3.1.2 EXPLICIT ANALYSIS OF ROADSIDE SAFETY FEATURES

3.1.2.1 What Does it Involve?

The design of the roadside environment is a complex problem. Evaluating alternative designs and choosing between them is a difficult task which involves degrees of uncertainty with respect to the occurrence of collisions, their outcomes in terms of severity, and the real costs of the property damage, injuries, and fatalities which can result. Nonetheless, as noted earlier, such analysis provides an explicit framework for considering design trade-offs - a much more desirable approach to roadside safety design than meeting arbitrary "standards" whose underpinnings may or may not be appropriate to a given situation. Such a framework is also a requisite foundation for the value engineering exercises which often form part of the road design process.

An explicit framework for roadside safety analysis must necessarily recognize local agency needs, policies and practices within the specifics of its approach. However, it is generally accepted that any such process will be built on two fundamental toolsets:

1. Predictive models which provide a way of estimating collision frequencies and severities under a wide variety ofconditions.

2. Cost-effectiveness models which provide a way of quantifying the life-cycle costs (and benefits) associated with any given set of safety measures.

Predictive models have been developed and deployed by a number of agencies in North America. While the latest AASHTO Roadside Design Guide[4] probably represents the most current and widely accepted effort in this regard, designers should be aware that the state of the art in this area is continually developing and should be monitored regularly for new models and techniques which may have application to their design challenges. The techniques presented in this Guide are founded in part on the AASHTO work, but also recognize a number of practices drawn, where possible, from the Canadian context.

The techniques of cost-effectiveness analysis are well established and are applied for a variety of purposes in transportation and highway design agencies. There are a number of alternative approaches that are available but most commonly, the tools used by transportation agencies are built on life-cycle costing models and use present worth or annualized cost techniques as their underlying analysis methodology. All of these approaches are built on fundamental assumptions regarding parameters such as discount rates and unit collision costs. In order to enforce consistent and comparable results across the transportation agency, these basic assumptions are usually set as a matter of policy and represent a "given" for designers to use in their analyses.

3.1.2.2 Overview of Collision Prediction Models

Predictive models are used to provide at least three levels of information to the designer:

1. Estimates of the numbers of encroachments – an errant vehicle leaving the roadway – likely to occur. Designers must recognize that these estimates are probabilistic in nature, in spite of the deterministic form which they often take and make appropriate provisions in their roadside designs for this fact. Additional discussion of this issue is provided later in this chapter.

2. Estimates of the number of collisions likely to occur as a result of the encroachments. Every encroachment does not necessarily result in a collision, since a number of vehicles will normally recover within a certain distance without incident. Critical factors used in these models include: the angle of departure from the roadway, the speed of the vehicles involved, and the type of vehicles involved. Again, the probabilistic nature of these models must be kept in mind.

3. Estimates of the severity of the collisions which occur. Once an estimate of the number of collisions which can be expected to occur at a given location is available, this is usually converted into an equivalent dollar cost through the use of a parameter such as a Severity Index (SI). This parameter usually varies with the speed and type of vehicle, the angle of incidence of the collision, and the type of object struck. Different scales are used by different agencies in estimating severity indices, however both AASHTO[4] and NCHRP[5] provide representative sets of these indices.

Additional discussion on these concepts, suggested models for their application, and worked examples of their use are provided in subsequent sections of this Guide.

3.1.2.3 Overview of Cost-Effectiveness Analysis

Transportation agencies have traditionally used cost-effectiveness analysis models to address many different types of investment decisions, including the analysis of site-specific alternative safety treatments. AASHTO, TAC, Provincial and State Transportation Agencies, and independent research efforts have all contributed to the state of knowledge in the use of these techniques. The engineering economy aspects of such models are usually based on life-cycle cost analysis, and designers must consider that analysis outcomes can be substantively influenced by assumptions with respect to both the specific technique used, and many of the basic input parameters. In many instances, a number of input parameters (discount rates; monetary values to be used for fatalities, personal injury and property damage types of collisions, etc.) should be defined in agency policy, reviewed on a regular basis to ensure their appropriateness, and revised and deployed promptly in order to ensure consistency with the agency and political objectives which necessarily influence such safety investment decisions.

3.1.2.4 Integrated Roadside Safety Cost-Effectiveness Analysis

Introduction

Many agencies have developed integrated approaches to roadside safety cost-effectiveness analysis [4, 6, 13, 17]. In general, all of the approaches involve four distinct elements:

- an encroachment module: to estimate the expected encroachment frequency given road and traffic data
- a collision prediction module: to assess if an encroachment would result in a crash
- a severity prediction module: to estimate the severity and estimated costs for each crash
- a benefit-cost module; to calculate the incremental benefit/cost ratios between each pair of safety alternatives

A typical example of the structure of such an approach is described below, based on the Roadside Safety Analysis Program[6] (RSAP) process.

Benefit-Cost Analysis Overview

Benefit-cost analysis is an analytical approach to solving problems of choice. To carry out the analysis, objectives must be defined, alternative ways of achieving each objective need to be identified, and for each objective, the alternative which yields the required level of benefits at the lowest cost are determined.[18] Strictly speaking, the term cost-effectiveness analysis is often used as a synonym for benefit-cost techniques when the benefits or outputs of the alternatives cannot be quantified in terms of dollars. However, more recently, the two terms have been used interchangeably, and road safety cost-effectiveness analysis techniques uniformly quantify outputs in monetary terms. In addition, the road safety use of the term “cost-effectiveness” analysis generally also implies a suite of techniques incorporating the four modules noted above.

Table 3.1.3.2 Horizontal Curve Adjustments for Clear Zone Distances[4]

Radius	Design Speed					
(m)	60	70	80	90	100	110 +
900	1.1	1.1	1.1	1.2	1.2	1.2
700	1.1	1.1	1.2	1.2	1.2	1.3
600	1.1	1.2	1.2	1.2	1.3	1.4
500	1.1	1.2	1.2	1.3	1.3	1.4
450	1.2	1.2	1.3	1.3	1.4	1.5
400	1.2	1.2	1.3	1.3	1.4	
350	1.2	1.2	1.3	1.4	1.5	
300	1.2	1.3	1.4	1.5	1.5	
250	1.3	1.3	1.4	1.5		
200	1.3	1.4	1.5			
150	1.4	1.5				
100	1.5					

Note: The clear zone horizontal curve adjustment factor is applied to the outside of curves only. Curves flatter than 900 m do not require an adjusted clear zone.

sufficient by themselves to define the design domain for the clear zone. These numbers must be applied in the context of situation-specific factors and good design practice. The following design heuristics are included as one means of illustrating such practice and providing additional definition to the design domain for this parameter:

1. The figures in Table 3.1.3.1 provide only a framework for the designer to work with in looking at ranges of clear zone dimensions to use. They are not absolute, and must be considered in the context of site-specific conditions and practicality.

2. The evaluation of alternative clear zone design approaches should be carried out using a well-defined cost-effectiveness analysis procedure such as that provided by AASHTO's "Roadside" procedure and related software available with the 1996 AASHTO Roadside Design Guide, the more recent FHWA developed successor to Roadside, called RSAP or other similar procedures. Such analyses would normally consider alternatives, such as the use of roadside barrier, if provision of the recommended clear zone is not cost effective.

3. On unshielded, traversable 3:1 slopes, determination of the width of the recovery area at the toe of slope should take into consideration right-of-way availability, environmental concerns, economic factors, safety needs, and collision histories. In addition, the distance between the edge of the travel lane and the beginning of the 3:1 slope should influence the recovery area provided at the toe of slope.

4. Increasing inadequate superelevation on curves provides an alternative way of increasing road safety within a horizontal curve except where snow and ice conditions limit the use of such increases.

5. Applying the clear zone concept on flat and level roadsides is relatively simple. In fill or cut sections where roadside slope may be either positive, negative or variable, or where roadside channels exist, the situation is more problematic. Designers should refer to the discussions of Section 3.1.4 for additional guidance in such situations.

Benefit-Cost Analysis for Clear Zone: Example

This example calculation illustrates the application of benefit-cost analysis using a manual technique employed by the Ministry of Transportation of Ontario in their Prioritized Contract Content Guidelines[17]. While this technique differs in approach in terms of collision cost determinations from that described previously, it is equally valid and produces a similar incremental benefit-cost analysis result that allows for the explicit evaluation of safety consequences.

Problem Statement:

A rigid base luminaire support (concrete pole) is located 3.0 m from the edge of pavement, which is also the edge of the through lane. Since the hazard is within the clear zone this pole must be either relocated, shielded or made forgiving. Determine the most cost-effective of the following treatments:

A. leave the pole as is
B. replace the pole with a breakaway base pole
C. relocate the pole outside the clear zone
D. protect the pole with a roadside barrier

Given:

- two-lane undivided rural highway
- 3.5 m lanes, unpaved shoulders
- design speed: 90 km/h
- AADT: 10,000 (50:50 directional split)
- sideslopes: 5:1
- clear zone requirement 9.0 m, Table 3.1.3.1
- traffic growth rate: 0%
- encroachment rate: 0.00045 events/km/year
- concrete pole diameter: 0.5 m
- projected service life: 10 years
- breakaway base pole cost: $5,000 (Alternative B)
- relocation cost: $11,000 (Alternative C)
- relocation distance: 6.0 m (9.0 m total from the edge of pavement)
- protection cost: $11,500 (Alternative D)
- POLICY CRITERIA: If more than one alternative has B/C ratio greater then 2.0, an incremental benefit/cost analysis is required to determine which alternative is most cost-effective.

Collision Frequency Model:

$C_f = (E_f/2000)\ [(L+19.2)\ P\ [Y \geq A] + 5.14 \sum P\ [Y \geq (A+1.8+ (2J-1)/2)]]$

Where:

C_f = collision frequency (collisions/year)
E_f = number of encroachments/year/direction
L = horizontal length of the roadside obstacle (m)
W = width of obstacle (m)
A = lateral distance of the roadside object to edge of pavement (m)
$P[Y \geq ..]$ = probability of a vehicle lateral displacement greater than some value
J = the number of 1.0 m wide obstacle-width increments (the number of J units equal to W rounded to the nearest whole number)

Σ = mathematical summation with summation index range from J = 1 to J=W (in 1.0 m steps)

Calculations:

For all alternatives

E_f = encroachment rate x directional split x ADT
E_f = (0.00045)(0.5)(10,000) = 2.25 collisions/km/direction

Alternative A: Do Nothing

First, calculate the collision frequency from:

$C_f = (E_f/2000)[(L+19.2)P[Y \geq A]+5.14\Sigma P[Y \geq (A+1.8+(2j-1)/2)]]$

with: L = 0.5 m
A = 3.0 m (adjacent lane), A = 6.5 m (opposite lane)
W = 1.0 m (rounded to nearest m)

C_f = (2.25/2000[(0.5+19.2)(0.56) + (5.14)(0.36)] + (2.25/2000)[0.5+19.2)(0.3) + (5.14)(0.21)]
= 0.022 collisions/year

Notes: 1. the probabilities (P[Y...]) come from MTO source material[13].
2. both encroachment directions combined.

Now, calculate the annual cost of collisions:

$CC_A = C_f$ x cost per collision
CC_A = (0.022 collisions/year)($162,000/collision)
= $3,564 per year

Note: unit collision cost for concrete pole, $162,000, is derived from the severity index: 5.5.

Now, convert the annual cost to a net present value:

$NPV_A = CC_A$(P/A, discount rate, traffic growth, service life)

Note: (P/A,...) from MTO Source Material[17]

NPV_A = $3,564 (P/A, 6%, 0% growth, 10 years)
= $26,231

Alternative B: Replace

For this alternative, the collisions per year remain at 0.022 and the severity index for a breakaway pole reduces to 2.8, reducing the unit collision cost to $13,800.

CC_B = 0.022 x \$13,800
= \$304 per year

then:

NPV_B = \$304 (P/A, 6%, 0% growth, 10 year life)
NPV_B = \$2,237

Alternative C: Relocate

In this alternative, the relocation increases the distance between the road and the pole, reduces the probability of a collision taking place, and reduces the collision rate to 0.011 collisions/year. The severity index, 5.5, and the unit collision cost, \$162,000 remain the same since the concrete pole is retained. Calculating this alternative gives:

CC_A = 0.011 x \$162,000
= \$1,782 per year

then:

NPV_C = \$1,782 (P/A, 6%, 0% growth, 10 year life)
NPV_C = \$13,115

Alternative D: Protect

In this alternative, the probability of collisions increase because of the greatly increased length of obstacle presented by the barrier and its location relative to the road. The collision rate increases to 0.059 collisions/year. The severity index is reduced to 3.0, reducing the unit collision cost to \$15,088 due to the nature of the new obstacle (traffic barrier as opposed to a concrete pole). Calculating this alternative gives:

CC_D = 0.059 x \$15,088
= \$890 per year

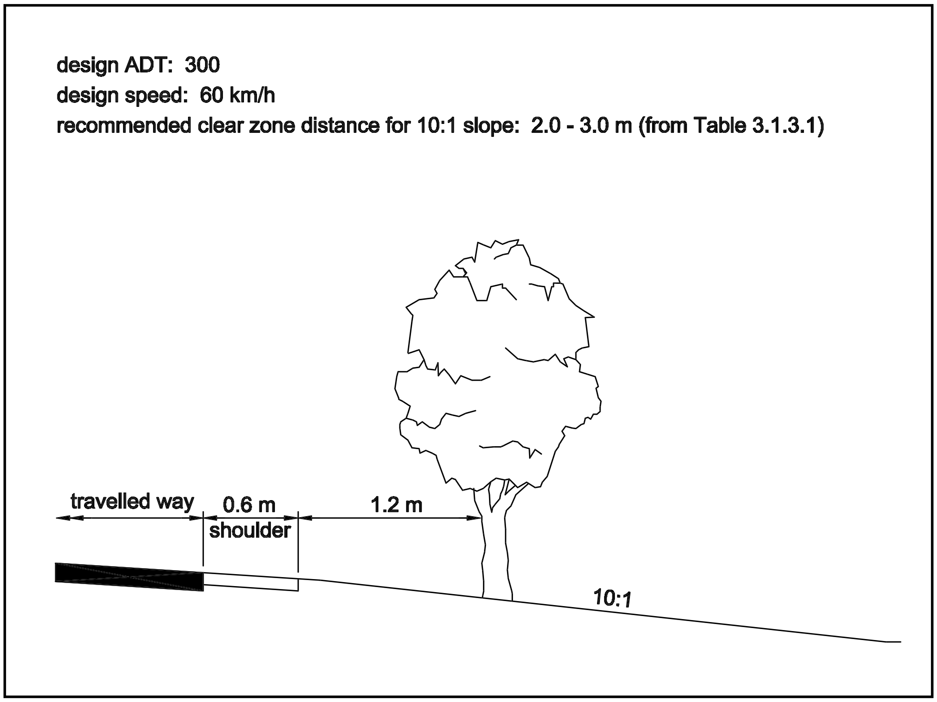
NPV_D = \$890 (P/A, 6%, 0% growth, 10 year life)
NPV_D = \$6,552

Now the benefit-cost analysis can be carried out based on the summary of alternatives below:

Alternative	Net Present Value annual collision cost	Installation Cost (Given)
A	\$26,231	\$5,000 (in 10 years)
B	\$2,237	\$5,000 (now)
C	\$13,115	\$11,000 (now)
D	\$6,552	\$11,500 (now)

Using A as the base case the following relative benefits and costs can be calculated:

EXAMPLE B[4]

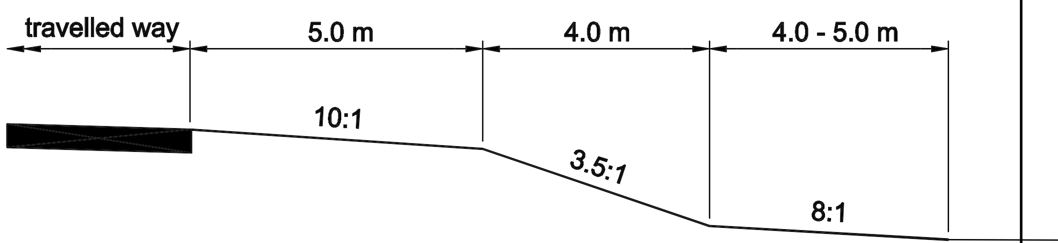


Discussion:

The available clear zone distance is 1.8 m, 0.2 to 1.2 m less than the recommended recovery area. When an area has a significant number of run-off-the-road collisions, it may be appropriate to consider shielding or removing the tree within the collision area. If this section of road has no significant collision history and is heavily forested with most of the other trees only slightly farther from the road, this tree would probably not require treatment. However, if none of the other trees are closer to the roadway than, for example 4.5 m, this individual tree represents a more significant hazard and should be considered for removal. If a tree were 4.5 m from the edge of the travelled way, and all or most of the other trees were 7.5 m or more, its removal might still be appropriate. This example emphasizes that the clear zone distance is an approximate number at best and that individual objects should be analyzed in relation to other nearby obstacles.

EXAMPLE C[4]

design ADT : 7000
design speed : 100 km/h
recommended clear zone distance for 10:1 slope: 9.0 - 10 m (from Table 3.1.3.1)
recommended clear zone distance for 8:1 slope: 9.0 - 10 m (from Table 3.1.3.1)
available recovery distance before breakpoint of non-recoverable slope: 5.0 m
clear runout area at toe of slope: 9.0 - 10 m minus 5.0 m or 4.0 - 5.0 m

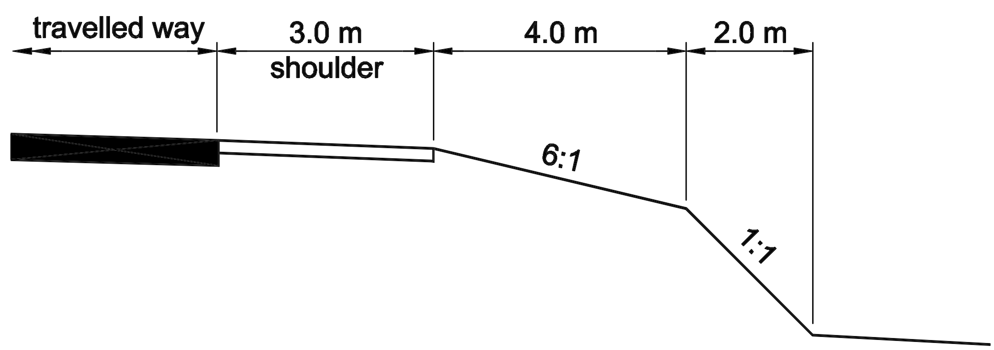


Discussion:

Since the non-recoverable slope is within the required clear zone distance of the 10:1 slope, a runout area beyond the toe of the non-recoverable slope is required. Using the steepest recoverable slope before or after the non-recoverable slope, a clear zone distance is selected from Table 3.1.3.1. In this example, the 8:1 slope beyond the base of the fill dictates a 9.0 to 10 m clear zone distance. Since 5.0 m are available at the top, an additional 4.0 to 5.0 m should be provided at the bottom. All slope breaks should be rounded and no fixed objects would be built within the upper or lower portions of the clear zone or on the intervening slope.

EXAMPLE D[4]

design ADT : 12000
design speed : 110 km/h
recommended clear zone distance for 6:1 slope: 9.0 - 10.5 m (from Table 3.1.3.1)



Discussion:

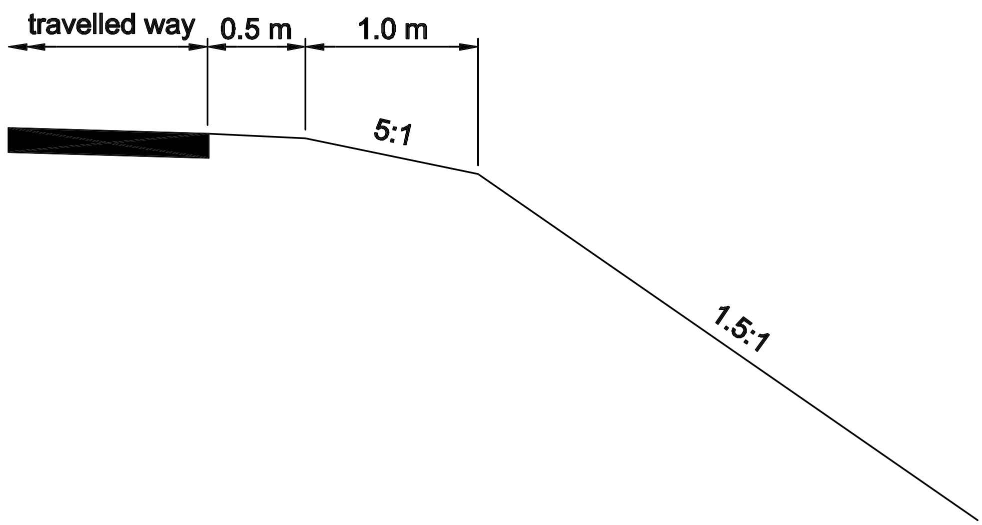
Since the critical slope is only 7.0 m from the travelled way, instead of the suggested 9.0 to 10.5 m, it should be flattened or shielded.

EXAMPLE E[6]

design ADT: 350

design speed: 60 km/h

recommended clear zone distance for 5:1 slope: 2.0 - 3.0 m (from Table 3.1.3.1)



Discussion:

The available 1.5 m is 0.5 to 1.5 m less than the recommended recovery clear zone. If much of this roadway has a similar cross section and no significant run-off-the-road collision history, neither slope flattening nor a traffic barrier would be recommended. On the other hand, even if the 5:1 slope was 3.0 m wide and the clear zone requirement was met, a traffic barrier might be appropriate if this location had noticeably less clear zone than the rest of the road and the embankment was unusually high.

3.2 ACCESS

Tables

The following is an example of a seven level access category system[5]:

- Access Level 1; access via interchanges with public roads only
- Access Level 2; access via at-grade public road intersections or at interchanges
- Access Level 3; right-turn access driveway only
- Access Level 4; right and left-turn access in, right-turn access out
- Access Level 5; right and left-turn access into and out of an activity centre – left-turn lanes required
- Access Level 6; right and left-turn access into and out of an activity centre – left-turn lanes optional
- Access Level 7; right and left-turn access into and out of activity centre – driveway spacing limited by safety requirements only

The seven access levels may be modified to reflect design practices of specific agencies.

A general approach to assigning access categories or levels to a road system is given in Table 3.2.2.1. This table shows how each of the seven types of allowable access relates to the six basic road classes – freeways, expressways, major arterials, minor arterials, collectors, and local roads, and the general design features associated with each class.

It can be seen from the table that direct property access is prohibited from freeways and expressways, access levels 1 and 2. Direct property access should be denied or restricted from access levels 3 and 4, major arterials, respectively. However, access may be provided where no reasonable alternative access is available, or where it is in the general public interest to do so. Where access must be provided, it should be limited to right turns only for access level 3, and to right- and left-turn entry and right-turn exit for access level 4. Direct property access may be permitted for access levels 5 and 6; it is desirable at level 7.

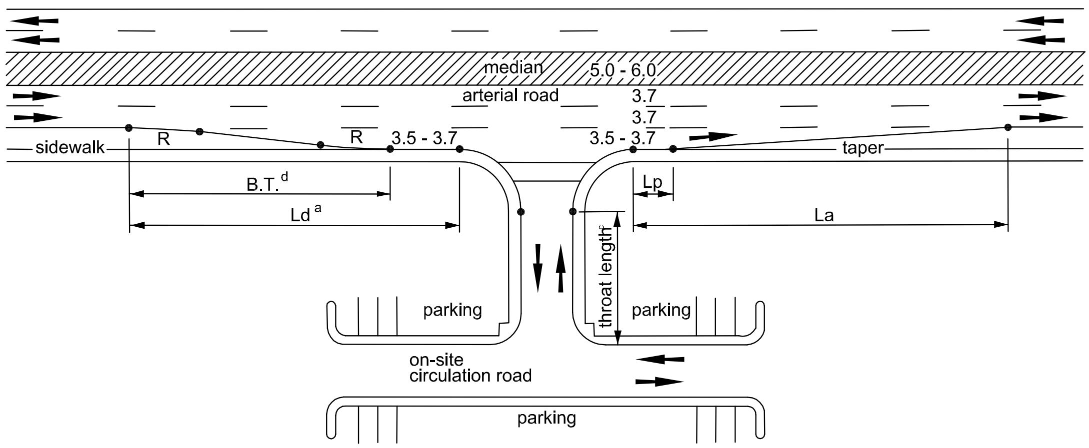
Higher access categories can be selected for rural and suburban areas or new corridors where existing strip development has not yet eroded the function of the road. In areas with existing high density development, the assignment of lower categories and therefore, lower or ambient standards may be more practical. Keep in mind, however, that in existing high development corridors where there is support for improving mobility and safety, a higher standard can be selected and over time, the redevelopment in the corridor will reflect that higher standard. In general, for each road segment, the highest standard which can be implemented should be selected.

Table 3.2.2.1 Access Categories Keyed to Road Type

Access Category	Road Classification	Direct Property Access	General Design Features
1	Freeway	No	Multilane, Median
2	Expressway	No	Multilane, Median
3	Major Arterial	Restrict or Deny[a]	Multilane, Median
4	Major Arterial	Restrict or Deny[b]	Multilane, Median[c]
5	Minor Arterial	Yes	Multilane, or 2 Lanes
6	Collector	Yes	2 Lanes
7	Local/Frontage	Yes	2 Lanes

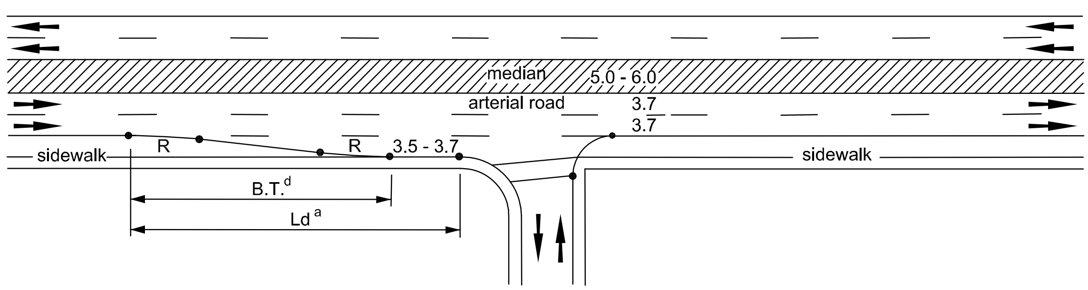
Note:
a. Right turns only when provided.
b. Right and left turn entry and right turn only exit when provided.
c. Might be two-lanes in some rural areas.

Figure 3.2.5.2 Auxiliary Lane Mid-Block Access for Major Developments



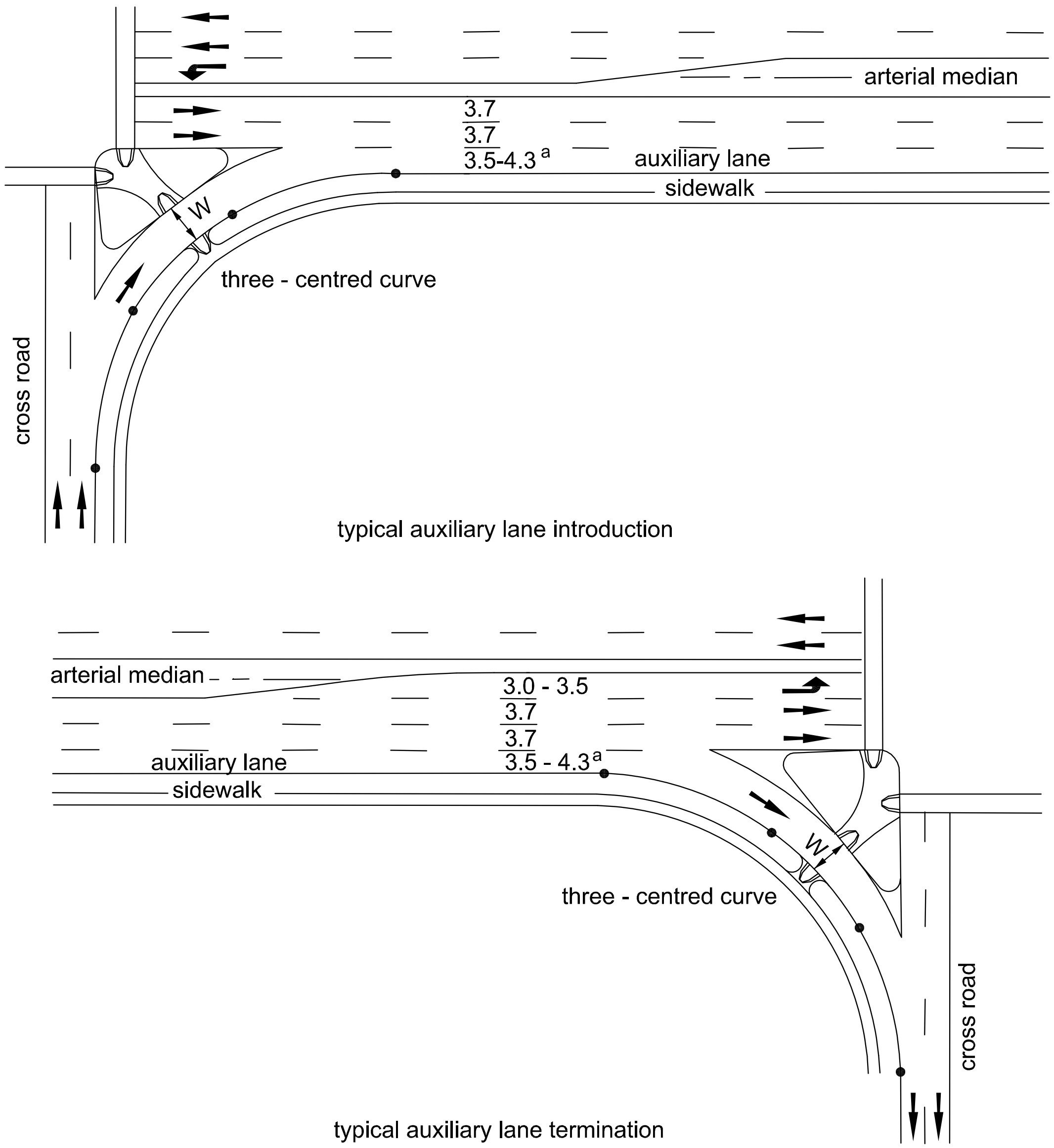
design speed km/h	approach dimensions m			departure dimensions m		
	B.T.	R	Ld[b] (min)	Lp	taper rate	La[b]
50	40	90	75		no taper	
60	50	150	90	26.0	20:1	100
70	60	220	110	57.5	25:1	150
80	70	280	130	57.5	25:1	150

Notes:
a. Ld (min) is normally increased where storage of vehicles is a typical requirement, due to pedestrian movements or vehicle queuing in the entrance.
b. The Ld (min) and La distances based on grades of 2 percent or less.
c. Refer to Table 3.2.9.3 for minimum throat lengths.
d. Straight taper, without curves, also acceptable.



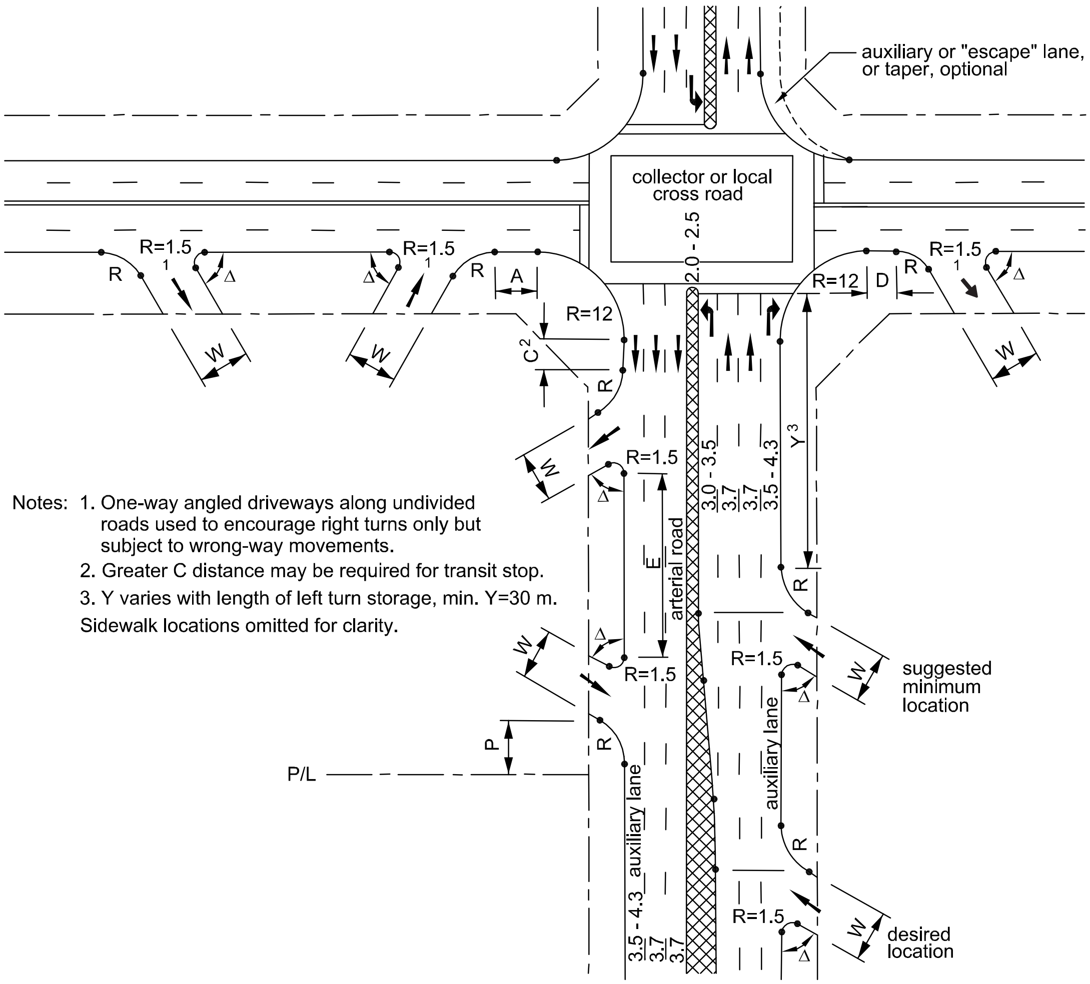
alternative arrangement - no acceleration lane

Figure 3.2.5.3 Typical Auxiliary Lane Introduction and Termination



Notes: a. 4.3 m for lanes shared with bicycles.
For W, see Chapter 2.2
Alternative introduction and termination treatments shown in Figures 3.2.5.2, 3.2.5.7 and 3.2.5.8

Figure 3.2.5.7 Simple Radius Intersection Arrangement With One-Way Angled Accesses Along Auxiliary Lane of a Divided Arterial



Notes: 1. One-way angled driveways along undivided roads used to encourage right turns only but subject to wrong-way movements.
2. Greater C distance may be required for transit stop.
3. Y varies with length of left turn storage, min. Y=30 m.
Sidewalk locations omitted for clarity.

typical dimensions			
criteria	dimension ref.	commercial	industrial
width	W	4.5 to 7.5 m	5.0 to 9.0 m
right-turn radius	R[a]	4.5 to 12.0 m	6.0 to 15.0 m
minimum spacing			
-from property line	P[b]	≥ R	≥ R
-from street corner			
• far side	C	5.0 m	5.0 m
• cross street approach	A[c,d]	5.0 m	5.0 m
• cross street departure	D[d]	5.0 m	5.0 m
-between driveways	E	25.0 m	25.0 m
minimum angle	Δ[e]	60° to 70°	45° to 60°

Notes: a. Values at or near the high end of the range to be used for major generator driveways and where trucks turn.
b. Also established in consideration of the distance to the first driveway on the adjacent property.
c. If the cross road is divided, the driveway is normally located in advance of the cross road left-turn bay.
d. If intersection is signalized, greater dimension suggested- refer to Figure 3.2.8.2
e. Minimum angle of 70° desirable where pedestrians routinely cross.

- the two-way left-turn lane is on a multi-lane arterial roadway with frequent signalized intersections

Overhead signs are typically placed at one-quarter or one-half points between major cross roads. They are positioned a minimum of 50 m away from the intersections to assist in adequate visibility.

Two-way left-turn lanes are generally not extended through a major intersection. They are terminated prior to the intersection and replaced with a single exclusive left-turn lane. Appropriate pavement markings or divisional islands should be used to terminate the two-way left-turn lane in advance of the exclusive left-turn lane at the major intersection.

3.2.6.2 Width

Widths for 2WLTLs are generally the same as the adjacent through lane, but not less than 3.5 m for design speeds equal to or less than 60 km/h. A width of 4.0 m is desirable for design speeds greater than 60 km/h. The additional width over the adjacent lane recognizes that vehicles are making turning manoeuvres from both directions simultaneously, and adds a measure of safety. Widths greater than 5.0 m are generally avoided due to operational problems.

3.2.6.3 Application

Since opportunities for a left-turning vehicle to decelerate within the limits of a 2WLTL may be restricted by access spacing and the potential for conflicting vehicle movements, a 2WLTL is best suited for urban roads with operating speeds of 50 to 60 km/h. Operating speeds up to 70 km/h may be tolerated where most other conditions are favourable.

Two-way left-turn lanes operate successfully over a wide range of arterial road volumes. A survey of major Canadian cities indicates successful operation of 2WLTLs for arterial roads with volumes of up to 35 000 veh/d and a seven-lane cross section. The successful operation is the result of a number of interrelated factors including:

- horizontal and vertical alignments
- sight distance
- cross section dimensions
- through traffic volumes
- left-turning traffic volumes
- frequency of traffic signals
- frequency of cross roads
- frequency of accesses
- driver familiarity

Due to the complexity and number of design factors to be considered at any specific site, it is difficult to stipulate a set of limiting conditions for the effective operation of 2WLTLs. The physical conditions at each potential site are normally examined by experienced geometric design and traffic operations personnel, and engineering judgement is used to determine the potential for and improvements required to successfully implement a 2WLTL.

Two-way left-turn lanes may be prone to improper use, particularly in jurisdictions where few 2WLTLs exist and driver unfamiliarity is a problem. Some of the potential operational problems are as follows:

- vehicles may make angle turns across the 2WLTL, leaving the rear of the turning vehicle encroaching into the adjacent through lane while waiting for a gap to merge with or cross the through traffic stream
- left-turning vehicles may enter the 2WLTL too far in advance of the access where the left turn is to be made, and thereby impede or risk collision with opposing left-turning traffic in the 2WLTL
- through vehicles may use the 2WLTL as a passing lane to overtake slower moving traffic in the through lanes
- left-turning vehicles may not use the 2WLTL to decelerate from the operating speed of the arterial, but decelerate substantially in the through lane before entering the 2WLTL

- cyclists may perceive the 2WLTL as a relatively protected area, and ride along it for long distances

- pedestrians may be placed at a greater risk, due to the wide cross section and the lack of a physical refuge area

Proper education and enforcement programs can be effective to achieve a significant reduction in improper use. The general advantages and disadvantages of 2WLTLs are summarized in Table 3.2.6.1.

Explicit Evaluation of Safety

Two-way left-turn lanes (2WLTLs) diminish conflicts with vehicles turning left from the main roadway and provide a refuge for vehicles turning left onto the main road. Approximately half of the collisions involving vehicles entering or exiting driveways are associated with left-turn manoeuvres. Almost all research articles relating to the safety effect of 2WLTLs are for multi-lane roadways in urban and suburban settings. The relationship between the reduction in number of collisions versus the density of access points is given by Equation 3.2.1:

$$CMF = 1-0.35\,(0.0047X + 0.0039X^2)/(0.745 + 0.0047X + 0.0039X^2) \quad (3.2.1)$$

where CMF = the Collision Modification Factor

X = the number of access points per kilometre (total of both directions)

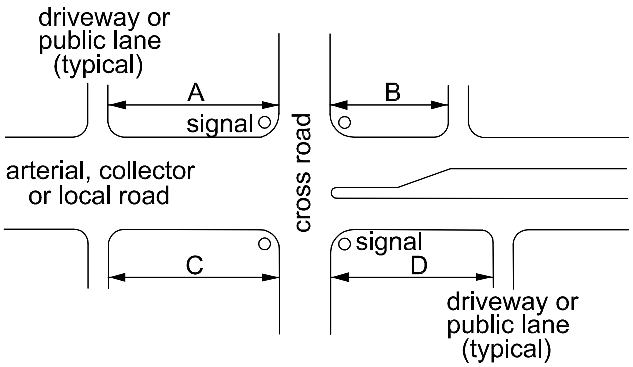
The Collision Modification Factor for 2WLTLs is depicted graphically in Figure 3.2.6.2.

For example, if there are 24 driveways on a 1.5 km section of undivided roadway, the number of driveways per kilometre is 24/1.5 or 16 access points per kilometre. By using either Equation 3.2.1 or Figure 3.2.6.2, the Collision Modification Factor is determined to be 0.79. The percentage reduction in collisions which could be anticipated if a 2WLTL was installed would be (1-0.79) x 100 = 21%. The cost of installing a two-way left-turn lane can be compared to the benefit of the reduction in collision costs to determine the advisability of the installation.

Table 3.2.6.1 Advantages and Disadvantages of Two-Way Left-Turn Lanes

Advantages	Disadvantages
• well suited to strip development with frequent low to medium volume driveways • remove turning traffic from the through lanes, significantly improving traffic safety and capacity • not as restrictive to access as a raised median • implementation costs and right-of-way requirements are less than that of a raised median	• generally not suited for operating speeds >70 km/h • not suitable to high volume driveways, exclusive turn lanes preferred • left-turn paths not clearly defined and turning conflicts can occur • limited to tangent alignments with good sight distance • traffic level of service lower as compared to divided roadway • opposing traffic flow not physically separated as with a raised median • pedestrians required to cross wide roadway without a physical central refuge area • operation may not be clearly understood by the unfamiliar driver

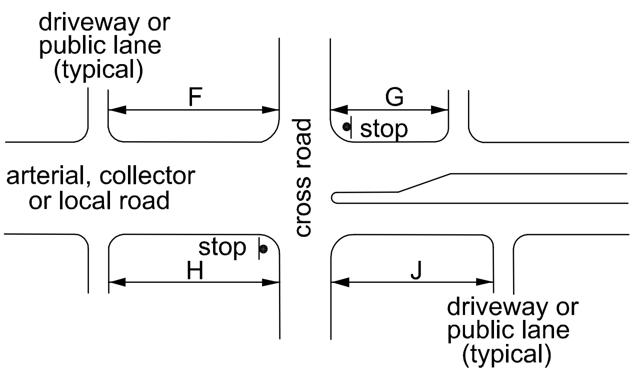
Figure 3.2.8.2 Suggested Minimum Corner Clearances to Accesses or Public Lanes at Major Intersections



item	min. clearance, m[d]		
	arterial	collector[b]	local[b]
A	70[c]	55	15
B	#[a]	25	15
C	70	55	15
D	70[c]	55	15

Notes: a. Distance (#) positions driveway or public lane in advance of the left turn storage length (min.) plus bay taper (des.).
b. Lesser values reflect lower volumes and reduces level of service on collectors and locals.
c. Reduced distances feasible if auxiliary lane implemented, see Section 3.2.5
d. Values based on operating speed of 50km/h, higher values desirable for higher speeds or may be warranted by traffic conditions.

signals at the cross road



item	min. clearance, m[d]		
	arterial	collector[b]	local[b]
F	35	20	15
G	#[a]	25	15
H	25	25	15
J	35	20	15

Notes: a. Distance (#) positions driveway or public lane in advance of the left turn storage length (min.) plus bay taper (des.).
b. Lesser values reflect lower volumes and reduces level of service on collectors and locals.

stop control at the cross road

roadway. As a minimum, B is equal to or greater than the storage length, but desirably B is equal to or greater than the storage length plus the bay taper.

The lower half of Figure 3.2.8.2 presents suggested minimum corner clearance dimensions adjacent to an intersection with stop control, rather than signals, at the cross road. Dimensions F and H are applicable to an undivided roadway, G and J to a divided road. Dimensions F and J are based on right-turning vehicles at the intersection being able to perceive and react to a conflict at the first access. Dimension H is based on providing space for three passenger vehicles to be queued at the stop control without blocking the driveway. Dimension G is based on the same philosophy as dimension B in the upper half of the figure.

The lesser values shown on Figure 3.2.8.2 for collector and local roads reflect the reduced needs associated with lower traffic volumes and a decreased expectation in level of service.

Due to small corner parcel sizes and the legal requirements for access provision, it may not be feasible to provide the suggested minimum corner clearances. Engineering judgement and a good understanding of traffic operations are needed to determine the most suitable access layout and related roadway provisions for the prevailing conditions.

Table 3.2.9.1 Typical Driveway[c] Dimensions

Dimension (m)	Land Use		
	Residential	Commercial	Industrial
width (W)			
• one-way	3.0[a] – 4.3	4.5[a] – 7.5	5.0[a] – 9.0
• two-way	3.0.[a] – 7.3	7.2[a] – 12.0[b]	9.0[a] – 15.0[b]
right-turn radius (R)	3.0 – 4.5	4.5 – 12.0	9.0 – 15.0

Notes:
a. Minimum widths are normally used with radii at or near the upper end of the specified range.
b. Increased widths may be considered for capacity purposes; where up to 3 exit lanes and 2 entry lanes are employed, 17.0 m is the max. width, exclusive of any median.
c. Applicable to driveways only, not road intersections.

corner clearance. A minimum dimension (C) of 5.0 m is suggested to separate the conflict zones and to provide for a greater manoeuvring area for turning trucks. For an industrial area, this then results in a minimum corner clearance of about 25.0 m (11.0 m for the minimum corner curb radius, the 5.0 m dimension (C), and a 9.0 m minimum driveway curb radius).

A high volume driveway on the near side of an intersection may warrant a left-turn storage area on the roadway to accommodate left turning traffic into the driveway. If this is the case, the driveway is located in consideration of the total distance needed for the back-to-back left-turn bays created on the roadway. The combined left- turn storage and taper requirements significantly increases the corner clearance requirements.

3.2.9.8 Spacing of Adjacent Driveways

In addition to the corner clearance considerations described in Subsection 3.2.9.7, driveways are normally located in consideration of their physical relationships to existing or possible future driveways. The following three criteria need to be considered:

- minimum spacing between driveways
- minimum offset to property line
- maximum number of driveways based on property frontage

The application of these design criteria assists in meeting the following objectives:

- to clearly identify to the user which property each driveway serves
- to ensure that sufficient space is available between driveways for the positioning of traffic signs, lighting poles and other surface utility fixtures, and road hardware
- to separate the conflict areas for each driveway
- to provide appropriate space between driveways for on-street parallel parking, where permitted and in consideration of sight line requirements
- to increase the length of potentially collision free pedestrian areas by minimizing the number and width of driveways

Roadway retrofit projects often provide the opportunity to improve existing driveway spacing.

The minimum spacing between driveways is measured between the end and start of the curb returns on the adjacent driveways, shown as dimension (E) on Figure 3.2.9.3. A 1.0 m minimum spacing is recommended between adjacent low volume driveways for residential properties, along local and collector roadways, while a 3.0 m minimum is the suggested dimension for both commercial and industrial

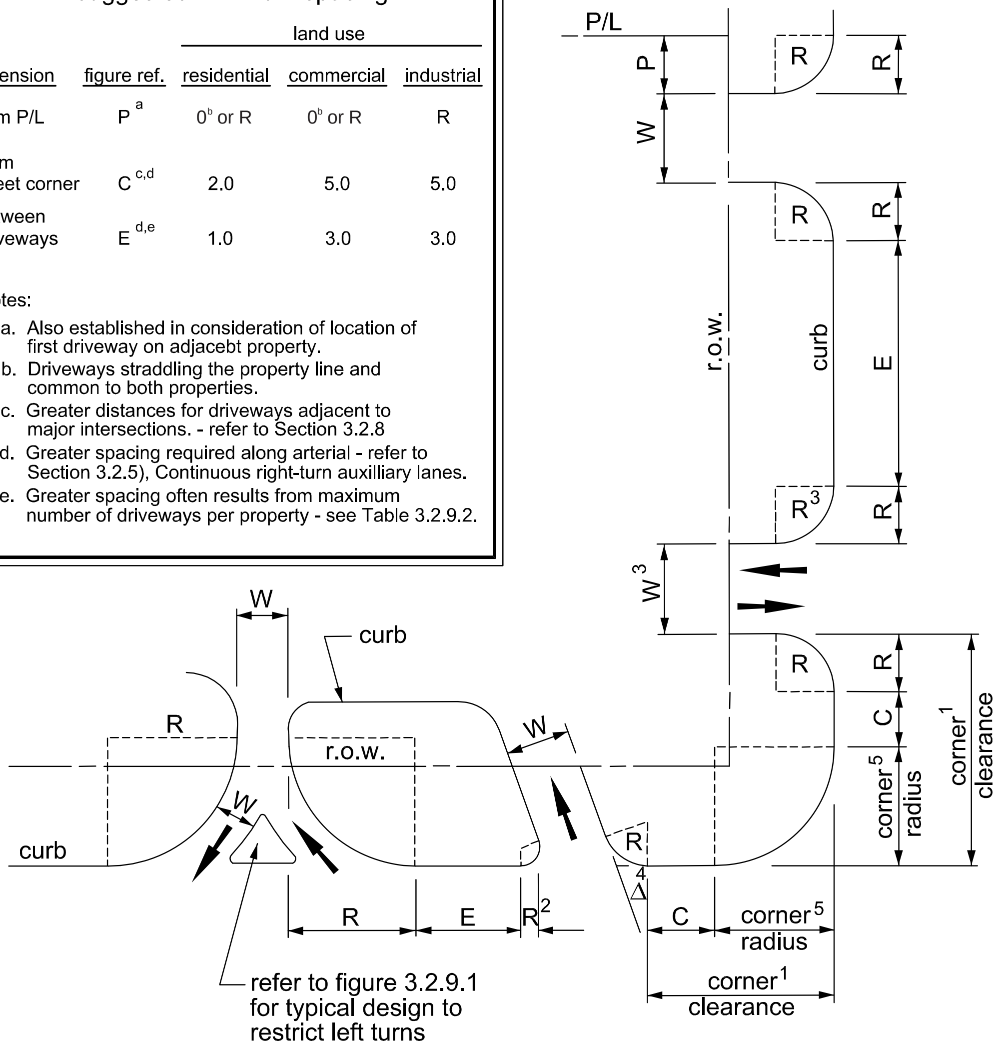
Figure 3.2.9.3 Driveway Spacing Guidelines - Locals and Collectors

suggested minimum spacing

dimension	figure ref.	land use: residential	land use: commercial	land use: industrial
from P/L	P [a]	0[b] or R	0[b] or R	R
from street corner	C [c,d]	2.0	5.0	5.0
between driveways	E [d,e]	1.0	3.0	3.0

Notes:

a. Also established in consideration of location of first driveway on adjacebt property.
b. Driveways straddling the property line and common to both properties.
c. Greater distances for driveways adjacent to major intersections. - refer to Section 3.2.8
d. Greater spacing required along arterial - refer to Section 3.2.5), Continuous right-turn auxilliary lanes.
e. Greater spacing often results from maximum number of driveways per property - see Table 3.2.9.2.



Notes:
1. For suggested minimum corner clearance at major intersections, see Figure 3.2.8.2
2. Where turns are not permitted, R=1.5 m assists in discouraging wrong-way movements.
3. For typical R and W dimensions, refer to Table 3.2.9.1
4. Minimum angle of 70° desirable where pedestrians routinely cross driveway, 45° minimum otherwise.

Table 3.2.9.2 Maximum Number of Driveways Based on Property Frontage[9]

Frontage (m)	Maximum Number of Driveways[a]
15	1[b]
16 – 50	2
51 – 150	3[c]
> 150	4 or more[c]

Notes: a. Subject to spacing guidelines presented in Subsections 3.2.5.2 and in Figure 3.2.9.3.
b. Single family residential properties normally restricted to one driveway, irrespective of frontage.
c. For large developments the location and design elements of driveways are normally determined by a detailed traffic impact study.

Where inter-development traffic is expected to be significant, and signalization of the driveway intersection is not desirable, the manoeuvre required to cross the entire width of a busy roadway in a single continuous movement may be difficult. In this case, it is often advantageous to offset the opposing driveways to eliminate the concentrated conflict zone. A minimum offset of 100 m between driveway centrelines is desirable, as illustrated in Figure 3.2.9.4. This technique does, however, increase the number of slow moving vehicles making ingress, egress and weaving manoeuvres on the roadway, which may present other operational concerns. The relative impact is assessed to determine the best design decision.

Retrofitting of existing driveway locations may be warranted over time as traffic conditions change along a roadway and at individual driveways. Alternatives to existing driveway locations, or driveway consolidation to improve spacing, may provide effective solutions to traffic operational concerns.

3.2.9.10 Clear Throat Lengths

In order for major driveways to operate efficiently, both from the road side and internally, it is desirable to provide a no conflict and storage zone within the driveway. This zone is commonly referred to as the clear throat length or set-back distance and is measured from the ends of the driveway curb return radii at the roadway and the point of first conflict on-site. Figure 3.2.5.2 illustrates how a throat length is measured. Failure to provide sufficient throat distance results in frequent blocking of on-site circulation roads which can in turn create queues of entering vehicles. The provision of appropriate clear throat length or storage space is particularly important for drive-in service developments where the customers remain in their vehicles while waiting to be served. These types of developments include drive-in restaurants and banks, automatic car washes, and parking facilities with entry control.

For large developments, the appropriate throat length is best determined by a detailed traffic analysis based on the traffic control provided at the road and the anticipated volumes and types of traffic. Table 3.2.9.3[7] is a guideline for suggested minimum clear throat lengths for various types of developments.

3.2.9.11 Grades

When selecting the most suitable grades for a driveway, a number of considerations are important including:

- road classification
- driveway volume
- maximum grade for the driveway within the right of way where it intersects the roadway
- minimum grade for the driveway within this same zone
- maximum driveway grade on-site
- maximum rate of grade change
- pedestrian crossing cross-slope

Table 3.2.9.3 Suggested Minimum Clear Throat Lengths for Major Driveways[7]

Land Use	Development Size	Minimum Clear Throat Length (m) Collector	Arterial
Light industrial	<10 000 m^2	8	15
	10 000 – 45 000 m^2	15	30
	>45 000 m^2	15	60
Discount store	>3 000 m^2	8	15
		8	25
Shopping centre	<25 000 m^2	8	15
	25 000 – 45 000 m^2	15	25
	45 001 – 70 000 m^2	25	60
	>70 000 m^2	40	75
Supermarket	<2 000 m^2	15	25
	>2 000 m^2	25	40
Apartments	<100 units	8	15
	100 – 200 units	15	25
	>200 units	25	40
Quality restaurant	<1 500 m^2	8	15
	>1 500 m^2	8	25
Fast food restaurant	<200 m^2	8	25
	>200 m^2	15	30
General office	<5 000 m^2	8	15
	5 000 – 10 000 m^2	8	25
	10 001 – 20 000 m^2	15	30
	20 001 – 45 000 m^2	30	45
	>45 000 m^2	40	75
Motel	<150 rooms	8	25
	>150 rooms	8	30

Notes: 1. Refer to Figure 3.2.5.2 for method of measurement.
2. For major developments, it is desirable to determine throat lengths and queue on the basis of a site-specific traffic study.

- roadway, driveway, roadside and property drainage
- cyclist accommodation

Desirable maximum grade changes, between the roadway cross-slope and the driveway grade, vary in accordance with the road classification. For the higher classification road, it is desirable to minimize the grade change at the roadway edge, thereby encouraging high speed turns into the driveway and reducing the deceleration and interference with the through traffic on the major road. This is particularly important for high volume driveways. Figure 3.2.9.5 provides guidelines for limiting the grade change at the road edge. For high volume driveways on arterial roads, a maximum grade change of 3% is acceptable. For low volume driveways on local roads, a maximum of 8% is acceptable.

Driveways are constructed at an incline from the roadway in order to prevent surface drainage along the roadway from discharging down a driveway and onto private property. Where this is impractical, curb drainage across the driveway can be effectively controlled by using a slightly deeper gutter and adjacent catch basins. It is also common practice to limit the amount of property drainage that drains onto the roadway via the driveway by providing separate on-site drainage systems.

Assuming a normal roadway cross-slope of 2.0% and the desirable maximum grade changes defined above, the resulting maximum driveway grades within the boulevard and

3.3.3 OUTDOOR PEDESTRIAN RAMPS, STAIRS AND HANDRAILS

Model requirements for outdoor pedestrian ramps, stairs and handrails are provided in the National Building Code,[2] as prepared by National Research Council Canada. Regulatory requirements are the responsibility of each province and territorial government and may vary from the National Building Code. Provincial and municipal regulations may supplement or replace the National Building Code guidelines. It is therefore advisable for the designer to be knowledgeable of the local requirements.

Pedestrian travel can be accommodated along a significant slope using one of three methods:

- a ramp or inclined sidewalk
- a series of steps or stairs
- a combination of ramps and steps

Ramps having a maximum slope of 12:1 (8.33%) and appropriate level landings can be easily negotiated by most pedestrians and persons in wheelchairs. For barrier-free travel by persons in wheelchairs,[3] sidewalks with an incline greater than 20:1 (5.0%) are typically designed as ramps with landings. Ramps are provided with a level landing area at least 1.2 m long at intervals of not more than 9.0 m along their length, and where there is an abrupt change in ramp direction. For barrier-free ramps, a minimum length of 1.5 m is usually provided for the landing area. Exterior ramps are normally a minimum of 1.1 m in width, or 1.5 m in width where space is required for two wheelchairs to pass, and are constructed of materials that provide a slip-resistant, continuous and even surface. Ramps are particularly well suited to serving large crowds of people. It is not always feasible to provide sidewalk grades suitable for barrier-free travel, such as where street grades exceed 5.0%. In these cases, it is desirable to provide an alternate suitable route, if possible.

Stairs are effective in traversing significant vertical heights within a minimum horizontal distance. Flexibility in stair design permits adjustments to suit varying site constraints. Stairs, however, present a barrier to persons using wheelchairs, people pushing strollers or buggies, and cyclists. The elderly and people with walking impairments may also be severely restricted by any appreciable height of stairs. Stairs slow normal pedestrian travel speeds by up to 30% and represent a hazard for large crowds.

It is desirable to provide stairs at a constant grade, with uniform run and rise dimensions, through the length of the stairway. Alternate routes are normally provided for persons with wheelchairs or walking impairments. Stairways typically have three or more risers to ensure visibility to the pedestrian and to prevent tripping occurrences associated with unexpected single steps. Stairs with open risers or nosings are generally avoided to prevent possible problems with toe snagging. Overhead lighting placed to one side and at the top of a stairway is usually effective in illuminating the edge of each tread at night.

The minimum width of a stairway is normally 1.1 m. The maximum vertical rise without provision of a landing is normally 3.7 m. Where landings are provided, each landing is typically as wide as the stairway with a minimum length of 1.1 m. If awnings or other shelters are provided above the stairway, the minimum vertical clearance provided is usually 2.05 m.

Stair runs are normally not less than 230 mm and not more than 355 mm, exclusive of nosings. Risers are typically not less than 125 mm and not more than 200 mm. A 355 mm run combined with a 150 mm riser allows relatively steep slope to be traversed while affording a comfortably wide run. For barrier-free design, stair runs of 280 mm minimum and risers of 180 mm maximum are usually provided. The front edge of the stair treads are oriented at right angles to the direction of travel for pedestrian safety. Runs and landings are provided with a slip-resistant finish or are provided with slip-resistant strips protruding not more than 1 mm above the surface of each tread. A slope of 1% toward the forward edge of each run is desirable to assure drainage.

Where ramps or stairways are 1.1 m or more in width, it is advantageous to provide handrails on both sides. In cases where ramps or stairways are more than 2.2 m in width, intermediate handrails are desirable and are normally positioned to ensure that not more than 1.65 m exists between adjacent handrails.

Handrail heights are normally in the range of 800 mm to 920 mm, as measured vertically from a line drawn along the outer edge of the stair treads or the surface of the ramp. At least one of the handrails is normally continuous throughout the length of the stairway or ramp including landings. It is also typical to extend at least one of the handrails 300 mm to 450 mm beyond the top and bottom of the stairway or ramp.

Where stairways are provided within a street right-of-way, it may also be desirable to provide a parallel ramp to allow bicycles and strollers to be pushed up immediately adjacent to the stairs.

Figure 3.3.3.1 portrays a number of the design elements pertinent to stairways.[1]

agency normally has specific clearance requirements which are to be honoured. In many cases, it is desirable to relocate the overhead utility underground to avoid the conflict, while improving the aesthetics of the street right of way.

With respect to street lights, it is desirable to examine the blocking effect of the future mature tree canopies on the illumination levels intended for the roadway and the pedestrian areas. Strategic spacing, consideration of tree canopy form, and pruning of trees relative to the luminaries is often sufficient to avoid significant problems for the roadway illumination. It is desirable to position trees midway between streetlight poles and to prune the lower branches so that unobstructed light reaches a point a minimum of 1.8 m above the mid-span points as illustrated on Figure 3.3.4.1.[4] It is also desirable to select trees with thin, permeable canopies to reduce the needs of pruning. Separate pedestrian style lighting may be required to provide the levels of illuminance necessary for the safety and security of the pedestrian area.

For the climatic conditions in most Canadian urban centres, it is important not to provide a dense tree cover adjacent to pedestrian areas. The warmth provided by the sun is generally beneficial for pedestrian comfort in temperate climates. The strategic placement of deciduous trees is effective in providing shade protection during hot summer afternoons while allowing the sun to provide warmth during winter months.

Tree spacing and location are dependent on a variety of factors including:

- desired visual affect
- species of tree
- physical constraints, such as utilities, lighting, and underground structures
- vehicle/vehicle and vehicle/pedestrian sight line requirements

Each streetscaping project is normally assessed on the basis of the existing conditions and constraints toward identifying the most effective planting plan.

Tree grates are normally provided for trees planted within a hard surfaced pedestrian area. Their use maximizes the available pedestrian travel space while allowing proper gas exchange at the air/soil contact zone. Tree grates are typically manufactured using durable concrete or cast-iron materials. It is important to provide surface drainage away from the tree trunk to assist in minimizing the intrusion of roadway salt and other harmful materials into the soil and root system. Raised curbs around each tree pit may be effective in limiting salt intrusion but are more restrictive to pedestrian travel than grates which are flush with adjacent sidewalks. Tree or other grates not flush with the sidewalk or with openings greater than 13 mm in diameter may pose a hazard for persons with reduced mobility, and are normally located laterally beyond the line of travel and clear sidewalk width.

Appropriate vegetation can be selected for use on embankment and cut slopes to effectively create soft aesthetics, reduce maintenance and increase slope stability.

3.3.4.3 Vehicular Traffic Considerations

The location and configuration of vegetation are determined so as to maintain the sight lines required at street intersections and other pedestrian crossing areas. Sight distance requirements are discussed in Chapter 2.3 – Intersections. Shrubs less than 1.0 m in height and trees with canopies providing at least 2.4 m of vertical clearance may be considered within an intersection sight triangle. Hedges, high shrubs and coniferous trees which block sight lines are avoided in these critical areas. Generally, it is desirable to eliminate, or at least restrict, the type and amount of vegetation within the critical sight triangles at intersection areas to maintain pedestrian and vehicular safety.

The locations of traffic signs, particularly regulatory and warning signs, and traffic signals are normally co-ordinated with the planting layout. Vegetation at locations that may block the driver's view of traffic signs and signals are avoided.

When creating a tree planting plan, consultation with the adjacent property owners and business operators is desirable. It may be desirable not to block important or interesting buildings from the view of the passing vehicular traffic. Certain buildings may be equally or more important to the visual quality and character of the street than planted vegetation.

Trees with trunk diameters greater than 150 mm at maturity are considered fixed objects. Groups of small trees or shrubs closely spaced may have the same effect as a single large tree. Therefore, for larger trees and tree groups, it is advisable to locate trees in accordance with the clear zone guidelines, based on design speed, from the edge of the travel lane (existing or future) to the tree trunk, as outlined in Chapter 3.1. Permitted locations for trees are often a matter of local policy, but it is generally desirable to avoid having large trees in close proximity of vehicular travel lanes, such as within boulevards less than 2.0 m and medians less than 4.5 m in width. When parking is provided along an urban street where streetscaping is employed, the parking area generally provides a suitable buffer between the traffic lanes and the boulevard trees and other fixed objects, and therefore the curb to fixed object dimension is less critical.

A minimum setback of 750 mm from the curb face to the tree trunk face is generally desirable in all cases. This provides a suitable minimum clearance for passengers to open a vehicle door and exit/enter reasonably unimpeded, reduces the frequency of splashed salt and other harmful materials onto the tree trunk and minimizes the intrusion of root growth into the road subgrade.

3.3.4.4 Roadside Treatments

The selection of the most appropriate roadside area treatments is influenced by a number of factors including:

- the total curb to property line (roadside) width available
- the clear sidewalk width required to accommodate the anticipated pedestrian characteristics and volumes
- the nature and characteristics of the adjacent land use
- the volume, speed and type of vehicular traffic along the adjacent roadway
- the location of overhead and underground utilities

Generally, four options are available to the designer as illustrated in Figure 3.3.4.2.[5] Each landscaping project is assessed on its individual characteristics. It may be advantageous on certain projects to implement combinations of options to suit varying land uses, vehicular traffic and parking conditions, and the physical constraints; combination of options may be more visually stimulating in making the motorists more aware of the driving environment and thus enhancing safety. Where a change is made from one option to another, it is important to design the pedestrian route transitions to be obvious and unobstructed.

A description and typical application of the four options are outlined by the following:

<u>Option A – Boulevard and Border Vertical Features</u>

This treatment is particularly beneficial where wide expanses of paved areas exist on either side of the roadside. An example is a pedestrian area between a multi-lane road and a parking area for a regional shopping centre. The vertical features and the created enclosure effectively buffer the pedestrian from both the busy road and the adjacent land use. To provide an appropriate pedestrian scale, the ratio of pedestrian area width to height of the vertical features is normally in the range of 1:1 to 1:2.

<u>Option B – Border Vertical Features Only</u>

With this option, a buffer is introduced only between the pedestrian area and the adjacent development and therefore provides a physical separation between the pedestrian area and the adjacent land uses. Common applications are along off-street parking areas and adjacent to industrial land uses, where large and unsightly areas are visible to the pedestrian. The aesthetics of residential land uses may also be improved by this style of treatment. This arrangement may also be the only feasible option where overhead utility lines exist along the

3.4.5 ALIGNMENT ELEMENTS

The alignment elements in the following paragraphs are generally applicable to bike paths. Other classifications of bikeways are designed for motor vehicle traffic and those standards are adequate for bicycles, with the exception of stopping sight distance. Stopping sight distance is greater for bicycles than motor vehicles, particularly in the case of steep downgrades, and should be considered in the designation of bike lanes and bike routes.

3.4.5.1 General Approach

As for any transportation facility, there is a responsibility to generate a collision free design. The standards and practices for bikeways that follow are intended to assist designers to meet this responsibility. However, the bicycle is a distinct vehicle which is often used in locations of substandard geometrics. In such cases, providing suitable warning signs along bikeways is a significant consideration in maintaining safety.

3.4.5.2 Design Vehicles

The suggested dimensions of a bicycle to be used in the design of bikeways are:

- length, 1.75 m
- width at pedals, 400 mm
- height to lowest pedal position, 100 mm
- width at handlebars, 800 mm
- height to handlebars, 1.25 m
- height to top of seated riders, 2.0 m

Desirable bikeway widths for design are as follows:

- one-way, 1.20 m to 1.60 m
- two-way, 2.20 m to 2.60 m

3.4.5.3 Design Speed

The speed at which a cyclist travels is dependent on several factors, including the type and condition of the bicycle, the purpose of the trip, the condition and location of the path, the speed and direction of the wind, and the physical condition of the cyclist. Paved bike paths are designed for a selected speed that is at least as high as the preferred speed of the faster cyclists. In general, a minimum design speed of 30 km/h is used; however, when the downgrade exceeds 4%, or if strong tailwinds prevail, a design speed of 50 km/h is advisable.

On unpaved paths, where cyclists tend to ride more slowly, a lower design speed of 25 km/h can be used or where the grades or the prevailing winds dictate, a higher design speed of 40 km/h can be used. Since bicycles have a higher tendency to skid on unpaved surfaces, horizontal curvature design should take into account lower coefficients of friction.

3.4.5.4 Stopping Sight Distance

Minimum stopping sight distance for bicycles is the distance required to bring a bicycle to a controlled full stop. It is a function of the cyclists' perception and brake reaction time, the initial speed of the bicycle, the coefficient of friction between the tires and the bikeway surface and the braking capability of the bicycle. The stopping sight distance is given by the expression:

$$SSD = 0.694V + \frac{V^2}{255(f + g/100)} \qquad (3.4.1)$$

Where: SSD = stopping sight distance (m)

V = design speed (km/h)

f = coefficient of friction

G = grade (% up grade is positive and down grade is negative)

The expression is based on a perception-reaction time of 2.5 s. Table 3.4.5.1 illustrates minimum stopping sight distance for a range of speeds from 10 to 50 km/h and grades up to 12%. For two-way facilities, the values for the descending direction control the design. Coefficient of friction (f) is taken to be 0.25 for paved surfaces, which accounts for the poor wet weather braking characteristics of many bicycles.[2]

Table 3.4.5.1 Minimum Stopping Sight Distance for Bicycles (Paved Surface, Wet Conditions)

Grade (%)	Minimum Stopping Sight Distance (m) — Design Speed (km/h) 10	15	20	25	30	35	40	45	50
12	8	13	18	-	-	-	-	-	-
10	8	13	18	24	-	-	-	-	-
8	8	13	19	25	32	-	-	-	-
6	8	13	19	25	32	40	-	-	-
4	8	13	19	26	33	41	49	-	-
2	8	14	20	26	34	42	51	61	-
0	9	14	20	27	35	44	53	63	74
-2	9	14	21	28	36	45	55	66	77
-4	9	15	21	29	38	47	58	69	81
-6	9	15	22	30	39	50	61	73	86
-8	9	16	23	32	42	53	65	68	92
-10	10	16	24	34	44	53	70	84	100
-12	10	17	26	36	48	61	76	92	110

Notes: For the purposes of measuring stopping sight distance the height of eye is normally taken to be 1.37 m and the height of object zero, to provide for impediments to bicycles at pavement level, such as potholes.

For selection of design speed, refer to Subsection 3.4.5.3.

3.4.5.5 Horizontal Alignment

Radius and Superelevation

The minimum radius of a circular curve for a bikeway is a function of bicycle speed, superelevation, and coefficient of friction. These variables are related by the expression:

$$R = \frac{V^2}{127(e + f)} \qquad (3.4.2)$$

Where: R = radius (m)

V = design speed (km/h)

e = superelevation (m/m)

f = coefficient of lateral friction

This relationship is used to determine the minimum design radius for given design speeds. For most applications and conditions, the superelevation rate will range from a minimum of 0.02 to 0.05 m/m. The coefficient of lateral friction used for design of paved bikeways varies from 0.3 at 25 km/h to 0.22 at 50 km/h. For the design of unpaved surfaces, lateral friction factors are reduced to 50% of those of paved surfaces. Table 3.4.5.2 gives coefficient of lateral friction and minimum radius for a range of design speeds based on superelevation rates of 0.02 and 0.05 m/m.

Where curve radii less than those in Table 3.4.5.2 are used, or superelevation is unavailable, warning signs in advance of the curve are appropriate.

Lateral Clearance on Horizontal Curves

Lateral clearance to obstructions on the inside of horizontal curves is based on the need to

Table 3.4.5.2 Minimum Radii for Paved Bikeways

Design Speed (km/h)	Coefficient of Lateral Friction	Minimum Radius for Design (m)	
		e = 0.02 m/m	e = 0.05 m/m
25	0.30	15	14
30	0.28	24	21
35	0.27	33	30
40	0.25	47	42
45	0.23	64	57
50	0.22	82	73

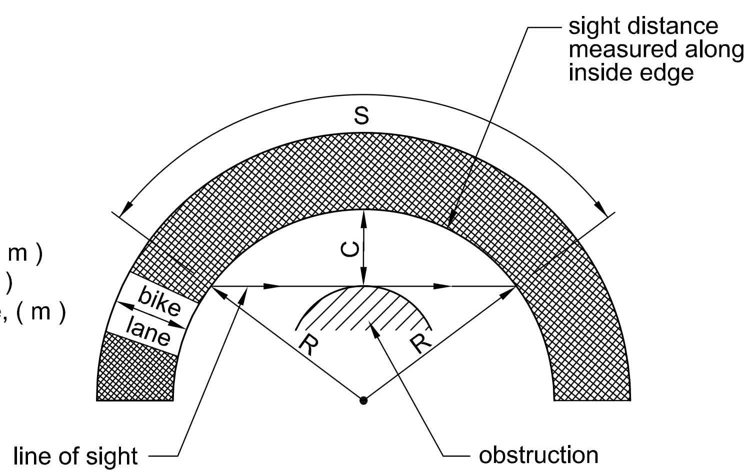
provide sufficient sight distance to an object on the intended path of the bicycle for which the rider has a need to stop. The line of sight to the object is taken to the corner of the visual obstruction, and the stopping distance is measured along the intended path, which is taken to be the inside edge of the inner lane.

Figure 3.4.5.1 illustrates the method of measurement and gives a mathematical expression for the calculation of lateral clearance. Table 3.4.5.3 gives the lateral clearance for a range of radii from 10 to 80 m and stopping sight distances from 10 to 100 m.

Figure 3.4.5.1 Lateral Clearance for Stopping Sight Distance

$$C = R\left[1 - \cos\left(\frac{90S}{\pi R}\right)\right]$$

S = stopping sight distance, (m)
R = radius of inside lane, (m)
C = distance from inside lane, (m)



Note: Formula applies only when "S" ≤ length of circular curve.

Table 3.4.5.3 Lateral Clearance for Bicycles on Horizontal Curves

Radius (m)	Clearance (m) Stopping Sight Distance (m) 10	20	30	40	50	60	70	80	90	100
10	1.2	4.6	9.3	-	-	-	-	-	-	-
15	0.8	3.2	6.9	11.5	-	-	-	-	-	-
20	0.6	2.4	5.4	9.2	13.7	18.6	-	-	-	-
25	0.5	2.0	4.4	7.6	11.5	15.9	20.8	-	-	-
30	0.4	1.7	3.7	6.4	9.8	13.8	18.2	22.9	27.9	-
35	0.4	1.4	3.2	5.6	8.6	12.1	16.1	20.5	25.2	30.0
40	0.3	1.2	2.8	4.9	7.6	10.7	14.4	18.4	22.8	27.4
45	0.3	1.1	2.5	4.4	6.8	9.6	12.9	16.6	20.7	25.0
50	0.2	1.0	2.2	3.9	6.1	8.7	11.8	15.2	18.9	23.0
55	0.2	0.9	2.0	3.6	5.6	8.0	10.8	13.9	17.4	21.2
60	0.2	0.8	1.9	3.3	5.1	7.3	9.9	12.8	16.1	19.7
65	0.2	0.8	1.7	3.1	4.7	6.8	9.2	11.9	15.0	18.3
70	0.2	0.7	1.6	2.8	4.4	6.3	8.2	11.1	14.0	17.1
75	0.2	0.7	1.5	2.7	4.1	5.9	8.0	10.4	13.1	16.1
80	0.2	0.6	1.4	2.5	3.9	5.6	7.5	9.8	10.8	15.1

Note: No value is shown where deflection angle exceeds 180° (stopping sight distance > R).

The lateral clearance values shown occur at the mid point of the curve.

3.4.5.6 Vertical Alignment

Grades

Grades greater than 5% are normally avoided on bikeways. Where there are compelling reasons for exceeding 5%, the length is kept as short as possible and higher design speeds are desirable to accommodate higher speeds in the downhill direction. On long steep upgrades it is desirable to have a relatively flat area of grade, in the order of less than 3%, every 100 m for rest. Where new bike path is proposed, it is preferable to make the route longer to maintain lower grades, than shorter with higher grades.

The normal minimum longitudinal gradient for bikeways is 0.6%. Where surface drainage is provided by adequate cross-slope and lateral slope of the ground away from the bikeway, the minimum grade may be zero.

Crest Curves

The minimum length for crest curves is based on providing at least minimum stopping sight distance (S), as described in Subsection 3.4.5.4. The eye height is taken to be 1.37 m and the object height is taken to be zero, based on the ability to see a fault in the bike path surface sufficiently soon to be able to stop. Where design is predicated on a significant usage by children, a lower eye height may be appropriate. Table 3.4.5.4 gives minimum lengths for design speeds up to 50 km/h and algebraic differences in grade (A) up to 25%. If lengths are required for intermediate values, the table may be interpolated or the formula used. Stopping sight distance is based on level grade. Where there is a significant difference in approach and departure grades, adjustment to the length of curve to account for significant grade differences may be appropriate, based on the values in Table 3.4.5.1.

Sag Curves

Where bike paths are used in the hours of darkness, they are normally, for security reasons, illuminated. There is little need to apply sag curves sufficiently flat to provide sight distance by headlight as for motorized vehicles in non-illuminated areas. The criterion for bicycles, therefore, is comfort. The sag curve

These inexperienced cyclists have the option of making a two-legged left turn by riding a course similar to that followed by a pedestrian, or dismounting from their bicycles and walking them across the crosswalks (Figure 3.4.7.3).

3.4.7.5 Bikeway Ramps

At the intersection of bike paths and roadways, bikeway ramps are typically installed to facilitate movement between the two. Where the potential for conflict between bicycles and motor vehicle traffic is high, consideration is given to appropriate signs warning drivers and cyclists of the conflict area.

Configurations for a typical corner layout and a typical mid-block layout are shown in Figure 3.4.7.6. Alternatives to ramps at bike path / roadway intersections include raised crosswalks and platform intersections, which favour the cyclist over the motor vehicle.

3.4.7.6 Bikeways Crossing Freeway Interchange Ramps

Background

Motorized traffic on interchange ramps tend to operate at fairly high free flow speeds; this tends to increase the risk of more severe collisions. It is critical to provide good visibility and manoeuvring space to allow cyclists and motorists to function / interface effectively and safely. It is therefore desirable to accomplish crossings in the shortest distance feasible, utilizing the highest intersecting angle feasible.

The following sections discuss recommended at-grade crossings; however, grade separation designs utilizing a bicycle path could be used if the approach ramp elevations are appropriate, and if bicycle volumes are fairly high and motor traffic volumes are high. Standard bicycle path geometric guidelines would be applied to the approaches to a grade separated crossing for a bikeway.

Width and Layout

It is desirable to control the location of cyclists in an interchange area on a crossing roadway by utilizing a separate shoulder / bike lane in order to regulate cyclists crossing an interchange ramp, and thus ensure cyclists and motorists each know what to expect of one another. Since manoeuvring space is also a critical factor in reducing the risk of collisions, it is necessary to provide at least a 1.5 metre wide bikeway for each direction of travel within the interchange limits.

Cyclists should yield to motor vehicle traffic, hence, a bike yield sign should be erected to control cyclists; a fairly tight design radius may be used since cyclists should slow and yield at ramp crossings.

Freeway / Expressway Exit (Crossing Roadway Entrance) Type Ramps (Figure 3.4.7.7)

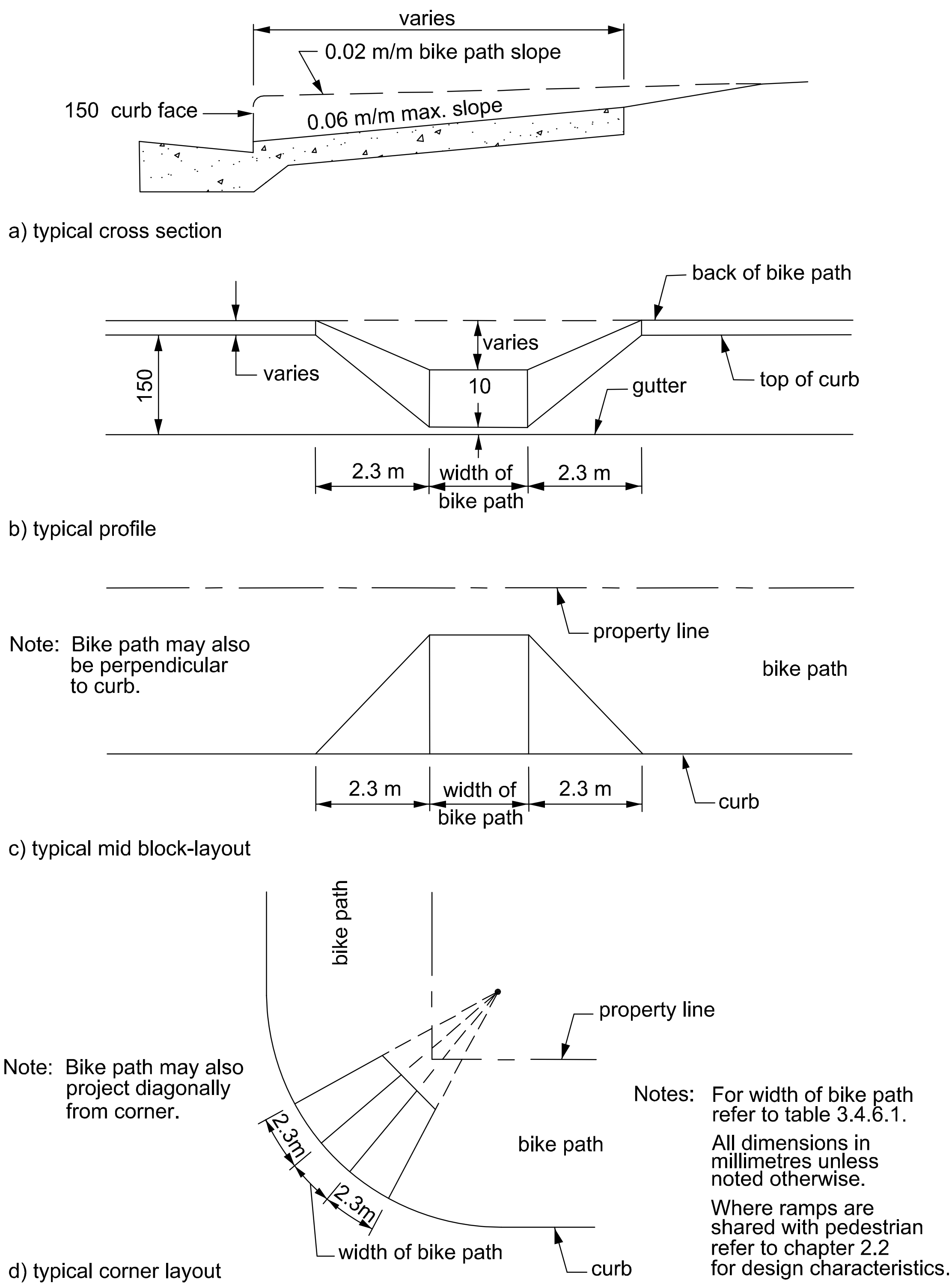
The freeway exit type ramp represents a less hazardous crossing than the freeway entrance type ramp; cyclists can see car / truck traffic, which the cyclist must manoeuvre through, on the freeway exit ramp entering a crossing roadway. As a result, a fairly tight turning radius design for the bikeway can be accepted.

The minimum recommended design parameters are displayed in Figure 3.4.7.7.

Freeway / Expressway Entrance (Crossing Roadway Exit) Type Ramps (Figure 3.4.7.8)

The freeway entrance type ramp is potentially more dangerous than the exit type ramp; cyclists must look over their shoulder to establish the presence of oncoming motor vehicle traffic in order to weave across a ramp lane. It is necessary to guide cyclists to intersect the ramp at an angle which will encourage them to slow and check behind, with adequate distance from the crossing, for vehicles. The cyclists can then establish whether there are any turning cars which may necessitate caution / stopping, or whether the way is clear and they can proceed across the ramp. In order to provide an adequate operating intersection angle, and adequate stopping distance, the cyclists should be guided away from the ramp and then back toward the ramp at an acceptable crossing angle. The design configuration appears in Figure 3.4.7.8 with dimension details given in Table 3.4.7.2.

Figure 3.4.7.6 Typical Bikeway Ramps

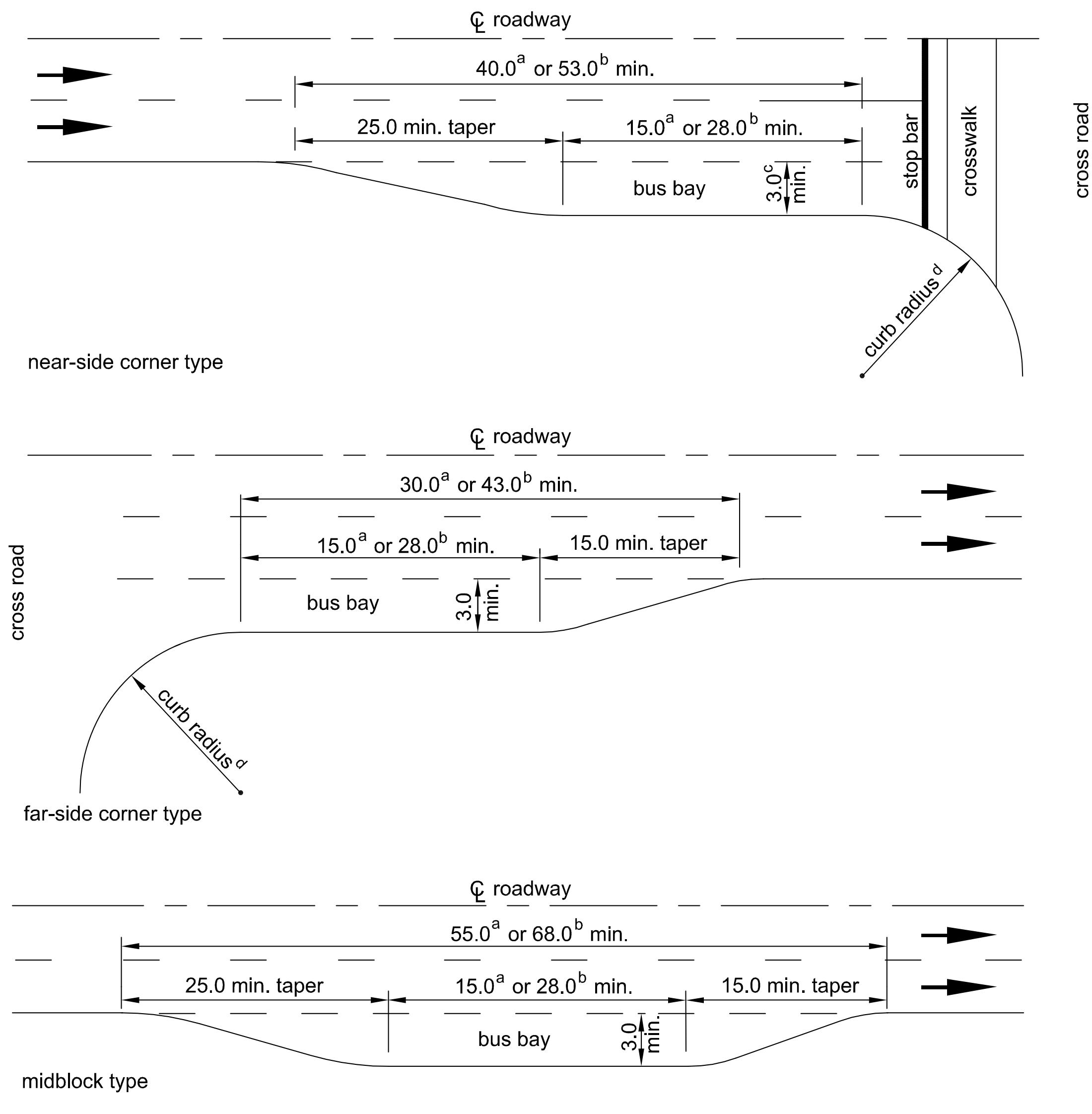


Notes: For width of bike path refer to table 3.4.6.1.

All dimensions in millimetres unless noted otherwise.

Where ramps are shared with pedestrian refer to chapter 2.2 for design characteristics.

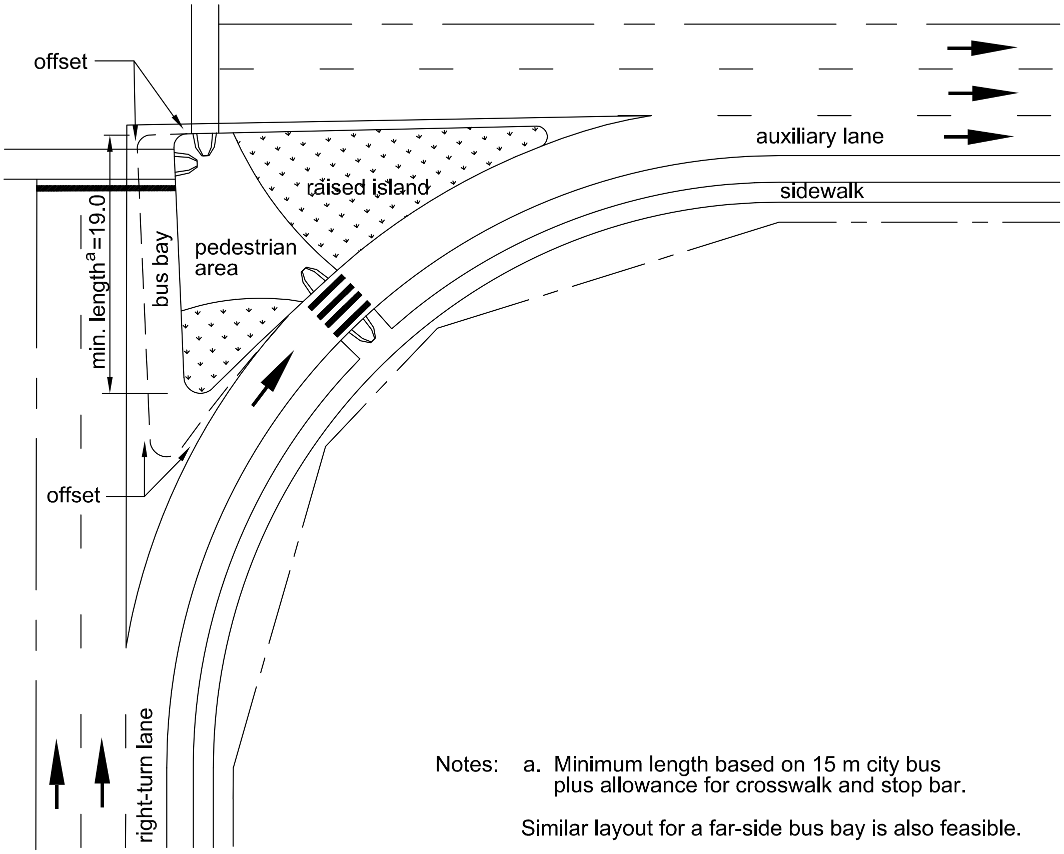
Figure 3.5.2.1 Minimum Bus Bay Dimensions



Notes:
a. Dimensions to accomodate a single city bus.
b. Dimensions to accomodate two city buses.
c. Dimension increased to 3.5 m if used as a combination bus bay, right-turn bay.
d. Refer to Chapter 1.2 for design vehicle turning radius.

For an articulated bus, a bus bay length of 25 m and 30 m tapers are suggested minimum dimensions.

Figure 3.5.2.2 Typical Island Type Bus Bay



Notes: a. Minimum length based on 15 m city bus plus allowance for crosswalk and stop bar.

Similar layout for a far-side bus bay is also feasible.