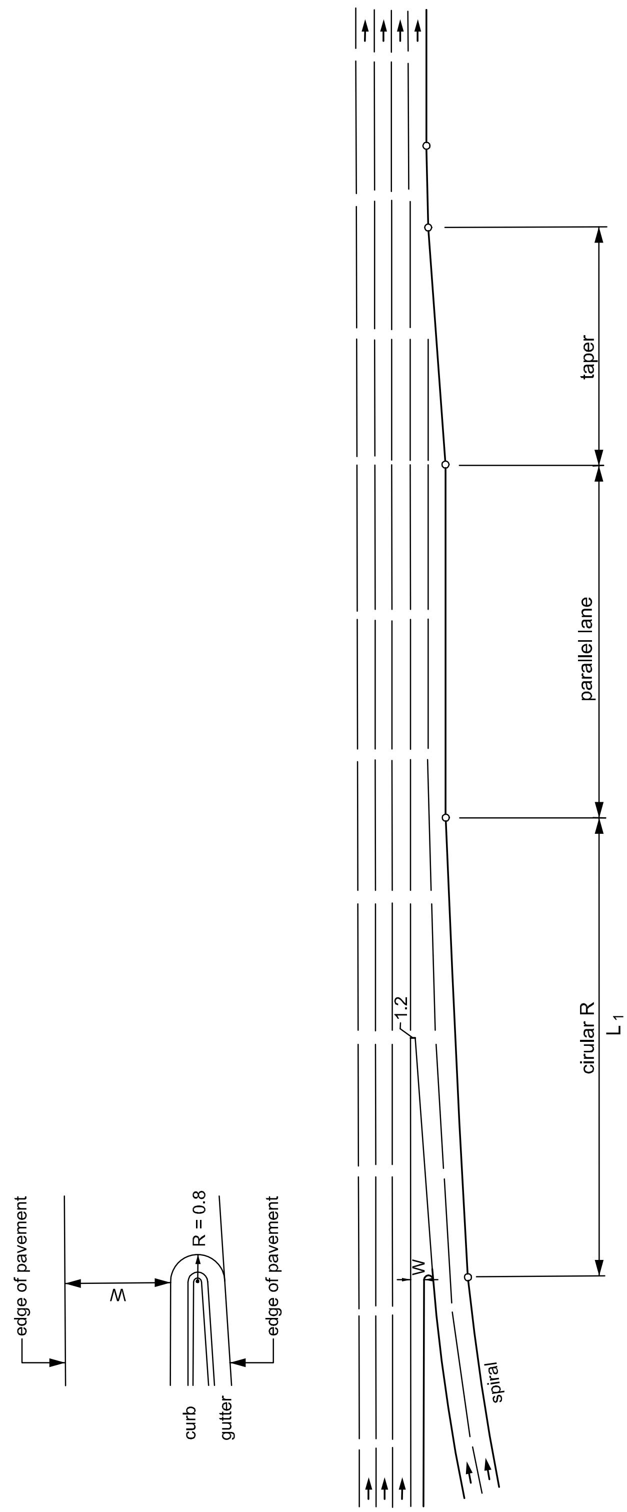
2.4.7.5 Grading and Landscaping Development

Grading at an interchange is determined mainly by the alignments, profiles, cross-sections and drainage requirements for the intersecting roads and ramps. Contour grading can be designed to increase safety and enhance aesthetics. Flat slopes should be used where feasible, to increase safety and to enhance the appearance of the area. V-ditches and small ditches with steep side slopes are usually avoided. Drainage channels and related structures should be as inconspicuous and maintenance-free as feasible. The need for a ‘forgiving’ roadside on the outside of curves is particularly important for tighter interchange ramps.

Contour design is usually applied to residual areas of land in interchange areas between ramps to create noise berms and for material disposal areas. These areas can be graded with varying slopes to give an undulating and natural looking appearance. Contour design is carried out in conjunction with drainage design with consideration for safety and, where appropriate, in conjunction with landscaping. Residual pockets of land in interchange areas, particularly loop ramps can be used to dispose of surplus material and to minimize spoil. Conversely, they may be used to generate additional excavation and to minimize borrow. In placing fill in loop ramp areas, care must be taken not to compromise sight distance requirements.

Grading of areas on the back slope can be contoured to match the adjacent topography and can greatly improve the aesthetics of the road. This can be accomplished, in part, by rounding the edge of the interface between the cut of slope and the adjacent hillside, and

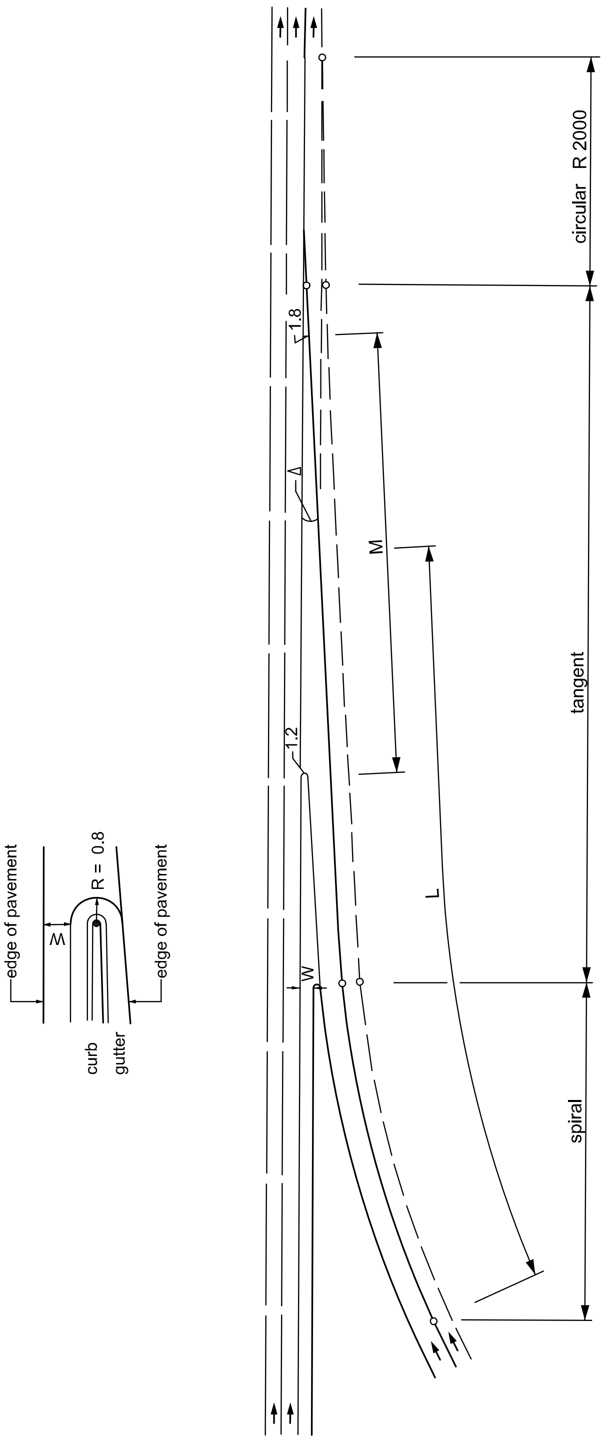
Figure 2.4.8.6 Typical Design Entrance Terminal Parallel Two Lane



class	main line design speed (km/h)	W Offset (m)	circular R R (m)	circular R L_1 (m)	parallel lane (m)	taper (m)
UAD	100	0	2000	135	210	85
	80	3.0	2000	135	25	70
RFD	100	3.0	3000	165	125	85
UFD	120	3.0	4000	190	280	95
	130	3.0	5000	215	335	100

UCU 60 AND UCU 80 values use tapered design, see Figure 2.4.8.7

Figure 2.4.8.7 Typical Design Entrance Terminal Tapered Single and Two Lane



class	main line design speed (km/h)	Δ	M (m)	W (m)	L (m) ramp design speed (km/h) 40	50	60	70	80
RCU RAD UAD	100 120	1°-30' 1°-15'	172 206	0.75 0.75	330 500	290 470	230 430	- 360	- -
RCU RAD UCU UAU UAD	60 80	2°-30' 2°-00'	103 129	0 0	60 170	20 140	- 90	- -	- -
RFD UFD	80 100 120 130	1°-30' 1°-15' 1°-00' 1°-00'	172 206 258 258	3.0 3.0 3.0 3.0	170 330 500 -	140 290 470 550	90 230 430 520	- - 360 460	- - - 380