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Chapter 13

Intersections at Grade

Manual Contents

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Latest Chapter 13 Amendment

Revision Register

Issue/ Rev No.	Reference Section	Description of Revision	Authorised by	Date
6	NA	Glossary of terms moved to the start of the chapter and additional terms added.	Steering Committee	18 Oct 2006
	Various	Inclusion of the results of Arndt (2004) – Relationship Between Unsignalised Intersection Geometry and Accident Rates.		
	Various	Greater awareness of the requirements of cyclists provided. Additional references provided to Austroads (1999b) – Bicycles.		
	13.3.1.1	More information on the design and check vehicle provided.		
	13.4.2	More generic intersection diagrams added at the start of the chapter, in lieu of the previous detailed drawings. All detailed information from the previous drawings moved into the Section on detailed geometric design.		
	13.4.2.2 & 13.7	CHR(S) and AUL(S) turn treatments introduced. Type AUR turn treatment no longer used.		
	13.4.3.5	More explanation on selecting the most appropriate form of traffic control provided.		
	13.4.4	New warrants for unsignalised intersection turn treatments developed.		
	13.5	New section 'Capacity of Intersections' added from Austroads (2005) Intersections at-Grade.		
	13.6.3 & 13.6.4.1	Approach sight distance to a 0.2m high object added for use in some instances.		
	13.6.4	Formulae for sight distance added.		
	13.6.4.6	New section ‘Pedestrian Sight Distance Requirements' added from Austroads (1995) Pedestrians.		
	13.7	Numerous dimensions of the turn treatments have been modified and new diagrams produced.		

Issue/ Rev No.	Reference Section	Description of Revision	Authorised by	Date
	13.7.2	Some guidance provided on the selection of raised medians/islands over painted medians/islands.		
	13.7.2	Clearances between kerbs, medians and islands revised.		
	13.7.2	Clearances between kerbs and edge lines revised.		
	13.7.4.1	Criteria for selecting the length of deceleration lane revised.		
	13.7.4.3	New section 'Acceleration Lane Design for Trucks' added from Austroads (2005) Intersections at-Grade.		
	13.7.10.4	Diagrams of retrofitting CHR turn treatments to four way intersections provided.		
	13.9.2	New criteria for sight distances at accesses introduced.		
	13.9.4	Rural property access requirements modified.		
	13.10	Extended Design Domain (EDD) criteria for intersections moved from Chapter 4 (EDD for Sight Distance and EDD for Turn Treatments) to this chapter.		
	13.10.2	Criteria for Extended Design Domain for sight distance at domestic accesses developed.		
	Appendix G	The section 'Evaluation of Options' moved into an appendix.		

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Chapter 13

Intersections at Grade

Glossary

Special terminology used in this Chapter is given below. Some of the terms are detailed in Appendix 13B. Other definitions may be found in the Glossary of Terms section of the various chapters in this manual. Reference documents, which provide a fuller explanation of some of the concepts contained in this Chapter, are listed under "References" at the end of this Chapter.

accident rate Unless otherwise stated in this chapter, the term 'accident rate' refers to the number of accidents per year. When this term is used for comparative purposes, all variables are held constant (eg traffic volumes, speed, visibility) except those as stated.

approach sight distance ASD Stopping sight distance on the approaches to an at grade intersection. See Section 13.6.4.1.

channelisation A system of controlling traffic by the introduction of an island or islands, or markings on a carriageway to direct traffic into predetermined path, usually at an intersection or junction.

channelised intersection An intersection provided with medians and/or islands for defining the trafficable area and to control specific movement.

entering sight distance (ESD) The sight distance required for a vehicle to enter from a side street and accelerate such that it would not impede traffic on a non-terminating approach, travelling in the same direction. See Section 13.6.4.

furniture At an intersection, this is the equipment, such as sign posts, median kerbs, lighting poles, etc., which is installed to make the intersection work more effectively.

holding line A broken transverse pavement marking which shows motorists where the front of their vehicle should be if they have to wait to enter part of an intersection.

inner lane The lane adjacent to the median in multilane divided roads.

leg A right of way that forms an approach and/or departure to an intersection. The right of way may contain roadways, footways and bikeways.

locking up A traffic phenomenon at intersections in which entering traffic blocks the route of traffic exiting, and brings all movements to a standstill.

major road/major leg When used in the context of an unsignalised intersection, these terms refer to the legs that are continuous through the intersection and have priority ie they do not contain stop or giveway signs.

medians/islands See Section 13.7.1.1.

minimum gap sight distance (MGSD) The sight distance acceptable to a driver to enter or cross a conflicting traffic stream.

minor road/minor leg/side road When used in the context of an unsignalised intersection, these terms refer to the legs

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that do not have priority ie they either end at a T-intersection (where the standard T-intersection road rule applies) or they contain stop or giveway signs.

MUTCD Manual of Uniform Traffic Control Devices (Qld).

near side The left side of a vehicle moving forward (nearest the kerb).

off side The right side of a vehicle moving forward.

outer lane The lane adjacent to the left hand shoulder on multilane divided roads.

platoon A group of vehicles, travelling at substantially the same speed. Platoons often occur with a group of vehicles which were previously queued at a red traffic signal.

point of conflict The road space desired by one vehicle or traffic movement, which is simultaneously required by another vehicle or traffic movement.

road hierarchy A tributary system in which roads are ranked in terms of their functional classification. See Section 13.2.

RUM An acronym for Road User Movement. See Appendix 13G.

safe intersection sight distance (SISD) The distance required for the driver of a vehicle on the non-terminating approach to observe a vehicle entering from a side street, decelerate and stop prior to a point of conflict.

stop line An unbroken transverse pavement marking requiring motorists to stop before entering an intersection and showing where the front of the stopped vehicle should be.

TWRTL An acronym for Two Way Right Turn Lane.

“Y” values The sum of the critical movement flow ratios for the whole of the intersection See Appendix 13B.

Acknowledgement

The work done by staff of the Roads and Traffic Authority (RTA) of New South Wales in the development of this chapter and the assistance of RTA is gratefully acknowledged.

13.1 Introduction

13.1.1 Scope

This Chapter of the Road Planning & Design Manual has been developed from the following sources:

- RTA Road Design Guide - Section 4. Various sections of this chapter that are based the RTA content have been tailored to Queensland conditions and practice.
- The results of an unsignalised intersection study undertaken by Main Roads (Arndt, 2004). The specific accident types referred to this chapter are those identified in Arndt (2004), as shown in Appendix 13F.

This chapter provides details of the planning and design procedure to be followed, and the details necessary to develop a set of working drawings for an intersection at grade.

The procedure is arranged in the same order as the "Design Flow Chart" given as Figure 13.1

The procedure is based on the following basic factors:

- understanding the position of each leg of the intersection in the local road hierarchy;
- identifying and including ALL users of an intersection in the design considerations;
- selecting a "design vehicle" for each movement, and using the turning templates for this vehicle at the speed which is appropriate for the movement;
- using a "check" vehicle (an over dimensional vehicle or other large vehicle that uses the intersection occasionally) to ensure that the vehicle can traverse the intersection without damaging the roadside furniture or other installations;
- developing the geometry of the intersection to orientate vehicles so that visibility is enhanced and relative speeds are reduced;
- using gap acceptance and sight distance criteria as the principal design factors; and
- requiring safety, delay, site suitability, funding and economic criteria to be taken into account in selecting the layout and form of control.

13.1.2 Exclusions

Only "at grade" intersections are covered in this section. It should be read in conjunction with the following:

- Chapter 14 – 'Roundabouts' in this Manual;
- Austroads (1993) 'Guide to Traffic Engineering Practice – Part 6 – Roundabouts';
- Design Vehicles and Turning Path Templates (see Chapter 5);
- "aaSIDRA" user manual;
- Chapter 17 – 'Lighting' in this Manual.

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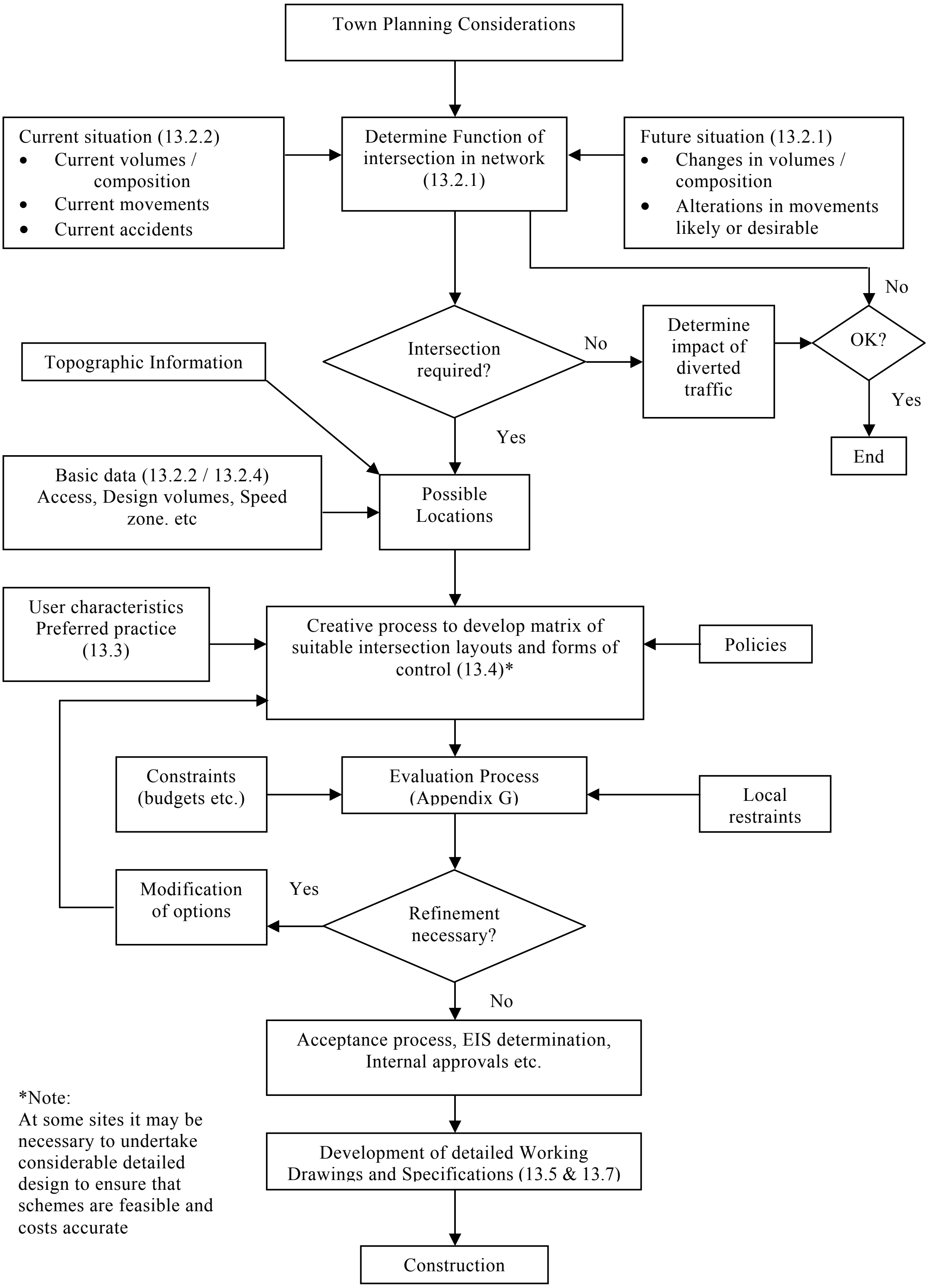

Figure 13.1 Design Flow Chart in the Modelling Process to Produce a Set of Working Drawings for Construction or Modification of an Intersection (numbers in brackets are the reference text detailing that area)

For appropriate signposting and linemarking of intersections at grade refer to either the Guide to Pavement Markings or the Manual of Uniform Traffic Control Devices (MUTCD).

13.1.3 Policy

The operating and safe working characteristics of a road network depend more on the performance of intersections, than on any other single feature. Intersections generally determine the capacity of the network (particularly in urban areas); they are over represented in accident statistics (about 50% of accidents in urban areas are at intersections).

An intersection should be planned and designed for an effective operational life of at least 20 years. This may include a staged implementation (for example ducting initially installed with traffic signals and/or traffic islands to be constructed at a later date).

Road designers and planners should always consider the intersection site as an integral part of the road network. It should not be considered in isolation.

The function of the intersection in the road network should be defined and future changes should be taken into consideration.

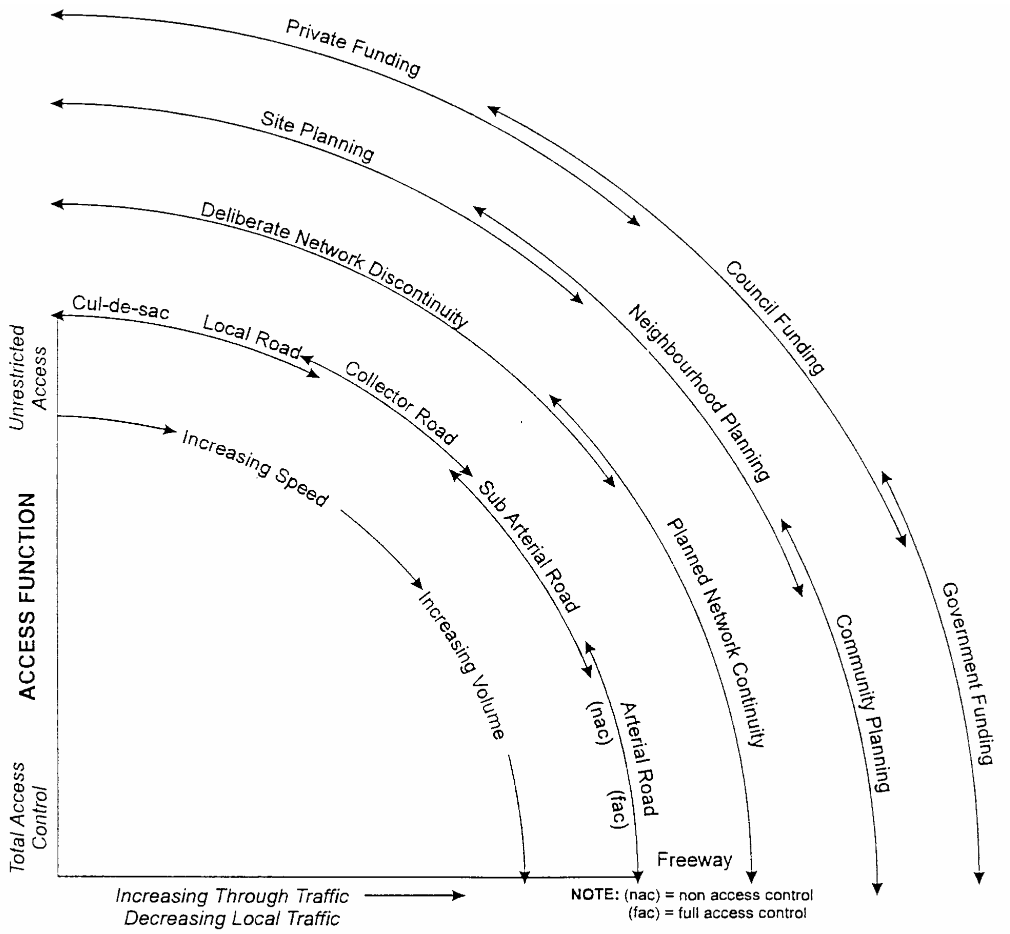

Figure 13.2 Road Hierarchy showing Functions, Characteristics, and Major Sources of Funds

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13.2 Network Considerations

A road network provides for the movement of people and goods. Its relationship to land use is fundamental. Traffic is a function of land use; land use is a function of access. A road network is an integral part of land use and cannot be considered independently.

Traditionally, roads have operated in a dual function mode. A typical road provides for both through traffic movement (movement function) and the movements necessary to support the adjoining land use (access function).

When traffic volumes are low, the dual function can be accepted; as traffic volumes increase, the problems associated with this duality of operation become very important. This can lead to breakdowns in the service provided in both functions as manifested by delays, accidents, and other malfunctions of the network.

It is not always necessary for a road to provide both a movement function and an access function. Current thinking is to allocate functions to roads, based on a hierarchy, as shown in Table 13.1.

Table 13.1 Road Function Based on Placement within Hierarchy

Type	Movement Function	Access Function
Arterial (fac)	Sole Function	Nil
Arterial (nac)	Major	Minimal
Sub Arterial	Significant	Minor
Collector	Minor	Significant
Local	Minimal	Major
(fac) = full access control		
(nac) = partial or no access control		

The structure of road hierarchy is further discussed in Chapter 1.

Intersections are a fundamental part of a road hierarchy. They are the nodes of the system, and determine how effectively the network operates. They govern how effectively each road can perform its allocated function in the hierarchy. Because of this, each intersection has a profound influence on land use and development options.

Three questions have to be answered before any detailed design of an intersection is possible. These questions attempt to establish the purpose of the intersection in the road network.

They include:

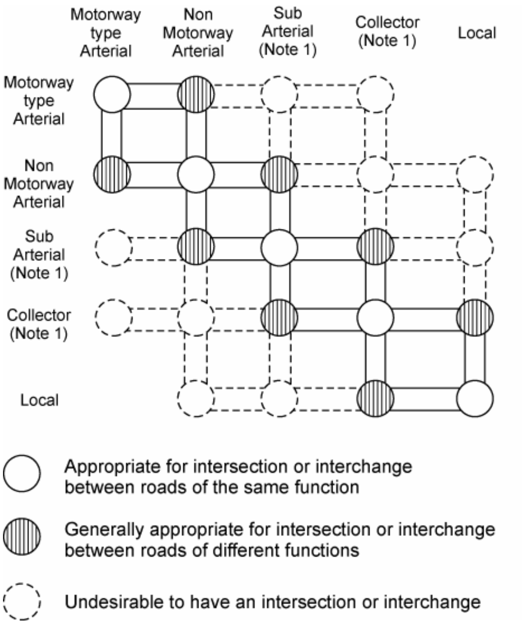
- What function should the intersection fulfil?
- What happens now?
- What changes could occur in the future?

13.2.1 What Function should the intersection Fulfil?

An intersection is not an entity in itself. It must have a purpose defined by the network. In turn, the road network is an integral part of community land use, and cannot be developed independently.

Figure 13.3 shows that there are intersections which connect facilities of the same rank i.e. arterial to arterial, sub arterial to sub arterial, etc. These are shown by the open circles. As well, there are intersections which connect facilities at different levels in the hierarchy, such as arterial to sub arterial, sub arterial to collector, collector to local road. These are shown by shaded circles. Other intersections within the hierarchy may be inappropriate. These are shown by the dashed lines.

It should also be recognised that not all the traffic on the network is in the form of motor vehicles. Pedestrian movements and bicycle networks should be identified so that the points where these modes intersect with each other and the motor vehicle network are known.



Notes:
1: For rural networks it may be difficult to differentiate between Sub Arterial/Arterial roads, and Collector/Local roads.
2: Unfortunately, the administrative classification of a road may not reflect its function in the network.

Figure 13.3 Appropriate, and Inappropriate Intersections or Interchanges as Determined by the Hierarchy (adapted from Bovy, 1972)

Hence, the important first step is to identify the function of each road leg, the conflict points with non motorised modes and determine whether each should form part of the intersection. The network consequences of decisions to delete a traffic movement or a leg from an intersection, or delete the whole intersection, should also be examined before such a decision is confirmed. These consequences are to include the effect on pedestrian and cyclist movement desire lines.

The output from this step in the process is a series of decisions about which legs should form the intersection (if any), the function of each leg in the road hierarchy, and the movements which are allowed and those which should have priority.

13.2.2 What Happens Now?

With the composition of the intersection defined and the function of each movement and leg clarified, the current situation must be quantified at the likely site(s). Basic data that should be collected includes:-

1. Topography at the site(s);
2. Land use, access points to properties, and special site constraints (public utilities, awnings and balconies, trees, monuments, etc.);
3. The current traffic (including cyclists and pedestrians) defined in terms of:

- o Hourly through and turning movements during peak, inter-peak and non-business times,
- o Volumes during special, regular events (holiday periods, sporting fixtures etc.),
- o Composition and approach speed during each of these times (taking into account speed limits),
- o Movements currently given priority or denied,
- o Details of public transport (especially bus and taxi desire lines),
- o Requirements for priority for public transport (if any),
- o Existing preferred bicycle routes,
- o Proportions of through and local traffic,

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- o Performance (acceleration, braking, turning, walking speed, etc.) of the various users;

4. The network effects of the intersection (impacts on adjacent intersections and on the progression of vehicles along major arterial roads);
5. Special network functions existing, or proposed, such as freight routes, bus routes, bicycle routes, etc;
6. Values of economic factors (operating and delay costs, rates for construction and maintenance work, property values, utility adjustment costs, etc.) to be used in the analysis stage;
7. Budget limits; and
8. Special constraints (political commitments, flood levels, etc.).

13.2.3 What Changes could occur in the Future?

The objective here is to scan the possible future to define likely events which will have an impact on the operation of the intersection. Issues include:-

- Changes in traffic volumes or composition;
- Alterations to the road hierarchy;
- Alterations in turning movement volumes;
- Changes in land use;
- Adjustments to the speed zones;
- Changes in the form of traffic control (e.g. will traffic signals ever be installed?);
- Foreshadowed amendments to traffic regulations; and
- Planned route changes for trucks, buses and/or bicycles (including changes to the bicycle network).

13.2.4 Output

The output from the process of considering the intersection as part of the network is a report documenting:

(a) the decision on whether an intersection is required at all;

(b) the determination about the legs which are to form the intersection, the function of each leg (especially the type of access controls which will apply at the intersection site, and on each approach/departure leg) and a determination of any restrictions in movements that might be omitted from the intersection;

(c) confirmation of the speed zoning which will apply on the approach and departure side of each leg in the immediate and longer term, and the desirable speed of turning movements (particularly during times of low flow);

(d) the hourly traffic volumes (vehicles, bicycles and pedestrians) to be used for the design, the movements which should receive preference, and the movements which are to be positively discouraged, or denied;

(e) the network impacts of any discouragement or denial of a movement(s) at the intersection;

(f) the definition of public transport requirements (priority, movements, where services should stop, if the services should stop in a bus bay or in the traffic lane, etc.);

(g) the decision on the special provisions for movements which should be incorporated (especially emergency services);

(h) the constraints which apply (site and/or other);

(i) the need for future changes in the form of traffic control; and

(j) details of network influences (such as downstream intersections) that will affect the operation of the intersection at the site.

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13.3 User Considerations

13.3.1 Basic Vehicle Characteristics

13.3.1.1 Design and Check Vehicles

The design vehicle is the largest vehicle likely to regularly use a movement. On most arterial roads this would be a 19.0m long articulated vehicle, although a “B-Double” or “Road Train” may be the design vehicle at some locations. In urban areas a 14.5m restricted route bus may be the design vehicle.

The check vehicle is larger than the design vehicle and is one that may occasionally use a movement. The check vehicle may be permitted to run over kerbs and encroach on adjacent lanes. “Permit” vehicles (i.e. over-width or over-length vehicles travelling with a permit) are not to be used as a design vehicle unless they regularly feature on the route. Instead, "permit" vehicles are often used as the check vehicle.

Dimensions of various vehicles are given in Chapter 5.

13.3.1.2 Design Vehicle Swept Path Template

The computer program VPATH, in conjunction with turning path templates, forms the basis of geometric design of intersections. All intersection layouts MUST allow the design vehicle to negotiate the intersection with the required minimum clearances to roadside features eg kerb lines. This must be shown by application of turning path templates for the design vehicle. A wide range of turning path templates, at various scales, is available in Austroads Design Vehicles and Turning Path Templates. Each template is marked with the operating speed.

Vehicle turning paths must also be generated for the check vehicle to ensure it can negotiate the intersection.

The desirable minimum turning path radius is given by the template covering speeds of 5 km/h to 15 km/h. This gives a minimum radius of 8.0m for cars, and 15.0m for heavy vehicles (20m for B-Doubles). General use of an absolute minimum 12.5m radius for heavy vehicles should be avoided as this can only be achieved by turning onto full steering lock at very low speeds. Conversely, large curve radii at intersections may promote higher than desirable speeds. Accordingly, the desirable speed for each turning movement must be established.

Vehicle turning path templates should be applied to accommodate the swept path, plus a minimum offset of 0.6m from the extremities of the vehicle path to a kerb or pavement edge. This offset can be overlooked for local access/minor roads in rural areas where the shoulder is partly sealed. However, the turning path should not cross the centreline of the minor road in general (see Figure 13.71 for an exception to this). Note that a vehicle, 7.5m or more in length, travelling wholly in the lane adjacent to the nearside lane may turn left subject to displaying the appropriate signs on the rear of the vehicle. The same applies to a right hand turn from a one way street / carriageway. Templates should only be applied for these circumstances where designs cannot achieve left or right turns from the proper lane because of physical or economic constraints and low volumes of heavy vehicle turning movements occur.

The computer program VPATH should be used to confirm designs. This program is particularly useful for checking complex combinations of geometric elements and unusually configured or overwidth/ overlength vehicles. See Appendix 13C for a description of VPATH.

Note that other software (e.g. Auto-Turn and Auto-Track) may be able to produce satisfactory results.

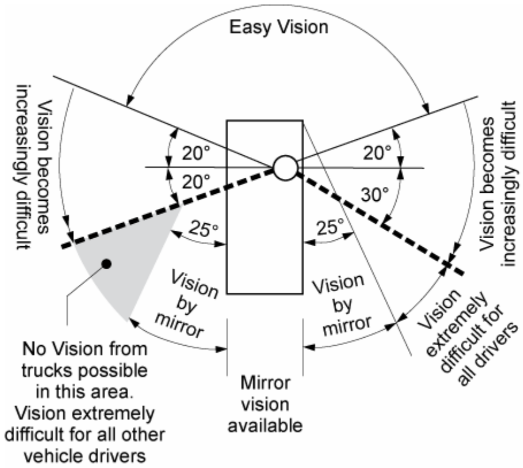
13.3.1.3 Visibility from Vehicles

Observation Angle

There are no design rules dealing with visibility from vehicles. Ackerman (1989) provides the visibility angles shown in Figure 13.4. At each conflict point, the vehicle paths, and orientation should be developed with these visibility angles in mind. The maximum desirable angles are shown by the dotted lines.

Policy is that road centre lines should be designed to intersect at between 70° and 110° in both urban and rural situations. For curved alignment the angle of the tangent(s) at the intersection point should not be less than 70°, or more than 110°. The orientation of vehicles at all points of conflict (such as left and right merges and left turn slip lanes) are to be arranged with Figure 13.4 in mind.

Arndt (2004) found that larger observation angles increased Angle-Minor vehicle accident rates (accidents resulting from minor road drivers failing to give way and colliding with drivers on the major road – refer Appendix F for more details). The observation angle was measured between a line representing the instantaneous direction of travel of minor road drivers 4m behind the holding line and a line tangential to the major road. This relationship, shown in Figure 13.5, confirms the need to limit the observation angle, and therefore, the skew of the intersection.



See Section 13.7.5 for design requirements for vehicles turning right.
See Section 13.7.11 for design requirements for vehicles turning left.

Figure 13.4 Sight Restrictions due to Vehicle Design (adapted from Ackerman, 1989)

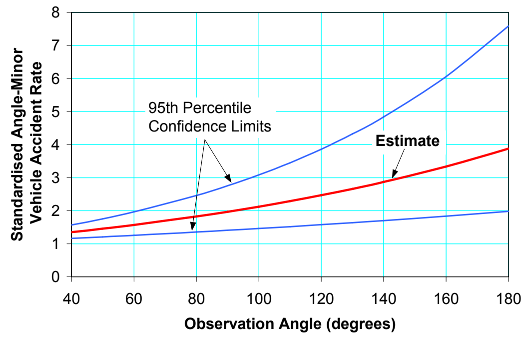


Figure 13.5 Effect of Observation Angle on Angle-Minor Vehicle Accident Rates (Arndt, 2004)

Number of Stand-up Lanes on the Minor Road at Unsignalised Intersections

Arndt (2004) showed that the Angle-Minor vehicle accident rate at unsignalised intersections with two stand-up lanes on the minor road is significantly higher than for one stand-up lane. A free left turn lane did not constitute an additional stand-up lane.

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The higher accident rate can be attributed to vehicles in the offside stand-up lane blocking visibility for vehicles in the nearside lane, and vice versa, as illustrated in Figure 13.6.

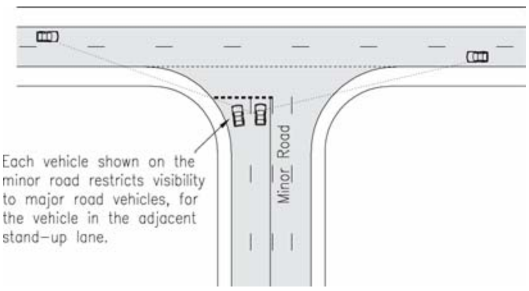


Figure 13.6 Restricted Visibility at an Unsignalised Intersection Comprising Two Stand-Up Lanes on the Minor Road

The Angle-Minor vehicle accident rate was found to be 1.5 times higher for those conflict points where there was an adjacent minor-road stand-up lane in the direction of the relevant oncoming major road vehicles.

At Tee intersections, this may not be a major problem because Angle-Minor vehicle accident rates for the various conflicts are generally low (except for one conflict type which is not affected by the visibility restrictions due to adjacent vehicles).

At cross intersections, however, visibility restrictions due to adjacent vehicles will substantially increase an already high Angle-Minor vehicle accident rate for conflicts involving through movements from the minor road.

For the above reasons, only one stand-up lane should be provided on minor road approaches at unsignalised intersections, particularly at four-leg intersections with heavy through movements from the minor legs. Where two lanes are required for capacity reasons, installation of a left-turn slip lane or signalisation of the intersection should be considered.

Queuing through Intersections

Arndt (2004) found that Angle-Major vehicle accident rates (accidents resulting from a right turning major road driver failing to give way and colliding with an oncoming vehicle – refer to Appendix F for more details) can be up to three times higher on multi-lane roads where queuing occurred on the opposite major road leg. Such queues typically formed due to a “blockage” downstream of the unsignalised intersection, eg an intersection with traffic signals. A diagram of this scenario is shown in Figure 13.7.

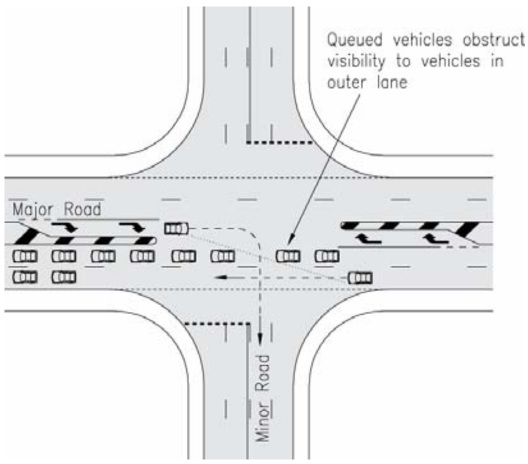


Figure 13.7 Queuing Through an Unsignalised Intersection due to a Downstream Blockage

The higher accident rate can be attributed to the queue in the offside lane obstructing the view to vehicles travelling at speed in the nearside lane.

To mitigate this problem, queuing from downstream blockages on multi-lane roads should not extend through upstream intersections. This may require an analysis of the downstream blockage to identify treatments to prevent the queue extending into the intersection (eg if the downstream blockage is a signalised intersection, change the phasing or increase the number of stand-up lanes).

13.3.2 Important Characteristics of People

13.3.2.1 Primary Requirement

The primary requirement in all road design is to remember that people are involved in roles varying from driver to cyclist and pedestrian.

The application of this requirement at intersections results in two basic principles:

- People (especially tourists) should not be surprised by the location of the intersection, or the layout, and
- They must not be severely disadvantaged for making errors, nor rewarded for deliberately committing unsafe acts.

The first of these principles requires actions such as close attention to advance signposting, (particularly where lane drops occur), a consistency of treatment along routes (so that drivers can have reasonable expectations about intersection treatments), and so on. The second principal requires designers to imagine the actions of those drivers who have made an error at the junction and ensure that recovery action is not unduly hazardous.

The layout should also meet general driver expectations and good design practice, including the provision of adequate distance for lane drops and run-out areas and avoiding right to left merges. However, the layout should also discourage unsafe acts, such as overtaking through narrow intersections on the near side.

13.3.2.2 People as Pedestrians/ Cyclists

The free speed and the speed of people walking in groups can be determined from Chapter 5 - Traffic Parameters and Human Factors. Grade requirements for pedestrians are also covered.

Chapter 5 also deals with the skills and needs of cyclists.

13.3.2.3 People as Drivers

Sight Distance

There are few aspects of intersection design more important to drivers than sight distance. Drivers must clearly see:

- the path they have to follow;
- the position of holding lines, lane lines and turning lines;
- points of conflict with other users;
- prospects of a collision occurring; and
- gaps of sufficient length to be able to make the proposed manoeuvre.

Details of the sight distance required at all intersections are given in Section 13.6.4.

Gap Acceptance Behaviour of Drivers

Gap acceptance behaviour is complex, and results of several studies are inconsistent. Some observers have noted that some form of “trade- off in risk” is part of the process.

For example, at intersections where sight distance is restricted, a driver may accept a shorter gap than in a more open environment.

As well, the mathematical models available are usually restricted to either fully random arrivals, or arrivals at a uniform rate. These may be invalid when there is some mechanism for "platoons" of vehicles to arrive, with large gaps between each platoon (e.g. downstream of a signalised intersection).

The minimum gap sizes which are accepted by the 85th percentile driver t_a are given in Table 13.7 (see Section 13.6.4.4). Also shown is the gap required by the next queued vehicle(s) t_f. These values, t_a and t_f can be used in the absence of site specific information. Figure 13A.1 (Appendix 13A) shows the likely number of gaps in various flows and the traffic volume which may enter for various values of t_a and t_f.

Values of gaps may be further modified by sighting requirements at turns. Such modifications are detailed in Sections 13.7.6 for right turns, and 13.7.11 for left turns.

Merge/Diverge Behaviour

Merging from the right side of the carriageway is undesirable and should not be used unless no other options are available. In undertaking such a merge, truck drivers rely solely on mirrors to view vehicles in the adjacent lane. Preferred practice is to provide a dedicated lane for a right turn movement which is required to run simultaneously with a through movement. Where this requires a lane drop, the possibility of merging the near side lane should be examined.

An emergency run off area free from hazards is required at the end of all merges (see Figure 13.45 in Section 13.7.4.2).

13.3.3 Safety

Intersections involve traffic conflicts. Where there are conflicts there is risk of accidents. Obviously, denying movements, or even closure of the intersection, will improve safety at a site. Such actions would be at the expense of other factors (such as local access). It may result in the accident problem being moved to another site, with no improvement to the overall network safety, and a possibility that total network safety is reduced. The objective is to obtain the appropriate balance between risk and the other network performance parameters that apply.

Any existing accident data must be critically reviewed to determine whether past events are likely to be repeated. The best indicator that remedial action is necessary at an existing site is that accident rates are consistently high. Even then, it is important to distinguish between the types of accidents occurring. For instance, collisions which have high relative speeds generally result in more severe accidents; where pedestrians are involved there is a high probability that an accident will be serious, even if the relative speed is low. At roundabouts, the total number of accidents may be high but the use of such devices in urban areas generally results in a significant improvement in casualty accident rates.

Methods of modelling intersection safety are not particularly well developed (Arndt and Troutbeck, 2001). Safety at intersections appears to be a function of exposure, speed, number of conflict points, sight distance, and other factors which are site specific. A high percentage of accidents involve some degree of human error by drivers. The likelihood and consequences of such human error can be reduced by the consistent application of the following safety principles:

- *Exposure Control* - reducing risks by reducing the amount of movement, or by substituting safer activities for relatively unsafe actions (e.g.

separating motor vehicles from unprotected road users);

- *Crash prevention* - system design, operation and condition, reducing risk by meeting driver expectation;
- *Behaviour modification* - user education and appropriate enforcement;
- *Injury control* - reduce severity of crash by vehicle design and removal of hazards from roadside; and
- *Injury management* - provide treatment quickly to those who have been injured.

The application of these safety principles at intersections results in the four general rules which follow. Safety principle(s) met are bracketed.

13.3.3.1 Reducing and Separating the Points of Conflict (exposure control, injury control)

The reduction, or minimisation of conflicts is particularly important. As shown in Figure 13.8, there are four basic types of intersection manoeuvres involving conflicts, namely:

- diverging in which the vehicle following is forced to slow (D);
- merging (M);
- crossing (C); and
- weaving (W).

An example of the number of conflicts under various arrangements is given in Table 13.2. (See also Appendix 13B).

Points of conflict can be separated/reduced by the addition of deceleration lanes, realignment of the intersection, etc. Figure 13.9 gives examples of conflict reduction.

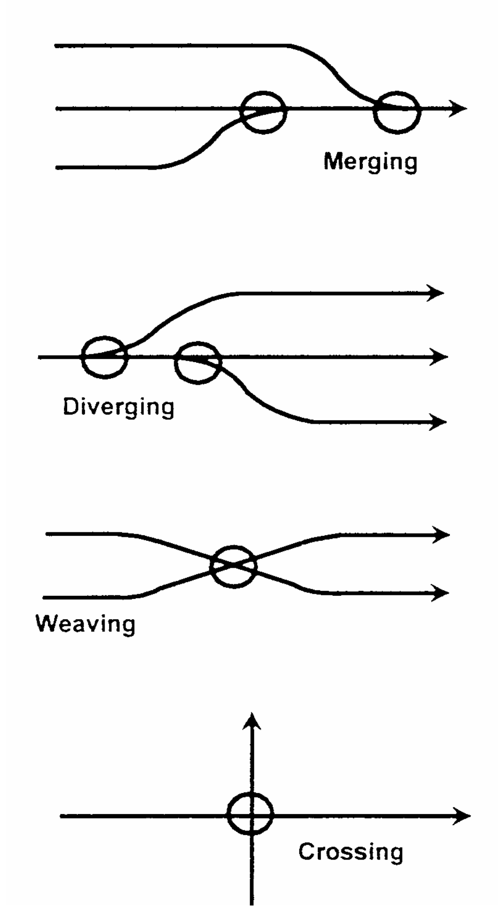


Figure 13.8 Basic Forms of Points of Conflict

Table 13.2 Number and Types of Conflicts

Intersection Type	Method of Control: No Positive Control	Method of Control: Signals *	Method of Control: Round-about
3 Way	3D, 3M, 3C (9)	3D, 1M, 1C (5)	3D, 3M or 3W (6)
4 Way	8D, 12M, 12C (32)	8D, 4M, 4C (16)	4D, 4M or 4W (8)

D = diverge conflict
M = merge conflict
C = cross conflict
W= weave
() = Total conflicts
Note: * Basis of assessment is two phase operation; conflicts can be reduced or minimised under split phase operation.

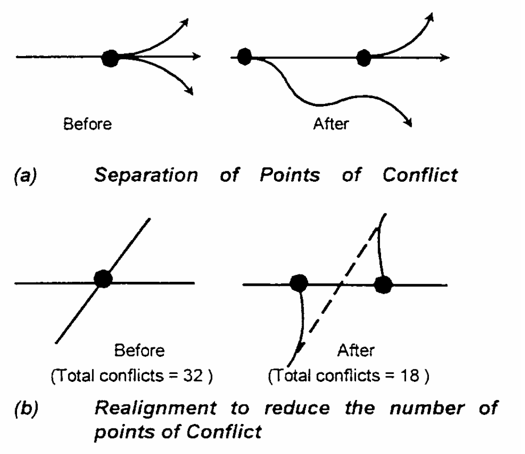


Figure 13.9 Examples of Separating the number of Points of Conflict at an Intersection

The number and types of conflicts given in Table 13.2 are often used to explain why certain intersection types are safer than others. For example, a 4-way roundabout is safer than a 4-way unsignalised intersection because of the lower number of conflict points (8 as opposed to 32 respectively).

This comparison is very simplistic and should only be used as a general rule. It can not explain how some intersection comparisons do not follow the indicated safety performance. For example, it cannot explain how a 4-way roundabout under certain conditions can sometimes record a higher accident rate than 4-way unsignalised and signalised intersections.

There are several reasons why some intersection comparisons do not follow this general rule. Some of these are as follows:

- Some of the major accident types occurring at intersections are not included in Table 13.2. For example, single vehicle accidents and rear-end vehicle accidents on the entry curve can be the predominant accident type at roundabouts in high speed areas. Table 13.2 does not consider such conflict types.
- Not all of the conflict points in Table 13.2 have the same exposure. Certain conflicts may involve low traffic volumes, thus being much less likely to record an accident.
- There are several other parameters that strongly influence accident rates that are not considered in Table 13.2 eg visibility and relative speed. The influence of these parameters can vary between conflict points and between intersection types. Some of these parameters are discussed in the following sections.

Arndt (2004) found that all of the traffic flow variables in the unsignalised intersection study were to a power less than one. This suggested the following hypothesis. For a particular section of roadway, total safety is improved by providing a smaller number of intersections that carry higher side road volumes, rather than providing a greater number of intersections that carry lower side road volumes (the total traffic flow from the minor roads being the same in both cases).

This hypothesis supports the notion that the number of intersections along a roadway should be limited, subject to capacity and delay considerations. In effect, this is another method of reducing the number of conflict points.

13.3.3.2 Keeping it Simple (exposure control, crash prevention)

Complicated intersections have poor accident records. A fundamental check is to imagine what a driver, using the intersection for the first time, would do. Two requirements are paramount:

- No driver should need special knowledge of how to negotiate the intersection.
- There should be a clear order of precedence within the intersection.

Arndt (2004) found that those conflict points at unsignalised intersections with the highest driver workload recorded the highest accident rates. One of the causes of a higher driver workload was a greater number of legs (ie drivers on a minor leg at a four-way intersection have to observe gaps in a greater number of traffic streams than drivers on the minor leg at a T-intersection).

13.3.3.3 Minimising the Area of Conflict (exposure control, crash prevention)

Minimising the area of conflict is achieved by reducing the area of pavement where conflict can occur by defining vehicle paths as shown in Figure 13.10.

13.3.3.4 Controlling Speed (exposure control, crash prevention, behaviour modification and injury control)

Alteration of approach alignment and channelisation can reduce approach speeds and the relative speed. A properly designed roundabout is a good example of this treatment. The improvement in relative speed is illustrated in Figure 13.11 and Figure 13.12. (See also Appendix 13B).

Arndt (2004) found that conflict points at unsignalised intersections with the highest relative speeds recorded the highest accident rates.

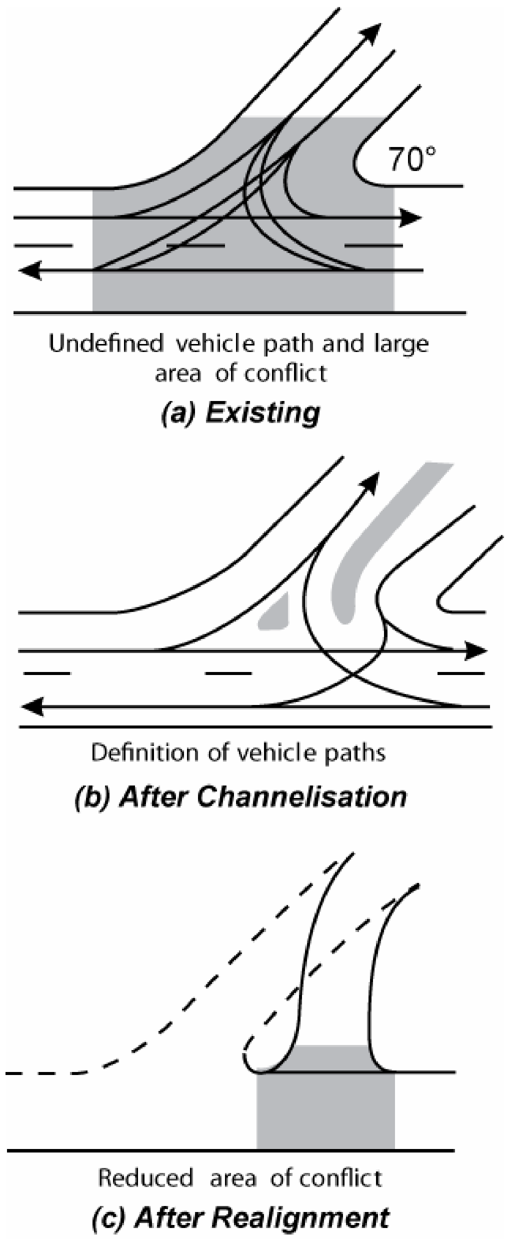


Figure 13.10 Sketch showing the Way Existing Points of Conflict can be Localised by Channelisation and the Area of Conflict Reduced by Realignment

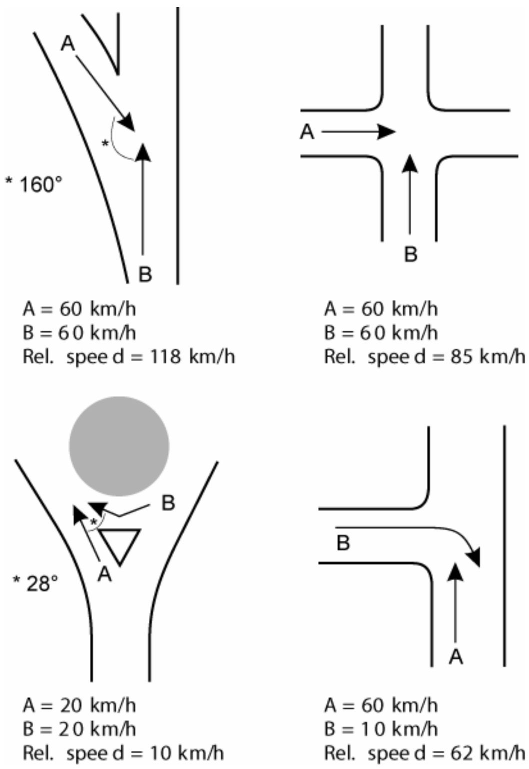


Figure 13.11 Potential Relative Speed at Intersections

(Source: Ogden and Bennett, 1989)

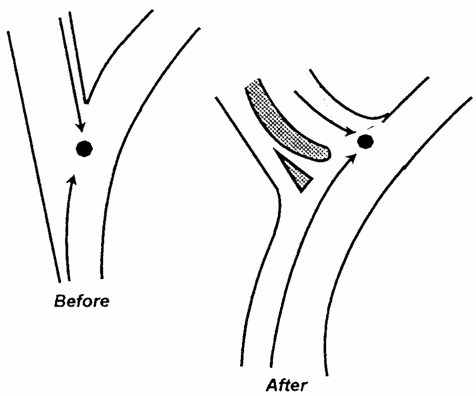


Figure 13.12 Improvement in Approach and Relative Speed due to Realignment and Channelisation

Behaviour of drivers

The uniform application of intersection control devices is an essential factor in the safe and efficient operation of the road system. Drivers tend to establish an expectancy with regard to the type of devices to be used in various situations. To achieve such consistency, the following guidelines are important:

- avoid situations where a through lane (especially next to the median) becomes an exclusive turn lane. Where this cannot be avoided there must be clear signposting, sited well before the intersection, diagrammatically showing what to expect.
- avoid right to left merges. Where they can not be avoided, provide control devices and signage to provide separation between the merging traffic streams.
- avoid merge lanes that are too short on the departure side and merge lanes that do not have adequate run-out areas.
- maintain consistency in the appearance of intersection types and forms of traffic control.
- ensure that forms of traffic control are appropriate to the site and not disobeyed with impunity.
- remove advertising which imitates traffic control devices or which give directions to traffic.
- provide adequate recovery areas for drivers who "get it wrong".
- monitor intersections to identify unusual movements, or where "decision overload" situations are occurring.
- ensure that the priority of each intersecting stream is obvious to drivers and other users.

13.3.4 Summary

Intersections must be designed with ALL users, and their vehicles (where appropriate) in mind. This means:

- Locating the interaction to allow drivers to see the intersection, identify the paths to follow, read important messages, locate points of conflict with other users, and determine priority;
- Providing general intersection geometry to suit the dimensions, approach and departure speed, and capability of the design vehicle(s);
- Orienting vehicles in the intersection, using appropriate geometry, to allow drivers to readily see other road users;
- Allowing sufficient gaps to occur so that delays are kept to reasonable levels (for both pedestrians and vehicles);
- Promoting safety by keeping intersections as simple as possible, reducing and separating points of conflict, reducing the area of pavement where conflict can occur and controlling speed;
- Making the layout “forgiving” so that people who get it wrong are not severely disadvantaged without rewarding poor driving practice;
- Ensuring that the network function is recognised and supported;
- Establishing a consistency of treatment along the route so that people who are unfamiliar with the location can make intersection decisions quickly and accurately; and
- Reviewing layouts so that emergency vehicles can negotiate the intersection with minimal difficulty.

13.4 Options for Intersection Layout and Form of Control

13.4.1 General

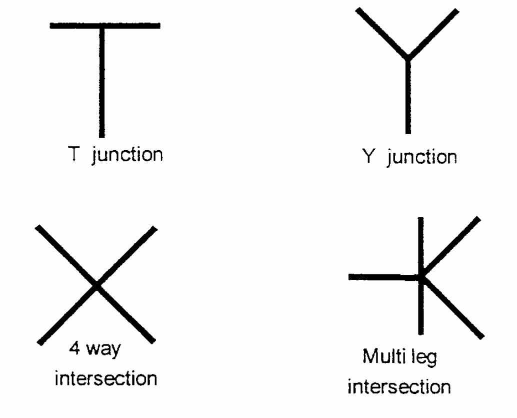
The purpose of this part of the process is to consider all layout options and forms of traffic control weighing up the associated advantages and disadvantages of each. The output will be a list of options which should be further investigated. A list of options discarded, together with reasons for their rejection, should also be part of the output.

As part of this process, the designer must take a critical look at the sites available so that topographic and other location advantages and disadvantages associated with each layout can be identified. As it is difficult to make any intersection layout work properly at a poor location, time is well spent ensuring that the best available site is adopted.

In urban situations, the choice of location of the intersection is usually limited by the layout of streets and the constraints of property development.

There is a variety of layout options available for an intersection. These can be broadly classified as "at grade" or "grade separated". It will be noted that grade separated layouts are automatically selected by legs being designated as full access control arterials (ie. motorways). Grade separated layouts are dealt with in Chapter 16.

The number of legs at an intersection, and the angle at which they meet, can vary. One or more legs can be curved. Some terminology is illustrated in Figure 13.13.



NOTE: Some, or all legs may be curved. Combination of these basic elements are often used, eg. a staggered T.

Figure 13.13 Terminology used to Describe the Number, and the Angle of the Legs of an Intersection

The layout of intersections in urban areas will be highly dependent on the pattern of the road system, the volumes and directions of the traffic using the intersection, and the constraints of the site and surrounding development. Options in urban areas may not always be a standard solution and must be developed for the conditions prevailing at the particular site using the principles described in this section.

A brief discussion on the advantages, and disadvantages of each type of layout is given in Section 13.4.2. Sketches are sometimes sub-divided into left (L) and right (R) turn treatments to emphasise that it is the volume of turning traffic on an approach that determines the layout for that movement. It is appropriate to mix layouts (e.g. to have two type BAL intersections to form a four-way intersection) at a particular site.

Arrangements at property entrances are discussed in Section 13.9.

Choice of the form of control is from the following list:

- Traffic regulations, which can be augmented by signs to clarify priority. This is one type of unsignalised control.
- Priority signage, by using signs such as "give way" or "stop" to over-ride regulations. This is often required to give priority to the major movement. This is another type of unsignalised control.
- Traffic signals.
- Roundabouts (which are a specialised form of channelised intersection having their own set of regulations) - see Chapter 14.

Discussion on forms of traffic control is given in Section 13.4.3. Table 13.3 provides a summary.

13.4.2 Advantages and Disadvantages of Various Layout Options

This section discusses advantages and disadvantages of the various layout options. Whilst this section predominantly discusses layouts for unsignalised intersections, some elements are applicable to the other forms of control. The layout options given in this section are applicable to both urban and rural sites.

13.4.2.1 Basic Intersection (type BA)

This is the simplest layout. It is designed to be as compact (and inexpensive) as possible. It is most appropriately used where the volume of turning and through traffic is low.

Carriageways intersect with an appropriate corner radius and taper to suit the swept path of the design vehicle. It can be used with any wearing surface.

"BA" Turn Treatments

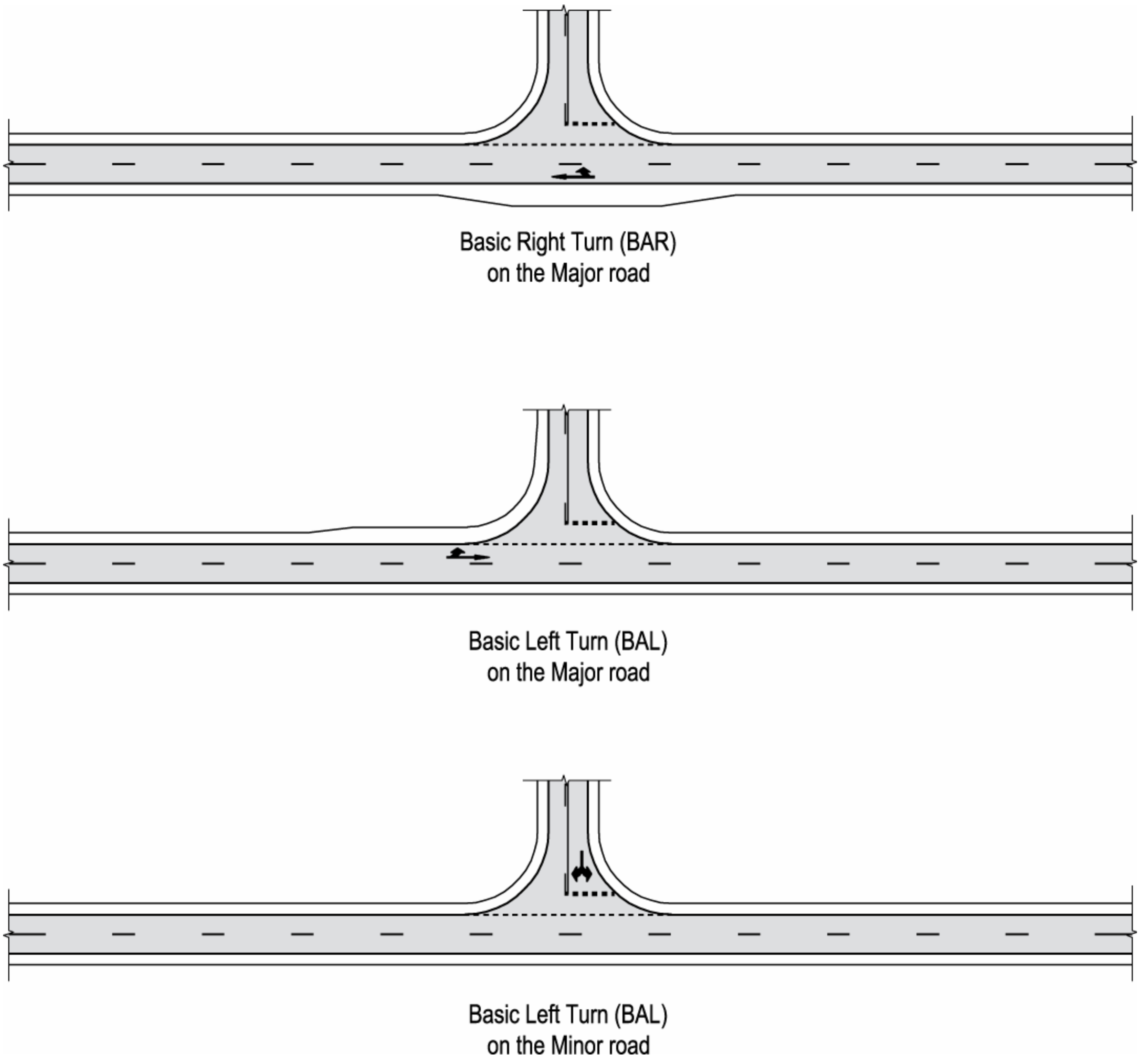
A "BA" intersection comprises the following turn treatments:

- Basic Right Turn treatment (BAR) on the major road;
- Basic Left Turn treatment (BAL) on the major road; and
- Basic Left Turn treatment (BAL) on the minor road.

These treatments are shown in Figure 13.14. Often, not all of the treatments will be used together at a single intersection.

BAR turn treatments are used on two-lane roadways only i.e. they do not apply to multi-lane roadways. A feature of the BAR turn treatment is a widened shoulder on the major road that allows through vehicles, having slowed, to pass turning vehicles. A feature of the BAL turn treatment on the major road is a widened shoulder, which assists turning vehicles to move further off the through carriageway making it easier for through vehicles to pass. Where the major road is sealed, it is preferred that the widened shoulders are sealed, unless the shoulders can be maintained with a sound and even surface in all weather conditions.

Rear-End-Major vehicle accidents are generally rear-end type accidents resulting from a through driver colliding with a driver turning right from the major road - refer to Appendix F for more details. Arndt (2004) found that Type BAR turn treatments record a Rear-End-Major vehicle accident rate 52 times higher than do CHR turn treatments (CHR turn treatments are discussed in Section 13.4.2.3). That is why BAR turn treatments are usually limited to intersections with low volumes only.



Note: Arrows indicate movements relevant to the turn type. They do not represent actual pavement markings.

Figure 13.14 Basic Intersection Turn Treatments "BA"

Arndt (2004) found that some BAR turn treatments (and AUR – refer Section 13.4.2.2) in the study comprised a narrow median. The Rear-End-Major vehicle accident rate was found to decrease substantially with median width, regardless of the type of median (painted, raised or depressed). The median enables the right turning vehicle to be positioned further away from the point of conflict in the through lane, lowering the probability of the vehicle being struck.

Providing a median at a BAR turn treatment is unlikely to be a practical design consideration in many cases. However, there may be scope at some existing BAR treatments to consider introducing such a median by reducing the shoulder width. This may be a low cost option of achieving a reduction in the Rear-End-Major vehicle accident rate.

"MNR" Turn Treatments

A basic right-turn treatment on a multi-lane undivided road is the MNR turn treatment (multi-lane undivided road with no specific right-turn facility). A layout of this type is shown in Figure 13.15.

Arndt (2004) found that MNR turn treatments record the highest Rear-End-Major vehicle accident rate of all the turn treatments (100 times higher than CHR turn treatments). This result likely reflects the fact that MNR turn treatments, unlike any other turn treatment, provide no specific facilities for through vehicles to avoid turning vehicles.

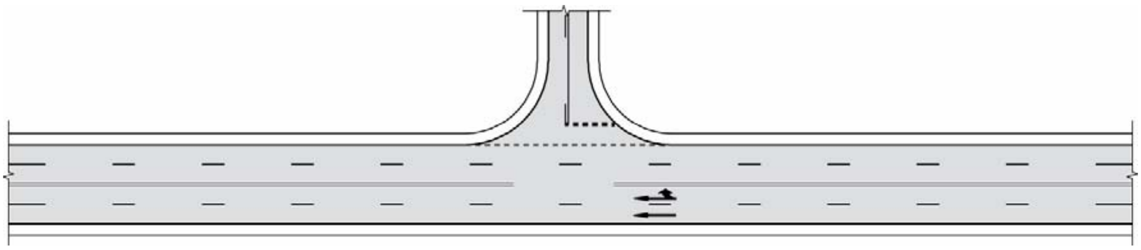
MNR turn treatments should only be retained at existing sites where no other solutions are feasible. They should not be incorporated into new unsignalised intersection designs.

13.4.2.2 Intersections with Auxiliary Lanes (type AU)

Type AU intersections comprise short lengths of auxiliary lane to improve safety, especially on high speed roads. Such layouts allow traffic to bypass a vehicle waiting to turn right, or a lane for left turning traffic, or both. This layout can only be used on legs which are sealed.

This layout can be confused with an auxiliary lane for overtaking and should only be used at locations where the driver can appreciate the purpose of the lane. Situating such intersections near auxiliary lanes used for overtaking must be avoided.

AU type layouts have been used at intersections where an arterial meets with sub-arterials, collectors, or local roads (particularly in rural areas where high speed, low volume traffic occurs and the volume of turning traffic is sufficient to make a conflict likely). They are more expensive than basic intersections, but can work out more cheaply when long term accident costs are included in the estimating. As there are pavement markings associated with this option, approach sight distance (1.15 to zero) must be obtained.



Note 1: This turn type is not to be used at new unsignalised intersections.
Note 2: Arrows indicate movements relevant to the turn type. They do not represent actual pavement markings.

Figure 13.15 Multi-lane Undivided Road with No Specific Right Turn Facility "MNR"

"AU" Turn Treatments

An "AU" intersection comprises the following turn treatments:

- Auxiliary Right Turn treatment (AUR) on the major road;
- Auxiliary Left Turn treatment (AUL) on the major road; and
- Auxiliary Left Turn treatment (AUL) on the minor road.

These treatments are shown in Figure 13.16. Often, not all of the treatments will be used together at a single intersection.

Warrants for the various turn treatments are given in Section 13.4.4 and have been developed using the results of Arndt (2004). The warrants have been produced by identifying the location at which the benefits of providing a higher-level treatment (the reduction in estimated accident costs) are made equal to a proportion of the additional construction costs.

The new warrants show that it is not beneficial to provide AUR turn treatments. Instead, Channelised Right Turn Treatments with reduced length of right turn slots (CHR(S) – refer to Sections 13.7.9.2 and 13.7.10.2) are preferred. Basically, CHR(S) treatments offer significantly better value for money than do AUR turn treatments, in terms of safety benefits versus construction cost. Arndt (2004) found that Type AUR turn treatments record a Rear-End-Major vehicle accident rate 30 times higher than do CHR [and CHR(S)] turn treatments.

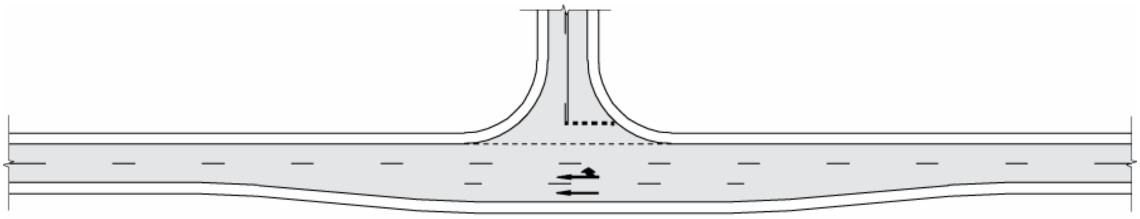
Other advantages of using CHR(S) turn treatments in lieu of AUR turn treatments include the following:

- Reduction in Overtaking-Intersection vehicle accidents (where a right turn vehicle is hit by an overtaking vehicle);
- Provision of more consistent intersection layouts;
- Increase in the average design life of turn treatments ie compared to AUR turn treatments, CHR(S) treatments will be able to function for longer periods before an upgrade is required; and
- Address concerns from the motoring public that more CHR turn treatments should be provided on high-speed roads to improve safety.

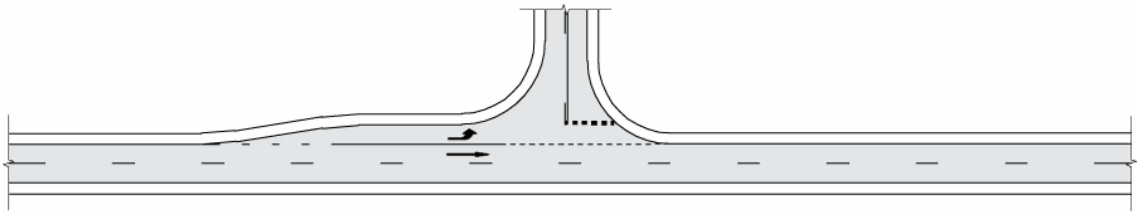
For the above reason, AUR turn treatments should not be used for the design of new unsignalised intersections. These treatments are not detailed in this chapter.

As discussed in Section 13.3.1.3, the accident rate for vehicles entering the major road from the minor road at an unsignalised intersection is significantly higher when there are two stand-up lanes on the minor road (ie when there is an auxiliary lane). An AUL turn treatment on the minor road is not preferred for this reason, particularly at four-way unsignalised intersections.

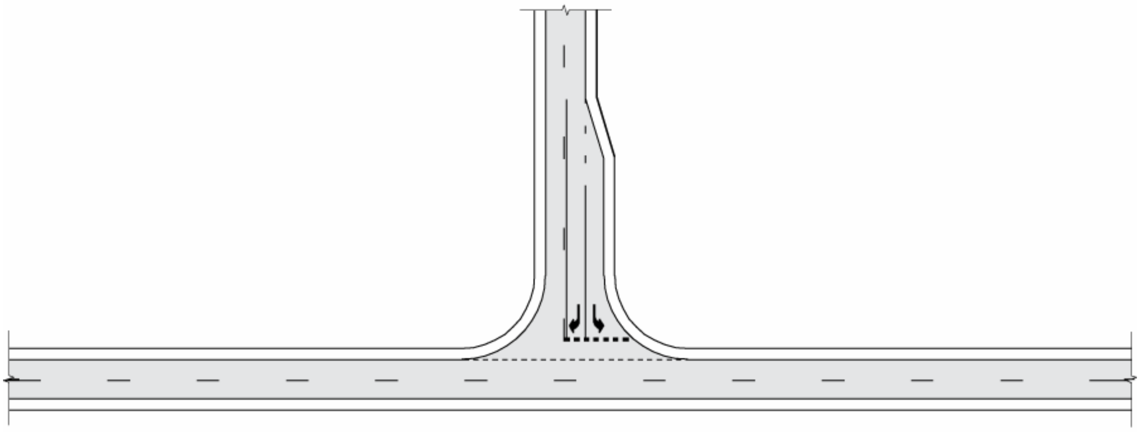
For the above reason, AUL turn treatments on the minor road should not be used for the design of new unsignalised intersections. These treatments are not detailed in this chapter. A channelised left turn treatment (CHL) or signalisation of the intersection are preferred solutions in this instance.



Auxiliary Right Turn (AUR)
on the Major road
(This turn type not to be used at unsignalised intersections)



Auxiliary Left Turn (AUL)
on the Major Road



Auxiliarly Left Turn (AUL)
on the Minor Road
(This turn type not to be used at unsignalised intersections)

Note: Arrows indicate movements relevant to the turn type. They do not represent actual pavement markings.

Figure 13.16 Auxiliary Lane Intersection Turn Treatments "AU"

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13.4.2.3 Channelised Intersections (type CH)

A channelised intersection is one where conflicting vehicle travel paths are separated by raised, depressed, or painted medians and/or islands. Auxiliary lanes are often used in conjunction with channelisation.

Channelisation has particular application in the following areas:

- Intersections at odd angles (Y-junctions, skewed cross roads), or multi leg intersections (generally only appropriate if the intersection is realigned and/or if traffic signal control is used).
- Sites where turning traffic movements are particularly heavy.
- Locations where the safety record of an intersection is shown to be susceptible to particular accident types, such as opposing side swipe and head on crashes, right turn opposing, and high speed rear end collisions.
- Sites where a refuge area for pedestrians is desirable.
- Sites where unusual manoeuvres are occurring, or where unwanted movements are to be eliminated.

A channelised layout may be the only solution appropriate at some sites. These include some multi-lane divided roads, and sites where it is necessary to provide positive protection of the furniture (signs, traffic signal posts, etc.) associated with the form of traffic control adopted.

This type of layout is the most expensive form of an at grade intersection. The associated furniture (particularly raised medians) can be regarded as a hazard, which means that the increased risk must be clearly outweighed by other advantages.

All channelised intersections with raised medians and kerbed islands must be lit in accordance with the standards set out in Chapter 17.

Channelised intersections always require good sight distance to the starting point of the median (especially raised). The median or island may have to be extended to meet this requirement. A few large islands are always preferable to a large number of small islands.

An operational problem with these layouts on two lane-two way roads can be the loss of opportunities to overtake, and this must be taken into account in the route strategy.

Drainage of raised medians and islands can be expensive. Regular sweeping may be necessary.

Where traffic volumes are high, the number of approach lanes, including auxiliary lanes, will increase and channelisation (in some form) becomes inevitable. Preliminary approach lane requirements may be assessed using “Y” values (see Appendix 13B). Verification and refinement of approach treatment can be done using computer programs. Detailed design requirements for medians and islands are given in Section 13.7.2. As urban channelised intersections are often controlled by traffic signals, the possibility of this form of control should be established early in the process so that appropriate provision can be made.

"CH" Turn Treatments

A "CH" intersection comprises the following turn treatments:

- Channelised Right Turn treatment (CHR) on the major road;
- Channelised Left Turn treatment (CHL) on the major road; and
- Channelised Left Turn treatment (CHL) on the minor road.

These treatments are shown in Figure 13.17. Often, not all of the treatments will be used together at a single intersection.

Arndt (2004) showed that CHR turn treatments record a much lower Rear-End-Major accident rate than BAR, AUR and MNR turn treatments. This is predominantly due to the separation of the turning movement from the through traffic.

Arndt (2004) also found that Rear-End-Major vehicle accident rates at CHR turn treatments with short lengths of turn lane were not significantly higher than for full length turn lanes. For this reason, Channelised Right Turn treatments with short turn lanes [CHR(S)] were developed for lower trafficked areas. This treatment is discussed in Sections 13.7.9.2 and 13.7.10.2.

There are two types of CHL turn treatments. One is the high entry angle turn treatment as shown in Figure 13.17. The other comprises a multiple radii return with a full length acceleration lane (refer Sections 13.7.12.5 and 13.7.13.6). Both of these treatments are also described as free left turn lanes.

Arndt (2004) found that all types of CHL turn treatments were associated with an increase in single vehicle accident rates, as compared to BAL turn treatments (and rear-end accident rates for CHL turn treatments on the minor road). This reduced safety performance is expected to result from the higher speeds at which left-turning drivers were observed to travel at on CHL turn treatments.

Although CHL turn treatments record increased rates of these accident types, the rates are relatively low as compared to most other accident and conflict types. Therefore, warrants for CHL turn treatments should not be selected on the basis of safety. Instead, they may be justified by circumstances such as:

- Improving capacity and delays at the intersection.
- Improving safety for other conflict types. CHL treatments on the major road may provide greater visibility for drivers on the minor road as per Austroads (2002).
- Providing a bypass facility for left-turning vehicles at traffic signals.
- Changing the give way rule in favour of other manoeuvres at the intersection.
- Defining more appropriately the driving path by reducing the area of bitumen surfacing, especially at skewed intersections catering for large and over dimensional vehicles.

There are various types of Channelised intersections, and these are discussed in the following sections and in Section 13.7.

Two Staged Crossing

A two staged crossing allows right turning traffic from the minor road of an unsignalised intersection to undertake the manoeuvre in two stages, which has benefits when volumes on the major road are high and volumes on the minor road are low. A diagram of a two staged crossing is shown in Figure 13.18.

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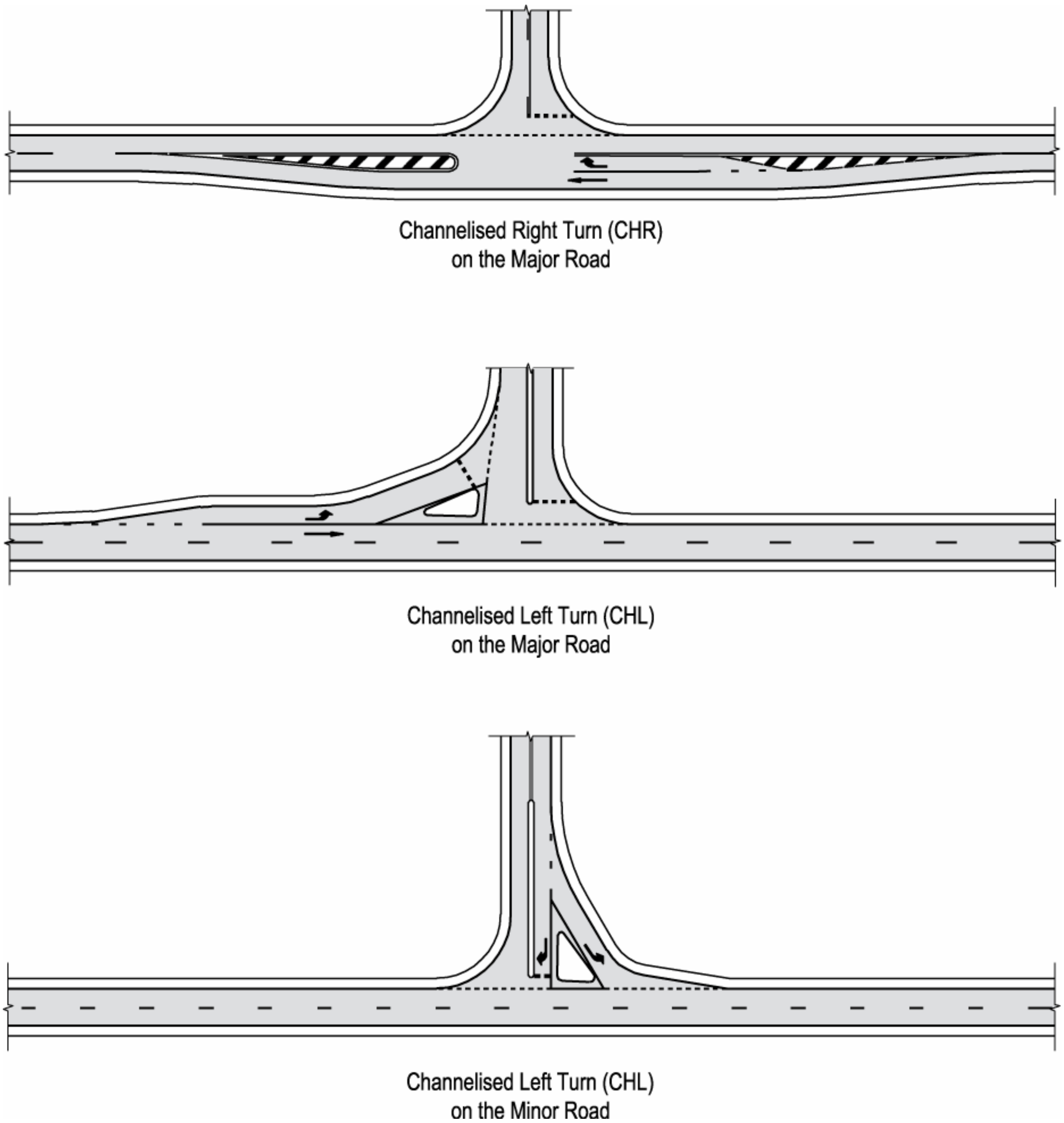
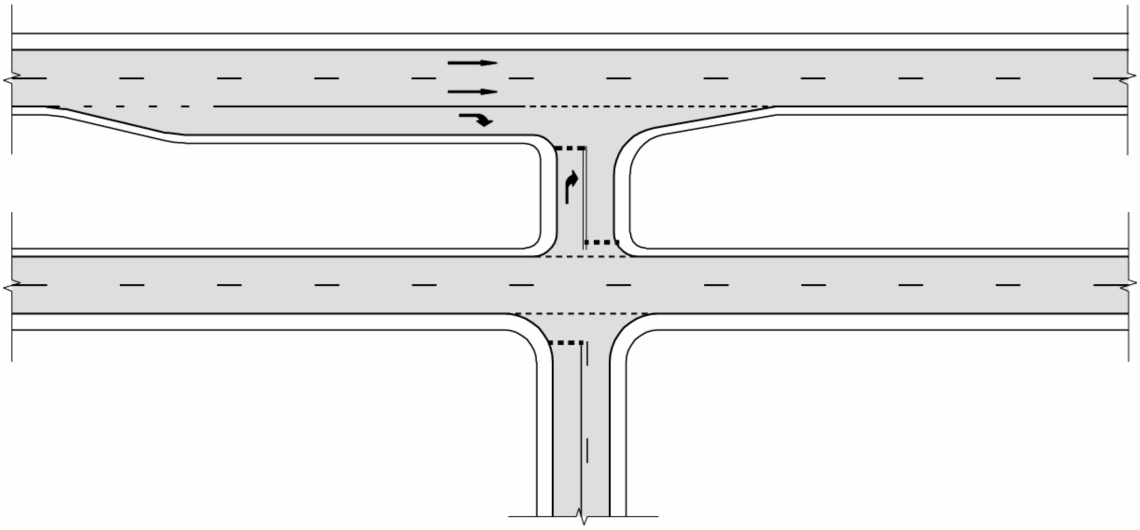


Note 1: An alternative to the high entry angle CHL turn treatments shown above is the three centred return CHL with full length acceleration lane, as shown in Sections 13.7.12 and 13.7.13.

Note 2: Arrows indicate movements relevant to the turn type. They do not represent actual pavement markings.

Figure 13.17 Channelised Intersection Turn Treatments "CH"



Note: Arrows indicate movements relevant to the turn type. They do not represent actual pavement markings.

Figure 13.18 Two Staged Crossing

Staggered T-intersection

Staggered T-intersections are used as a safer alternative to four-way unsignalised intersections. As discussed in Section 13.4.3.5, 4-way intersections with priority signage record high accidents rates for the through movements from the minor road, particularly if the minor legs are aligned.

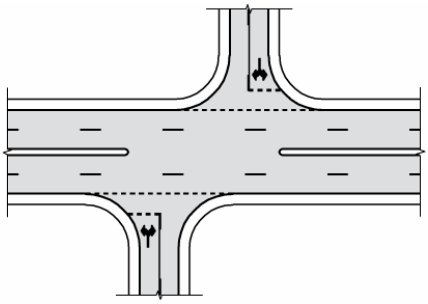
At four-way intersections where the minor legs are fully aligned, drivers can overlook the presence of the intersection and can perceive the minor road continuing straight ahead. This can be especially true in a rural setting.

Staggered T-intersections attempt to minimise this safety problem by offsetting the minor road legs. This requires motorists travelling through from a minor leg to initially turn onto the major road followed by turning onto the opposite minor road leg. Conflict points (involving through movements from the minor legs) generated by staggered T-intersections are deemed to be safer than those generated by 4-way intersections.

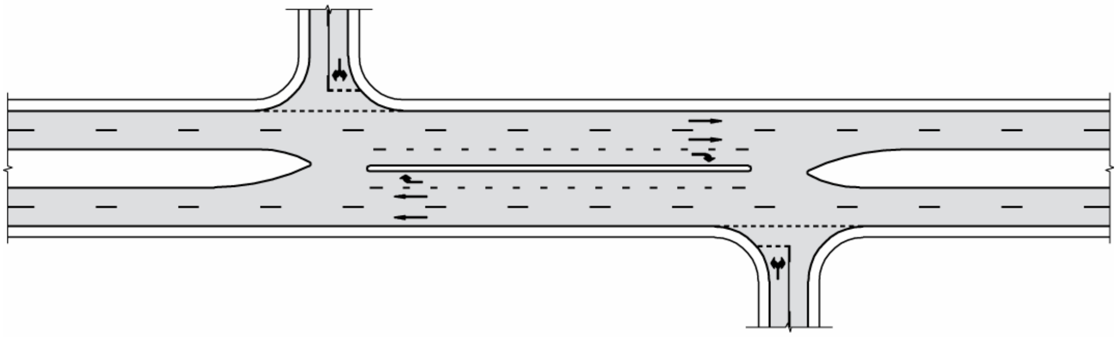
There are two types of staggered T-intersections as shown in Figure 13.19. One is a Left-Right Stagger, where motorists initially turn left onto the major road, then right onto the opposite minor road leg. It is most desirable that a right turn slot be introduced for the motorists turning right from the major road.

The other type of Staggered T-intersection is the Right-Left stagger, where motorists initially turn right onto the major road, then left onto the opposite minor road leg. This treatment is often more cost effective than a Left-Right stagger if converting from a four-way cross intersection.

Arndt (2004) suggested that a Left-Right stagger may be safer than Right-Left Stagger, due to less hazardous conflict points being generated.



Right - Left Staggered T-Intersection



Left - Right Staggered T-Intersection

Note: Arrows indicate movements relevant to the turn type. They do not represent actual pavement markings.

Figure 13.19 Staggered T-intersection

Seagull

A diagram of a seagull layout is shown in Figure 13.20. Seagull intersections usually work well where right turning traffic from the minor road would be delayed for extended periods due to the small number of coincident gaps on the major road, particularly if upstream events on both of the major road legs cause traffic to arrive at the intersection in platoons.

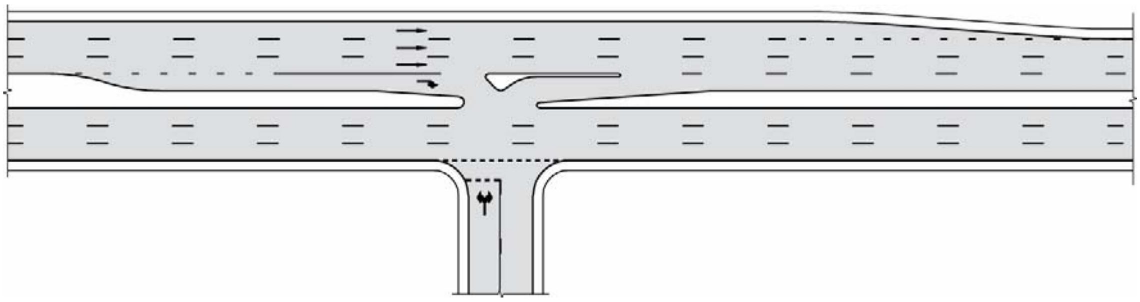
When the volume of right turning traffic is small, it is preferable to store vehicles, one at a time, in the median. This requires a two staged crossing treatment as shown in Figure 13.18.

Channelised Intersections with Right-Turn Bans

Channelised intersections can be designed to restrict certain right turn movements. Examples of such channelisation are shown in Figure 13.21.

13.4.2.4 Roundabouts

A roundabout is a special form of channelisation. It has particular application in areas where safety and amenity can be improved by controlling traffic speed. Refer to Chapter 14 of this manual for details.



Note: Arrows indicate movements relevant to the turn type. They do not represent actual pavement markings.

Figure 13.20 Seagull

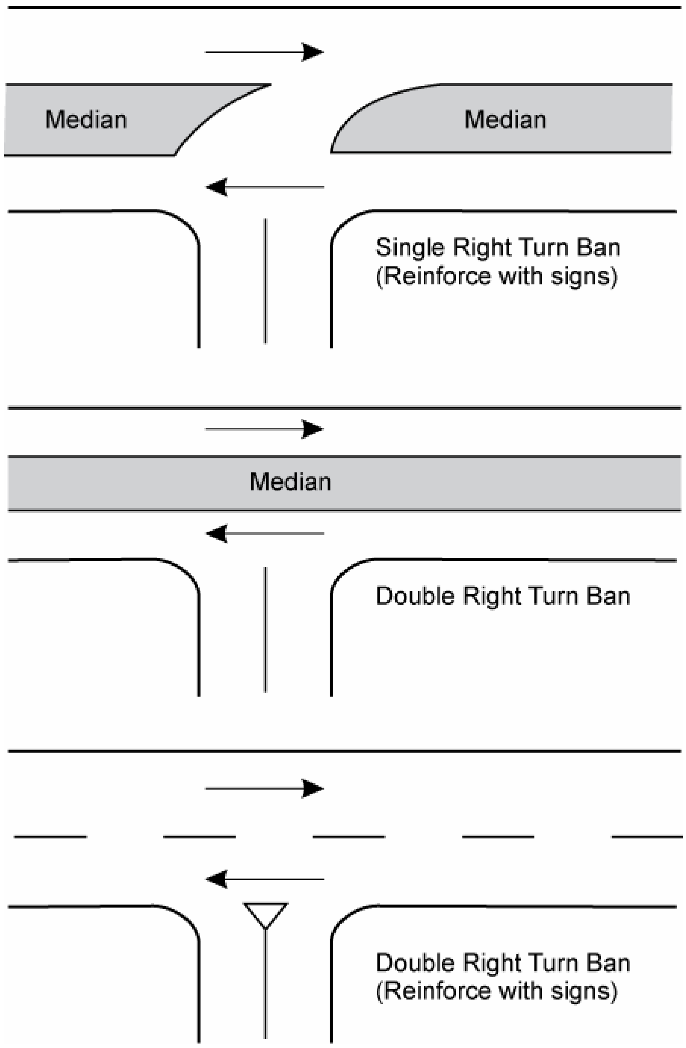


Figure 13.21 Typical Right Turn Bans

13.4.3 Traffic Control Devices

The purpose of a traffic control device is to allocate priority, or to guide users as they negotiate an intersection. The devices, in order of increasing cost, are:-

- Traffic regulations, which can be augmented by signs designed to emphasise or to clarify;
- Priority Signage - regulatory signs and/or pavement markings installed to override normal regulations;
- Roundabouts; and
- Traffic signals.

13.4.3.1 Traffic Regulations

In the absence of regulatory signs, traffic is controlled by regulations (rules of the road).

For example, traffic entering an intersection, other than at a T-junction, has to give way to traffic on its right; traffic on the non-terminating leg of a T- junction has right of way over traffic on the stem; left turning traffic has right of way over conflicting right turning traffic.

“Give Way” signs, and pavement markings (eg. turning arrows) can be used to:-

- identify who has right of way (particularly at four leg, and multi-leg intersections); and
- reinforce normal priority rules where the site has an accident history of failure to observe regulations.

Traffic regulations are one type of unsignalised control.

13.4.3.2 Priority Signage

Priority signage consists of regulatory signs and pavement markings and is another type of an unsignalised control.

The purpose of providing regulatory signs (signs such as STOP, turn bans etc, where a penalty applies) is to:

- override normal priority rules to promote a movement dictated by the road hierarchy;
- provide positive discouragement to traffic using local roads instead of arterial roads; and/or
- manage sites with deficient sight distance.

Pavement markings can be used to organise and/or control vehicular and pedestrian traffic. Example are:-

- Barrier lines;
- Separation lines;
- Turn lines;
- Continuity lines; and
- Marked pedestrian crossings.

These forms of control require good sight distance to each device used. They work best in areas where low volumes occur. A sealed surface is necessary where pavement markings are planned. They have regular application where traffic signals or roundabouts are not provided.

The “Guide to Pavement Markings” and the Manual of Uniform Traffic Control Devices contain details of recommended procedures for this form of control.

13.4.3.3 Roundabouts

Chapter 14 provides a comprehensive treatment of the design of roundabouts.

13.4.3.4 Traffic Signals

Traffic signals provide control by separating conflicting vehicle movements on a time basis. Pedestrian control can be incorporated, or installed separately at isolated mid-block sites.

The primary factor in selecting this form of intersection control has to do with the availability of gaps. If the gaps in the major street flow can safely accommodate entering traffic from side streets for the majority of time, it is reasonable to assume that traffic signals are not required. However, as vehicle volumes increase, the likelihood of having to provide traffic signals increases. Traffic signals are perceived by pedestrians as an effective and safe method of crossing the road. This perception is heightened when young or elderly pedestrians are involved. However, the inter peak and out of peak delays can be unreasonable and the sum of delay and operating costs must be carefully considered.

Traffic signals should only be provided at locations where it can be demonstrated that a significant benefit will occur that cannot be achieved by other means.

Whilst the capital cost of signals is relatively low, the ongoing costs of maintenance and operations can double the initial cost over the design life. Driver discipline is an essential feature of traffic signals, so installations at inappropriate sites (where driver frustration would probably occur) can lead to non compliance by drivers.

Traffic signals may be installed on roads with posted speeds up to 80 km/h.

Installation of traffic signals is subject to warrants given in the "Traffic Signals Design Manual" (January 1999). (See also Chapter 18 of this Manual.)

13.4.3.5 Selecting the Form of Traffic Control

Selection of the most appropriate form of traffic control depends on many factors. There are no numerical warrants for the choice of intersection control. Choice of control type depends on factors including:

- **Capacity**. Unsignalised intersections are suited to lower traffic flows as they have the lowest capacity. However, unsignalised intersections can be staged to allow future upgrading to traffic signals. Signalised intersections have the highest capacity because more stand-up lanes can be added as required (subject to available right-of-way). However, they can cause increased delays in off-peak periods. Roundabouts can have high capacity if multiple lanes are used. However, roundabouts with more than two lanes may cause safety problems eg cutting across lanes, viewing through adjacent vehicles on multi-lane entries. Signals can be fitted to roundabouts to prolong their life.
- **Traffic movements**. Unsignalised intersections work well when the minor road traffic volume is low compared to the major road volume (provided delays on the minor legs are not excessive). Signalised intersections work well under most combinations of traffic movements. An exception to this is where high volumes of right turning vehicles would cause excessive delays. Another exception is where low traffic volumes on the side roads are causing excessive delays to the major road traffic. Roundabouts perform best at the intersection of roads with roughly similar traffic flows, particularly those with a high proportion of right turning

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traffic. This is for reasons of both traffic throughput and safety.

- **Number of Heavy Vehicles**. Roundabouts are usually not preferred if there are high volumes of heavy vehicles, especially if multi-combination vehicles are present.
- **Number of Pedestrians**. Unsignalised intersections and roundabouts may not be appropriate solutions where there are moderate to high volumes of pedestrians, particularly if traffic volumes through the intersection are high. Signalised intersections are preferred in this instance.
- **Number of Cyclists**. Unsignalised and signalised intersections generally cater appropriately for cyclists. Roundabouts may not be appropriate solutions where there are significant volumes of cyclists. This is especially true for multi-lane roundabouts and roundabouts in areas of high operating speeds. Cyclists are well over represented in accidents at roundabouts.
- **Operating speed of the roadways**. Unsignalised intersections and roundabouts are acceptable on roadways in all speed areas. These control types generally need some measures to gradually reduce speed on high speed legs without priority. Signalised intersections are not used on roadways with a speed limit above 80km/h.
- **Available right-of-way**. Well designed roundabouts that achieve a high level of safety require significant amounts of right-of-way. This is to achieve the necessary diameter and entry path curvature. Greater amounts of right-of-way will be required in areas of higher operating speeds and/or when used by multi-combination vehicles. Whether sufficient right-of-way can be obtained will depend on factors such as property values, cost of service relocations, community acceptance of a loss of parking and topography.
- **Form of Control at Adjacent Intersections**. On a road with a networked set of traffic signals, it would likely be inappropriate to add a roundabout, as drivers would not expect it. In addition, the roundabout may well destroy the efficiency of the traffic throughput. However, unsignalised intersections may work appropriately if located between signalised intersections, provided sufficient gaps are provided in the traffic stream by the plattooning effects of the signals.
- **Future considerations**. Future alterations to road hierarchy, changes in land use, planned route changes for particular vehicle types (eg trucks, buses, bicycles) may dictate the form of traffic control that has to be initially selected.
- **Cost**. This includes the initial, operating and maintenance cost of the treatment. Unsignalised intersections have the lowest initial cost. Signalised intersections have the highest operating costs. The cost of crashes during the life of the facility should be included.
- **Number of legs & angle between legs.** This issue is discussed in the next section.

Table 13.3 may also be used to assist in determining suitable forms of control. This table shows a general assessment of the suitability of forms of control for various layouts under four combinations of speed and volume. The judgements are meant as a guide only, and not a substitute for a more objective, site specific analysis.

A more detailed evaluation of options is given in the process in Appendix G. This evaluation uses a system to weight the following criteria:

- Safety
- Delay
- Site Suitability
- Cost

Caution is advised when using this process. Accident rates for the various forms of control and intersection types can vary enormously depending on the particular traffic movements, the operating speed of the legs and numerous other parameters. Using the average accident rates given in Appendix G may well result in selecting a value much different to the actual accident rate that would occur. It is suggested that a more qualitative approach that considers the factors previously given in this section may produce a better result.

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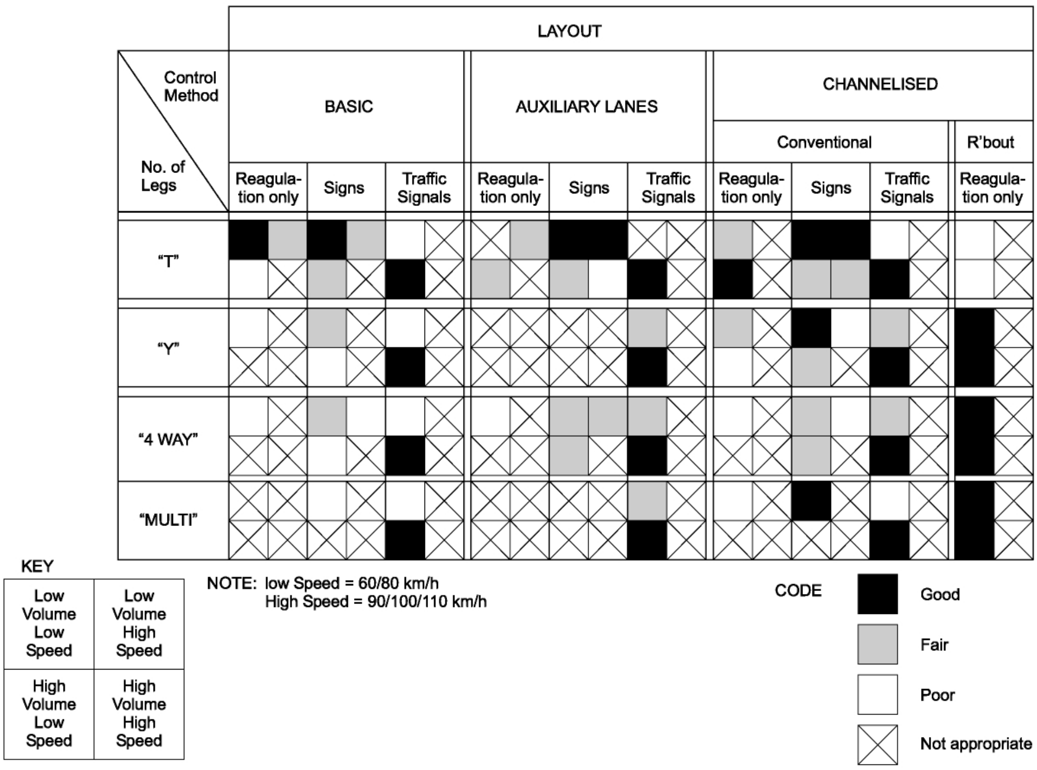

Table 13.3 General Assessment of Suitability of Forms of Control (to be used a general guide only)

Number of Legs and Angle Between Legs

Not all forms of control suit each of the intersection layouts (the number and angle of legs of an intersection) given in Figure 13.13. Generally, only the following combinations should be considered for the design of intersections on **new** roads:

- **T-junction - all forms of control generally work well**.
- **Four-way intersection – traffic signals and roundabouts generally work well.** Arndt (2004) found that four-way intersections with priority signage record high Angle-Minor vehicle accident rates (accidents due to minor road drivers failing to give way and colliding with a major road vehicle – refer to Appendix F for more details) for through conflicts from the minor leg, particularly if the minor legs are aligned. Four-way intersections with aligned minor legs were found to have Angle-Minor vehicle accident rates almost double that if the minor legs were not aligned. Aligned minor legs can deceive drivers as to the presence of the intersection. For drivers not concentrating adequately, the road can appear to continue straight ahead. Four-way intersections with traffic regulations only also record high accident rates for the through movements and can also cause significant driver confusion as to who has right-of-way. If a 4-way intersection is to be unsignalised, a staggered T-intersection is the preferred treatment.
- **Y-junction – traffic signals and roundabouts generally work well.** Y-junctions with priority signage do not provide a suitable observation angle for drivers on the minor road because of the large skew angle between legs. Y-junctions with traffic regulations only also have the problem with obtaining a suitable observation angle and can cause significant driver confusion as to who has right-of-way.
- **Multi-leg intersection – single lane roundabouts generally work well.** Multi-leg intersections with priority signage or traffic regulations only cause driver confusion as to who has right-of-way. Multi-leg intersections with traffic signals can record high accident rates, can result in inadequate sighting of lanterns, and produce a high proportion of inter-green time. Multi-leg, multi-lane roundabouts cause significant driver confusion as to appropriate lane choice for the intended movement.

13.4.4 Warrants for Unsignalised Intersection Turn Treatments

This section details warrants for turn treatments on the major road at unsignalised intersections (excluding roundabouts and seagull treatments). The warrants are for both urban and rural roads.

Development of the Warrants

Development of the warrants in this section is detailed in Arndt and Troutbeck (2005) and is briefly discussed below.

Arndt and Troutbeck (2005) provided the following reasons for the creation of the warrants:

- To improve the limitations and ambiguity of the previous warrants in Chapter 13 of the Road Planning and Design Manual and Austroads (2005) ‘Part 5 - Intersections at Grade’.

- To base the warrants directly on the measured safety performance of each turn type.
- To ensure that higher-order turn treatments are not warranted until higher traffic volumes on lower-speed roads. This is because turn treatments on lower speed roads record far fewer Rear-End-Major vehicle accidents (generally rear-end type accidents resulting from a through driver colliding with a driver turning right from the major road – refer to Appendix F for more details) than do turn treatments on high speed roads.
- To ensure that higher-order right-turn treatments are provided at lower traffic volumes than for higher-order left-turn treatments. This is because lower order right-turn treatments record far more Rear-End-Major vehicle accidents than lower order left-turn treatments.
- To incorporate CHR and AUL turn treatments with short length right-turn slots (refer to Figure 13.49, Figure 13.59, Figure 13.73 and Figure 13.81 for diagrams of these treatments). Such treatments have significant safety benefits over lower-order turn treatments.

The warrants have been produced by identifying the location at which the benefits of providing a higher-level treatment (the reduction in estimated accident costs) are made equal to a proportion of the additional construction costs. This proportion is the benefit cost ratio (BCR) and applies for an assumed design life. The benefits and costs of a higher-level treatment are compared to the base case (the minimum turn treatment).

For the right turn treatments, a design life of ten years and a BCR equal to one is assumed in the calculations. For the left turn treatments, however, using BCR values of one with a design life as high as 50 years, the warrants produced are such that traffic flows, on even the busiest roads, would never be high enough to justify using higher-level left-turn treatments. Omitting higher-level left turn treatments in all circumstances would not meet driver expectation and would cause operational problems, especially on the busier roads. Therefore, an alternative method of determining warrants for left-turn treatments was developed.

For the left-turn warrants, the curves produced for the right-turn treatments are adopted. As the major road traffic volume on the X-axis of the warrants is based on all relevant major road traffic flows, higher-order right-turn treatments are required at lower traffic volumes than for higher-order left-turn treatments. This process ensures that these warrants reasonably match driver expectations set through the previous warrants.

The warrants show that it is not beneficial to provide AUR turn treatments. Instead, Channelised Right Turn Treatments with reduced length of right turn slots [CHR(S)] are the preferred treatment. Basically, CHR(S) treatments offer significantly better value for money (in terms of the safety benefits versus the construction costs) than do AUR turn treatments.

Application of the Warrants

The warrants are based on the construction of intersections on new roads (ie greenfield sites). Therefore, their most appropriate application is to the selection of turn types for intersections on new roads.

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The warrants may also be used as a reference for the construction of new intersections on existing roads. However, there may be occasions when a prohibitive cost dictates that the indicated turn treatment is impractical (eg right-of-way limitation, large drainage structure exists, major utility service works are involved). In this case, a documented benefit/cost analysis should show why the cost is prohibitive. The analysis should include an estimation of the safety cost, which can be calculated by Equation 13.1 given in following section titled 'Estimate of the Safety Cost of Turn Treatments'.

The warrants may also be used as a reference for intervention levels when upgrading existing intersection turn treatments. Alternatively, requirements for upgrading existing intersections may be based on a documented benefit/cost analysis, such as that discussed above.

The warrants are not intended for direct application to accesses and driveways, although they may be used as a reference for such.

Warrants for Turn Treatments

The warrants for major road turn treatments at unsignalised intersections are given in Figure 13.22 and Figure 13.23. Figure 13.22 is for the selection of turn treatments on roads with a design speed greater than or equal to 100km/h. This figure is particularly appropriate for high speed rural roads.

Figure 13.23 is for the selection of turn treatments on roads with a design speed less than 100km/h. This figure is particularly appropriate for urban roads, including those on the urban fringe, and lower speed rural roads.

If a particular turn from a major road is associated with some geometric minima (for example, limited sight distance, steep grade), consideration should be given to the adoption of a turn treatment of a higher order than that indicated by the warrants. For example, if the warrants indicate that a BAR turn treatment is acceptable for the relevant traffic volumes, but limited visibility to the right-turning vehicle is available, consideration should be given to the adoption of a CHR(S) or CHR turn treatment instead.

Another example is as follows. If a major road is on a short steep downgrade, and numerous heavy vehicles travel quickly down the grade, it would not be appropriate to adopt a BAL turn treatment. Instead, an AUL(S) or an AUL would be a preferred treatment.

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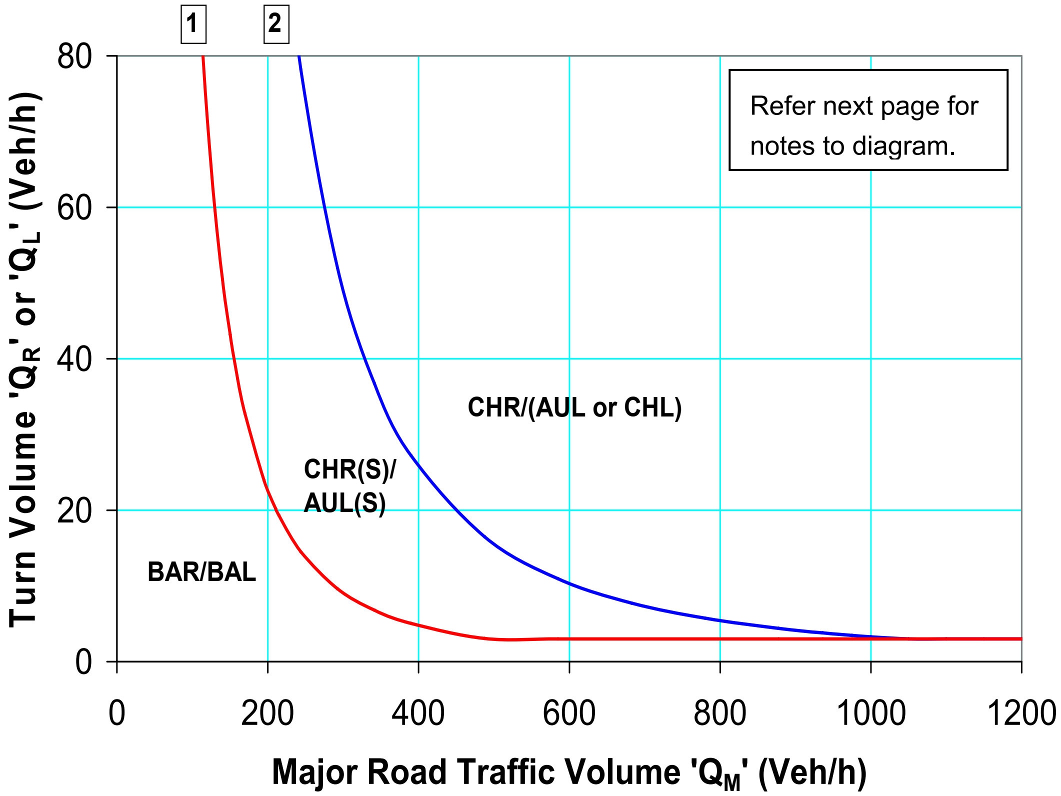

Figure 13.22 Warrants for Turn Treatments on Roads with a Design Speed ≥100km/h

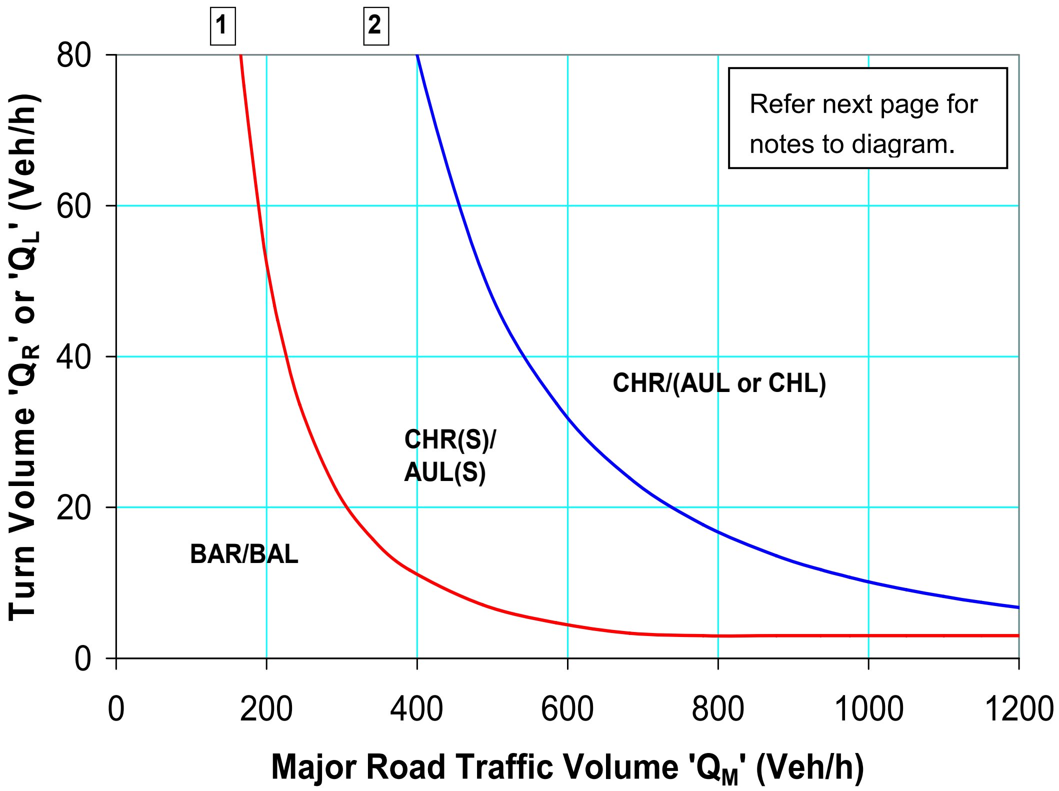

Figure 13.23 Warrants for Turn Treatments on Roads with a Design Speed < 100km/h

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The following notes apply to the warrants in Figure 13.22 and Figure 13.23:

1. Curve 1 represents the boundary between a BAR and a CHR(S) turn treatment and between a BAL and an AUL(S) turn treatment.

2. Curve 2 represents the boundary between a CHR(S) and a CHR turn treatment and between an AUL(S) and an AUL or CHL turn treatment. The choice of CHL over an AUL will depend on factors such as the need to change the give way rule in favour of other manoeuvres at the intersection and the need to define more appropriately the driving path by reducing the area of bitumen surfacing.

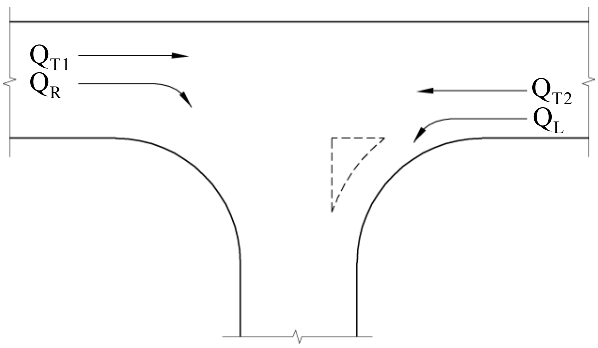
3. The warrants apply to turning movements from the major road only (the road with priority).

4. Use Figure 13.24 to calculate the value of the Major Road Traffic Volume Parameter (Q_M).

5. Traffic flows applicable to the warrants are peak hour flows, with each vehicle counted as one unit (ie do not use equivalent passenger car units [pcu's]). Where peak hour volumes or peak hour percentages are not available, assume the design peak hour volume equals 15% of the AADT for 500 hours each year, use 5% of the AADT for the rest of the year. See Chapter 5 for further details.

6. If more than 50% of the traffic approaching on a major road leg turns left or right, consideration needs to be given to possible realignment of the intersection to suit the major traffic movement. However route continuity issues must also be considered (for example, realigning a highway to suit the major traffic movement into and out of a side road would be unlikely to meet driver expectation).

7. If a turn is associated with other geometric minima, consideration should be given to the adoption of a turn treatment of a higher order than that indicated by the warrants.



Turn Type	Splitter Island	Q_M (veh/h)
Right	No	$= Q_{T1} + Q_{T2} + Q_L$
Right	Yes	$= Q_{T1} + Q_{T2}$
Left	No/Yes	$= Q_{T2}$

Figure 13.24 Calculation of the Major Road Traffic Volume Parameter 'Q_M'

Estimate of the Safety Cost of Turn Treatments

Equation 13.1 calculates the safety benefits of using a higher order left or right turn treatment. The safety benefits are the reduction in estimated accident costs, which may be used as part of a benefit/cost analysis to justify a particular turn treatment (for example, at an existing intersection). For greenfield sites, use the warrants shown in Figure 13.22 and Figure 13.23 (which were also developed using Equation 13.1, as detailed in Arndt and Troutbeck [2005]).

$$C_{RM} = 2.75 \times 10^{-12} \times C_A \times T_{DL} \times Q_i^{0.406} \times Q_M^{0.912} \times S_{MT}^{2.94} \times (e^{TT_M} - e^{TT_A}) \qquad (13.1)$$

where

C_{RM} = safety benefit of using the higher order turn treatment ($)

C_A = average cost of a Rear-End-Major vehicle accident = $38,974 from Arndt (2004)

T_{DL} = design life (years)

Q_i = turning traffic flow from the major leg (veh/h) (Q_R or Q_L as per Figure 13.24)

Q_M = traffic flow on the major legs according to Figure 13.24 (veh/h)

S_{MT} = 85th percentile through major road speed (km/h)

TT_M = type of lower-order turn treatment (values given below)

TT_A = type of higher-order turn treatment (values given below)

Turn treatment values are:

- BAR – 3.83
- CHR(S) and CHR – 0
- BAL – 0.666
- AUL(S) and AUL - 0.0493

Example Calculation

Determine the safety benefit of providing a CHR turn treatment in lieu of an existing BAR turn treatment for the following conditions:

Design life ‘T_{DL}’ = 20 years

Design right-turn traffic flow ‘Q_R’ = 60 veh/h

No splitter island opposite the right turn

Design approaching through traffic flow ‘Q_{T1}’ = 190 veh/h

Design opposing through traffic flow ‘Q_{T2}’ = 200 veh/h

Design opposing left-turn traffic flow ‘Q_L’ = 50 veh/h

85th percentile through speed ‘S_{MT}’= 70km/h

Answer

$Q_M = Q_{T1} + Q_{T2} + Q_L$ = 190 + 200 + 50 = 440veh/h from Figure 13.24 (for no splitter island).

Lower-order turn treatment 'TT_M' = 3.38 for a BAR

Higher-order turn treatment 'TT_A' = 0 for a CHR

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Using Equation 13.1:

$$C_{RM} = 2.75 \times 10^{-12} \times 38974 \times 20 \times 60^{0.406} \times 440^{0.912} \times 70^{2.94} \times \left(e^{3.38} - e^{0}\right) = \$34,858$$

Therefore, the safety benefit over 20 years of providing a CHR turn treatment over a BAR turn treatment at this site is $34, 858.

13.5 Capacity of Intersections

13.5.1 General

Interrupted traffic flow conditions predominate on most urban roads and on some major rural roads. Generally it is the major intersections, signalised or not, which determine the overall capacity and performance of the road network. Significant volumes of crossing or turning traffic at minor roads cause interruptions and capacity reductions, which can be lessened on some routes by channelisation and intersection control.

To determine the capacity of intersections, an analysis of one or more of the following is required:

- midblock or route capacity;
- unsignalised intersection capacity; and/or
- signalised intersection capacity.

The capacity of a route or intersection can be significantly affected by lane width, gradients and the presence of public transport and trucks in the traffic stream. Adjustment factors are applied in capacity analysis to allow for narrow lanes, steep gradients and high proportions of trucks.

Detailed information on capacity analysis is contained in Austroads (1988a) 'GTEP Part 2 - Roadway Capacity' and Austroads (2003a) 'GTEP Part 7 - Traffic Signals'. The US Highway Capacity Manual (TRB, 2000) is also a useful reference.

Austroads (1988c) 'GTEP Part 1 - Traffic Flow' and Austroads (2004) 'GTEP Part 3 - Traffic Studies' may also be relevant in the assessment of road capacity.

The following discussion does not cover the calculation of capacity of roundabouts, which is contained in Austroads (1993) 'GTEP Part 6 – Roundabouts'.

13.5.2 Design Traffic Volumes

Design hour traffic volumes (DHV) should be estimated using appropriate techniques such as projecting trends from historical data or traffic models for complex road networks.

In the absence of such information, the following steps can be taken to obtain a set of design traffic volumes on which the intersection design and capacity calculations may be based:

- Obtain relevant peak hour traffic movement counts of existing traffic, usually the morning and evening peak hours for the site or route under consideration.
- For each approach to the intersection determine a capacity, or desired service volume at some other desired level of service, for each 'design' hour under consideration. The capacity or service volume may relate to the existing cross section or a future cross section for each leg of the intersection.
- Adjust all existing traffic movement volumes at the intersection by a factor equal to the ratio of this assessed capacity (or service volume) to the existing volume on the relevant approach, for each 'design' hour under consideration.

13.5.3 Midblock or route capacity

To achieve balance, intersection design should take into account the capacity of the approach roads. Table 13G.6 of Appendix

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13G lists typical one-way, mid-block capacities for urban roads with unflared major intersections and interruptions from cross and turning traffic at minor roads. This volume also represents the maximum service volume on the major approaches of intersections along the route. When improvements to isolated intersections are being considered and the upstream conditions will remain unchanged, these figures can be assumed to be limiting values.

Peak hour traffic volumes can rise to 1200 - 1400 vehicles per hour, per mid-block lane, on any approach route where any combination of the following conditions may either exist, or be implemented:

- adequate flaring at major upstream intersections;
- uninterrupted flow from a wider carriageway, upstream of an intersection approach and flowing at capacity;
- control or absence of crossing or entering traffic at minor intersections by major road priority controls;
- control or absence of parking;
- control or absence of right turns by banning turning at difficult intersections;
- high volume flows of traffic from upstream intersections during more than one phase of a signal cycle; and/or
- co-ordination of signalised intersections.

In estimating approach road capacity thought should be given to the road function, for example, a collector road should not be expected to handle volumes comparable with those on an arterial road of similar cross section, at the same level of service.

‘Level of Service’ is a term that denotes any one of an infinite number of combinations of operating conditions that may occur on a given lane or roadway accommodating various traffic volumes. Level of service is a qualitative measure of the effect of a number of factors, which include speed and travel time, traffic interruptions, freedom to manoeuvre, safety, driving comfort and convenience and operating costs. In practice, selected specific levels of service are defined in terms of particular limiting values of a number of these factors.

13.5.4 Unsignalised Minor Road Intersections

At intersections carrying light crossing and turning volumes, the capacity figures for uninterrupted flow generally apply for the approach roads.

Table 13.4 indicates the maximum traffic volume combinations for uninterrupted flow conditions. It is unnecessary to flare intersection approaches or carry out an intersection analysis when the combinations of major road and minor road volumes are less than those in the table.

However, separate lanes for left or right-turning vehicles may be added to the major road for safety reasons (see Section 13.7). For more detailed information refer to Austroads (1988a) ‘GTEP Part 2 - Roadway Capacity’.

Table 13.4 Intersection Capacity – Uninterrupted Flow Conditions

Major Road Types[1]	Major Road Flow (vph)[2]	Minor Road Flow (vph)[3]
Two-Lane	400	250
	500	200
	650	100
Four-Lane	1000	100
	1500	50
	2000	25

Notes
1. Major road is through road i.e. has priority
2. Major road design volumes include through and turning movements
3. Minor road design volumes include through and turning volumes

13.5.5 Unsignalised Major Intersections

When the volumes given in Table 13.4 are exceeded or channelisation is being considered, the capacity of the intersection in the unsignalised state should be analysed.

Gap acceptance and delay criteria should be considered for the various flows or combination of flows in conflict.

Such an approach is warranted where:

- the intersecting volumes do not meet the warrants for signalisation (see Section 13.5.6), but there is doubt as to whether a particular movement or movements can be absorbed without excessive delay into or through a stream without alteration to the intersection design; or
- intersecting volumes meet the warrants for signalisation, but for economic or other reasons immediate installation of signals is not envisaged. The capacities and delays in the unsignalised state should then be checked, so that interim adjustments can be made to the intersection.

Appendix 13A provides a guide for estimating capacity of unsignalised intersections and an example calculation. Please note that aaSIDRA uses different default values for the gap acceptance parameters to those given in Appendix 13A.

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13.5.6 Signalised intersection warrants and capacity

13.5.6.1 Concepts and definitions

The control of traffic signals is complex and is not treated in this manual. A detailed description of traffic signal design and operation is provided in Austroads (2003a) ‘GTEP Part 7 - Traffic Signals’.

13.5.6.2 Warrant for Traffic Signals

Traffic signals are usually installed to:

- safely manage vehicle and pedestrian conflicts; and
- improve operational efficiency.

The following guidelines indicate those circumstances where signals could be of significant benefit (refer also to AS1742.2).

At signalised intersections, terms ‘major’ and ‘minor’ are used respectively to indicate the roads carrying the larger and smaller traffic volume:

(a) **Traffic volume**: Where the volume of traffic is the principal reason for providing a control device, traffic signals may be considered, subject to detailed analysis when the major road carries at least 600 vehicles/hour (two way) and the minor road concurrently carries at least 200 vehicles/hour (two way) on one approach over any 4 hours of an average day.

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(b) **Continuous traffic**: Where traffic on the major road is sufficient to cause undue delay or hazard for traffic on a minor road, traffic signals may be considered when the major road carries at least 900 vehicles/hour (two way) and the minor road concurrently carries at least 100 vehicles/hour (two way) on one approach, over any 4 hours of an average day. This warrant applies provided that the installation would not disrupt progressive traffic flow, and that no alternative and reasonably accessible signalised intersection is present on the major road.

(c) **Pedestrians:** To help pedestrians cross a road in safety, signals may be considered when over any four hours of an average day, the major road carries 600 vehicles/hour (two-way) and 150 pedestrians per hour or more cross the road. If a central median is available, pedestrians can move through traffic in two separate movements. A median width of 1.2m is the absolute minimum required to stage pedestrians and 1.5m is a more desirable minimum. In such cases the traffic volume warrant on the major road can be increased to 1000 vehicle/hour (two-way) over any four hours. Where the 85th percentile speed on the major road exceeds 75km/h, the 600 vehicle/hour and 1000 vehicle/hour traffic volume warrants should be decreased to 450 and 750 vehicle/hour respectively.

(d) **Crashes:** To reduce crashes, signals may be considered if there is a 3-year average of 3 or more reported crashes per year of a type which can be eliminated or reduced by traffic control, and the traffic volume is at least 0.8 times the volume warrants given in (a) or (b). While installation of traffic signals will reduce the overall crash rate and relatively severe right angle and right turn against crashes, the number of crashes of another type may increase (e.g. rear-end collisions). Signals should only be installed if simpler devices will not be effective in reducing the crash rate.

(e) **Combined factors:** In exceptional cases, signals occasionally may be justified where no single warrant specified above is satisfied but where two or more warrants are satisfied to the extent of 0.8 times or more of the stated values.

13.5.6.3 Capacity

The capacity of a traffic movement at the traffic signal depends on the saturation flow and the proportion of cycle time that is effectively green for that movement.

The saturation flow used to compute the capacity depends on a number of factors including the:

- type of road environment;
- type of lane (through, turning, combined);
- width of lane;
- gradient on approach; and
- composition of traffic (i.e. heavy trucks and/or public transport).

Saturation flow is defined in Section 13B.4 in Appendix B.

The last factor can have a substantial effect with heavy vehicles being equivalent to two cars on flat gradients. High percentages of heavy vehicles can therefore have a substantial effect on the capacity of an approach, particularly where an up-grade exists. The location and design of bus stops and the priority given to buses can also affect lane capacity Under these circumstances additional lanes or green time may be necessary to compensate for high numbers of trucks and/or public transport. Signalised intersection design and analysis software takes account of such requirements.

Where possible, signalised intersections that have to cater for high numbers of heavy vehicles should therefore be designed to provide:

- generous lane widths (up to 3.7m) - wider for kerbside shared cycle/vehicle lanes;
- adequate turning radii (15-30m);
- adequate minimum green times;
- flat gradients; and
- space for efficient turning manoeuvres.

Where signalised intersections need to cater for cyclists, cycle lanes (generally preferred) or wide kerbside lanes should be provided where possible. Where corridor width is tight, designers should consider providing additional kerbside lane width at the expense of width in the adjacent lanes to the extent that is practical – (median and middle lane widths down to 3.0m can be acceptable on existing urban roads).

The need for bus priority at traffic signals should also be considered.

At isolated sites the available green time is distributed between traffic and pedestrian movements according to demand. For linked signal systems other movements are accommodated within the coordination plan and the capacity that can be provided may be limited, with a consequent increase in delay.

Akcelik (1981) provides a manual method of computing signalised intersection capacity and timings, and queue lengths. However, computer programs such as aaSIDRA are usually used.

Some traffic models that may be relevant in simulation of traffic flow, capacity and road network performance include AIMSUM, PARAMICS, TSIS, CORSIM, TRANSYT and SATURN. In addition, traffic flow information and performance data for existing intersections is available through STREAMS.

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13.6 General Geometry of Approaches at an Intersection

13.6.1 General

The design of the preferred option starts with the alignment of the approach legs of the intersection.

Designers should take care in the planning and evaluation phases to ensure as far as practicable that all issues are identified at that time. A level of interaction with the requirements of this part of the process could help in avoiding surprises at the start of the detailed design.

13.6.2 Horizontal Geometry

Road centre lines are to be designed to intersect as close to 90° as practical; simple layouts are preferred. The best site for an intersection is in a sag vertical curve, with a straight alignment on each approach leg. Where this is not possible it is desirable that the horizontal alignment for the major movements should be as constant as possible (ie. either straight or curved through the intersection). This is particularly important at wide intersections to maintain good lane discipline.

Careful attention to the design of minor road legs of unsignalised intersections with high approach speeds is required. Arndt (2004) found that minor road approaches with high operating speeds recorded an accident rate approximately double that for approaches with lower operating speeds, for the following two accident types:

- Angle-Minor (where a minor road driver fails to give way and collides with a vehicle on the major road)
- Single-Minor-Turn (where a driver turning from the minor road loses control).

It is important to provide speed reducing treatments on minor legs with high approach speeds, particularly those where driver alertness or awareness is low. This may involve applying reverse curves on the approach and methods of carrying this out are provided in Chapter 14 - Roundabouts. Alternative treatments to reduce approach speed are also listed in Chapter 14.

Intersections on the inside of small radius horizontal curves produce difficult observation angles for drivers (refer Section 13.3.1.3) and should be avoided wherever feasible.

It is also preferable to avoid intersections on the outside of small radius horizontal curves, especially curves with a large deflection angle. Drivers entering on the minor leg of such intersections may have the following problems:

- Greater difficulty in perceiving the presence of through road, the vehicles on the road and the speed of the vehicles;
- Obscured visibility to oncoming major road vehicles by the vehicles travelling in the opposite direction on the major road; and
- Greater difficulty in perceiving the location of the intersection due to the superelevation that it normally required on the major road horizontal curve.

As with all intersections, adequate sight distance must be provided on all legs. Specific detailing may be required to ensure this happens.

Where curved horizontal alignments are involved it is inevitable that some reverse curvature will have to be provided. This creates many difficulties, particularly with crossfall. In urban environments a short length of straight, with a minimum 10 to 15m in length, should be provided between reverse curves. This provides approximately one to two seconds of travel allowing the driver to reverse the steering wheel. This is shown in Figure 13.25.

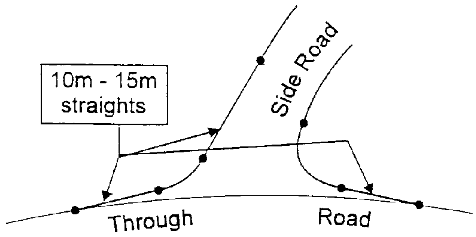

Figure 13.25 Typical Arrangement of Reverse Curves showing the Short Lengths of Straight often necessary in an Urban Environment

Each movement should be checked with a turning path template and the length of straight increased as necessary. Careful attention to the kerb profiles must be done interactively.

In rural situations a curved approach requires a length of straight to allow for plan, and/or cross fall transition. The desirable and minimum requirements are illustrated in Figure 13.26. If these requirements cannot be met, the length of straight in the side road should be extended. This will depend on the grade of the through road.

Where a short length of median is proposed on the side road it may be desirable to lengthen the straight to simplify construction of the median. The minimum length of median in these instances is 10m.

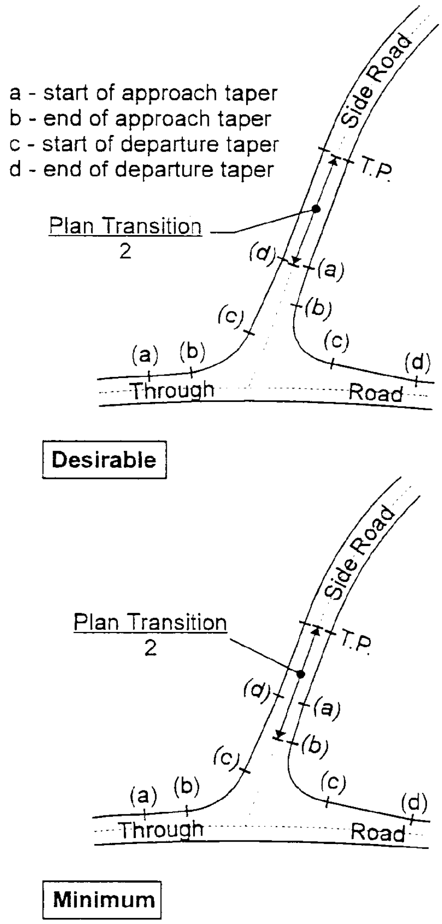

Figure 13.26 Desirable and Minimum Arrangements for a Curved Approach in a Rural Environment

13.6.3 Vertical Alignment

Designers should take care to avoid situations where sight distance is impaired when locating intersections. Where less than desirable situations arise, remedial treatment is required to ensure that the intersection will operate safely. These situations are described below:

- Near Crests: If an intersection must be located within a crest vertical curve, it should be on the top of the crest (not either side) and, preferably, on straight horizontal alignment.

- In Cuttings: Large volumes of additional excavation can result if adequate sight distance is to be provided for through and entering traffic.
- On high embankments: Large quantities of fill may be required to obtain the required geometry.
- Where one or more of the legs have a steep upgrade to the intersection: This results in both sight distance and operational problems. This situation is more critical for upgrades than down grades, and is illustrated in Figure 13.27. Upgrades greater than 3% on the minor approach to an intersection are undesirable, especially where traffic has to stop before entering, Where such approach grades are necessary, preferred practice is to grade the road for a minimum distance of 10m from the gutter lip or edge line, at 3% or less. A similar approach is preferred for downgrades on the major road.
- Where a road is identified as a key cycling route, a bike lane treatment on the uphill leg(s) may be appropriate to account for side-to-side movement of the bicycle and the large speed differential uphill. A bicycle lane is also desirable on the downhill leg(s).
- Where the leg of an intersection would be on the outside of a superelevated curve: The location of intersections on the back of curves invariably results in drivers approaching from that direction not being able to see the intersection and its layout. It is desirable that approach sight distance (ASD) be available to the road surface at all intersections. However, the grading shown as line 1 in Figure 13.28 is not generally favoured because of the extent and cost of the earthworks required to provide acceptable minimum sight distances. Where possible, locate the intersection away from the curve. Where this is not possible, the preferred solution is a grading in the form of line 2 in Figure 13.28. Line 2 provides a uniform approach grade with a short vertical curve used to join it to the cross-fall of the major road. This also results in a relatively flat 'standing' area similar to the 10m shown in Figure 13.27.

In the case of line 2, a raised median is provided on the minor road approach with appropriate signing to warn approaching drivers or the intersection ahead. Line 2 grading is preferred because it provides long sight distances to the median on which the Keep Left sign is located and to the T-junction sign located on the side of the road, and it is cheaper to construct than line 1.

The following requirements apply to designs based on line 2:

- at least 10m of the median shall be visible to approaching drivers for a distance equal to the approach sight distance (ASD) for the 85th percentile operating speed on the approach;
- the median should, as far as possible, be directly in the line of sight of drivers for a distance equal to the ASD for cars;
- the median should be kerbed to increase conspicuity;
- the average grade for vehicles at the stop line should be as flat as possible in order to facilitate acceleration into the major road and allow the vehicle to stand with minimal application of brakes;

- the short vertical curve shall not encroach on to the shoulder of the major road; and
- the island shall be visible for all approaching trucks at truck stopping distance.

These requirements are illustrated in Figure 13.29 and Figure 13.30.

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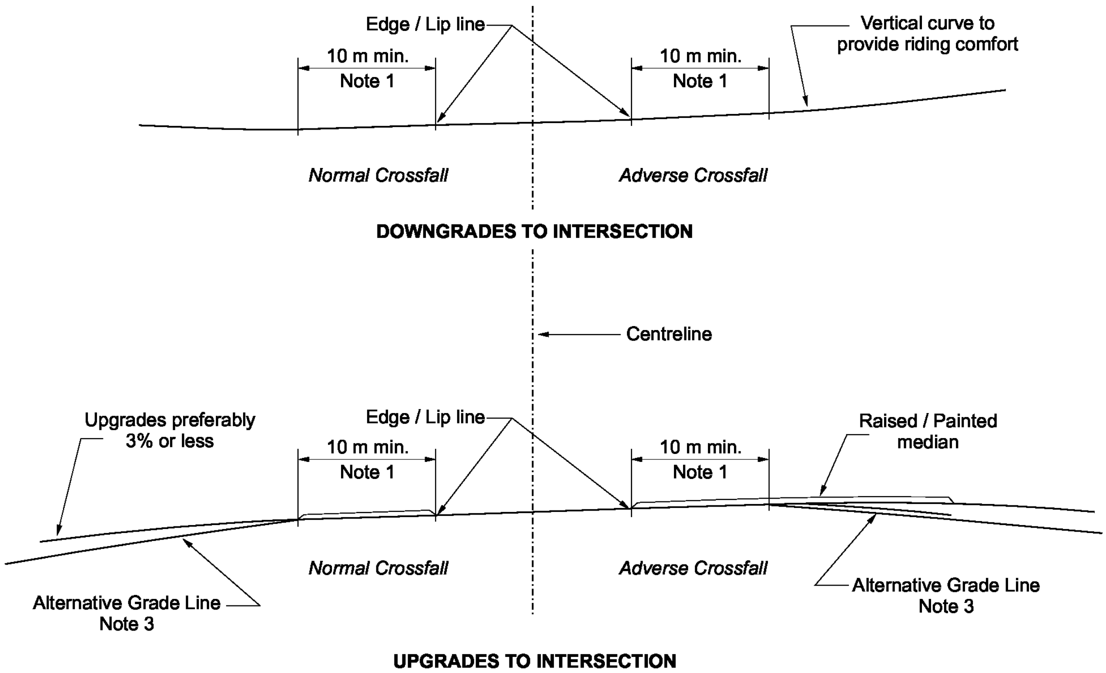

NOTES:
1. Grade should not exceed the crossfall for 10m in approach to edge/lip line.
2. SSD 1.15m to 0.0m to be provided to Stop / Give Way line and median nose.
3. Maximum algebraic change of grade for alternative grade line 12%.

The objective is to provide a reasonably flat section prior to the stop line so that a relatively easy entry condition occurs. The sight distance problems at such sites should be noted.

Figure 13.27 Cross Section of Major Road Showing Treatment of Legs which Approach a Grade

Figure 13.28 Sight Distance to an Intersection

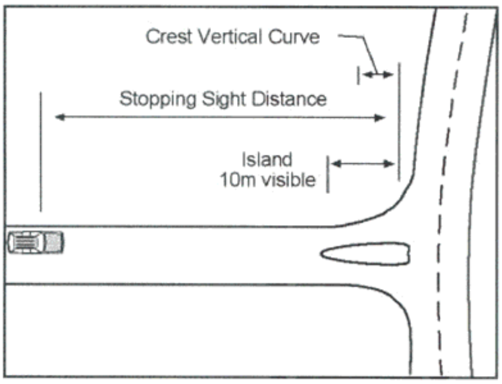

Figure 13.29 Sight Distance to T Intersections on Curves

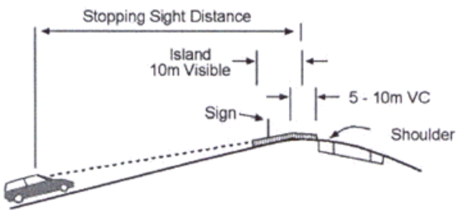

Figure 13.30 Approach Grading to T Intersections

13.6.4 Sight Distance Requirements

Intersection performance is dependent upon adequate horizontal and vertical sight distance for all entering traffic.

A feature of intersections is that sight lines are often required at large angles to the users' normal view point. In a motor vehicle, the driver may have to look through the side windows; pedestrians can be required to make observations over an arc of 180° (or more). As well, the paths travelled are often significantly curved which means that stopping distances (measured along the travel path) are more difficult to determine.

The type and extent of sight distance available will significantly influence the design and location of an intersection. Both horizontal and vertical sight lines must be taken into account to check for disruption by natural objects, (eg. trees), and structures, (eg. fences and buildings).

The same values of driver eye height contained in Chapter 9, are used, viz:

- cars - 1.15m; and
- trucks – 2.4m.

There are four sight distance criteria applicable to intersections:

- Approach Sight Distance (ASD)
- Safe Intersection Sight Distance (SISD)
- Entering Sight distance (ESD)
- Minimum Gap Sight Distance (MGSD)

These types of sight distance are discussed in the following sections. The minimum design requirements are ASD, SISD and MGSD. Providing ESD is desirable but often impractical. However, ESD should be provided if feasible.

In addition to the above specific intersection sight distance requirements, Stopping Sight Distance (SSD) must be available at all locations at the intersection, as detailed in Chapter 9. This requires the following:

- SSD for passenger cars using a 1.15m eye height and a 0.2m object height; and
- SSD for trucks using a 2.4m eye height and a 0.2m object height.

13.6.4.1 Approach Sight Distance (ASD) 1.15m to 0.0m

Approach Sight Distance (ASD) ensures that the approaching driver is able to appreciate the intersection geometry and pavement markings in order to negotiate the intersection or stop, which ever is required.

ASD is numerically equal to normal car stopping sight distance (SSD), which is defined as the distance travelled by a vehicle between the time when the driver receives a stimulus signifying a need to stop and the time the vehicle comes to rest.

Equation 13.2 provides the formula for ASD.

$$ASD = \frac{R_T \times V}{3.6} + \frac{V^2}{254 \times (d + 0.01 \times a)} \qquad (13.2)$$

Where

ASD = Approach Sight Distance (m)

R_T = reaction time (s) – refer Chapter 9 for values

V = operating (85th percentile) speed (km/h)

d = coefficient of deceleration – refer Chapter 9 for values

a = longitudinal grade in % (in direction of travel: positive for uphill grade, negative for downhill grade).

Values for ASD and corresponding minimum radii vertical crest curves relevant to various design speeds are given in Table 13.5. Corrections for grade are given in Table 13.6. The values in these tables refer to passenger cars only.

Approach Sight Distance, appropriate to the approach speed, should be provided on each leg.

The difference between ASD and SSD is the object height used in its calculation. ASD is measured from a driver's eye height (1.15m) to 0.0m, which ensures that a driver is able to see any line marking and kerbing at the intersection.

In circumstances where it is unreasonable or exceedingly difficult to achieve ASD to an object height of 0m, the design should provide, as an absolute minimum, SSD measured from a drivers eye height (1.15m) to an object height of 0.2m. This will ensure that signs and other road furniture at the intersection are clearly visible and provides a minimum standard to ensure that drivers are aware of the presence of an intersection.

13.6.4.2 Safe Intersection Sight Distance (SISD) 1.15m to 1.15m

Safe Intersection Sight Distance is the minimum sight distance which should be available from vehicles on intersection legs with priority to vehicles which could emerge from non signalised legs.

SISD comprises stopping sight distance plus three seconds of travel time (observation time).

This provides sufficient distance for a driver on an approach with priority to observe a vehicle entering from a side street, decelerate and stop prior to a point of conflict. It also provides sufficient sight distance to see an articulated vehicle, which has properly commenced a manoeuvre from a leg without priority, but still creating an obstruction because of its length. This requirement provides sufficient distance for an articulated vehicle to cross the non-terminating movement on two lane two way roads, or undertake two-stage crossings of dual carriageways with design speeds of 80km/h or more.

Equation 13.3 provides the formula for SISD.

$$SISD = \frac{D_T \times V}{3.6} + \frac{V^2}{254 \times (d + 0.01 \times a)} \qquad (13.3)$$

Where

SISD = Safe Intersection Sight Distance (m)

D_T = decision time (s) = observation time (3s) + reaction time (s): refer to Chapter 9 for values of reaction time

V = operating (85th percentile) speed (km/h)

d = coefficient of deceleration – refer to Chapter 9 for values

a = longitudinal grade in % (in direction of travel: positive for uphill grade, negative for downhill grade).

Table 13.5 Intersection Sight Distance for Level Pavement

Design Speed (km/h)	Dece-leration (g)[1]	Entering Sight Distance 1.15m to 1.15m (m)[3]	ASD - Approach Sight Distance (1.15m to 0.0m)				SISD - Safe Intersection Sight Distance (1.15m to 1.15m)			
			Absolute Minimum 2.0 secs[5]		Desirable 2.5 secs[5]		Absolute Minimum 2.0 secs[5]		Desirable 2.5 secs[5]	
			m[2]	min R[4]	m[2]	min R[4]	m[2]	min R[4]	m[2]	min R[4]
40	0.56	100	33	500	39	700	66	500	72	600
50	0.52	125	47	1000	54	1300	89	900	96	1000
60	0.48	160	63	1800	71	2200	113	1400	121	1600
70	0.45	220	82	2900	91	3600	140	2200	149	2500
80	0.43	305	103	4600	114	5700	170	3200	181	3600
90	0.41	400	128	7200	140	8600	203	4500	215	5100
100	0.39	500	157	10800	170	12600	240	6300	253	7000
110	0.37	500	190	15700	205	18300	282	8700	297	9600
120	0.35	500	229	22800	245	26100	329	11800	345	13000

Notes:
1. Average deceleration adopted, given in terms of acceleration due to gravity (g).
2. For grade corrections to ASD and SISD, see Table 13.6.
3. Limiting values of ESD based on the assumption that drivers are unlikely to seek gaps greater than 500m.
4. Crest vertical curve radius (m).
5. Reaction times.

Table 13.6 Grade corrections to ASD and SISD

(Source Austroads, 1988d: Guide to Traffic Engineering Practice Part 5)

Design Speed (km/h)	Correction							
	Upgrade				Downgrade			
	2%	4%	6	8%	2%	4%	6%	8%
40	-	-	-1	-1	-	-	1	2
50	-	-1	-2	-3	-	2	3	4
60	-1	-2	-3	-4	1	3	4	6
70	-2	-4	-5	-7	2	4	7	9
80	-3	-5	-7	-9	3	6	10	13
90	-4	-7	-10	-13	4	8	13	19
100	-5	-9	-14	-17	6	12	18	26
110	-7	-13	-18	-23	7	16	25	36
120	-9	-17	-24	-30	10	21	34	48

Note: Corrected stopping distances should be rounded conservatively to the nearest 5 metres

SISD is viewed between two points 1.15m above the road surface. One point is the driver's eye height on the leg with priority and the other represents the height of a vehicle in the side street, 5.0m (minimum of 3.0m) from the lip or edge line projection. Refer to Figure 13.31.

Values for SISD are given in Table 13.5. Corrections for grade are given in Table 13.6. The values in these tables refer to passenger cars only.

In the vertical plane SISD over crests is automatically achieved if SSD is provided for that crest.

The time gaps provided by applying the SISD model are generally sufficient for heavy vehicles to undertake the following movements:

- Left or right turn from the minor road onto the major road;
- Through movement from the minor road at a cross intersection; and
- Right turn from the major road into the minor road.

However, the time gaps may not be sufficient for heavy vehicles to undertake these movements in particular circumstances, eg:

- Where the design heavy vehicle is greater than a 19m semi-trailer;
- The major road is on a steep grade; and/or
- The major road comprises more than one lane in each direction.

Under such circumstances, specialist advice should be sought as to whether the minimum values of SISD are sufficient to cover the particular heavy vehicle movements.

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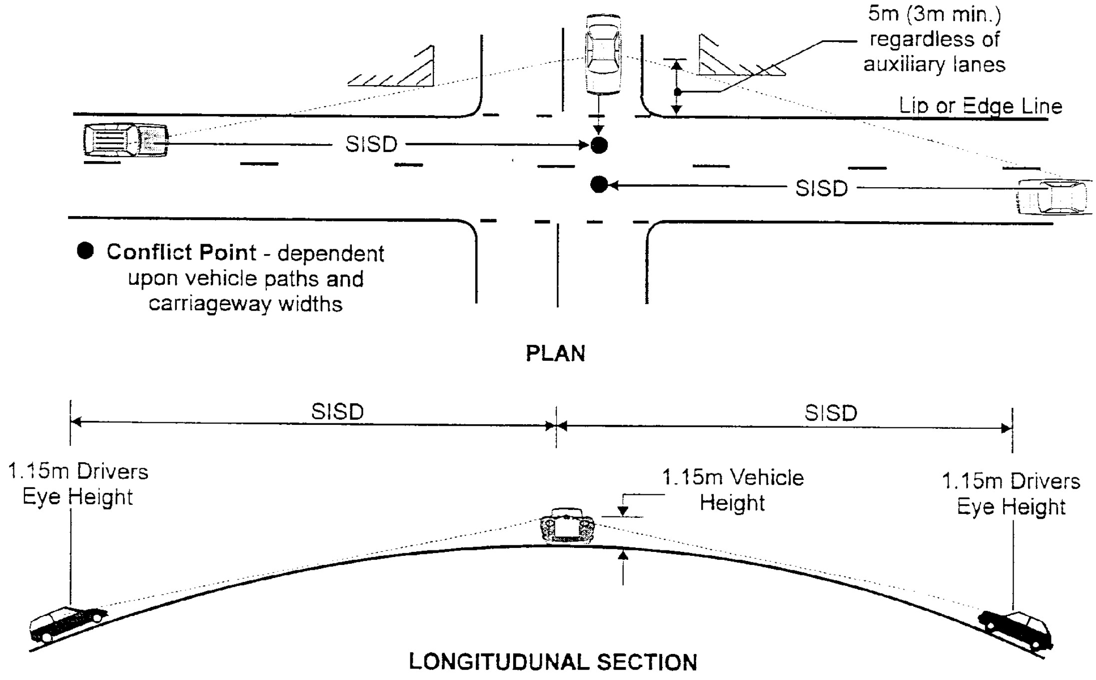


Figure 13.31 Sketch showing measurement of SISD

The diagram of safe intersection sight distance shown in Figure 13.31 shows sight lines between the vehicle entering from the minor road and the major road vehicles. Arndt (2004) shows that it can also be important to consider lines of sight for other conflict points and different geometric layouts. This is particularly the case where a minor road intersects on the outside of a tight horizontal curve, as shown in Figure 13.32. The SISD model should be applied to each of the cases shown in Figure 13.32 to ensure that adequate visibility is provided:

- Between vehicles approaching on the major road and vehicles turning right from the major road for BAR turn treatments (this is a similar requirement to the line of sight required between approaching major road vehicles and a stalled right-turning minor road vehicle at all types of right-turn treatments); and
- Between vehicles turning right from the major road and oncoming major road vehicles at all types of right-turn treatments.

13.6.4.3 Entering Sight Distance (ESD) 1.15m to 1.15m

Entering Sight Distance is required for traffic to enter from a side street and accelerate, so that it would not impede traffic on a non-terminating approach travelling in the same direction.

ESD is measured between two points 1.15m above the travelled way. One point represents the vehicle height on the non-terminating approach and the other the driver's eye height in the side street 5.0m (minimum 3.0m) from the lip or edge line projection.

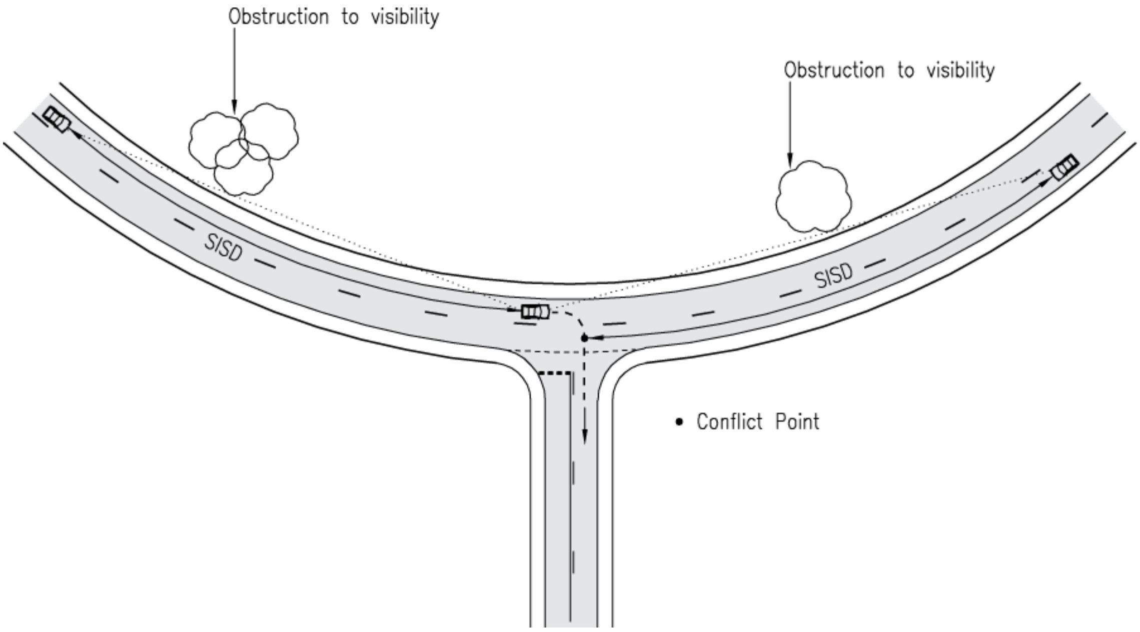


Figure 13.32 Application of the SISD Model for Minor Roads Intersecting on the Outside of Horizontal Curves

ESD is rarely achieved except in flat terrain or in a large sag V.C. and should be regarded as an optimum objective. Refer to Austroads (1988d) Section 5.2 'Sight Distance' for more detailed information on Entering Sight Distance.

13.6.4.4 Minimum Gap Sight Distance (MGSD) 1.15m to 0.6m

Gap acceptance behaviour was discussed in Section 13.3.2. (Refer also to Appendix 13A).

The MGSD model requires vehicles to be oriented so that drivers are able to see traffic that will conflict with their intended manoeuvre (both at the front and to the side or rear simultaneously) and judge whether there is a gap of sufficient length to make the manoeuvre.

As discussed in Section 13.3.1.3, Arndt (2004) showed that increased observation angles result in higher Angle-Minor vehicle accident rates; therefore supporting the following maximum sighting angles.

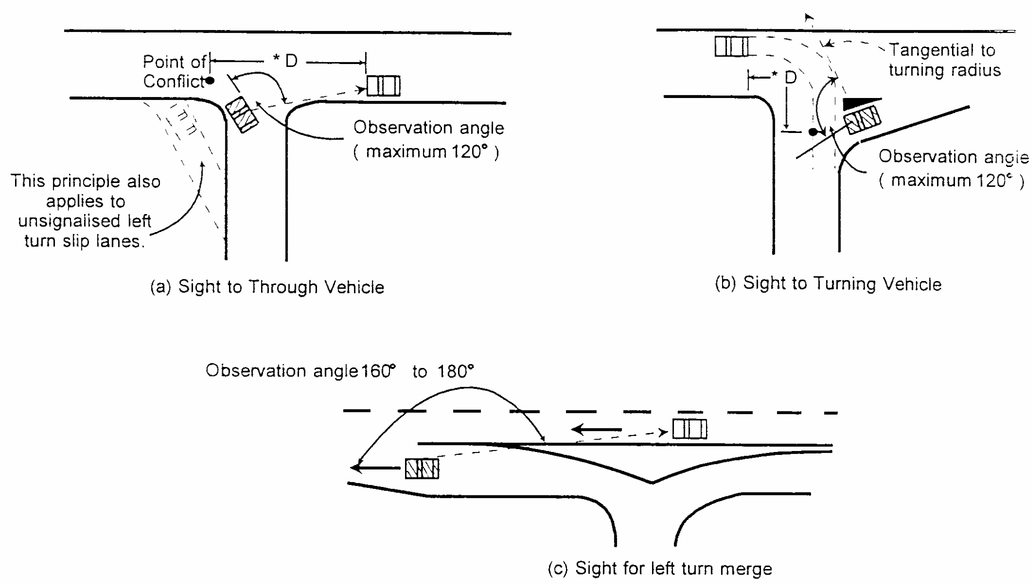
For left turns the sighting angles are restricted to a maximum of 120°, or between 160° to 180° for the left turn merge. Refer to Figure 13.33.

For right turns the sighting angles are restricted to a maximum of 110°, or between 170° to 180° for the right turn merge. Refer to Figure 13.34.

The sight distance required for an entering vehicle to see a gap in the conflicting streams sufficient to safely commence its desired manoeuvre is dependent upon:

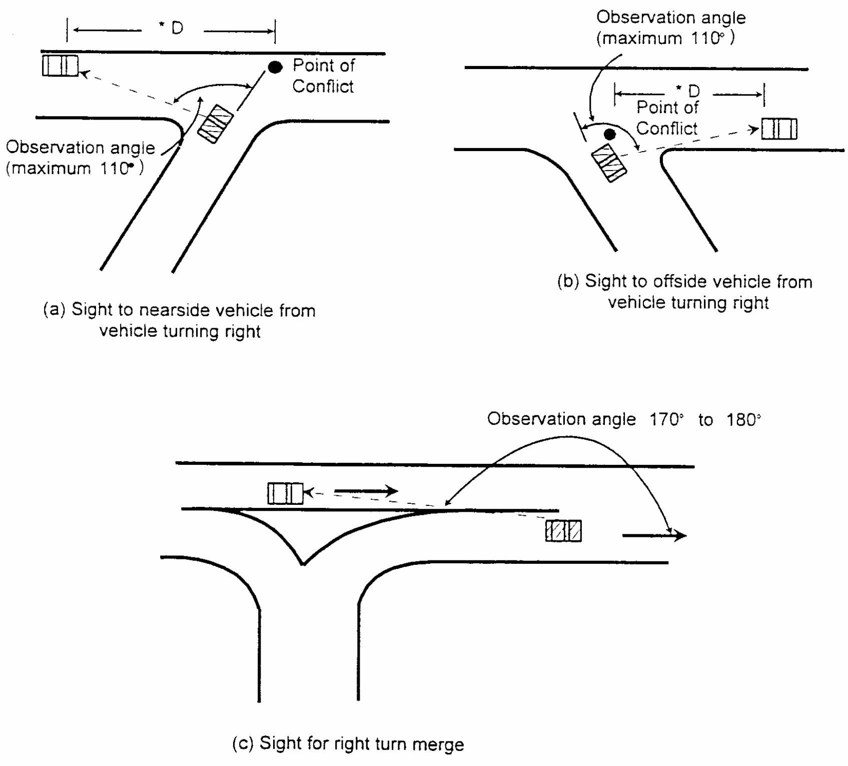
(a) length of the gap being sought (gap acceptance time t_a); and

(b) the observation angle to approaching traffic.



* D = Minimum Gap Sight Distance (m)

Figure 13.33 Sight Distance Requirements and Angles to Consider for Traffic Turning Left



* D = Minimum Gap Sight Distance (m)

Figure 13.34 Site Distance Requirements and Angles to Consider for Traffic Turning Right

MGSD is measured from the point of conflict (between approaching and entering vehicles) back along the centre of the travel lane of the approaching vehicle. This is shown as "D" in the sketches. It is measured from a point 1.15m (driver's eye height) to a point 0.6m (object height - typically the traffic indicator) above the travelled way.

Gap acceptance time will vary according to:

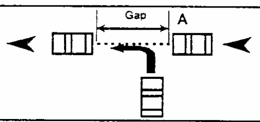
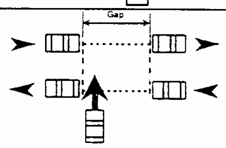
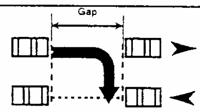
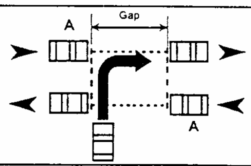
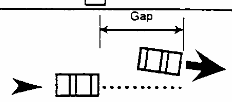
- type of manoeuvre - left turn/right turn/ crossing;
- width of carriageway - increased time required for greater widths;
- one way or two way traffic flows - increased time required to look both ways; and
- type and speed of vehicles, (for example, large heavy vehicles; the presence of bicycles).

Gap acceptance times for various manoeuvres into, from and across various through carriageway widths for both one way and two way traffic are shown in Table 13.7. The corresponding distances are given in Table 13.8.

13.6.4.5 Sight Distance Requirements at Signalised Intersections

Sight distance requirements at signalised intersections are given in Chapter 18 of this manual.

Table 13.7 Gap Acceptance Time for MGSD

Movement	Diagram	Description	t_a	t_f
Left Hand Turn		Not interfering with A Requiring A to slow	14-40 sec 5 sec	2-3 sec 2-3 sec
Crossing		Two lane / one way Three lane / one way Four lane / one way Two lane / two way Four lane / two way Six lane / two way	4 sec 6 sec 8 sec 5 sec 8 sec 8 sec	2 sec 3 sec 4 sec 3 sec 5 sec 5 sec
Right Hand Turn from major road		Across 1 lane Across 2 lanes Across 3 lanes	4sec 5sec 6sec	2 sec 3 sec 4 sec
Right Hand Turn from minor road		Not interfering with A One way Two lane / two way Four lane / two way Six lane / two way	14-40 sec 3 sec 5 sec 8 sec 8 sec	3 sec 3 sec 3 sec 5 sec 5 sec
Merge		Acceleration Lane	3 secs	2 secs

Note: $t_{(a)}$ *= Critical acceptance gap,* $t_{(f)}$ *= follow up headway*

Table 13.8 Table of Distance "D" (MGSD in metres) for Various Speeds

	85th Percentile Speed of Approaching Vehicle (km/h)											
		10	**20**	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**	**110**
Gap Acceptance Time (t_a) (secs)	**4**	11	22	33	44	55	67	78	89	100	111	122
	5	14	28	42	55	69	83	97	111	125	139	153
	6	17	33	50	67	83	100	117	133	150	167	183
	7	19	39	58	78	97	117	136	155	175	194	214
	8	22	44	67	89	111	133	155	178	200	222	244
	9	25	50	75	100	125	150	175	200	225	250	275
	10	28	56	83	111	139	167	194	222	250	278	305

13.6.4.6 Pedestrian Sight Distance Requirements

At intersections, pedestrian crossing facilities should be located where there is a clear view between approaching motorists and pedestrians on the crossing or waiting to cross a roadway. Approach Sight Distance (ASD) should be provided between approaching vehicles (1.15m eye height) and the surface of the roadway (0m) at the crossing. This ensures that, even if there is no pedestrian actually on the crossing, the driver should be aware of the crossing by seeing the associated pavement markings and other cues and therefore be alerted to take the appropriate action if a pedestrian steps onto the crossing without due attention to approaching traffic. Values of ASD are given in Table 13.5 (adjust for grade using values in Table 13.6).

In addition to providing ASD, it is necessary to ensure that the pedestrian can see approaching traffic in sufficient time to judge a safe gap and cross the roadway. The sight distance required for this is called 'Crossing Sight Distance' (CSD) and is calculated from the critical safe gap (in the traffic stream) and the speed of approaching traffic. Figure 13.35 shows a diagram of the provision of ASD and CSD at a pedestrian crossing facility.

The calculation of CSD is given by Equation 13.4.

$$CSD = t_c \times \frac{V}{3.6} \qquad (13.4)$$

Where

CSD = sight distance required for a pedestrian to safely cross the roadway

t_c = critical safe gap (s)

V = 85^{th} percentile approach speed (km/h)

The critical safe gap (t_c) is the time required for the pedestrian to safely cross the roadway. It is calculated from the crossing length and the walking speed. Chapter 5 provides guidance on selecting an appropriate walking speed.

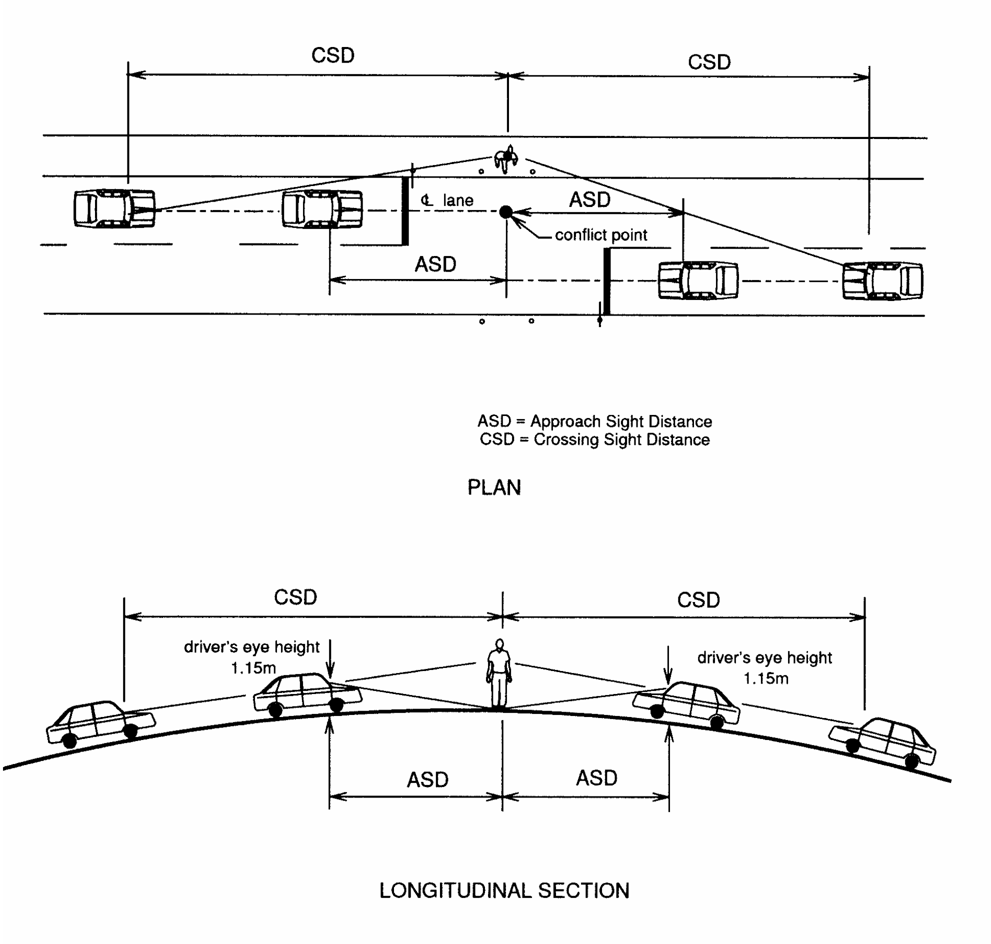


Figure 13.35 Sight Distance at Pedestrian Crossings

It is important that the line of sight not be obstructed by street furniture, such as poles, mailboxes, telephone booths, trees, decorative planters and so on. However, minor obstructions, such as posts, poles and tree trucks, less than 200mm diameter within the sight line may be ignored.

Particular attention needs to be given to parked vehicles which can pose visual obstructions, especially for children, wheelchair occupants or individuals of small stature. This may require banning parking for some distance on each side of the crossing, the distance being determined for each case to ensure that parked vehicles will not obscure the required sight lines. At those locations where there is a strong requirement by adjoining land uses to retain legal on-street parking, consideration should be given to extending the footpath.

Other design considerations for pedestrians crossing roads are given in Austroads (1995) 'Guide to Traffic Engineering Practice - Part 13 - Pedestrians'.

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13.7 Detailed Geometric Design

13.7.1.1 Lane Widths and Design Vehicles

Every intersection should be designed to accommodate the appropriate design and check vehicle. This has been discussed in Section 13.3.1.2.

Generally, turning lanes should be at least the same width as the adjacent through lanes given in Chapter 7. Where a lane (either through or auxiliary lane) is located between kerbs, the requirements of Section 13.7.2.6 apply.

Bicycle lanes through intersections should meet the requirements of Austroads (1999b) Part 14.

Motorcycles are overrepresented in multi vehicle crashes at intersections. Austroads (1999c) - Part 15 includes measures to minimise the risk for motorcycles and these should be included into the design of intersections. These measures will often reduce the potential risk for other motor vehicles.

13.7.2 Medians and Islands

13.7.2.1 General Requirements

Medians separate opposing traffic flow; islands separate traffic flowing in the same direction. Medians and islands can be raised, depressed, or defined by markings on the pavement.

Medians and islands defined in paint of various types, or depressed, have the same degree of legal control as raised installations if double lines are used, i.e. they should not be crossed by traffic.

There are no numerical warrants for the provision of raised medians in lieu of the painted medians (for both the major and minor road). Raised medians and islands are generally preferred in urban areas. In rural areas, provision of raised medians may be considered in situations such as the following:

- At the more important intersections eg those with high traffic volumes;
- At locations where consistency along a route is required;
- At locations where there are perception problems eg limited visibility to intersections on crests and tight curves;
- Where there is a need to minimise ‘corner cutting’ at the intersection; and/or
- Where there is a need to control movements into property accesses in the vicinity of the intersection.

The decision to use painted or raised medians and traffic islands in rural areas is not simple and each situation must be individually assessed.

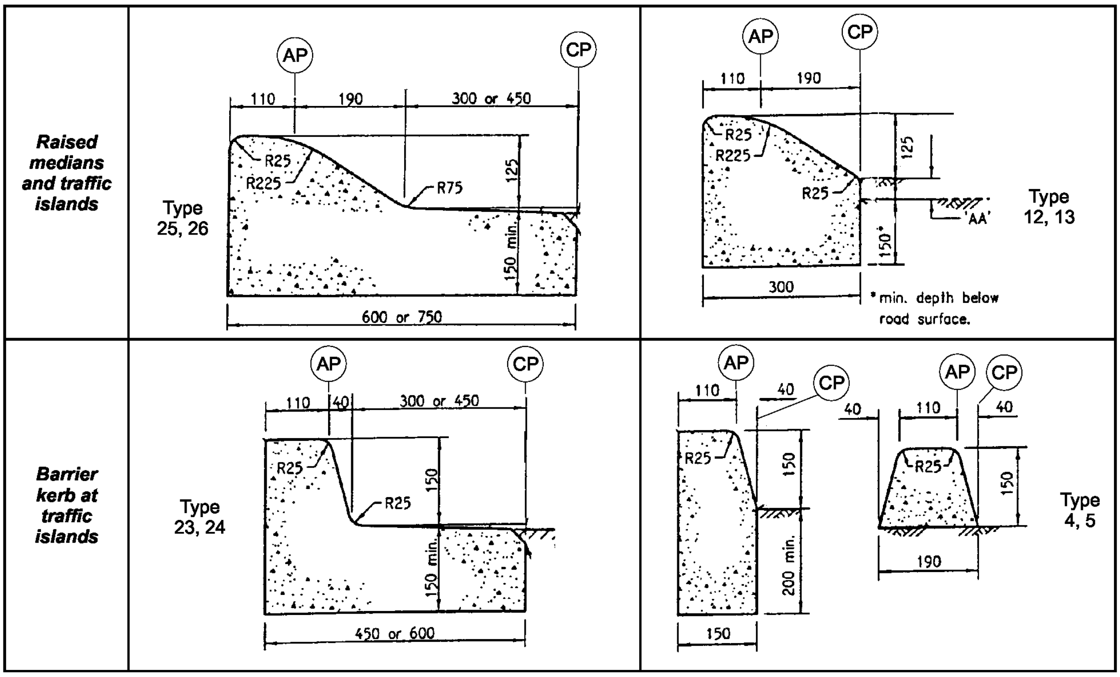
Raised medians are defined using semi-mountable or barrier kerbs. These are shown in Figure 13.36. Depressed medians can also be outlined using kerbs. The use of such kerbs has the following advantages:

- improved conspicuousness of the median;
- physically restricts turning or crossing movements;
- guides traffic around signposting and traffic signal hardware;
- defines refuge areas for pedestrians acting as a deterrent to vehicles;
- provides positive guidance for turning and through vehicles; and

- can concentrate and direct rainfall runoff into a gully reducing the risk of hydroplaning.

The use of such kerbs has the following disadvantages:

- requires lighting which can be expensive to install and maintain, especially in isolated areas;
- may require a greater network of drainage systems, increased maintenance activities and cost to cater for the concentrated rainfall runoff;
- could generate safety issues if struck by fast moving traffic;
- may require greater lane widths to cater for broken down vehicles;
- in instances where through traffic may block access into a right-turn lane at traffic signals, whereas a painted median would assist right turn manoeuvres by allowing drivers to drive over the median (within the limits of the road rules); and
- where traffic furniture on raised medians and islands is prone to damage by errant and over-dimensional vehicles.



The Clearance Point, marked "CP", is used to determine clear lane widths (see 13.7.2.2 Raised Medians and 13.7.2.4 Raised Islands). The Area Point marked "AP" is used to determine the areas of islands (see 13.7.2.4 Raised Islands). Refer Chapter 7 for details of kerb types.

Figure 13.36 Dimensions of Semi-mountable and Barrier Kerbs used on Median and Islands

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Semi-mountable kerbs are usually used (especially Type 11 semi-mountable) but in some locations, a barrier type may be appropriate (e.g. near Type 6 traffic signal posts). Semi-mountable kerbs have the advantage that they can be driven over by slow moving vehicles passing a broken down vehicle, where insufficient width is available on the road surface. However, kerbs are an obstruction on the road (especially barrier kerbs), so they must be highly visible and have properly designed and adequately maintained approach delineation, eg paint lines and reflectors.

Consequently, there are restrictions on the length, area and offset from the edge of lanes for medians and islands. These are detailed in the following sections.

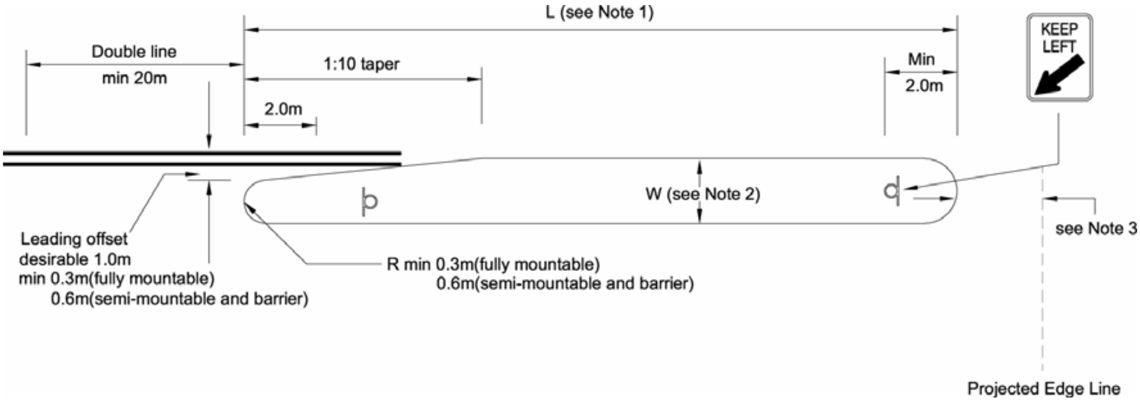
Islands at intersections should be designed to suit turning paths of design vehicles and maintain continuity of the major road through the intersection.

13.7.2.2 Raised Medians

(a) Minimum Dimensions

The layout of an isolated, raised median at a channelised intersection is given in Figure 13.37. Minimum lengths are given in Table 13.9.

Median widths are given in Table 13.10. Full median width shall be maintained for a distance of 2.0m each side of a pedestrian crossing.



Notes:
1. See Table 13.9.
2. See Table 13.10.
3. Nose located to suit turning paths. Where no specific bicycle facilities are provided on an urban road, the minimum offset is 1.0m. Where the projected edge line is the edge of an exclusive bicycle lane, minimum offset is 0.3m. For rural roads, the minimum offset is the greater of the shoulder width on the through road or 1.0m.

Figure 13.37 Minimum Treatment of an Isolated Median

Table 13.9 Minimum median Length (L) in Figure 13.37

Speed Prior to the Intersection (km/h)	Length (m)
60	10
80	20
100	40

Table 13.10 Minimum Widths of Medians at Intersections (W)

Conditions	Desirable Minimum Width	Absolute Minimum Width
No signs, signals or pedestrians	0.6	0.3*
Size A signs, no signs but some pedestrians, no signal posts	1.2	0.9
Single 200mm lanterns	2.4	1.2
Dual 200mm lanterns	2.4	1.5
Pedestrian Refuge/Size B signs	2.0	1.25
Refer to Figure 13.37 for the dimension 'W' * Painted medians only.		

(b) Desirable Clearances from Raised Medians to Lane Edges

Clearance from the point CP (Figure 13.36) to:-

- Offside edge of lane on the major road, speed zoned 80km/h or less: 0.0m
- Offside edge of lane on the major road, speed zoned more than 80km/h: 0.5m
- Offside edge of lane on the minor road: 0.0m

13.7.2.3 Painted Medians

(a) General

Painted medians can be used in the following situations:-

- approaches to a raised or depressed median;
- where an intersection is unlit; and
- where the resultant width between kerbs would be too narrow for a raised median.

(b) Minimum Dimensions for Painted Medians

The minimum length of a painted median at an intersection should be in accordance with Table 13.9. This length excludes any transition between the median width and centreline pavement marking.

Treatment for various widths of painted median is shown in Figure 13.38. The minimum width is 0.3m.

(c) Desirable Clearances from Painted Medians to Lane Edges

Clearances from edge of painted median line to offside edge of lane: 0.0m

13.7.2.4 Raised Islands

(a) General

For definition purposes raised islands should have a minimum area as follows:-

- urban - 8.0m² (unsignalised);
- urban - 25.0m² (signalised); and
- rural - 40.0m².

See Figure 13.36 for measurement points.

Dimensions are site specific. At signalised sites, where pedestrian crossings are to be provided, minimum lengths of island sides abutting the crossing should be in accordance with Figure 13.39. Islands must be large enough to accommodate all of the necessary equipment and treatments.

A clearance of 0.5m from a face of kerb to signal lantern target board is required (from both sides). Refer to Chapter 18.

(b) Desirable Clearances from Raised Islands to Lane Edges

Minimum clearances from point CP (see Figure 13.36) to adjacent edge of lanes are given in Figure 13.39. Clearances for vehicles at raised islands, where there are no specific bicycle facilities, are given in Figure 13.41 in Section 13.7.2.6.

13.7.2.5 Painted islands

(a) General

Painted islands can be used in the following cases where:-

- an intersection is unlit; and/or
- the resultant width between kerbs would be too narrow for a raised median.

Large islands, as shown in Figure 13.40(a), will accommodate standard pavement markings. Smaller islands, which will not accommodate standard pavement markings, should be fully painted as shown in Figure 13.40(b).

(b) Chevrons

Chevron treatment on approaches to medians and islands should be in accordance with the "Guide to Pavement Markings" Manual (Main Roads, 2000).

Where the approach to a median or island is too small to accommodate standard chevrons, the area should be fully painted, with the edges offset 200mm from the island outline. Figure 13.40(b) gives details.

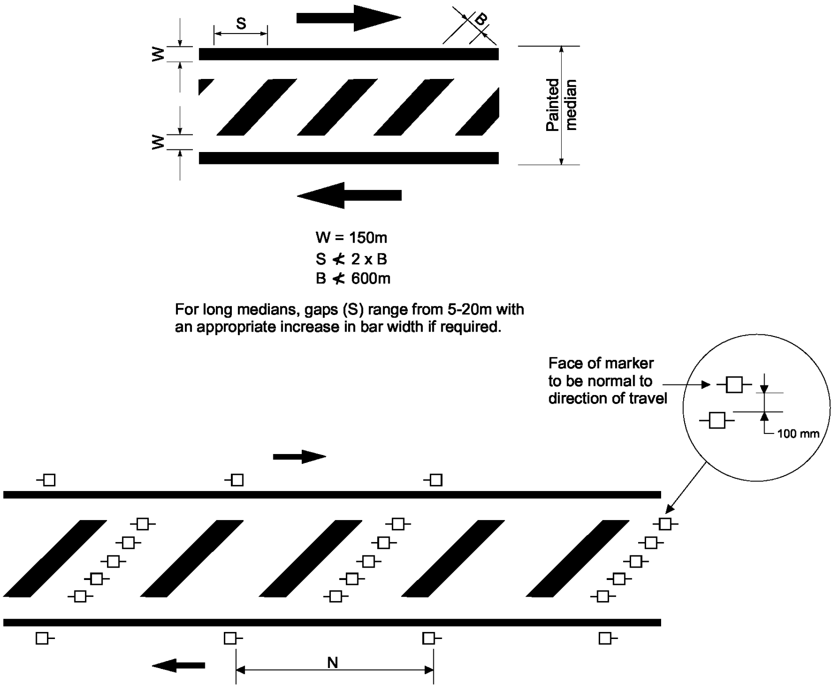
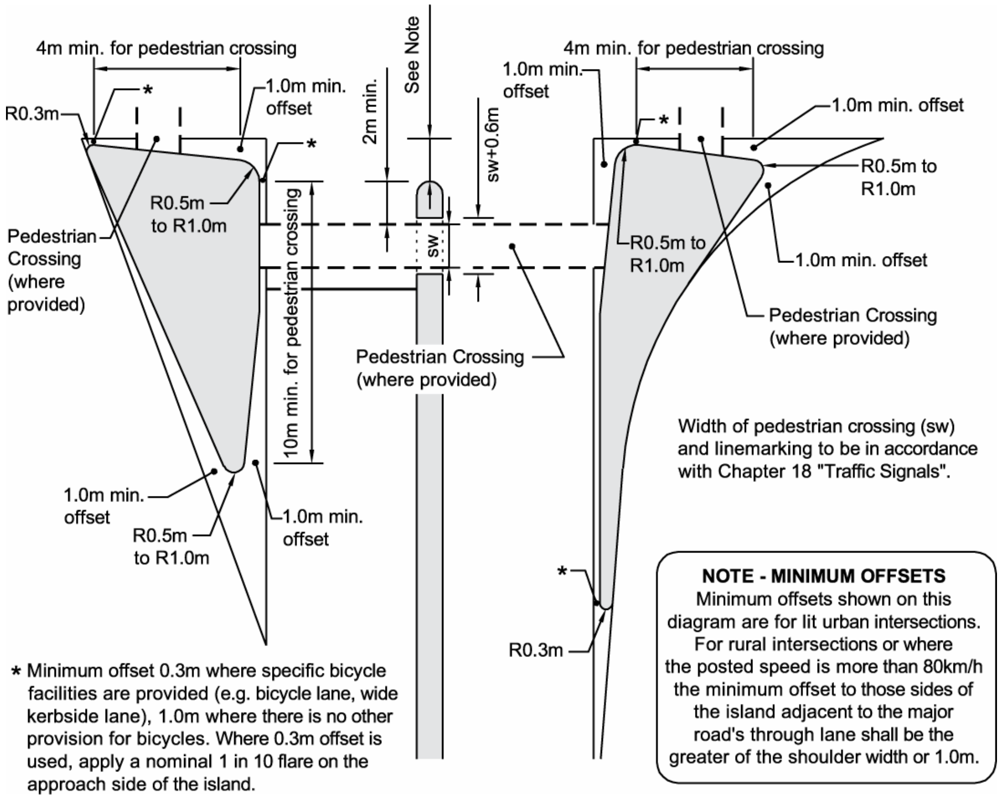


Figure 13.38 Treatment of Painted Medians



Note: Refer to Figure 13.37 for clearance to raised median

Figure 13.39 Detailed Island Treatment Showing Offsets

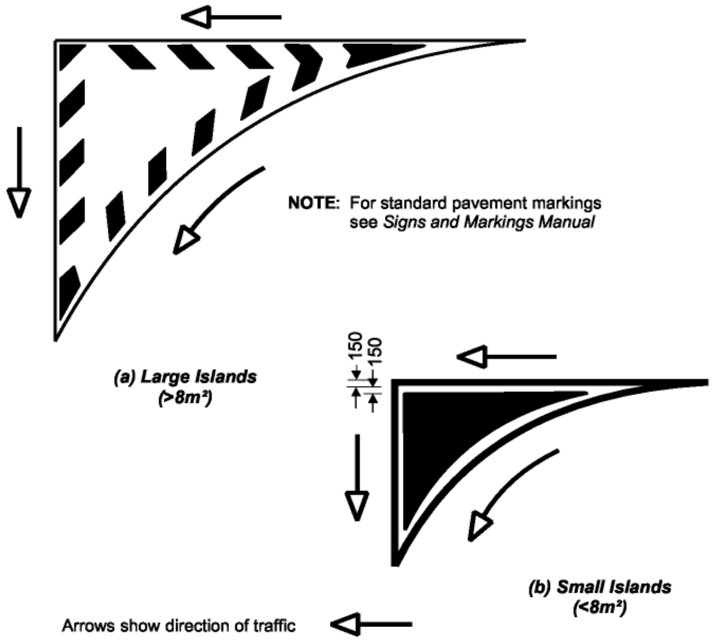


Figure 13.40 General Arrangement for Painted Islands

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13.7.2.6 Clearance between Kerbs and between Kerbs and Roadside Barriers

It is desirable to provide a clearance of 5.5m between kerbs and between kerbs and roadside barriers to allow passing of broken down vehicles. It is important to apply this width where long lengths of parallel kerbing (or kerbing and barrier) apply.

At other than the above location, the width of 5.5 m is not mandatory if other provisions for passing broken down vehicles are provided. Such provisions may include a very slow passing manoeuvre either partially or totally on an island or median. For this to occur, islands/medians would require mountable or semi-mountable kerbing with sufficient offset to hardware (eg signs, light poles and traffic signal posts).

The actual clearances provided between kerbs and between kerbs and barriers will need to accommodate the design vehicle swept path plus offsets and possibly the check vehicle (especially between barriers). Figure 13.41 shows how to provide for the design vehicle swept paths at an intersection comprising single lane carriageways. In some cases, the clearances required to cater for the design vehicle swept path are greater than 5.5m.

For roundabouts, use the minimum clearances between kerbs provided in Chapter 14. Generally, roundabouts are provided with smaller clearances so that the required entry path curvature and deflection can be more easily obtained (ie 5m minimum in lieu of 5.5m and no separate allowance for bicycles on the carriageway with bicycles being catered for off road).

13.7.2.7 Bicycles

Intersections are areas of high conflict and are difficult for bicycles to traverse. Austroads (1999b) and the Queensland Transport Cycle Notes provide guidance on specific design for bicycles.

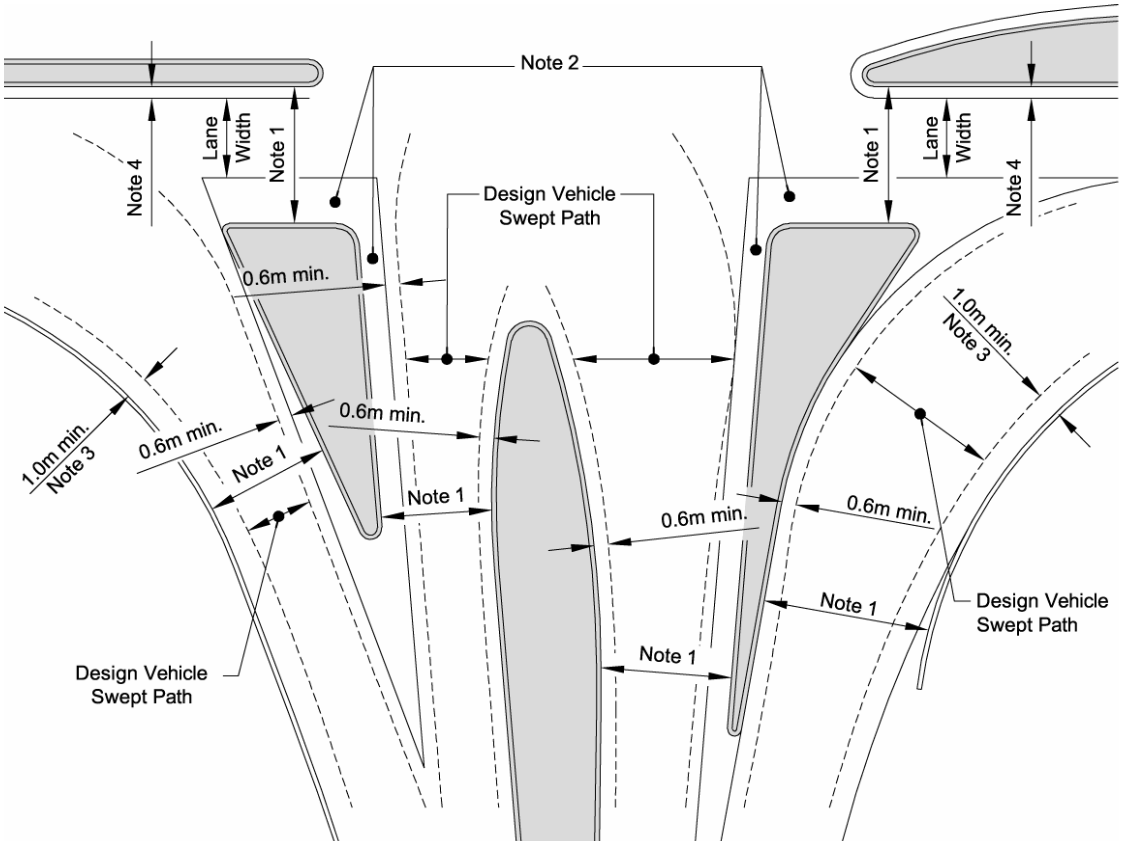
In accordance with the Main Roads policy 'Cycling on State Controlled Roads' (QDMR, 2004), road upgrades must incorporate 'cycle-friendly' designs. Along priority cycling routes, road upgrades must incorporate marked cycle lanes, cycle paths, shared paths or other facilities for cyclists.

A cycle-friendly design feature of urban intersections is the provision of 1.0m minimum offsets from the edge of lane to kerb faces where there is no other provision for cyclists eg there is no separate bicycle lane. This is to avoid cyclists having to negotiate 'squeeze points' at the intersection. On the major road in rural areas, the minimum offset must be the greater of the shoulder width and 1.0m.

If bicycle lanes are present either side of an intersection, specific cycle facilities must be provided to guide cyclists through the intersection.

Even a short marked cycle lane through an intersection that does not provide route continuity may provide safety advantages to cyclists provided that its termination point does not lead cyclists into an unsafe situation.

Where there are a high number of cyclists or an intersection has a poor cycle safety record, a green coloured pavement surface for the cycle lane might deliver added cycle safety.



1. Minimum desirable width between kerbs to allow for broken down vehicle is 5.5m. This width is not mandatory if other provisions for passing broken down vehicles are provided. Such provisions may include mountable or semi-mountable kerbing on islands/medians with sufficient offset to hardware (eg signs, light poles and traffic signal posts) for allow for a very slow passing manoeuvre.
2. Offsets between raised islands and adjacent edge lines are given in Figure 13.39. As no specific bicycle facilities exist in this example, a minimum 1m offset applies to cater for bicycles in urban areas. The 1m offset provides the capabilities listed in Note 3. On the major road in rural areas, the minimum offset must be the greater of the shoulder width and 1.0m.
3. The 1m offset provides:
- clearance from the kerb to the design vehicle swept path
- additional width for the check vehicle
- provision for cyclists
4. This diagram shows an intersection with no specific bicycle facilities. For diagrams of intersections with specific bicycle facilities (eg exclusive bicycle lanes), refer Austroads (1999b).

Figure 13.41 Detailed Island Treatment showing Clearances at an Intersection with Single Lane Carriageways and No Specific Bicycle Facilities

Hook turns for cyclists may be considered on multilane roadways at signalised intersections to reduce conflicts between on-road cyclists and motor vehicles. Hook turns eliminate the need for cyclists to weave across two or more lanes of traffic travelling at relatively high speed in order to make a right turn.

Approaches to left turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. Treatments to reduce the number of potential conflicts at left turn slip lanes are given in Austroads (1999b).

Wide kerbside lanes enable greater separation of cyclists and motor vehicles, creating a higher level of safety and increased operational efficiency. Wide kerbside lanes should be carried through intersections to avoid 'squeeze points'.

13.7.3 Through Lane Conversions

Conversion of an approach through lane of a multi-lane road into an exclusive left turn or right turn lane should be avoided as it may cause some through traffic to change lanes at the last moment, creating a potential for accidents, particularly in areas with high tourist or visitor populations. This treatment is not to be used in the design of a new intersection. Should such a conversion be unavoidable at an existing intersection, advance warning and guidance signs shall be erected informing drivers of what to expect. The signs shall be supplemented by pavement arrows.

13.7.4 Auxiliary Lanes at Intersections

At an intersection, an auxiliary lane is an additional lane (or lanes) added to the through carriageway for safety and/or intersection capacity purposes. Auxiliary lanes can be added to either the “near” or the “off” side (or both), and on the approach or departure (or both). On the approach side they are designed on the basis of various deceleration models; on the departure side, models of acceleration are used.

Auxiliary through lanes are often required to increase the capacity of an intersection (see also Chapter 18). These lanes are added on the approach to the intersection, carried through the intersection and dropped on the departure side.

Auxiliary lanes can provide an increase in potential conflict for cyclists. Section 5.2.2 of Austroads (1999b) outlines the process for evaluating such potential conflicts.

To maintain capacity of the through road, an auxiliary lane should be designed to operate with minimal interference to the through movement. The length of this lane will be governed by such issues as:

- deceleration from the approach speed to a stop for a right turn except on one way roads;
- deceleration from the approach speed to the turning speed applicable to the geometry of the turn as is the case for left turn slip lanes;
- additional length required for storage of vehicles queuing to turn; and
- acceleration from the turning speed applicable for the geometry of the turn to the speed of the through road.

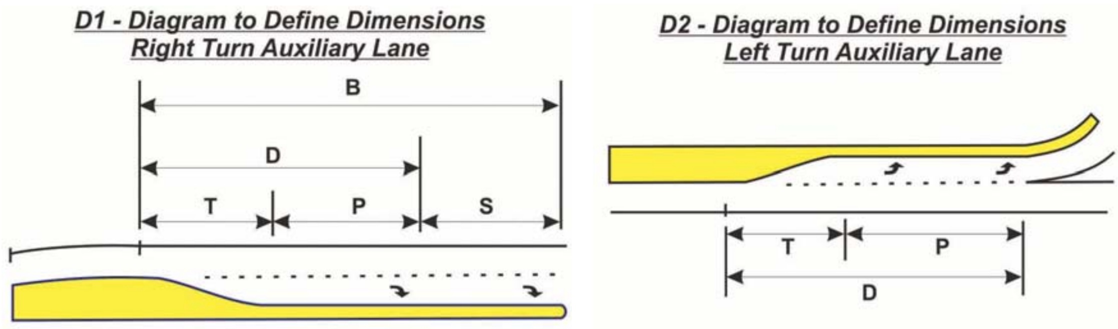
13.7.4.1 Deceleration Lanes

Design requirements for a separate auxiliary lane on the approach to an intersection will vary for urban and rural conditions. Basic design considerations include:

- diverge length;
- deceleration length;
- storage length;
- length of tapers.

For a deceleration lane the normal treatment is to have the length of auxiliary lane governed by full deceleration occurring within the lane from the start of the diverge taper, as shown in Figure 13.42. The diverge/deceleration length (D) in this figure is the greater of the diverge length (T_d) and the deceleration length. On level grades, these lengths are determined from Table 13.11.

A shorter length can be considered to reduce cost where turning volumes are low and/or the auxiliary lane has been provided principally to shelter a turning vehicle ie at CHR(S) and AUL(S) turn treatments. Deceleration will occur partially in the approach through lane with the resultant diverge length being shorter due to a lower entering travel speed.



B = total length of auxiliary lane, including taper, diverge/deceleration and storage (m)
D = diverge/deceleration length (m)
T = taper length (m)
S = storage length (m)
P = length of parallel lane (m)

Correction to Grade

Grade (%)	Ratio of length on grade to length on level	
	Upgrade	Downgrade
0 to 2	1.0	1.0
3 to 4	0.9	1.2
5 to 6	0.8	1.35

Left Turn Auxiliary Lanes Reduction to D when exit speed ≥ 20km/h

Approach speed (km/h)	Reduction in D (m)
20	10
30	15
40	25
50	40
60	55
70	75
80	100
90	125

Figure 13.42 Diagrams to Define the Dimensions of Auxiliary Lanes and Corrections to Deceleration Length 'D' to Allow for Grade

Table 13.11 Deceleration Distances Required for a Vehicle on a Level Grade

Design Speed of Approach Road (km/h)	Length of Deceleration (m) – including tapered approach where design speed of exit curve (km/h)										Diverge Length T_d (m) for lane widths	
	0 (Refer Note 2)		20	30	40	50	60	70	80	90	3.5m	3.0m
	Comf. 2.5m/s^2	Max 3.5m/s^2	Comfortable average rate of deceleration 2.5m/s^2									
50	40	30	30	25	15						50	40
60	55	40	50	40	30	15					60	50
70	75	55	70	60	50	40	20				70	60
80	100	70	95	85	75	60	45	25			80	65
90	125	90	120	110	100	85	70	50	25		90	75
100	155	110	150	140	130	115	100	80	55	30	100	85
110	185	135	180	175	160	150	130	110	90	60	110	95

Notes:
1. Distance T_d should be used in this zone
2. Adjust for grade using table in Figure 13.42 – Correction to D for Grade
3. Rates of deceleration: 2.5m/s^2 - Comfortable
3.5m/s^2 - Desirable Maximum

Diverge Length (T_d)

The diverge length is the distance required for a vehicle to diverge from the through lane into the auxiliary lane. It is calculated from Equation 13.5.

$$T_d = \frac{V \times Y}{3.6 \times S} \qquad (13.5)$$

Where:

T_d = diverge length (metres)

V = design speed* (km/h)

Y = width of lateral movement (metres)

S = rate of lateral movement (1.0m/sec)

*V - Where interference to the through lane is acceptable the approach speed is decreased due to deceleration occurring partially within the approach.

For most practical purposes diverge length T_d (m) = design speed V (km/h).

Diverge lengths for various lane widths are given in Table 13.11. Where the diverge length is greater than the deceleration length, the diverge/deceleration length (D) should be based on the diverge length.

Deceleration Length

The deceleration length is the distance required for a vehicle to decelerate from the design speed to stop, or to a turning speed governed by the turn radius. This length is determined on level grades from Table 13.11. Corrections for grade can be made using the ‘Correction to Grade’ table in Figure 13.42.

In most cases, the length of deceleration will be based on the comfortable deceleration criterion in Table 13.11. In very constrained situations, the length of deceleration may be based on the maximum deceleration criterion in this table.

Where the deceleration length is greater than the diverge length, the diverge/deceleration length (D) should be based on the deceleration length.

Deceleration from the design approach speed to zero should be used for all right turns and left turns unless the left turn:

- has priority (e.g. turning from through road where no slip lane is provided); or
- is under free flow conditions (e.g. slip lane with protected departure lane provided).

To determine the preferred length required for deceleration:

1. select appropriate turning speed from Figure 13.43 for turn radius;
2. using turning speed, determine deceleration length from Table 13.11;
3. adjust for grade using the 'Correction to Grade' table in Figure 13.42.

Turning speeds for various radii and crossfall are shown in Figure 13.43.

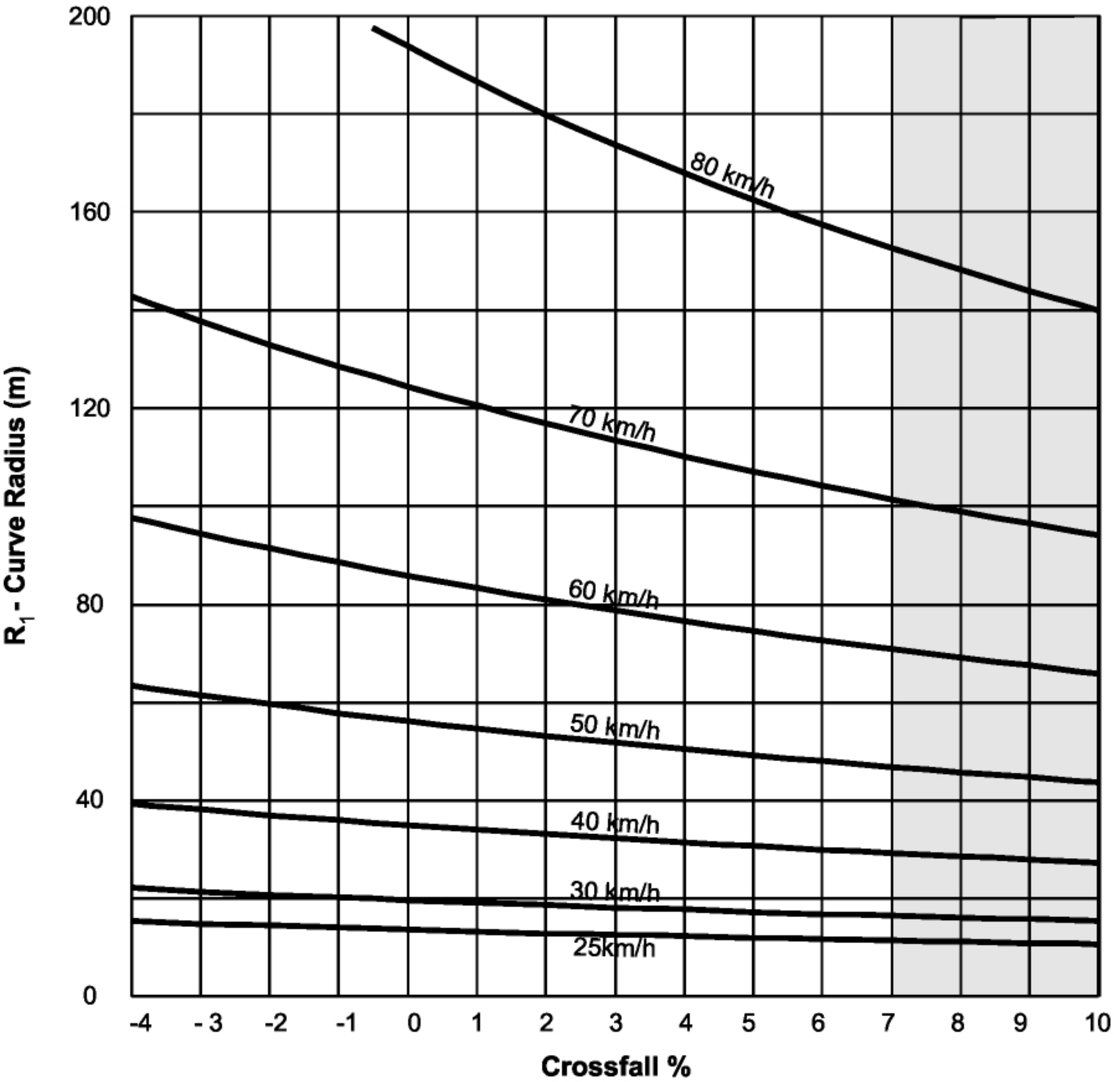


Figure 13.43 Turning Speeds for Various Combinations of Radius and Crossfall (f maximum – See Table 13.12)

The values of R_1 are calculated using Equation 13.6.

$$R_1 = \frac{V^2}{127 \times (e + f)} \qquad (13.6)$$

where:

R_1 = curve radius (m)

V = speed (km/h)

e = superelevation (m/m)

f = friction factor between vehicle tyres and the pavement (maximum in Table 13.12).

The crossfall adopted for turning roadways should not exceed +7% or -3% (absolute maximum -4%). For a turn executed at very slow speed (say<10km/h), the desirable maximum adverse crossfall is -5% with an absolute maximum -7%.

Table 13.12 Side Friction Factor – f

V (km/h)	f - maximum
25	0.36
30	0.36
40	0.36
50	0.35
60	0.33
70	0.31
80	0.26

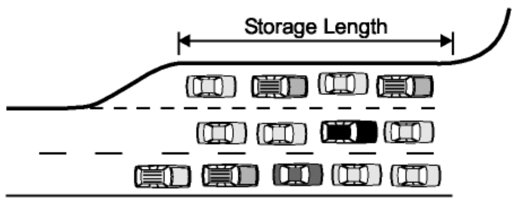
Storage

The storage length is the distance required to store vehicles in a lane while they are waiting to pass through the intersection.

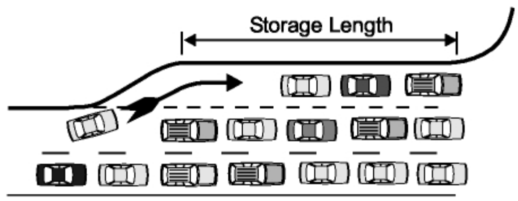
Storage lengths can be determined by simulating the operation of an intersection using various computer programs such as aaSIDRA. To use aaSIDRA, a traffic count and a preliminary intersection layout design is required. In the case of an unsignalised intersection, storage lengths can be determined using Appendix 13A "Computation Analyses for Non Signalised Intersections". Normally, a 95th percentile storage length is adopted.

Other computer programs can be used but the above is recommended. For co-ordinated signals, the cycle time for the system should be used for simulation purposes.

At signalised intersections, the auxiliary lane length is determined by adopting the storage length required for left turn vehicles, or through vehicles in the adjacent lane, whichever is the greater. This is shown in Figure 13.44.



(a) Left Turn Queue is critical



(b) Through Queue is critical

Figure 13.44 Details of the Definition of Storage Length Requirements

Both cases (a) and (b) in Figure 13.44 apply where the left lane must turn left, i.e. the auxiliary lane is not shared by through vehicles. In case (b) it is important to provide sufficient length so that stored through traffic does not block entry to the left turn lane causing the auxiliary lane to be under used.

Bicycle "head start" storage areas may also be considered. Guidelines for design are provided in Section 5.4.2.3 of Austroads (1999b).

Taper Lengths

The taper of a deceleration lane is the length over which the kerb, or edge line, transitions at the beginning of the lane.

It is desirable that taper lengths shorter than the normal diverge length be provided on deceleration lanes to give additional storage capability for those times when the 95th percentile queue length is exceeded. Where storage length is controlled by site constraints, a short taper length will also maximise the available space. Short taper lengths also better define the start of the deceleration lane. Equation 13.7 provides desirable taper lengths of deceleration lanes in urban and rural areas.

$$T = \frac{0.33 \times V \times W_T}{3.6} \qquad (13.7)$$

Where

T = taper length (m)

V = design speed of major road approach (km/h)

W_T = width of turn lane (m)

Using Equation 13.7, the taper length in urban areas is around 15 - 20m long for a 3m wide turn lane. The taper can comprise small radius reverse curves joined by a short straight (say 10-15m) to allow for reversal of steering. Using Equation 13.7, the taper length in high speed rural areas is around 30m long for a 3m wide turn lane.

Care must be taken where the auxiliary lane commences on a curve. Use of a short straight taper will better define the commencement of the auxiliary lane under such conditions.

It must be noted that the taper length is a site for potential conflict between through travelling bicycles and left turning motor vehicles. Guidelines for design are provided in Section 5.2.2 of Austroads (1999b).

13.7.4.2 Acceleration Lane Design for Passenger Cars

An added auxiliary lane on the departure side of a left or right turn may be provided if traffic is unable to join safely and/or efficiently with adjacent through traffic flow.

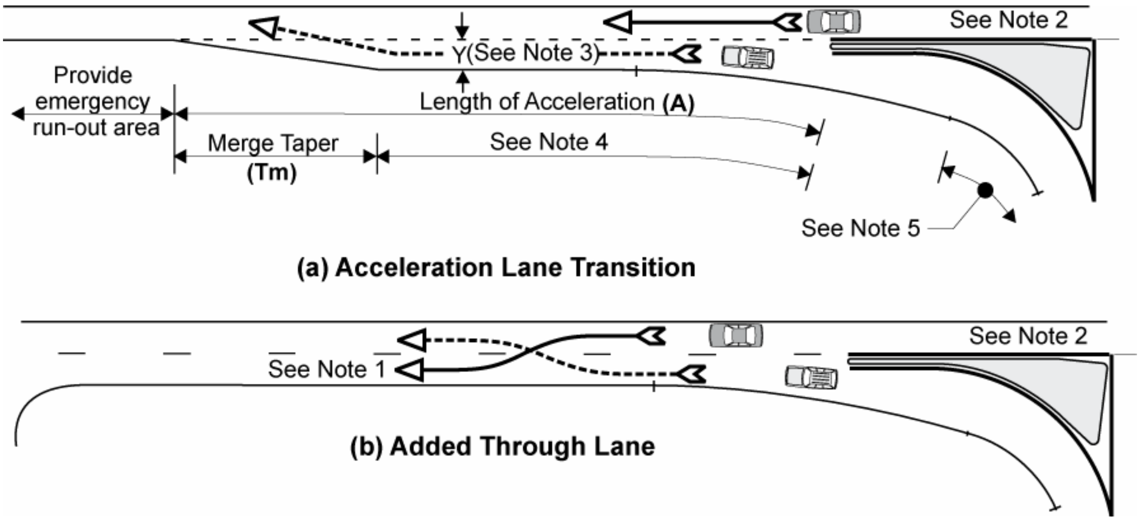
For an acceleration lane the length of auxiliary lane is governed by full acceleration to the through travel speed by the end of the merge taper. If this cannot be achieved the "lane" should act as a storage bay for one vehicle only, allowing the driver a two stage manoeuvre.

On roads nearing capacity, it may not be possible to terminate the auxiliary lane when the required acceleration length has been reached. Instead, the auxiliary lane would need to continue on until traffic conditions allowed it to be terminated. The proximity of other intersections must be taken into account to determine whether unsafe weaving manoeuvres would occur. The length available for weaving manoeuvres, and the number of lanes across which the weaving will occur, must be critically examined. The Highway Capacity Manual (TRB, 2000) can provide further information.

For an acceleration lane three basic design requirements need to be considered:

- acceleration length;
- merge taper length; and
- a run-off area.

These are illustrated in Figure 13.45.



Notes:
1. Proximity of downstream intersection may be critical for weaving manoeuvres.
2. Approaches to left turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. Treatments to reduce the number of potential conflicts at left turn slip lanes are given in Austroads (1999b).
3. Y= Width of lateral movement.
4. Length not less than 4 seconds of travel time at the through road speed - refer to Table 13.13.
5. Refer to Figure 13.43 for turning speed.

Figure 13.45 Options for Auxiliary Lanes on the Departure Side of an Intersection

Length of Acceleration Lane

The length of acceleration 'A' is the distance required for a vehicle to accelerate to the mean free speed of the road (which is often approximately equal to the speed limit) from either a stationary position, or from the turning speed governed by the turn radius. This is shown in Figure 13.45(a). The length of acceleration for level grades is determined from Table 13.13 with corrections for grade being applied as detailed in Table 13.14.

Where the acceleration lane is preceded by a turning movement, the turning speed can be obtained from Figure 13.43.

To determine the length required to accelerate to the mean free speed of the entered road:

1. select appropriate turning speed from Figure 13.43 for turn radius and crossfall;
2. using this turning speed, determine acceleration length from Table 13.13;
3. adjust for grade using Table 13.14.

This length includes the merge taper. However, the length of the merge taper is not adjusted by the grade correction factor.

The same acceleration lane design criteria are applied to urban and rural conditions.

Table 13.13 Length of Acceleration Lanes on a Level Grade

Speed Reached (km/h) #	Length of Acceleration A (m) (including length of pavement taper) where design speed of entry curve (km/h) is								4 sec travel (m)	Merge T_m (m)	Min desir. length 4 sec+T_m
	0**	20	30	40	50	60	70	80			
50	70	55	45	30					55	50	105
60	110	95	85	70	40				65	60	125
70	165	150	140	125	95	55			80	70	150
80	235	220	210	195	165	125	75		90	80	170
90	330	315	305	290	260	220	170	95	100	90	190
100	450	435	425	410	380	340	290	220	110	100	210
110	610	595	585	570	540	520	480	420	120	110	230

(shaded) Adopt minimum desirable length = 4 sec travel + T_m

\# For the purpose of calculating acceleration lane lengths at intersections, the speed reached is usually made equal to the mean free speed (which is often approximately equal to the speed limit)

** Length required where a vehicle accelerates from zero speed

Table 13.14 Correction of Acceleration Distances as a Result of Grade

Design Speed of Road Entered (km/h)	Ratio of Length on Grade to Length on Level * for: Design Speed of Turning Roadway Curve (km/h)									
	3 to 4% Upgrade					5 to 6% Upgrade				
	Stop	30	50	60	80	Stop	30	50	60	80
50	1.3	1.3				1.4	1.5			
60	1.3	1.3	1.3			1.5	1.5	1.5		
80	1.3	1.3	1.4	1.4		1.5	1.5	1.7	1.9	
100	1.3	1.4	1.5	1.5	1.6	1.6	1.7	1.9	2.2	2.5
110	1.4	1.5	1.6	1.6	1.8	1.8	2.0	2.2	2.6	3.0
	3 to 4% Downgrade					5 to 6% Downgrade				
	All Speeds					All Speeds				
50	0.7					0.6				
60	0.7					0.6				
80	0.65					0.55				
100	0.6					0.5				
110	0.6					0.5				

* Ratio from this table multiplied by length in Table 13.13 gives length of speed change lane on grade.

The length of the acceleration lane to be provided is a minimum of:

- The acceleration length as calculated above; and
- The distance travelled in four seconds at the speed of the through road (usually made equal to the mean free speed) plus the length of the merge taper.

Use Table 13.13 to determine these lengths.

Merge Taper Length (Departure)

The merge taper length is the distance required for a vehicle to merge from the auxiliary lane into the adjacent through lane. The merge length can be calculated from Equation 13.8:

$$T_m = \frac{V \times Y}{3.6 \times S} \qquad (13.8)$$

where:

T_m = merge length (m)

V = mean free speed (km/h)

Y = width of lateral movement (m)

S = rate of lateral movement

Acceleration Lane Merge - 1.0m/s

Through Lane Merge - 0.6 m/s

For most practical purposes

- acceleration lane merges $T_m = V$
- through lane merges $T_m = 1.6V$

The merge length should be included in the overall acceleration length for an auxiliary lane.

For safety reasons merges should never be applied:

- over a crest with sight distance below ASD; or
- to the inside of a horizontal curve where the radius of the offside lane line is less than
  - o 185m for 60km/h design speed
  - o 330m for 80km/h design speed
  - o 515m for 100km/h design speed
  - o 620m for 110km/h design speed (based on the minimum of 3.5m lane and 2 sec gap; 3m lanes require a 20% larger radius).

Approach Sight Distance (ASD) should be available at all points along the merge length to allow drivers to observe the linemarking with sufficient time to react.

Merge transitions around horizontal curves should be developed as for a straight alignment and transferred to the curved alignment by the distance and offset method. This is shown in Figure 13.46.

A circular curve should not be used for the merge taper, as the rate of lateral movement and reduction in width will not be uniform. Concentric circular arcs cannot be joined tangentially by a third circular arc. The curve within the merge length in Figure 13.46 is obtained by linear interpolation, not a circular arc.

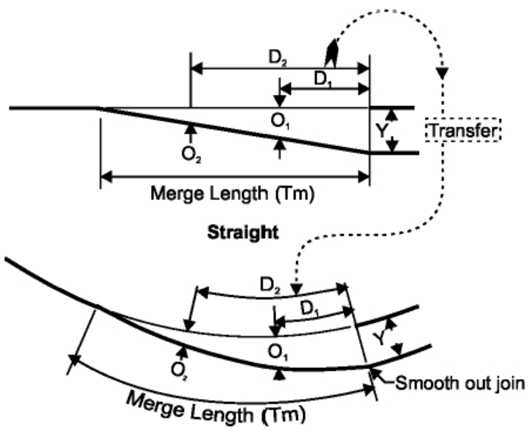


Figure 13.46 Procedure to Determine Offsets on a Curved Alignment Using Data from a Straight Road

Run-out Area

It is important to provide drivers with a run-out area at the end of the merge. The run-out area will accommodate those vehicles prevented from merging as they approach the narrowed section. Details for run-out areas are given in Chapter 15.

13.7.4.3 Acceleration Lane Design for Trucks

The speed of heavy vehicles needs to be considered when designing acceleration lanes. For the design of new acceleration lanes in greenfield sites, it is preferable that the design heavy vehicle has sufficient length to accelerate to a speed of 20km/h below the mean free speed of the through road (or a higher speed), particularly if the acceleration lane is on a dedicated heavy vehicle route.

If the speed of trucks nearing the end of an acceleration lane is too low, it can be very difficult for drivers on the through road to determine whether to brake and follow a merging truck or accelerate and move ahead of the truck. For this reason, the speed at which heavy vehicles will merge should be determined when designing acceleration lanes.

If the speed of heavy vehicles at the merge is much lower than the speed of the through traffic (say 30 – 40km/h difference or more), consideration should be given to extending the length of the acceleration lane. If this cannot be achieved, consideration should be given to installing a left-turn comprising a give-way or stop situation (ie a BAL or high entry angle CHR). Although the latter situation results in slow moving heavy vehicles on the through road, it is generally easy for through drivers to perceive these vehicles and slow for them.

Figure 13.47 plots the speed of a design semi-trailer on various lengths of graded roadway. It should be noted that these speed profiles are for a semi-trailer starting from rest on constant grades.

Table 13.15 provides acceleration lane lengths that would be required for semi-trailers to accelerate from rest to a specified decrement below the through lane speed. It should be noted that the table provides values only for flat conditions and downgrades. It is seldom practical to provide an acceleration lane of sufficient length on upgrades to enable trucks to accelerate to the design speed for through lanes.

The computer software package VEHSIM can be also used to determine truck speed at the end of an acceleration lane. Inputs required include the start speed, the vertical alignment and the heavy vehicle type. This program is particularly useful where the design heavy vehicle is other than a semi-trailer and/or the vertical alignment does not comprise a single grade.

13.7.4.4 Auxiliary Through Lanes

Auxiliary through lanes are often required at urban intersections to provide greater capacity, particularly if traffic signals are installed. These lanes are similar in appearance to auxiliary turn lanes but should be designed using the following principles.

Length of Auxiliary Through lanes

The length of auxiliary through lane required on the approach to an intersection depends on the amount of storage required for through vehicles waiting to enter the intersection.

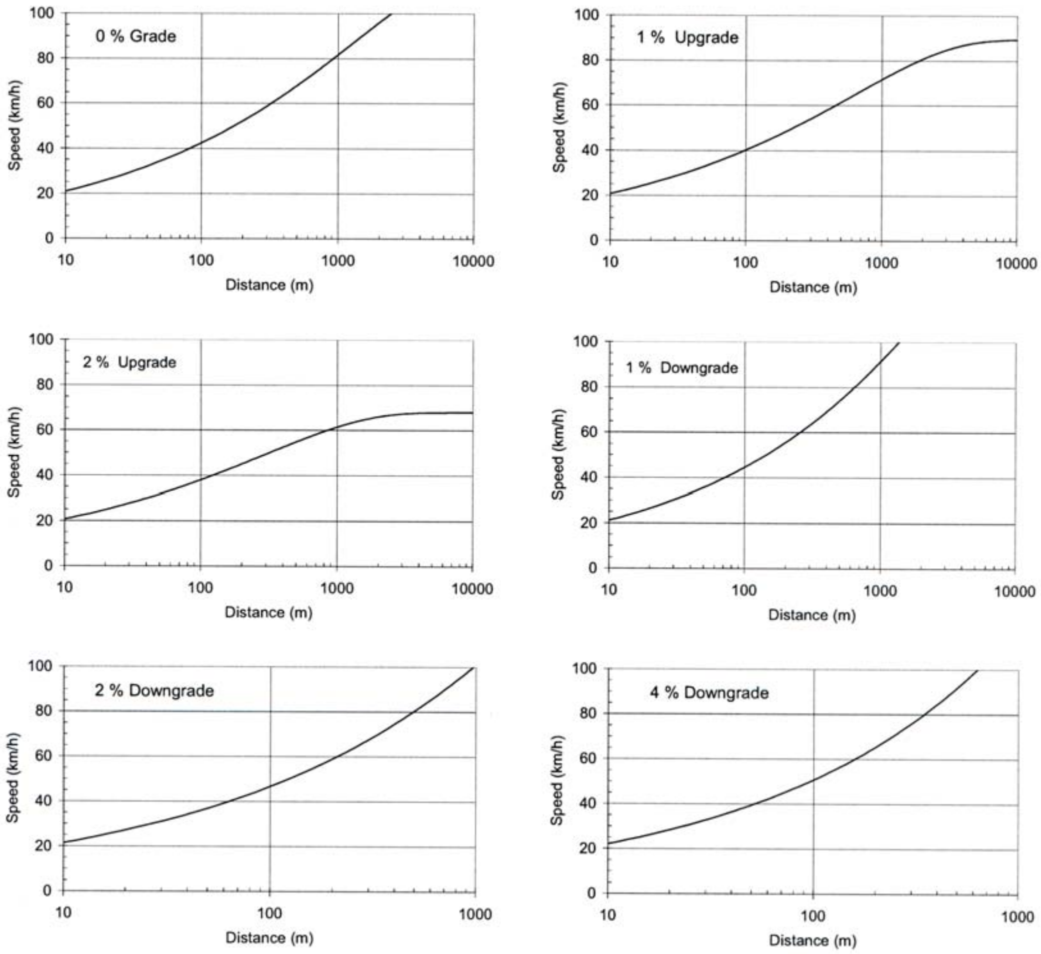


Figure 13.47 Profiles for a Semi-trailer Starting from Rest on Constant Grades

Table 13.15 Acceleration Lane Lengths (m) for Semi-trailers to Accelerate from Rest to a Specified Decrement below the Through Lane Speed (Source - McLean et al, 2002)

Through Road Speed (km/h) #	100 km/h			80 km/h		
Downgrade (%)	0 km/h	10 km/h	20 km/h	0 km/h	10 km/h	20 km/h
0	2,400	1,500	910	910	550	320
1	1,400	940	640	640	410	250
2	970	700	500	500	330	210
3	760	560	400	400	280	180

\# For the purpose of calculating acceleration lane lengths at intersections, the through road speed is usually made equal to the mean free speed (which is often approximately equal to the speed limit)

Additional through lanes should be dropped clear of intersections, desirably 100m minimum beyond an intersection. This distance includes the taper.

Auxiliary Through Lane Tapers

Tapers required at the start of auxiliary through lanes on intersections approaches, and at the end of such lanes beyond intersections, are more generous than those used on auxiliary turn lanes. This provides for more comfortable rates of lateral movement for drivers using the additional lane.

Where possible tapers should not be located on curves. Where this is not possible, care needs to be taken in the design of the taper to ensure that the effectiveness of the merge or diverge is not prejudiced by the road curvature

Diverging Tapers

Tapers for diverging movements at the commencement of additional through lanes should provide for a rate of lateral movement of 1.0m per second. Refer to Section 13.7.4.1 for the method of calculating the required length of taper.

Merging Tapers

Tapers for merging movements where additional through lanes are dropped should provide for a rate of lateral movement of 0.6m per second.

Care must be exercised with the location design of all merging tapers to ensure that there is sufficient sight distance for the approaching driver to realise the existence and geometry of the merge and have adequate time for relative speed adjustment and gap selection for merging. Refer to Chapter 15 for the method of calculating the required length of taper.

13.7.5 General Requirements at Intersections

At intersections, the following design considerations are mandatory:

- As discussed in Section 13.6.4, ASD, MGSD and SISD are to be checked for the 85th percentile speed for each leg.
- As with any design, road furniture and landscaping should be located and designed as to not interfere with the sight distance requirements.
- All non-frangible road furniture and drainage structures are to be located outside the clear zone, else be suitably protected, as per the requirements of Chapters 7 and 8 of the RPDM.
- Turning speeds at all intersections is 5-15km/h, however can be reduced to 0-5km/h when the vehicle is required to stop as determined by site conditions.
- Unless specifically shown otherwise, lane and shoulder widths on each leg shall be in accordance with Chapter 7 and where curve widening is required, it shall be in accordance with Chapter 11.
- Line marking for all intersections should be as per the requirements of the Manual of Uniform Traffic Devices and the Guide to Pavement Line Marking.
- Lighting in accordance with Chapter 17 shall be provided at intersections where raised medians/islands are provided. In addition, lighting should be provided where there is a doubt of drivers being able to determine the required path to travel through the intersection.

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13.7.6 Right Turn Treatments - General

Right turn treatments are provided for safety, delay, and capacity reasons. The treatment will vary according to requirements at each site. Treatments can vary from “do nothing” to major channelisation with traffic signal control. Types of right turn treatments have been discussed in Section 13.4.2.

Arndt (2004) has shown that there are major safety advantages of providing Channelised Right Turn Treatments (CHR) over other forms of right turn treatments. These right turn treatments can be defined by raised or painted medians and pavement arrows.

Demand, together with available pavement width, will determine the treatment required for right turn movement by bicycles. Refer to QDMR (2004) and Austroads (1999b).

For safety reasons, it is desirable that exclusive right turn lanes be protected by a median to guide through vehicles clear of stationary right turn traffic. Where a through lane must be converted into an exclusive right turn lane or left turn lane, clear signposting, alerting drivers of the layout, must be erected well in advance so that sufficient time is available to decide to change lanes.

The design of the turning path must be carefully considered and the effective radius and crossfall assessed. Appropriate design parameters are discussed in Chapter 7 and Section 13.7.4.1.

In providing right turn capacity, take care not to create a squeeze point for cyclists continuing along the roadway. Maintain the road shoulder or continuous wide kerbside lane on the left.

Cyclists’ right turn requirements also need to be considered. This is due to potential for conflict with continuing through traffic, traffic turning with the cyclist or traffic turning right from the opposite direction. These potential conflicts and suggested remedies are presented in Austroads (1999b) - Sections 5.4.2.5 and 5.4.2.6.

13.7.7 Intersections with Local Streets

Vehicles turning right into local side streets may create safety and/or capacity problems for through traffic.

At all sites where two-lane arterials/sub-arterials meet with local roads, a BAR turn treatment or better, (refer to Figure 13.48 in Section 13.7.9.1) should be provided subject to the warrants given in Section 13.4.4. For a divided road, where a right turn is to be allowed, a minimum of a short right turn bay (CHR(S) – refer to Figure 13.49 in Section 13.7.9.2) should be provided if the carriageway, or median, is wide enough. This is also subject to the warrants given in Section 13.4.4, which may show that a CHR turn treatment is required instead.

Where sight distance is poor, other options would be to ban the right turn, or to erect advance signs warning of turning traffic. Warning signs can also be used with the BAR turn treatment, or turn bays, where safety problems occur.

Circumstances may require the construction of a through lane (or path) for cyclists at T-junctions. This treatment is discussed in Austroads (1999b) - Section 5.4.2.7.

Layouts must be designed using the design vehicle and check vehicle as discussed in Section 13.3.1.1. The design vehicle for all legs should not be smaller than a single unit

(SU) vehicle, 12.5m long. The minimum turning radius is to be 15m.

13.7.8 Collector / Arterial Road Intersections

At these intersections, right turn traffic is often significant on one, or more, movements.

Turn bay geometry needs to be designed for the volume and composition of right turn traffic. This can result in providing more than one lane for right turn vehicles.

The number of lanes, and length of storage can be determined by modelling the intersection using an appropriate computer program (e.g. aaSIDRA).

Layouts must be designed using the design vehicle and check vehicle as discussed in Section 13.3.1.1. The design vehicle should not be smaller than an articulated vehicle 19.0m long as the minimum requirement for single lane turning movements (or other larger vehicle if such vehicles regularly use the road). Combined articulated vehicle/car turning paths should be used for multiple lane turns. Minimum turning radius is to be 15m.

13.7.9 Right Turn Treatments - Urban Conditions

13.7.9.1 Basic Right Turn Treatment (BAR) on a Two-Lane Urban Road

The BAR turn treatment described in this section is applicable at intersections of two-lane urban roads and minor local roads where traffic volumes do not warrant a higher order treatment.

The BAR turn treatment should provide sufficient pavement width for the design through vehicle to pass a vehicle waiting to turn right. The absolute minimum pavement width on a horizontal straight should be 6.0m between the centreline and the edge of pavement or kerb line whilst 6.5m is the preferred minimum. The latter is adequate for heavy vehicles (excluding road trains) to pass right turning vehicles (refer to Figure 13.48).

A turning radius of 10m to 15m should be used and the design turning vehicle's swept path determining the length of approach and departure widening for the site geometrics, i.e. angle of intersection, width of carriageways, etc.

No lane lines, or right turn arrows should be marked on the pavement for a BAR turn treatment.

13.7.9.2 Channelised Right Turn Treatment with a Short Turn Slot [CHR(S)] on a Two-Lane Urban Road

The BAR turn treatment on a two-lane urban road shown in Figure 13.48 has limited applications. It is mainly applicable at the junction of local roads with collector or arterial roads.

A more desirable treatment at such sites is a CHR(S) turn treatment as shown in Figure 13.49.

Although some deceleration of the right turning vehicles occurs in the through lane, this treatment records far fewer Rear-End-Major vehicle accidents (generally rear-end type accidents resulting from a through driver colliding with a driver turning right from the major road – refer to Appendix F for more details) than do BAR turn treatments. The good safety performance occurs by removing potentially stationary turning vehicles from the through traffic stream. This treatment is suitable where there are low to moderate through and turning volumes. For higher volume sites, a full length CHR turn treatment is preferred.

CHR(S) turn treatments can only be used with line marking. It is not intended to be used with raised or depressed islands. Right turning drivers often travel onto the painted chevron to exit the through traffic stream as soon as possible. This is a desirable feature, as it reduces the likelihood of Rear-End-Major vehicle accidents.

For the CHR(S) turn treatment, all through traffic is required to deviate, hence the deviation must be designed to suit the operating speed. Because of this deviation, parking limits may need to be applied as shown in Figure 13.49.

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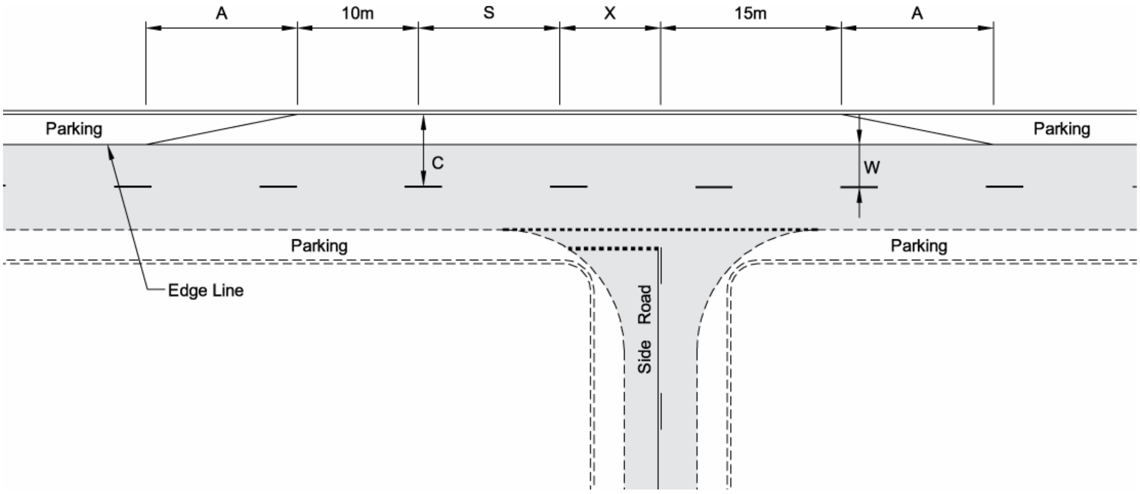


W= Nominal through lane width (m) (including widening for curves). Width to be continuous through the intersection.

C = On straights - 6.0m minimum
- 6.5m minimum for 19m semi-trailers and B-doubles
- 7.0m minimum for Type 1 & Type 2 Road Trains

On curves - widths as above + curve widening (based on widening for the design turning vehicle plus widening for the design through vehicle)

A = $\frac{0.5 \times V \times (C-W)}{3.6}$

Increase length A on tighter curves. Where the design through vehicle is larger than or equal to a 19m semi-trailer, the minimum speed used to calculate A is 80km/h.

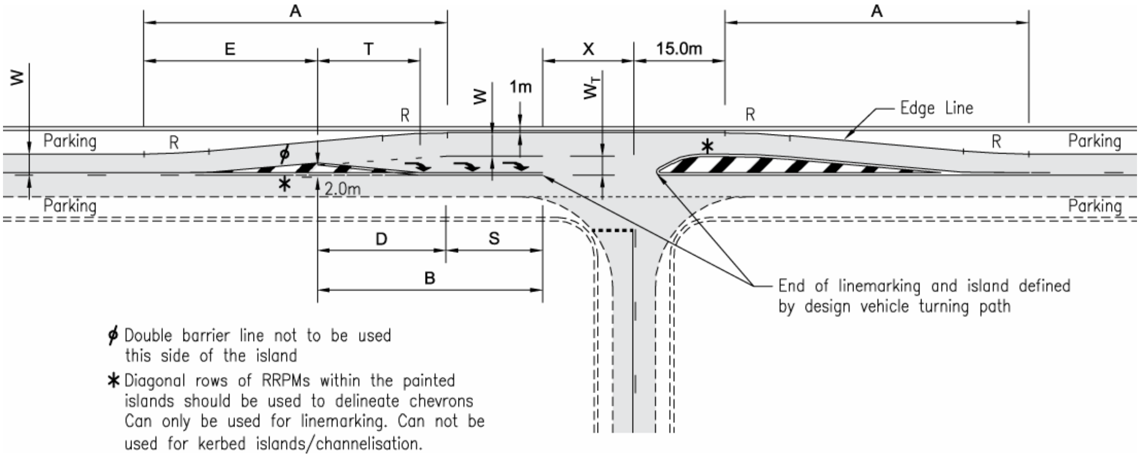
V = Design speed of major road approach (km/h)

S = Storage length to cater for one design turning vehicle (m) (minimum length 12.5m)

X = Distance based on design vehicle turning path, typically 10 - 15m

Note: This diagram does not show any specific bicycle facilities. Where specific bicycle facilities are required (eg exclusive bicycle lanes), refer Austroads (1999b).

Figure 13.48 Basic Right Turn Treatment (BAR) for a Two-Lane Urban Road



Design Speed of Major Road Approach (km/h)	Lateral Movement Length A (m) (1)	Diverge/ Deceleration length D (m) (2)	Desirable Radius R (m)	Taper Length T (m) (3)
50	40 (4)	15	110	15
60	50 (4)	25	175	15
70	60	35	240	20
80	65	45	280	20
90	75	55	350	25

(1) Based on a diverge rate of 1m/sec and a turn lane width of 3.0m Increase lateral movement length if turn lane width >3m. If the through road is on a tight curve, increase lateral movement length so that a minimal decrease in speed is required for the through movement.
(2) Based on a 20% reduction in through road speed at the start of the taper to a stopped condition using a value of deceleration of $3.5 m/s^2$. Adjust for grade using the 'Correction to Grade' table in Figure 13.42.
(3) Based on a turn lane width of 3.0m
(4) Where Type 2 road trains are required, minimum A = 60m

W = Nominal through lane width (m) (incl. widening for curves). For a new intersection on an existing road, the width is to be in accordance with the current link strategy.

W_T = Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle) = 3.0m minimum.

B = Total length of auxiliary lane including taper, diverge/deceleration and storage (m)

E = Distance from start of taper to 2.0m width (m) = $(A/W_T) \times 2$

S = Storage length to cater for design turning vehicle (m)

T = Taper length (m) = $\frac{0.33 \times V \times W_T}{3.6}$

V = Design speed of major road approach (km/h)

X = Distance based on design vehicle turning path, typically 10 – 15 m

Note: This diagram does not show any specific bicycle facilities. Where specific bicycle facilities are required (eg exclusive bicycle lanes), refer Austroads (1999b).

Figure 13.49 Channelised Right Turn Treatment with a Short Turn Slot [CHR(S)] on a Two-Lane Urban Road

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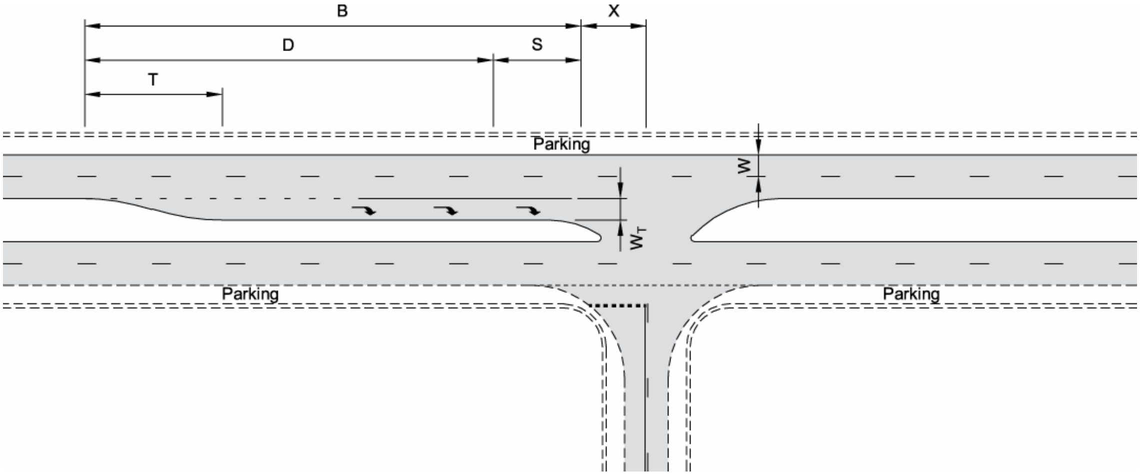
13.7.9.3 Channelised Right Turn (CHR) on an Urban Road

The most desirable treatment for right turns is a CHR turn treatment, as given in Figure 13.50.



W = Nominal through lane width (m) (incl. widening for curves).

W_T = Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle). Desirable minimum = W, absolute minimum = 3m.

B = Total length of auxiliary lane including taper, diverge/deceleration and storage (m)

D = Diverge/deceleration length including taper - refer to Table 13.11 (adjust for grade using the 'Correction to Grade' table in Figure 13.42).

S = Storage length (m), greater of: 1. The length of one design turning vehicle
2. (calculated car spaces –1) x 8m (refer to Appendix 13A or use computer program eg. aaSIDRA)

T = Taper length (m) = $\frac{0.33 \times V \times W_T}{3.6}$

V = Design speed of major road approach (km/h)

X = Distance based on design vehicle turning path

Note 1: This diagram does not show any specific bicycle facilities. Where specific bicycle facilities are required (eg exclusive bicycle lanes), refer Austroads (1999b).
Note 2: A raised concrete median in the minor road may be used with this treatment.

Figure 13.50 Channelised Right Turn Treatment (CHR) on an Urban Road

13.7.9.4 Seagull Right Turn Treatment on an Urban Road

Seagull turn treatments provide channelised protection for right turn traffic when departing or entering a through traffic stream at a T-junction. Treatment for urban conditions is shown in Figure 13.51.

The preferred practice is for right turn vehicles to enter their own lane in the through road. The advantages of this geometry include:

- it avoids the right to left merge problems associated with a shared lane arrangement;
- where a downstream intersection occurs, the conflicting merging movement (i.e. through traffic trying to merge left to right) is removed;
- storage of the merging vehicle is not required; and
- Truck merging is not compromised by vehicle/s in the blind spot (see Section 13.3.1.3).

Although not preferred, an alternative seagull layout with a right hand side merge and a full acceleration lane may also be used. Such a layout is shown for a rural road in Figure 13.64.

Although not desirable, the seagull layout can be used as a two stage right turn movement where turning volumes are light and there are insignificant articulated vehicle turning volumes. This treatment is shown in Figure 13.52. The exit allows for storage followed by a low speed right to left merge when a gap is available in the through traffic.

The layout in Figure 13.52 could be applicable where residential zone cul-de-sacs intersect with major roads. If turning volumes are high, there will be the resultant queuing in the exit creating an inefficient facility and the treatment should not be used.

This treatment is not preferred because of the awkward sighting angle that is produced. An offset right-turn lane (as shown for a rural road in Figure 13.66) is a preferred treatment in this instance, provided that sufficient median width is available.

With seagull layouts, careful attention is to be given in the event of a blockage. An absolute minimum width between kerbs of 4.5m is required with a desirable minimum width of 5.5m. However, such widths between kerbs may encourage drivers to form two lanes. Provision of edge lines may be necessary to minimise the risk of this happening.

Barrier kerbs should only be used where it is essential for other purposes; painted medians and islands should not be used. If a painted median is used for the seagull island, it should be further delineated using traffic guidance flaps.

Provision should be made for the installation of traffic signals in the layout, even if not immediately required.

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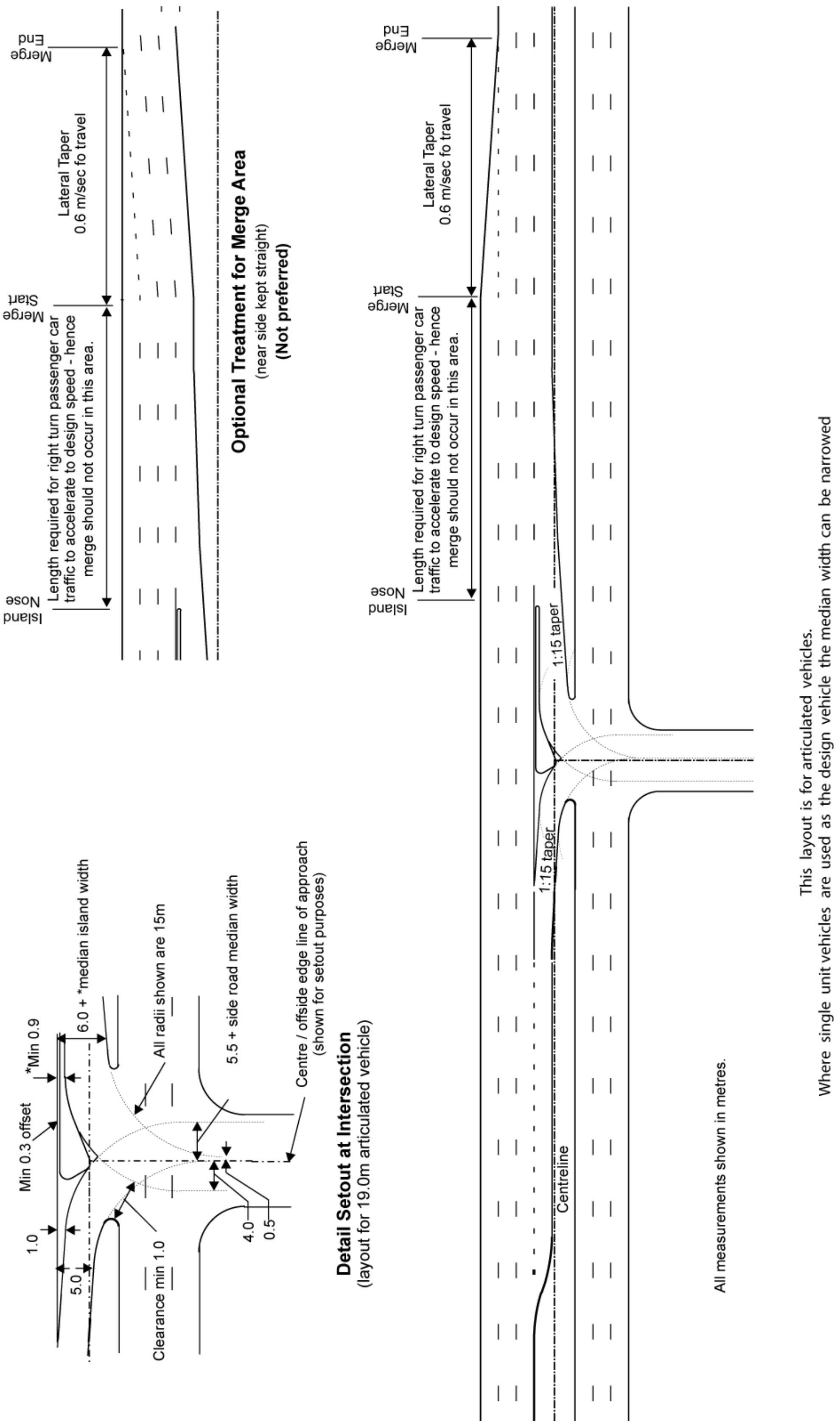
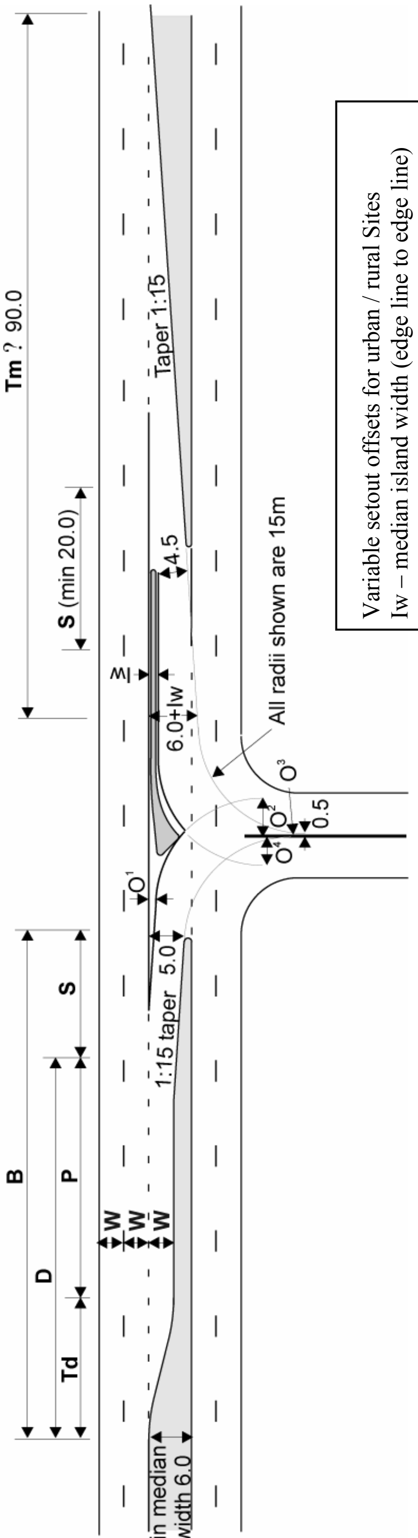


Note 1: This diagram does not show any specific bicycle facilities. Where specific bicycle facilities are required (eg exclusive bicycle lanes), refer Austroads (1999b).
Note 2: Although not preferred, a right hand side merge with a full length acceleration lane may also used. Figure 13.64 shows such an arrangement for a rural road.

Figure 13.51 Seagull Layout on an Urban Road (Road Lit)



W = Nominal through lane width (m) (including widening for curves)

B = Total length of auxiliary lane including taper, diverge/deceleration and storage (m)

D = Diverge/deceleration length including taper - refer to Table 13.11 (adjust for grade using the 'Correction to Grade' table in Figure 13.42.

P = Length of parallel lane (m).

S = Storage length (m), greater of 1. The length of one design turning vehicle

2. (calculated car spaces –1) x 8m (refer to Appendix 13A or use computer program eg. aaSIDRA)

Td = Diverge taper length (m) = $\frac{0.33 \times V \times W}{3.6}$

V = Design speed of major road approach (km/h)

Tm = Merge taper length (including storage – does not allow for acceleration)

Variable setout offsets for urban / rural Sites
Iw – median island width (edge line to edge line)
Urban – min 0.9
O^1 – Urban 1.0
O^2 – Urban 5.5
O^3 – Urban 1.0
O^4 – Urban 4.0
The offsets above suit turning paths of 19.0m articulated vehicle

Note 1: This diagram does not show any specific bicycle facilities. Where specific bicycle facilities are required (eg exclusive bicycle lanes), refer Austroads (1999b).
Note 2: The layout should **only be used where right turn movement from the side road is light** and there is insignificant articulated vehicle movement. It should not be used where the road alignment does not allow Minimum Gap Sight Distance of 5 seconds from the seagull exit storage bay (e.g. inside of horizontal curves).
Note 3: This treatment is not preferred because of the awkward sighting angle that is produced. An offset right-turn lane (as shown for a rural road in Figure 13.66) is a preferred treatment in this instance, provided that sufficient median width is available.

Figure 13.52 Seagull Layout on an Urban Road Utilised as Two Stage Right Turning Movement

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13.7.9.5 Offset Right Turn Lanes on an Urban Road

At four-way unsignalised or signalised intersections, it can be difficult for opposing right turn drivers on the major road to view oncoming through vehicles. This often occurs because the opposite right turning vehicle blocks visibility to the oncoming through vehicles. The problem is compounded for a right turning driver viewing from the inside of a horizontal curve. A solution to improve the visibility for the right turning drivers is to provide offset right turn lanes. A conceptual diagram of this treatment is shown in Figure 13.53.

As discussed in Section 13.4.3.5, it is undesirable to build new 4-way unsignalised intersections. The offset right turn lanes treatment may be applicable to retrofitting at existing 4-way intersections or for installations of traffic signals at new 4-way intersections. The treatment may also be suitable for application in some rural situations.

13.7.9.6 Median Turning Lanes or Two Way Right Turn Lanes (TWRTL) on an Urban Road

General

Median Turning Lanes or Two Way Right Turn Lanes (TWRTL) can be used to maintain capacity and level of service for the through lanes by removing the obstruction caused by a right turning vehicle. It has the added advantage of providing shelter for vehicles both entering and exiting from an access. A diagram of such a treatment is shown in Figure 13.54.

This treatment is particularly applicable in commercial and residential areas with closely spaced access points. It has been used successfully where arterial roads bisect country town business and industrial areas and access is required for motels, service centres commercial establishments and adjoining low traffic volume side streets.

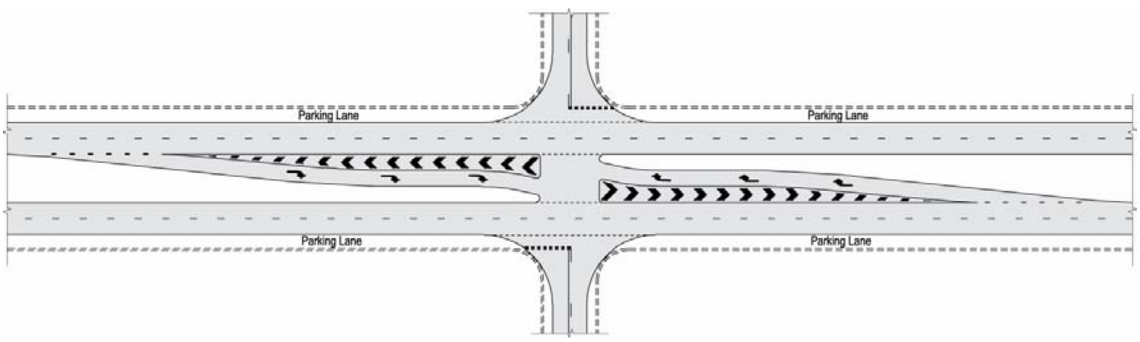
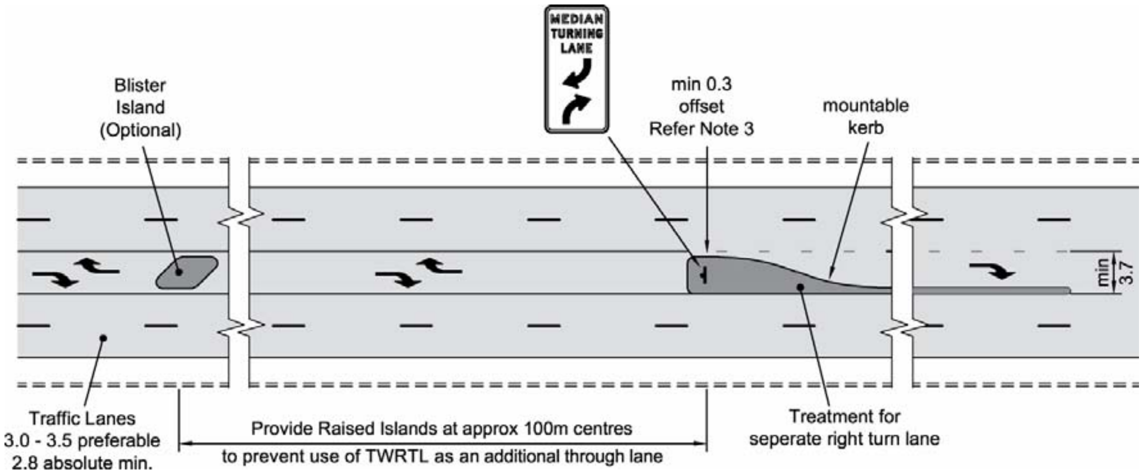


Figure 13.53 Conceptual Layout of Offset Right Turn Lanes on an Urban Road



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Notes:

1. This diagram does not show any specific bicycle facilities. Where specific bicycle facilities are required (eg exclusive bicycle lanes), refer Austroads (1999b).
2. See “Guide to Pavement Markings” for linemarking, spacings of pavement arrows, advance warning and regulatory signs.
3. Minimum offset is as per Figure 13.39.
4. Two lane carriageway shown, but can be used for single lane carriageway.

Figure 13.54 Two Way Right Turn Lanes on an Urban Road

TWRTLs should not be introduced without consideration of existing and future land use. They can add to arterial congestion by allowing unlimited and uncontrolled right turn movements. However, when used on roads with traffic signal control, TWRTLs may well provide sufficient gaps to adequately service low volume side properties with efficiency and safety. In non-access controlled areas they can encourage piecemeal land development with inappropriate accesses provided at developments.

On new heavily travelled arterial roads and commercial and industrial areas with widely spaced access points, median control of right turn movements is preferred.

TWRTLs should be restricted to the urban environment with travel speeds of 70 km/h or less. They should not be used in high density residential areas due to the potential conflict with uncontrolled pedestrian movements.

A TWRTL must not be used in conjunction with an intersection. The ends of the TWRTL treatment must not be closer than 10m from the start of any right turn lane at an intersection.

The through road should have no more than two lanes in each direction; resulting in a total of 5 lanes with the introduction of a TWRTL. Further research is being conducted on the operational characteristics of TWRTLs in a three lane carriageway situation.

Geometric Considerations

The TWRTL is to be paved flush with the adjacent lanes. To improve the definition of the lane a different coloured pavement material other than red (Bus Only lanes) or green (Cycle lanes) can be used.

The desirable width is 3.0m to 4.8m. An unbroken line is to be used on both sides of the TWRTL to prohibit it's use as an overtaking lane. TWRTLs and right turn auxiliary lanes within the same length of median must be separated by a raised island and adequately sign posted.

13.7.9.7 Pavement Arrows/Signs

Right turns can be controlled by pavement arrows and signs, with or without the use of a right turn bay. For shared lanes, right turn arrows may be used in conjunction with through, or left turn arrows.

If it becomes necessary for a through lane to be turned into an exclusive right turn lane, pavement arrows should be supplemented with regulatory signs. Where possible, an exclusive right turn lane should be protected by a raised or painted median.

Generally, pavement arrows are not used where the situation is covered by regulations under the Traffic Act, but they may be installed where drivers are not observing correct lane usage.

13.7.9.8 Turn Lines

Turn lines are generally provided between adjacent turns on multiple right turn facilities. However the offside of the inner lane may be marked (refer to Figure 13.55) where there are:

- observed lane discipline problems;
- the path to exit is unclear; or
- collisions occur between vehicles either travelling in the same or opposing direction.

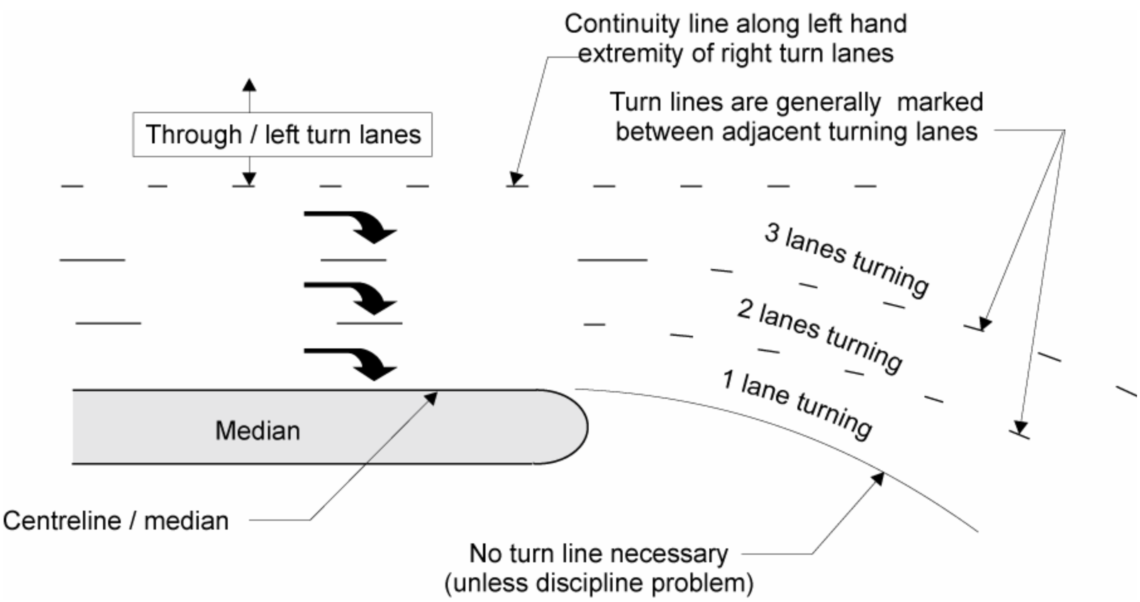


Figure 13.55 Details of Turn Line Treatments for Multiple Lanes

13.7.9.9 Opposed Right Turns

Where opposed right turns operate simultaneously, the turns should be designed to provide sufficient clearance between the nearside of opposing vehicles as follows:

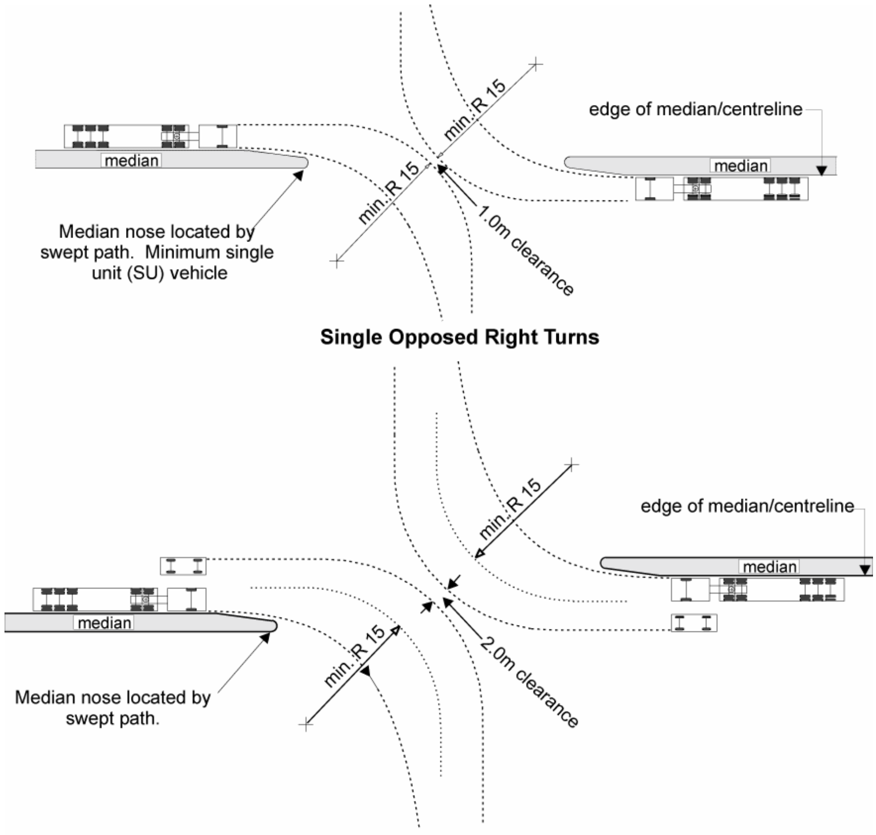
- Single turns 1.0m
- 1 Single, 1 Double 2.0m
- Double turns 2.0m

The following turning path templates should be used to design intersection geometry for opposed turns:

(a) Single turn - single unit (SU) vehicle, minimum radius of 15.0m; and

(b) Double turn - articulated vehicle/car abreast, minimum radius of 15.0m (See Figure 13.56).

Where lane width is insufficient to permit construction of a right turn lane for high volume bicycle traffic, provision of a hook turn facility for cyclists may be appropriate. This treatment is presented in Austroads (1999b) - Section 5.4.2.4.



Note: In constrained situations, an absolute minimum radius of 12.5m may be adopted for the right turn.

Figure 13.56 Opposed Right Turns

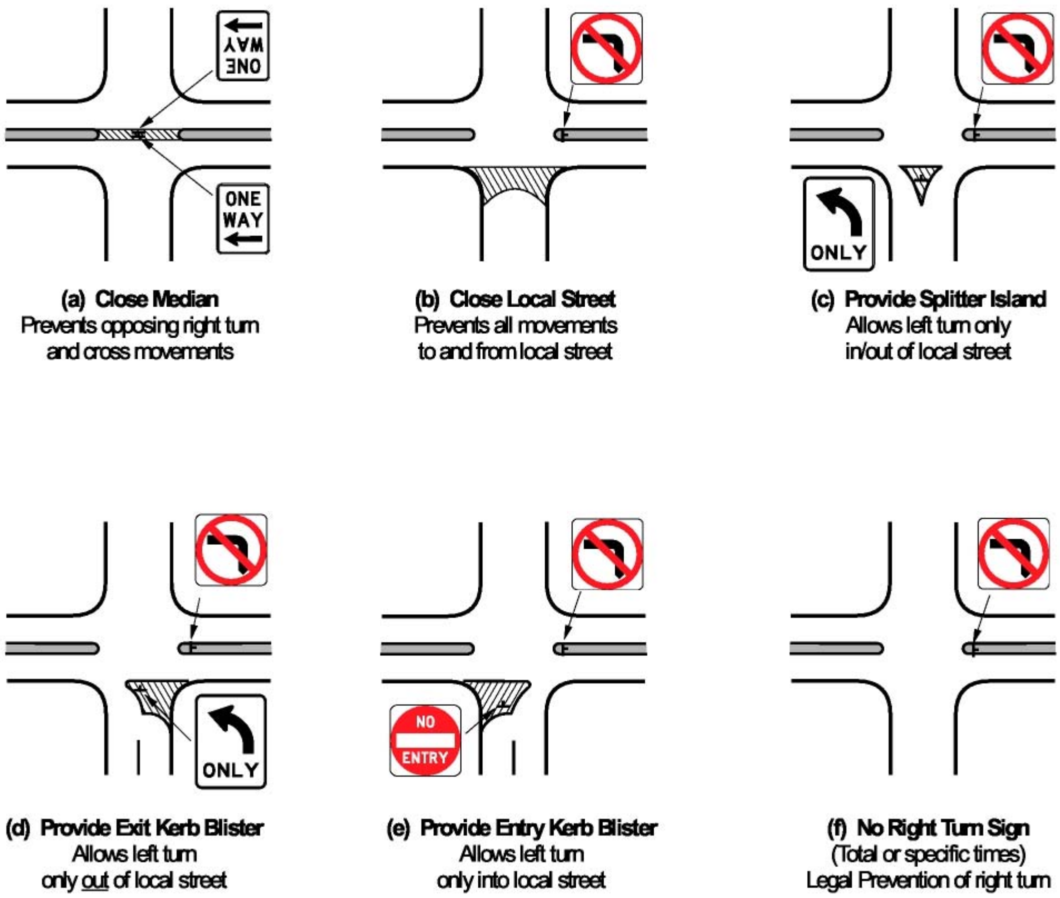
13.7.9.10 Right Turn Bans

Where a right turn bay cannot be provided, and safety and/or capacity problems exist, consideration should be given to banning the turn.

This action is essential where a filter turn opposes a trailing right turn phase at traffic signals.

Before any right turn is banned, convenient alternative access should be available or provided.

If the right turn can be banned, several options may be considered as shown in Figure 13.57.



Notes:
1. Treatments (b) to (e) can be used if there is no median i.e. centreline marking only.
2. For (f) without median, NO RIGHT TURN signs should be placed in footways.
3. When adopting a partial closure, site specific geometry (e.g. skew intersection) should be examined to ensure that unintended movements cannot be executed.

Figure 13.57 Right Turn Ban Treatments

13.7.10 Right Turn Treatments - Rural Conditions

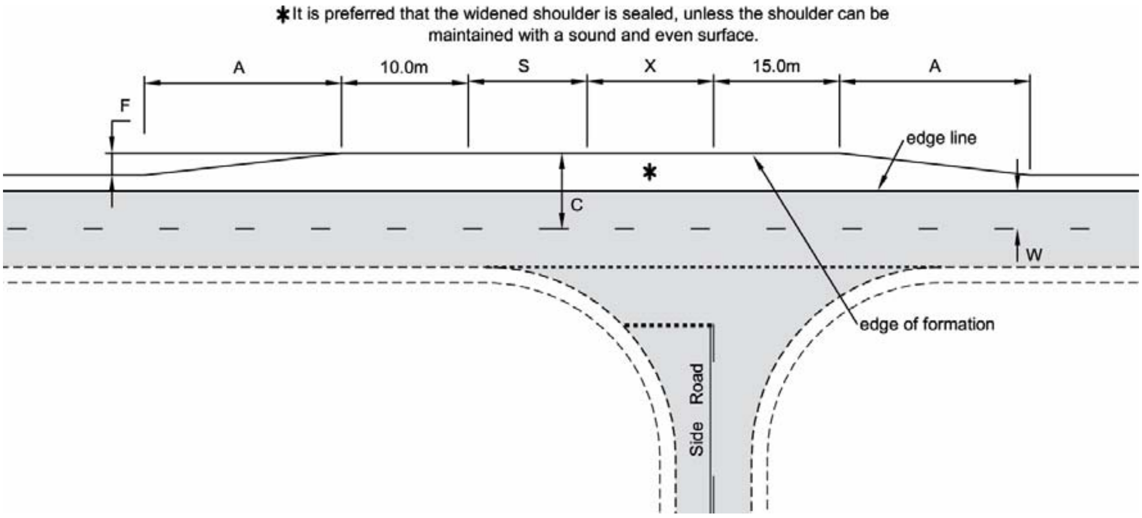
There are two fundamental types of right turn treatments for rural areas given in this section:

- Type BAR shoulder widening (minimum treatment);
- Type CHR pavement widening with a right turn bay. A subset of this type is the CHR(S).

Generally these treatments are applied to two-lane two-way rural roads.

13.7.10.1 Basic Right Turn Treatment (BAR) on a Two-Lane Rural Road

This is the minimum treatment for right turn movements from a through road to side roads and local access points. It is detailed in Figure 13.58.



W= Nominal through lane width (m) (including widening for curves). Width to be continuous through the intersection.

C = On straights - 6.5m minimum
- 7.0m minimum for Type 1 & Type 2 Road Trains
On curves - widths as above + curve widening (based on widening for the design turning vehicle plus widening for the design through vehicle)

A = $\frac{0.5 \times V \times F}{3.6}$
Increase length A on tighter curves. Where the design through vehicle is larger than or equal to a 19m semi-trailer, the minimum speed used to calculate A is 80km/h.

V = Design speed of major road approach (km/h)

F = Formation/carriageway widening (m)

S = Storage length to cater for one design turning vehicle (m) (minimum length 12.5rn)

X = Distance based on design vehicle turning path, typically 10 - 15m

Figure 13.58 Basic Right Turn Treatment (BAR) on a Two Lane Rural Road

This treatment provides sufficient trafficable width for the design through vehicle to pass on the left of a stationary turning vehicle. This is achieved by widening the shoulder to provide a minimum width sufficient to allow the vehicles to pass. Substantial speed reduction (potentially half of the design speed) is a feature of this layout.

On a terminating intersection leg no special provision is made for right hand turns.

This layout can be used on both sealed and unsealed roads.

It is preferred that the widened shoulder at BAR turn treatments is sealed, unless the shoulder can be maintained with a sound and even surface.

For locations where the safety record of a BAR turn treatment shows a high accident rate of rear-end collisions, a CHR(S) or CHR treatment is likely to be much more appropriate.

Where the pavement is sealed, line marking should be in accordance with the "Guide to Pavement Markings". Signposting should be in accordance with the "Manual of Uniform Traffic Control Devices" (MUTCD).

Where adequate through sight distance exists, BAR turn treatments will generally be marked with a broken centreline to allow overtaking on the major road through the intersection. This will not restrict overtaking opportunities, thereby minimising delays.

However, there may be instances where a BAR turn treatment on a section of road with good overtaking opportunities will yield a high likelihood of crashes resulting from inappropriate overtaking through the intersection (Overtaking-Intersection vehicle accident type – refer Appendix F). In such cases, a barrier line should be used. Examples of such instances include the following:

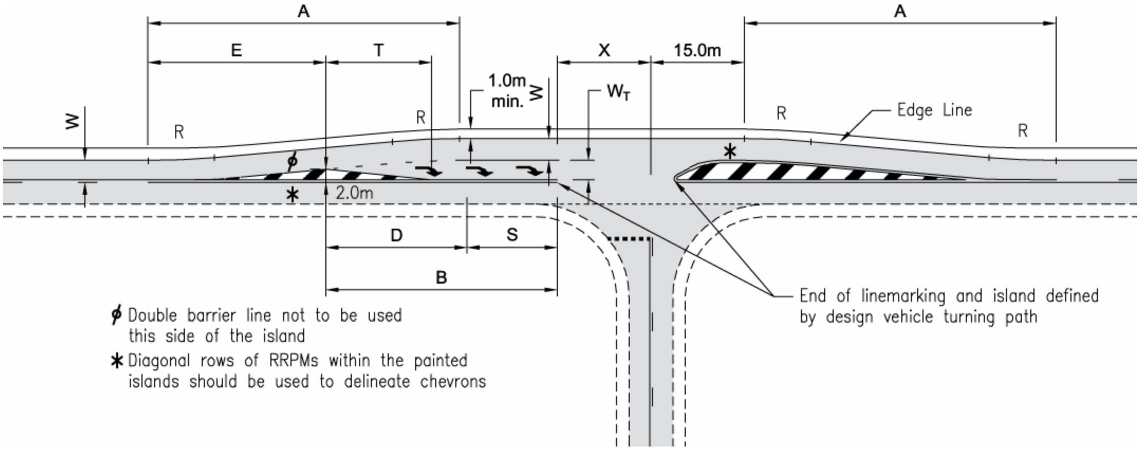
- The turn treatment is located after a significant length of roadway without overtaking opportunities. This geometry would result in drivers often overtaking through the intersection because of the large amount of time spent following other vehicles prior to the intersection. The increased exposure of overtaking may result in an excessively high Overtaking-Intersection vehicle accident rate.
- There are reasonably high right-turning volumes.
- The warrants dictate that a higher-level turn treatment is appropriate.

The issue of whether or not to use barrier lines is a trade off between safety and delay.

13.7.10.2 Channelised Right Turn Treatment with a Short Turn Slot [CHR(S)] on a Two Lane Rural Road

The BAR turn treatment on a two lane rural road shown in Figure 13.58 has limited applications. It is mainly applicable at the junction of side roads and rural arterial roads with lower traffic volumes. Such turn treatments can record high Rear-End-Major vehicle accident rates, especially in high speed areas.

A more desirable treatment at such sites is a CHR(S) turn treatment as shown in Figure 13.59. This treatment is suitable where there are low to moderate through and turning volumes. For higher volume sites, a full length CHR turn treatment is preferred.



Design Speed of Major Road Approach (km/h)	Lateral Movement Length A (m) (1)	Diverge/ Deceleration Length D (m) (2)	Desirable Radius R (m)	Taper Length T (m) (3)
50	40 (4)	15	110	15
60	50 (4)	25	175	15
70	60	35	240	20
80	65	45	280	20
90	75	55	350	25
100	85	70	425	30
110	95	85	500	30
120	100	100	600	35

(1) Based on a diverge rate of 1m/sec and a turn lane width of 3.0m Increase lateral movement length if turn lane width >3m. If the through road is on a tight horizontal curve, increase lateral movement length so that a minimal decrease in speed is required for the through movement.
(2) Based on a 20% reduction in through road speed at the start of the taper to a stopped condition using a value of deceleration of $3.5m/s^2$. Adjust for grade using the 'Correction to Grade' table in Figure 13.42.
(3) Based on a turn lane width of 3.0m
(4) Where Type 2 road trains are required, minimum A = 60m

W = Nominal through lane width (m) (incl. widening for curves). For a new intersection on an existing road, the width is to be in accordance with the current link strategy.

W_T = Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle) = 3.0m minimum.

B = Total length of auxiliary lane including taper, diverge/deceleration and storage (m)

E = Distance from start of taper to 2.0m width (m) = $(A/W_T) \times 2$

S = Storage length to cater for design turning vehicle (m)

T = Taper length (m) = $\frac{0.33 \times V \times W_T}{3.6}$

V = Design speed of major road approach (km/h)

X = Distance based on design vehicle turning path, typically 10 – 15 m

Figure 13.59 Channelised Right Turn Treatment with a Short Turn Slot [CHR(S)] on a Two Lane Rural Road

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This type of intersection can only be used with line marking. It is not to be used with raised or depressed islands. Right turning drivers often travel onto the painted chevron to exit the through traffic stream as soon as possible. This is a desirable feature, as it reduces the likelihood of Rear-End-Major vehicle accidents.

For the CHR(S) turn treatment, all through traffic is required to deviate, hence the deviation must be designed to suit the operating speed. A minimum of a 1m shoulder must be used on the through lane deviation.

The start of the right-turn taper occurs at a median width of 2m, in lieu of the full turning lane width as per a standard CHR treatment.

The length of turn slot is based on a right turning vehicle having a speed reduction of 20 percent in the through lane, prior to moving into the turn slot and decelerating. This is based on the assumption that drivers decelerate at a maximum value of $3.5 m/s^2$ from the start of the taper to the start of the storage length.

Although some deceleration of the right turning vehicles occur in the through lane, this treatment records far fewer Rear-End-Major vehicle accidents than do BAR turn treatments. The good safety performance occurs by removing stationary turning vehicles from the through traffic stream.

13.7.10.3 Channelised Right Turn Treatment (CHR) on a Two Lane Rural Road

For this layout, all traffic is required to deviate - hence the through road movement must be designed to suit the operating speed.

Because of this deviation the pavement can be widened to provide a right turn bay as shown in Figure 13.60.

Minimum lengths of deceleration for different design speeds are shown in Table 13.11. Refer to Appendix 13A or use computer programs such as aaSIDRA to determine the storage length (S).

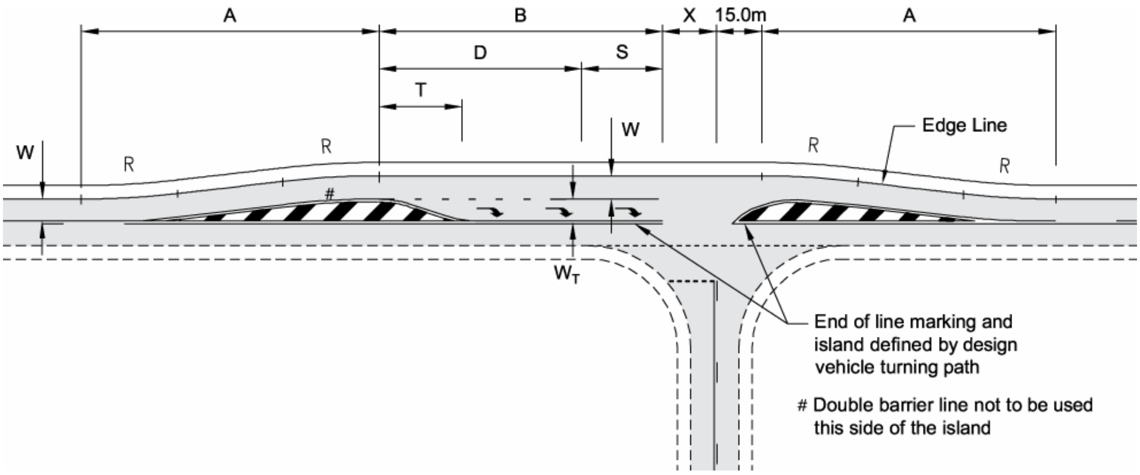
Details of the departure end of the right turn bay should be determined using turning path templates (minimum radius 15.0m). This will depend on the width, and the angle of intersection of the side street.

There are no numerical warrants for the provision of raised medians in lieu of the painted medians shown in Figure 13.60. Refer to Section 13.7.2.1 for a discussion on this issue.

Provision of raised medians is subject to the intersection being lit (refer to Chapter 17 for details).

Linemarking should be as shown in Figure 13.60. If the painted separation between opposing traffic flows is wider than a double white line, then it should be in accordance with painted medians shown in Figure 13.38.

Signposting should be in accordance with the MUTCD.



Design Speed of Major Road Approach (km/h)	Lateral Movement Length A (m) (1)		Desirable Radius R (m)
	W_T=3.5m	W_T=3.0m	
50	50 (2)	40 (2)	110
60	60	50 (2)	175
70	70	60	240
80	80	65	280
90	90	75	350
100	100	85	425
110	110	95	500
120	120	100	600
(1) Based on a diverge rate of 1m (sec). If the through road is on a tight horizontal curve, increase lateral movement length so that a minimal decrease in speed is required for the through movement. (2) Where Type 2 Road Trains are required minimum A = 60.0m			

W = Nominal through lane width (m) (incl. widening for curves). For a new intersection on an existing road, the width is to be in accordance with the current link strategy.

W_T = Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle). Desirable minimum = W, absolute minimum = 3m.

B = Total length of auxiliary lane including taper, diverge/deceleration and storage (m).

D = Diverge/deceleration length including taper - refer to Table 13.11 (adjust for grade using the 'Correction to Grade' table in Figure 13.42).

S = Storage length (m), greater of: 1. The length of one design turning vehicle
2. (calculated car spaces –1) x 8m (refer to Appendix 13A or use computer program eg. aaSIDRA)

T = Taper length (m) = $\frac{0.33 \times V \times W_T}{3.6}$

V = Design speed of major road approach (km/h)

X = Distance based on design vehicle turning path - typically 10 – 15m.

Note: 1. An alternative to the double white line on the offside edge of the right turn slot is a 1.0m painted median. The 1.0m median is particularly useful when the major road is on a tight horizontal curve and oncoming vehicles track across the centreline. Provision of this median will require the dimension 'A' to be increased.
2. A raised concrete median on the minor road may be used with this treatment to minimise 'corner cutting', particularly for higher turning volumes.

Figure 13.60 Channelised Right Turn (CHR) on a Two Lane Rural Road

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13.7.10.4 Channelised Right Turn Treatments at Four-Way Intersections

As discussed in Section 13.4.3.5, it is undesirable to build new four-way unsignalised intersections. However, many four-way rural unsignalised intersections exist. At some of these intersections, traffic volumes may dictate the need to retrofit CHR(S) or CHR turn treatments.

There are various options for applying the through lane deviation to retrofit these turn treatments at four-way intersections. Some of these options are shown in Figure 13.61 (for a CHR(S) turn treatment) and Figure 13.62 (for a CHR turn treatment). The deviation can be fully on one side of the intersection, as shown in (a) and (b) in these figures, or partly on each side, as shown in (c) of these figures. Site details will generally dictate which will be the best option.

The layouts shown in Figure 13.61 and Figure 13.62 may also be applicable to existing four-way urban intersections.

13.7.10.5 Seagull Right Turn Treatment on a Rural Road

A "Seagull" is a particular form of channelised layout for a T intersection.

Details of the preferred rural seagull layout are shown in Figure 13.63. The right turn vehicles enter their own lane in the through road. This avoids the bad sighting angle created if a right to left merge was provided immediately following the right turn and the difficultly of where the through traffic volumes and/or speed make gap acceptance difficult.

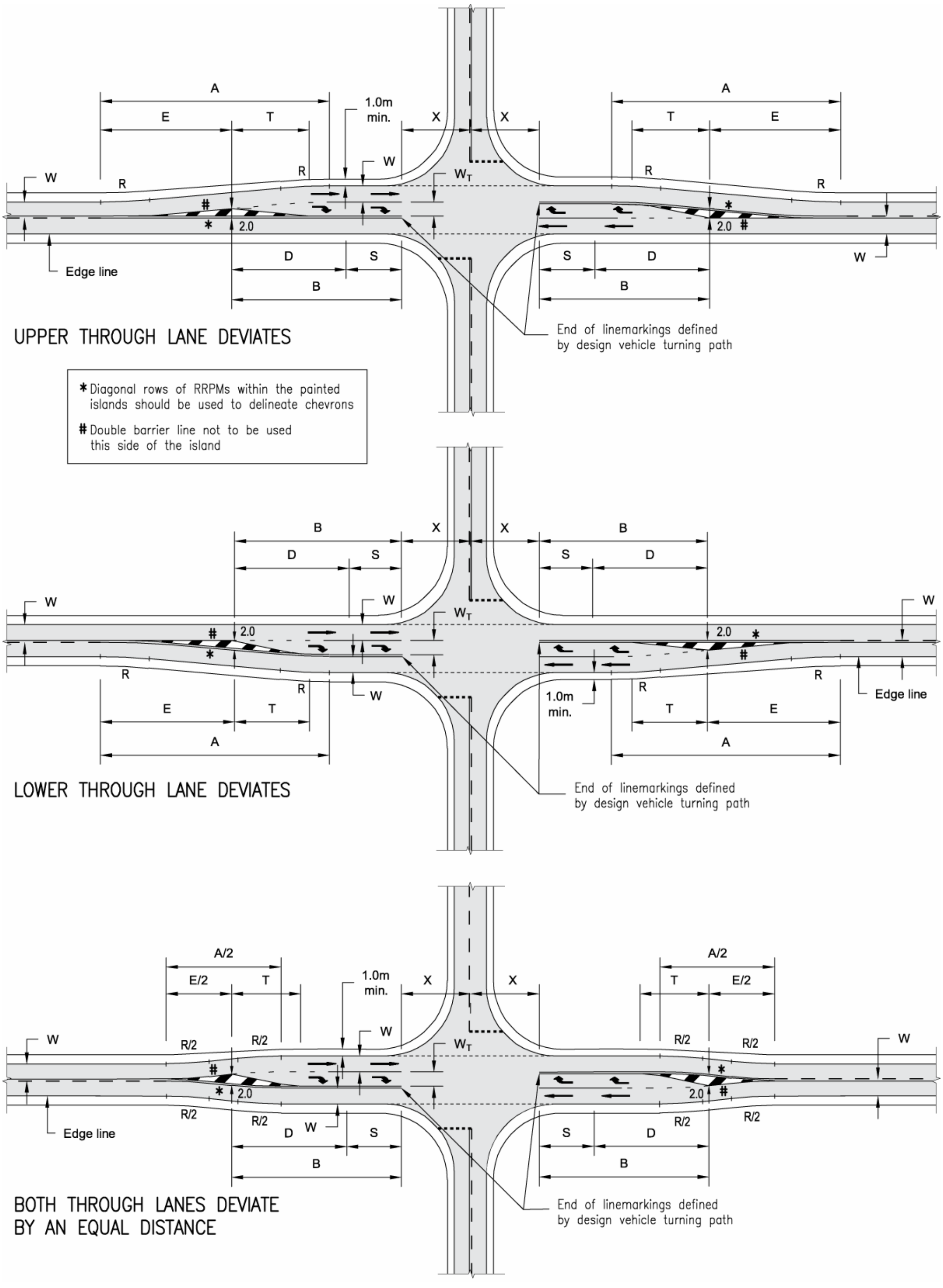
Although not desirable, the seagull layout can be used as a two stage right turn movement where turning volumes are light and there are insignificant articulated vehicle turning volumes. The exit on the through road can be used as a storage bay followed by a right to left merge when a gap can be found in the through traffic. This merge is done using the mirrors on the nearside of vehicles.

A minimum width median of 6.0m is required for this two staged turning manoeuvre. Details of this layout are shown in Figure 13.52 for an urban area. A similar layout is applicable in a rural area.

Due to the high speeds in the through lane, and the inherently more difficult right to left merge from a low speed, this design option is not preferred and should only be used as a last resort. An offset right-turn lane (as shown in Figure 13.66) is a preferred treatment in this instance, provided that sufficient median width is available.

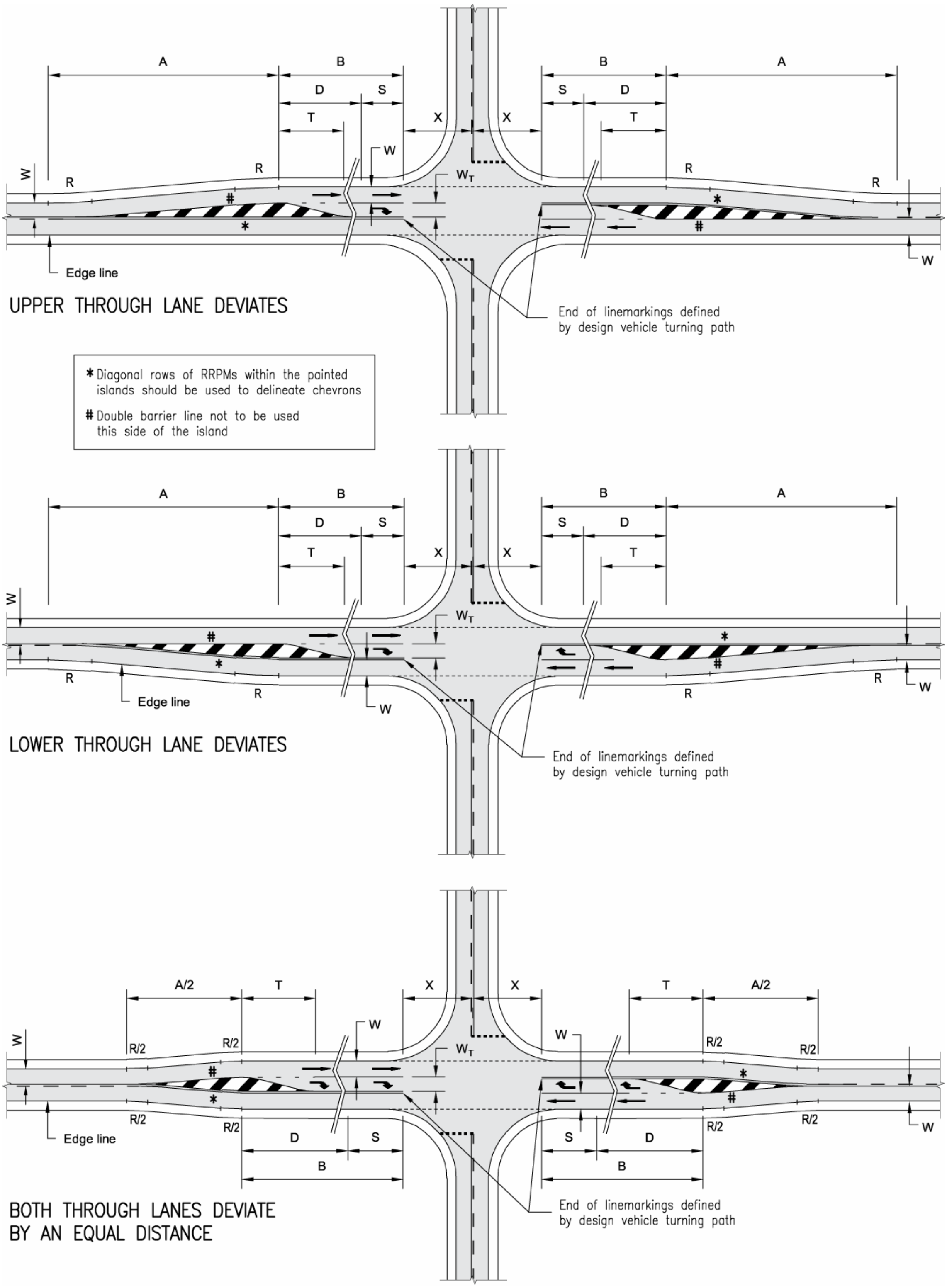
Where turning movements from the side road are high, or where the through traffic volumes and/or speed make gap acceptance difficult, then a dedicated exit lane should be provided (see Figure 13.63).

Where a seagull treatment provides a dedicated lane for exiting vehicles, the adjacent through lane(s) should be extended past the seagull to allow a passenger car vehicle to accelerate to the speed of through vehicles before the left to right merge is required. This is shown in Figure 13.63. It should be noted that providing a straight near-side edge line is not preferred as it lacks the visual queue of the edge line marking deviating for the merge manoeuvre.



Note: Refer to Figure 13.59 for the dimensions labelled in these diagrams.

Figure 13.61 Retrofitting CHR(S) Turn Treatments to an Existing 4-Way Intersection on a Two-Lane Rural Road



Note: Refer to Figure 13.60 for the dimensions labelled in these diagrams.

Figure 13.62 Retrofitting CHR Turn Treatments to an Existing 4-Way Intersection on a Two-Lane Rural Road

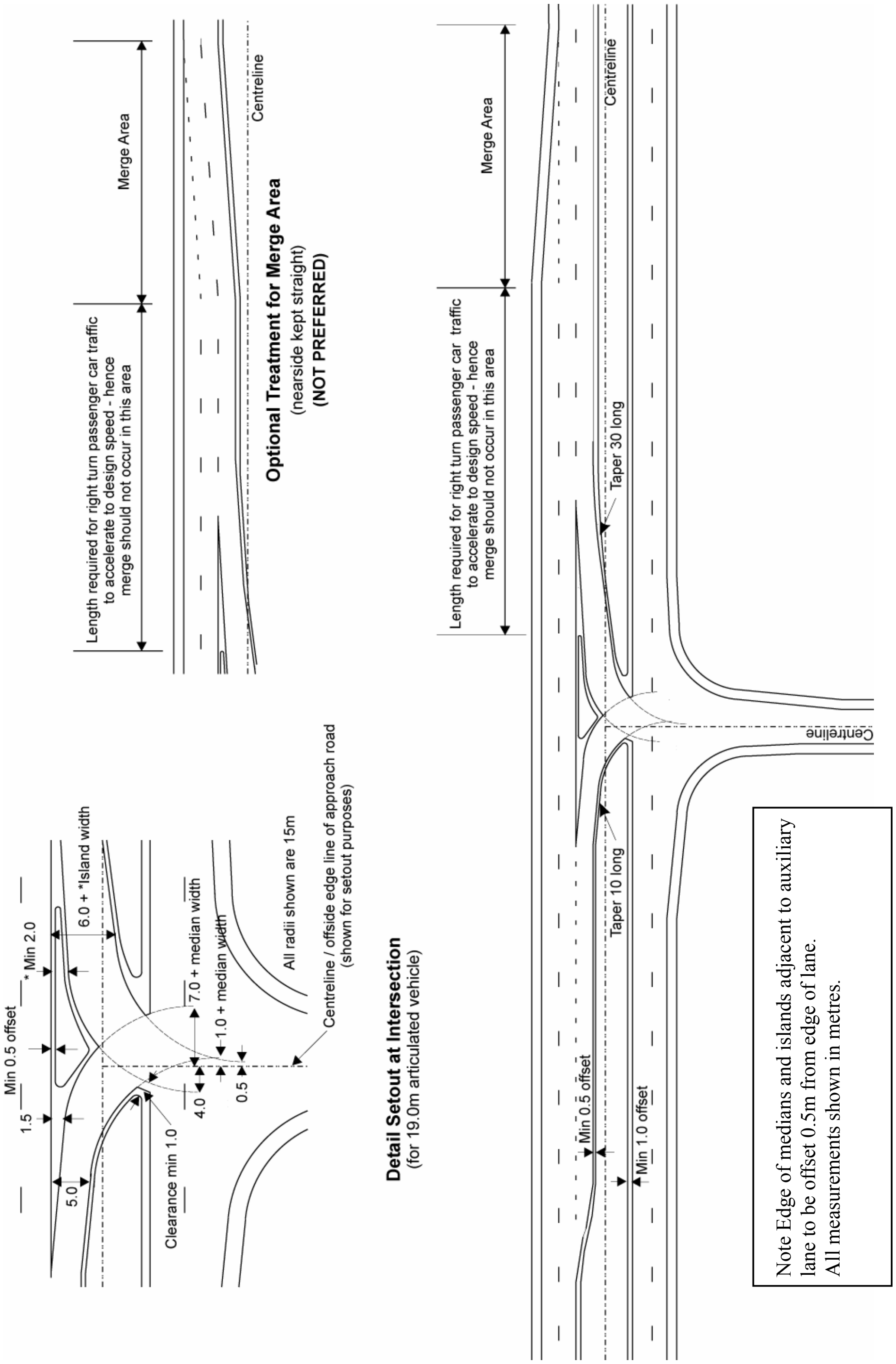


Figure 13.63 Preferred Seagull Layout on a Rural Road (Left Hand Side Merge)

An alternative seagull layout with a right hand side merge and a full acceleration lane is shown in Figure 13.64. Whilst not a preferred layout, there may be instances where such a layout provides a better overall result.

Seagull treatments require a minimum width of median to ensure that median and island noses are located to provide adequate control and guidance for traffic. Refer to Table 13.10 for details.

With seagull layouts, careful attention is to be given in the event of a blockage. An absolute minimum width between kerbs of 4.5m is required with a desirable minimum width of 5.5m. However, such widths between kerbs may encourage drivers to form two lanes. Provision of edge lines may be necessary to stop this happening.

Barrier kerbs should only be used where it is essential for other purposes; painted medians and islands should not be used. If a painted median is used for the seagull island, it should be further delineated using traffic guidance flaps.

13.7.10.6 Two Staged Crossing on a Rural Road

This layout is applicable on roadways with wide medians when the volume of right turning traffic is small and the traffic volumes on the through route are high. Right turning traffic from the minor road undertakes the turning manoeuvre in two stages. A layout of a two staged crossing is shown in Figure 13.65.

The width of the median should be sufficient to cater for the length of the turning design vehicle, as dimensioned in Figure 13.65. For the right turn from the major road, the median width should also cater for the calculated storage length. This is to provide drivers turning right from the minor road a clear view of approaching major road vehicles.

Turning paths are not to cross the centreline of the street being entered.

The layout shown in Figure 13.65 may also be applicable in some urban situations.

13.7.10.7 Offset Right-Turn Lane

If the width of the median at a two stagged crossing is insufficient for the calculated storage length, right turn drivers from the major road will store in the right-turn slot. Under these conditions, it can be extremely difficult for drivers turning right from the side road and stored in the median to obtain visibility around the stored vehicles to approaching traffic on the major road.

If the width of the median is insufficient for the calculated storage length, an offset right-turn lane (shown conceptually in Figure 13.66) is a preferred treatment. An offset right turn lane maximises visibility for right turn drivers from the side road stored in the median. The offset right turn lane should be positioned such that the right-turning passenger car from the side road, stored in the median, does not block the traffic stream in the right turn slot. This requires a minimum median width for the treatment to be operationally effective.

This treatment can also be applied in urban areas.

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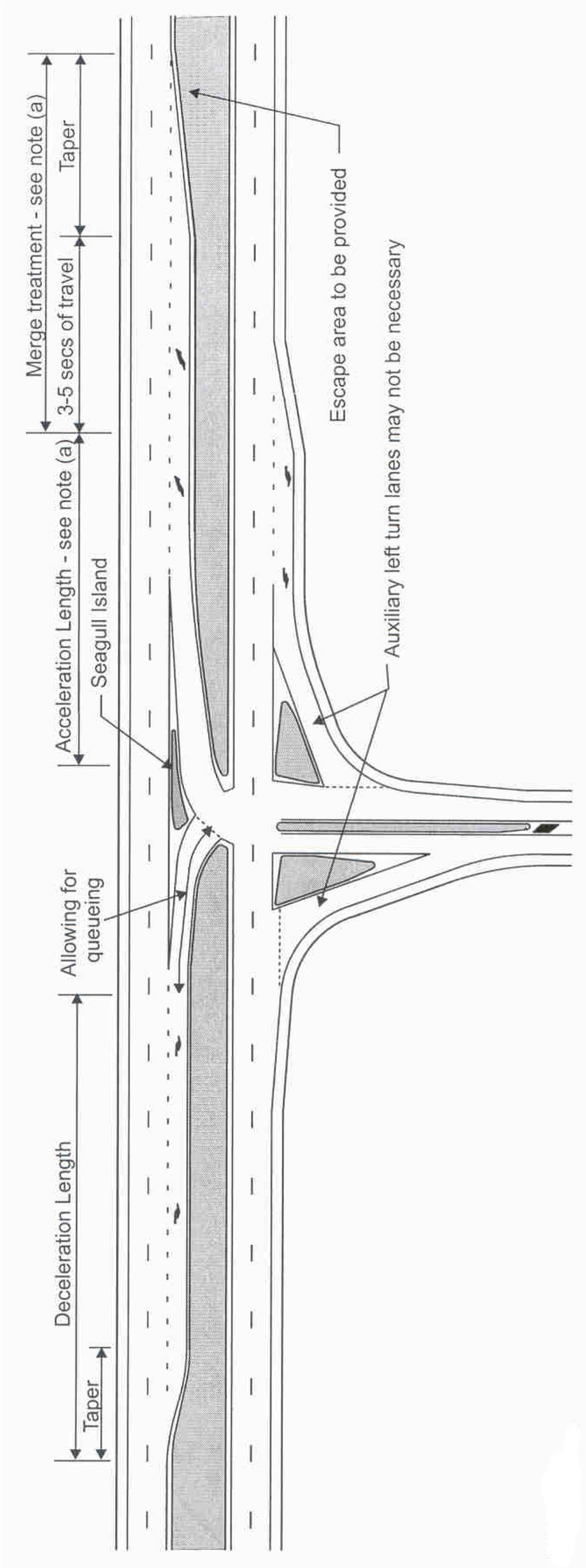
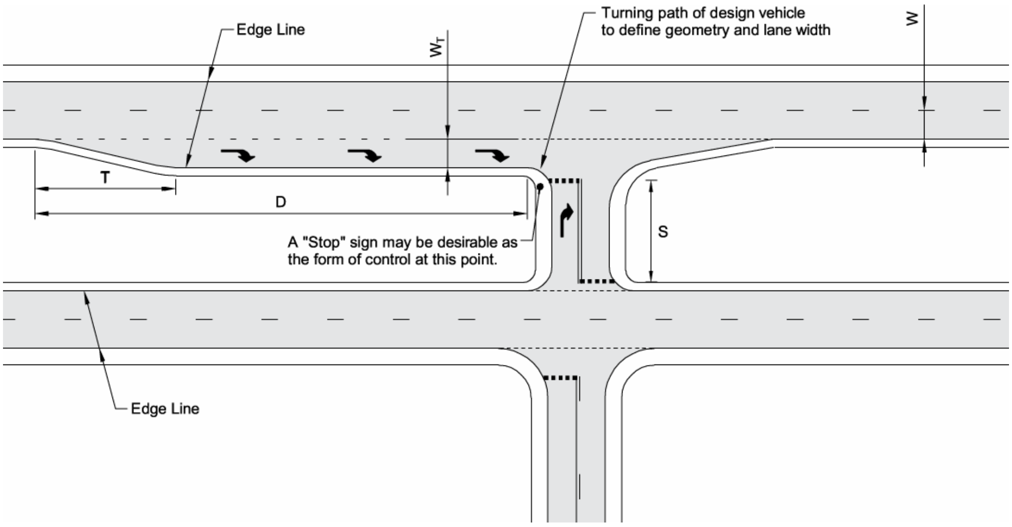


Note:
Due to higher speeds in the median lane and the inherently more difficult right hand side merge, the acceleration lane length should allow design speed to be attained prior to the start of the taper. Preferred practice is for the movement to discharge into a dedicated lane as shown in Figure 13.63.

Figure 13.64 Alternative Seagull Layout on a Rural Road (Right Hand Side Merge – Not Preferred)



W = Nominal through lane width (m) (incl. widening for curves)

W_T = Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle). Desirable minimum = W, absolute minimum = 3m.

D = Diverge/deceleration length including taper - refer to Table 13.11 (adjust for grade using the 'Correction to Grade' table in Figure 13.42).

S = Storage length (m), greater of:
1. The length of one design turning vehicle (for both right turn movements from the major and minor roads)
2. (calculated car spaces –1) x 8m (refer to Appendix 13A or use computer program eg. aaSIDRA) – only applicable for the right turn movement from the major road

T = Taper length (m) = $\frac{0.33 \times V \times W_T}{3.6}$

V = Design speed of major road approach (km/h)

Note: An offset right turn lane, as shown in Figure 13.66, is a more preferable solution for a two staged crossing. The offset right turn lane improves visibility for the right turn vehicle from the side road, once stored in the median

Figure 13.65 Two Stagged Crossing on a Rural Road

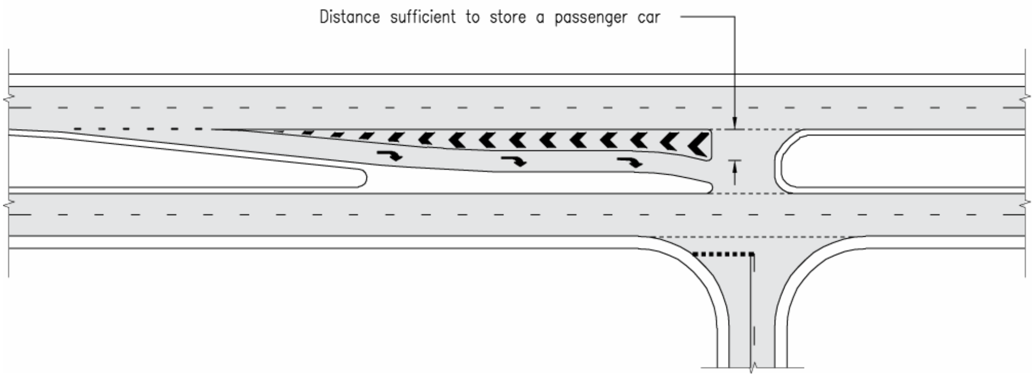


Figure 13.66 Conceptual Diagram of an Offset Right Turn Lane at an Intersection on a Rural Road

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13.7.10.8 Staggered T- intersection on a Rural Road

The purpose of a staggered T-intersection is to stop traffic on the minor legs inadvertently crossing an arterial, or sub-arterial road at high speed. This requires careful attention to the approach geometry and design to ensure that drivers appreciate what is expected, especially at night. Landscaping and additional furniture may be necessary to interrupt the line of sight from one side of the intersection to the other, clearly establishing the termination of the minor leg.

Figure 13.67 shows a diagram of a Right-Left stagger. This treatment is often more cost effective than a Left-Right stagger if converting from a four-way cross intersection.

This layout stores crossing vehicles on the minor legs. As traffic on the minor legs has to give way to both directions on the major route, calculations to establish delay may be necessary. Where right turn storage is already required on the major legs, this layout may be inappropriate and a Left-Right stagger preferred.

Figure 13.68 shows diagrams of Left-Right staggered intersections. These layouts allow cross traffic to undertake the manoeuvre in two stages, which has benefits when volumes on the through route are high. It is most desirable that a right turn slot be introduced for the motorists turning right from the major road.

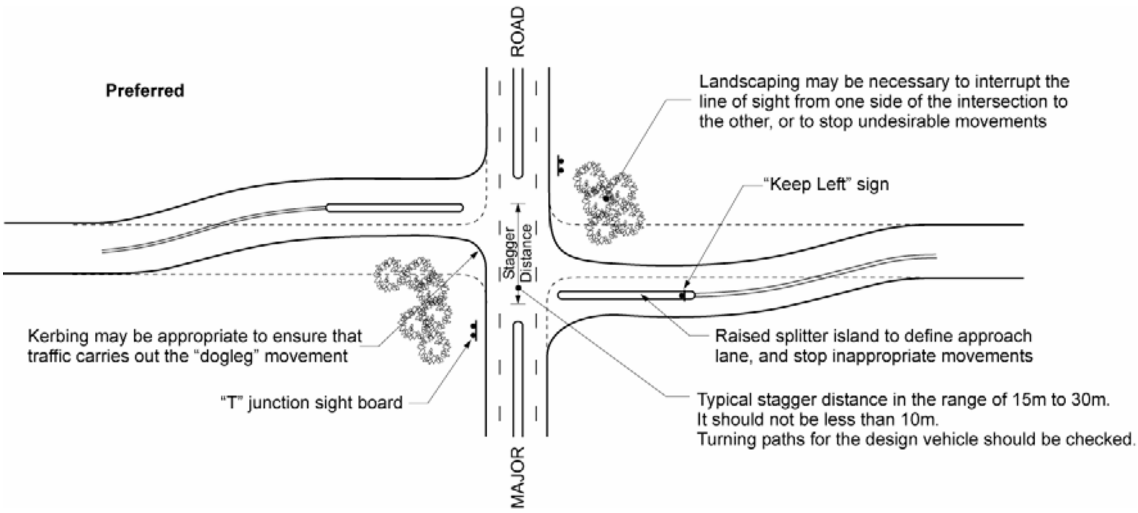

Figure 13.67 Right-Left Staggered T-Intersection on a Rural Road

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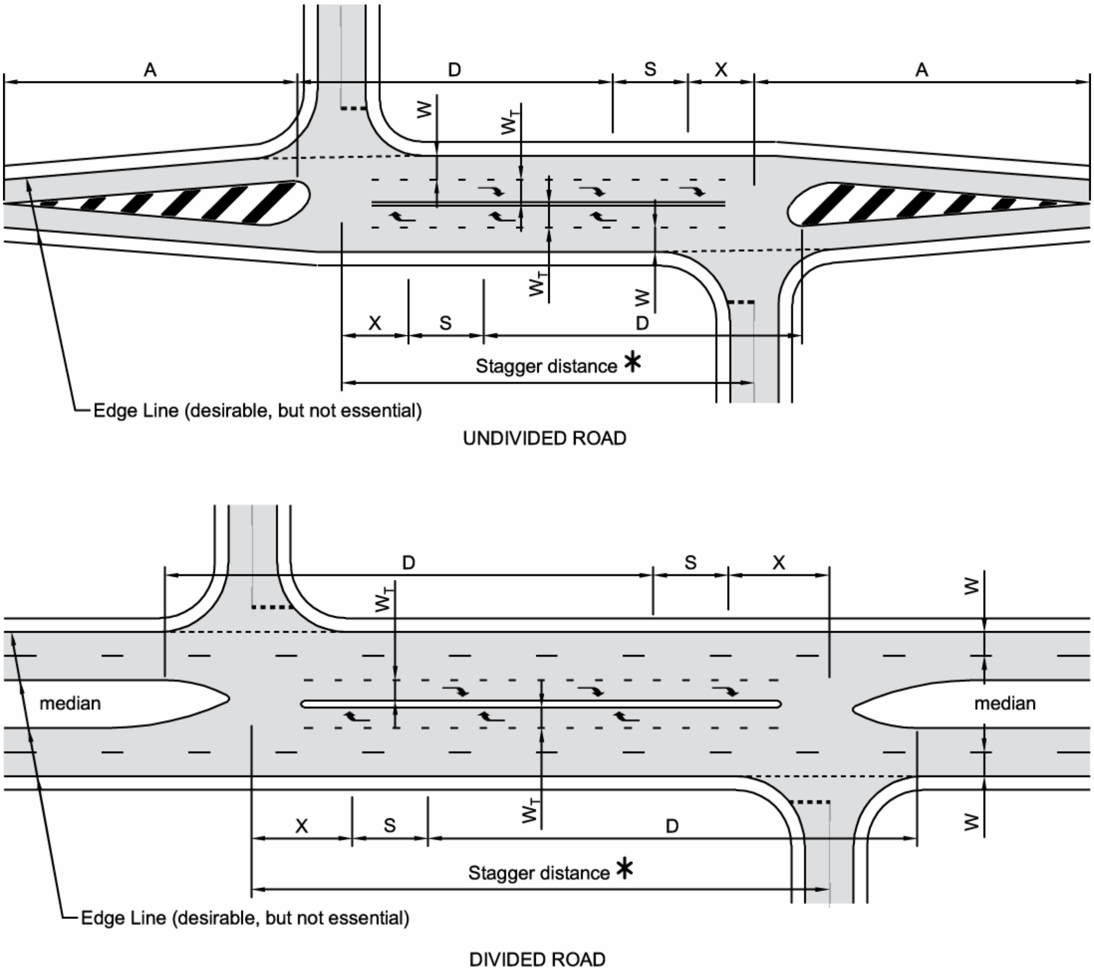

* The stagger distance must be sufficient to ensure that the ‘through’ design vehicle from the minor roads can store clear of the major road through lane when positioned in the right turn slot.

W = Nominal through lane width (m) (incl. widening for curves)

W_T = Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle). Desirable minimum = W, absolute minimum = 3m.

D = Diverge/deceleration length including taper - refer to Table 13.11 (adjust for grade using the 'Correction to Grade' table in Figure 13.42).

S = Storage length (m), greater of: 1. The length of one design turning vehicle
2. (calculated car spaces –1) x 8m (refer to Appendix 13A or use computer program eg. aaSIDRA)

X = Distance based on design vehicle turning path - typically 10 – 15m

Figure 13.68 Left-Right Staggered T-Intersection on a Rural Road

13.7.11 Left Turn Treatments - General

The design and layout of a left turn treatment depends on:

- volume and type of traffic making the turn;
- volume, speed and type of traffic with which the turn merges;
- estimated speed at entry, and desirable speeds through, and exiting from the turn;
- local restrictions such as turn angles, constraints, and so on;
- provision for through and turning cyclists; and
- priority of the turn.

These factors combine to determine the return radius, the width of the left turn lane and the need for a left turn splitter island. The return is the circular arc joining the kerb, or edge lines of intersecting roads. The width and direction of approach and departure to the turn will also influence the return radius or radii.

Treatments can vary from a single radius, to multi-radius returns. The radius, or radii of a return should be designed using appropriate design vehicle turning paths (refer Section 13.3.1.2).

Specific requirements are given in Sections 13.7.12 for urban conditions and 13.7.13 for rural. Layouts incorporating treatments for bicycles (mainly urban locations) are described in Austroads (1999b).

Having selected a return radius (or radii) designers should ensure that the turn:

- provides adequate sight to approaching vehicles;
- minimises areas of conflict; and
- keeps crossing widths for pedestrians to a minimum (mainly urban conditions).

The design of the turning path must be carefully considered and the effective radius and crossfall assessed. Appropriate design parameters are discussed in Chapter 7 and in Section 13.7.4.1.

13.7.11.1 Sight distance requirements

Sight requirements depend on the direction of approaching traffic and right-of-way regulations. For vehicles entering a priority road there are two sight lines to consider. One is to an approaching through vehicle, the other to a turning vehicle. These have been previously discussed in Section 13.6.4. Details are given in Figure 13.69.

The acceptable maximum observation angle of 120° is based on the visibility requirements from vehicles given in Section 13.3.1. This means that a driver would not be required to significantly change driving position to sight approaching traffic. When this happens it can result in a driver losing stereo vision, i.e. only being able to sight approaching traffic with the right eye thus losing depth of field vision. This makes it very difficult for a driver to accurately detect the position and speed of approaching traffic.

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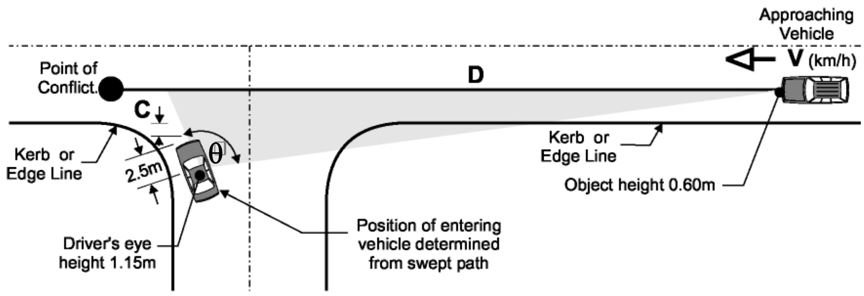

Notes:

θ Observation angle:
New or reconstructed work maximum 120°
Existing conditions – remedial treatment see Section 13.7.11.1.

C 0.5m from kerb or edge line projection or 1.0m from stop or give way line

D Minimum distance travelled by approaching vehicle on the through road in 5 seconds at design speed (**V** km/h).
D (meters) = **1.4V** (km/h)

Sight Envelop
Access Sight Distance both horizontally and vertically within this envelope
Rural Areas - No Obstruction to sight lines in this area.
Urban Areas - Fixed objects should not cause entering vehicles to lose sight of approaching vehicles.

Figure 13.69 Detailed Sight Distance Requirements to a Through Vehicle from a Vehicle Turning Left

In existing situations, where sighting requirements to approaching vehicles are below these criteria, remedial treatments should be considered. For example:

(a) reconstruct part, or relocate the intersection;

(b) ban the turn;

(c) reduce approach speed in the priority road; and/or

(d) provide traffic signal control.

Comments on each of these options follow.

Reconstruction

Reconstruction of the left turn to overcome sighting problems may be an option. By providing a protected acceleration lane on the departure side of the turn, observation angle criteria are no longer applicable and are replaced by merging requirements. Generally, acceleration lanes are associated with multi radii (three centred curve) returns. If a left turn slip lane exists without a protected acceleration lane, and the observation angle exceeds 120°, reconstruction to a high entry angle turn may be appropriate (refer to Sections 13.7.12 and 13.7.13 for details). Elimination of the slip lane, and provision of a single radius return, may be appropriate depending upon capacity requirements. Relocation of an intersection to overcome sighting problems is generally more practical in rural areas than in urban situations.

Banning the turn

This is an option provided convenient alternative access is available and the effect on the road network acceptable.

Reduce approach speed

Reduction of traffic speed so that the available sight distance meets sight requirements is generally only possible on local streets where effective speed control measures, such as speed humps, thresholds, or similar forms of speed control, can be appropriate. On collector, sub arterial and arterial roads, speed reduction can be achieved with a roundabout (mostly urban application) but different sight requirements will then apply. However, roundabouts can create problems where there are high volumes of other road users eg motorcyclists, pedestrians and cyclists.

Roundabouts should not be used solely as a speed control device to remedy sight distance deficiencies. Speed zoning over short, isolated lengths is not appropriate.

Provide traffic signals

Traffic signals can be used to resolve safety problems when sight distances are deficient. This solution can be costly and network consequences must be carefully examined (particularly in terms of delay).

13.7.11.2 Area of Conflict and Pedestrians

To minimise the area of conflict, the width of pedestrian crossing, and the relative speed of the vehicles and pedestrians, the turning radius should be kept to a minimum. Details of such radii are given in Figure 13.70 for urban conditions, and Section 13.7.13 for rural conditions. However, where the area of pavement is still excessive, and/or the width of pavement to be crossed by pedestrians is too long, other options include:

- a left turn island (min. area $8m^2$ urban, $40m^2$ rural); and
- median (min. 1.2m wide) for pedestrian refuge.

If a left turn island is provided, the observation angle to approaching through traffic will be exceeded for entering traffic if a single radius return above R11 is used, or the through road approach is straight for a distance less than five seconds of travel at the design speed. This applies to both urban and rural conditions even for provision of a minimum size island. To avoid this problem a high entry angle turn or protected acceleration lane on the departure should be used. This problem is addressed in more detail in later sections.

Auxiliary lanes can be used with any left turn treatment. Generally, approach auxiliary lanes are provided for capacity reasons in urban areas and for reasons of driver expectation at rural sites. Auxiliary lanes on departures must be protected from conflicting through movements by a left turn island, preferably raised.

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Vehicle Type	$\dfrac{\phi}{W(m)}$	MINIMUM KERB RADII (m)					
		1. Turning from lane adjacent to kerb side lane - approach width 6.4 metres			2. Turning from kerb side lane - approach width 3.2 metres		
		70°	90°	110°	70°	90°	110°
Semi Trailer 19.0m long	6.4	10	11	12	16	16	15
	5.5	12	14	14	18	18	16
Bus /Truck 12.5m long	6.4	3	6	8	12	12	12
	5.5	6	8	10	13	13	13

Notes:
1. Where approach and/or departure is curved, or widths vary from above, use turning templates to determine kerb radius and check observation sight distance.
2. Shading indicates kerb radii exceeds 11.0m - conditions apply, see Section 13.7.12.1.

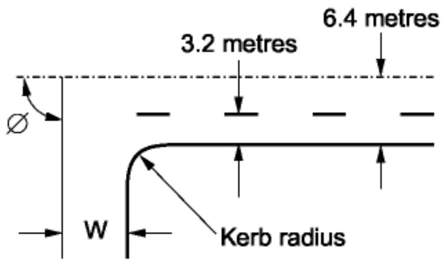

Figure 13.70 Guide to Minimum Kerb Radii under Some Typical Urban Conditions

13.7.11.3 Conflicts with Cyclists

Approaches to exclusive left turn treatments may create serious conflict points between cyclists and left turning motor vehicles. Accompanying bicycle treatments are presented in Austroads (1999b) – Sections 5.3 and 5.5.1.

On priority cycling routes (as defined in QDMR (2004) policy - Cycling on State Controlled Roads) where there are long deceleration or acceleration tapers, large radius curves and high speeds, it is desirable that a bicycle lane be marked through the diverge/merge area, clearly defining the presence of a cyclist.

Bicycle lane treatments through intersections could also be considered at other locations where cyclists would be at risk due to the geometric design requirements for motor vehicles. A short marked cycle lane through an intersection may provide safety advantages to cyclists provided that its termination point does not lead cyclists into an unsafe situation. Terminating near a sealed shoulder would normally deliver adequate safety.

13.7.12 Left Turn Treatments for urban applications

A right-angle in the kerb line is generally unacceptable, even where a turn is not permitted, because the arris is so prone to damage. Construction practice usually requires an absolute minimum radius of at least 0.5m.

Where a left turn is permitted, the minimum kerb return radius without a corner cut-off (which interferes with the adjoining property) is equal to the footway width provided. Such an arrangement is appropriate for cars and the occasional SU truck (such as the garbage truck) provided that there is sufficient pavement width available for the turning path.

This arrangement can be used for the left turn out of a local street into local and collector roads, especially if one-way conditions apply.

However, this layout is generally not acceptable at sub-arterial and arterial intersections. At such sites there may be a need to accommodate heavy vehicle turning movements. A corner cut-off is usually required. Turning path applications for urban conditions are shown in Figure 13.71. Where returns join kerb lines which are curving in the opposite direction a length of straight should be provided between the curves (refer Figure 13.25). The return may be a single or multi radius (two or three centred curve).

Types of turn treatments for urban areas are discussed in Sections 13.7.12.1 to 13.7.12.5. They apply to turn movements from both the minor to major road and from the major to minor road.

13.7.12.1 Basic Left Turn Treatment (BAL) on an Urban Road

A Basic Left Turn Treatment (BAL) on an urban road is where no specific facilities are provided for the left turn. Types of BAL turn treatments in urban areas are shown in Figure 13.71. Often, the kerb lines of the intersecting roads are joined by a single radius circular arc.

The return radius/radii can be selected using turning path templates. For a single radius return, Figure 13.70 may also be used.

The observation angle of 120° to approaching traffic will be exceeded when the kerb return radius exceeds 11 metres and the approach on the through road is straight for a distance equal to or greater than that travelled in five seconds at the design speed of the through road. Hence, the kerb return radius of 11 metres should only be exceeded when:

- entering on the outside of a horizontal curve;
- leaving a through road without a slip lane; or
- entering traffic only needs to sight turning traffic (refer Figure 13.33b).

The kerb return radius should be reduced when entering on the inside of a curve.

Observation angles for above conditions should be checked with the criteria shown in Figure 13.33 and Figure 13.69.

Pedestrian crossing widths are generally not a problem if minimum kerb return radii are used. It will be noted that the narrower the departure or approach lane width, the larger the return radius necessary. Hence, these two factors must be considered together. Options have been discussed earlier in Section 13.7.11.

It is preferable to provide a widened area for left movements from the major to minor road. Using part of the parking lane and providing a parking limit can achieve this.

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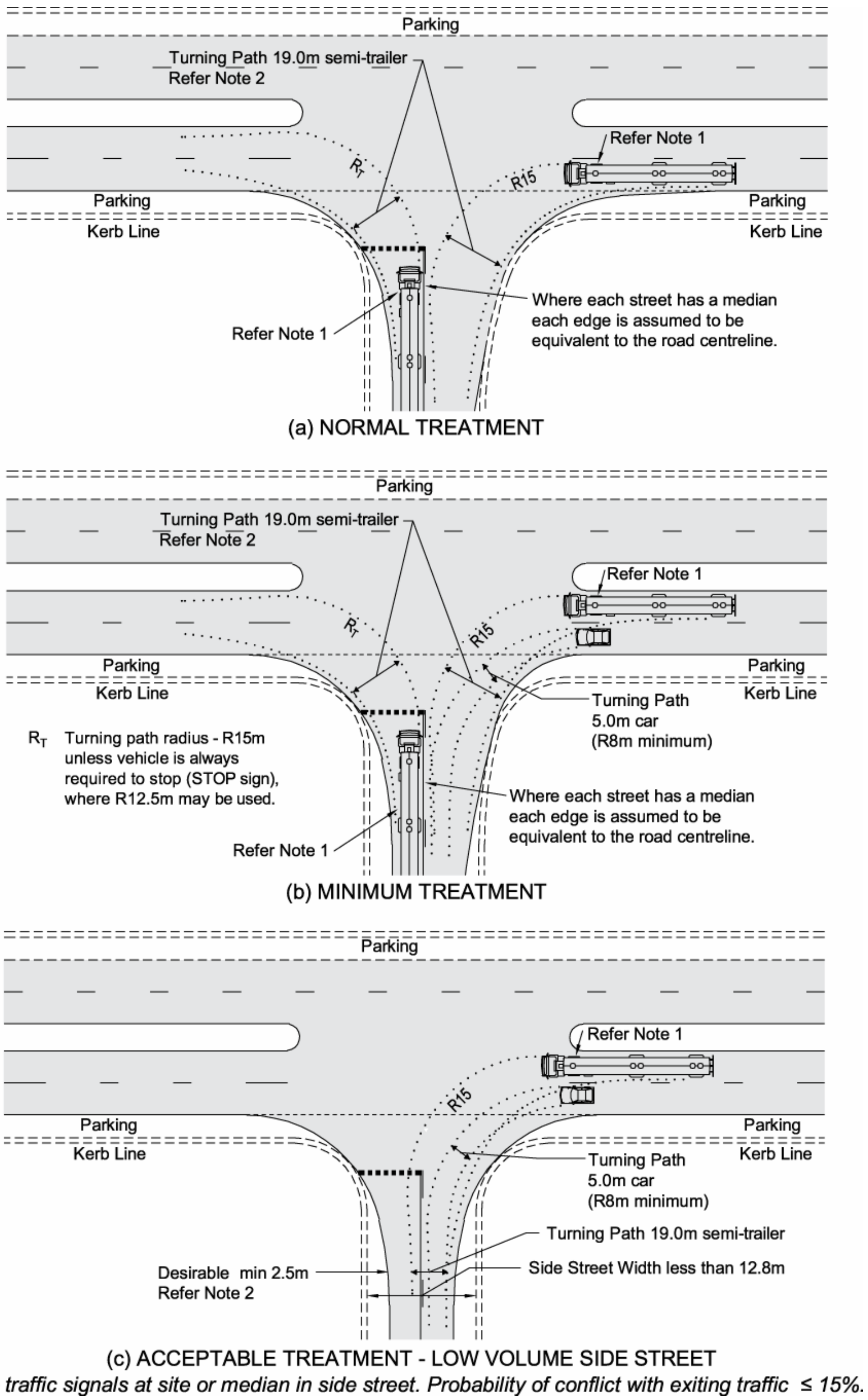

Note 1: Where approach is two lanes or more in widths, heavy vehicles (12.5m long or more) must turn from the kerbside or adjacent lane, unless otherwise controlled by signs and pavement arrows.

Note 2: Where side street approach and/or departure is not used by vehicles over 12.5m long, a turning path for a bus/truck may be used.

Note 3: This diagram does not show any specific bicycle facilities. Where specific bicycle facilities are required (eg exclusive bicycle lanes), refer Austroads (1999b).

Figure 13.71 Basic Left Turn Treatment (BAL) on an Urban Road

An alternative to the Basic Left Turn treatment is shown in Figure 13.72. For this arrangement, two centred curves can be used to advantage in avoiding physical restrictions, such as utilities in the footway. This treatment is most effective for acute angle turns.

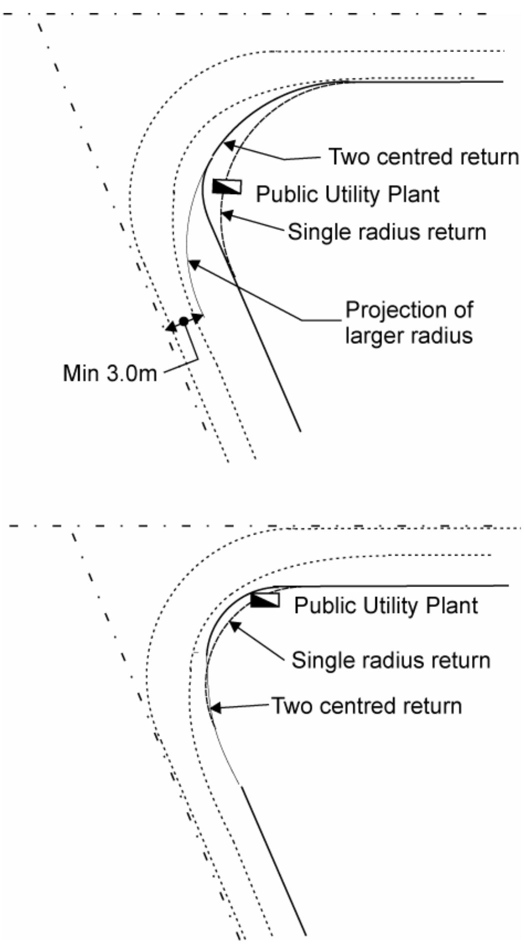

Figure 13.72 Arrangement of Compound Curves to Avoid an Obstruction which may be Expensive to Move

13.7.12.2 Auxiliary Left Turn Treatment with a Short Turn Slot [AUL(S)] on the Major Leg of an Urban Road

The BAL turn treatment from the major to minor road in Figure 13.71 is generally only suitable for lower turning volumes. A more desirable treatment at such sites is an AUL(S) turn treatment as shown in Figure 13.73. Although some deceleration of the left turning vehicles occur in the through lane, this treatment records very few Rear-End-Major vehicle accidents (generally rear-end type accidents resulting from a through driver colliding with a left turning major road driver). This treatment is suitable where there are low to moderate through and turning volumes. For higher volume sites, a full length AUL turn treatment is preferred.

13.7.12.3 Auxiliary Left Turn Treatment (AUL) on the Major Leg of an Urban Road

A diagram of an AUL turn treatment on the major leg of an urban road is shown in Figure 13.74.

The length of the auxiliary left turn lane should not be restricted to the minimum if there is little difficulty in making it longer and the demand warrants the treatment.

Pavement turn arrows are only necessary if site-specific problems are anticipated.

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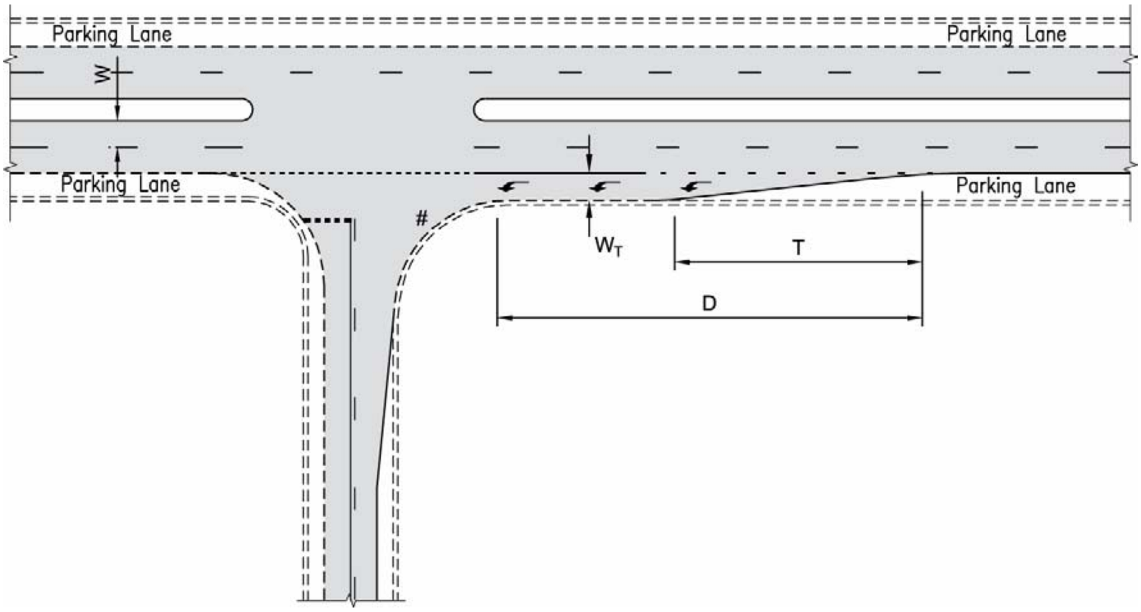


For setting out details of the left turn geometry, use vehicle turning path templates and/or Figure 13.70.

Design Speed of Major Road Approach (km/h)	Diverge/Deceleration Length D (m) (1)	Taper Length T (m) (2)
50	15	15
60	25	15
70	35	20
80	45	20
90	55	25
(1) Based on a 20% reduction in through road speed at the start of the taper and a value of deceleration of 3.5m/s^2. Adjust for grade using the 'Correction to Grade' table in Figure 13.42. (2) Based on a turn lane width of 3.0m		

W = Nominal through lane width (m) (incl. widening for curves)

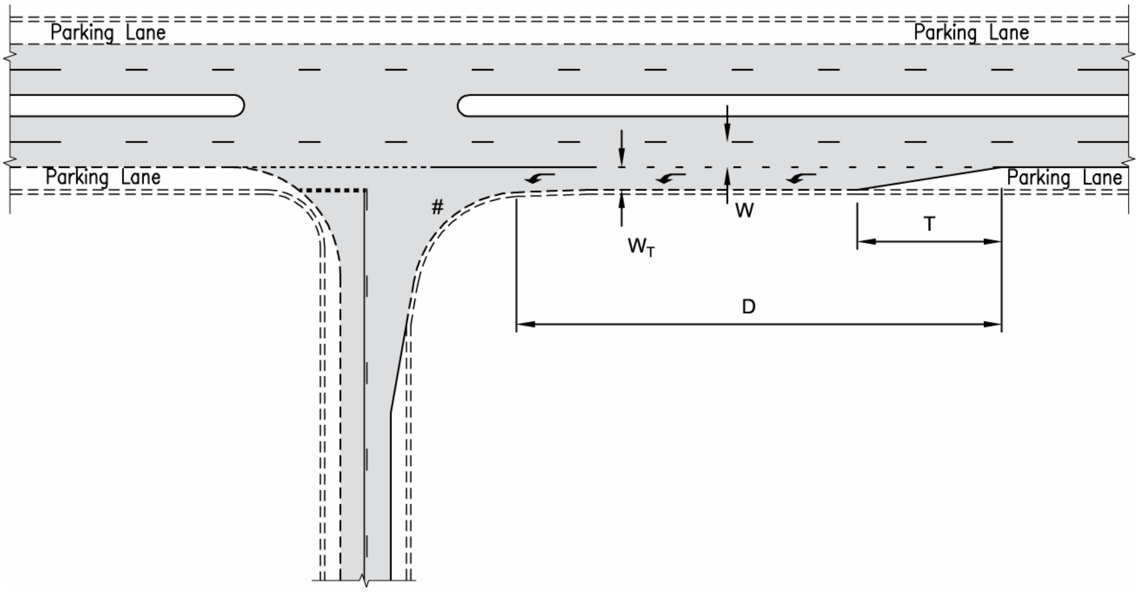
W_T = Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle). Desirable minimum = 3.0m, absolute minimum = 2.8m.

T = Taper length (m) = $\frac{0.33 \times V \times W_T}{3.6}$

V = Design speed of major road approach (km/h)

Note: Approaches to left turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. Treatments to reduce the number of potential conflicts at left turn slip lanes are given in Austroads (1999b).

Figure 13.73 Auxiliary Left Turn Treatment [AUL(S)] on the Major Leg of an Urban Road



For setting out details of the left turn geometry, use vehicle turning path templates and/or Figure 13.70.

W = Nominal through lane width (m) (incl. widening for curves)

W_T = Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle). Desirable minimum = W, absolute minimum = 3m.

D = Diverge/deceleration length including taper - refer to Table 13.11 (adjust for grade using the 'Correction to Grade' table in Figure 13.42).

T = Taper length (m) = $\frac{0.33 \times V \times W_T}{3.6}$

V = Design speed of major road approach (km/h)

Note: Approaches to left turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. Treatments to reduce the number of potential conflicts at left turn slip lanes are given in Austroads (1999b).

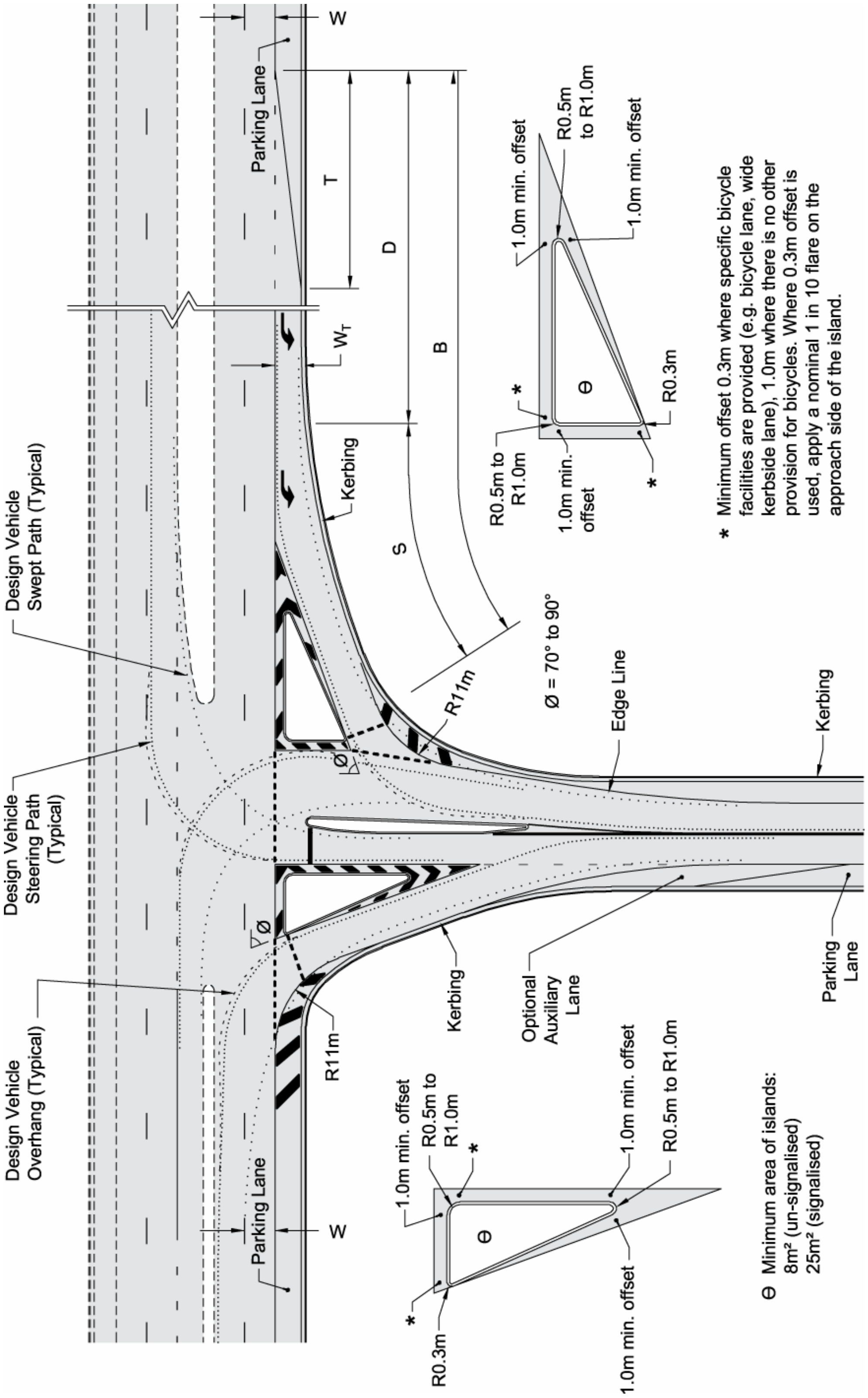
Figure 13.74 Auxiliary Left Turn Treatment (AUL) on the Major Leg of an Urban Road

13.7.12.4 Channelised Left Turn Treatment (CHL) with High Entry Angle on an Urban Road

Where kerb lines intersect in the range 70° to 110°, and a Channelised Left Turn Treatment (comprising a left turn island) is required in conjunction with a single radius return, a high entry angle treatment is necessary. This is the only way to achieve an island 8 m² (or more) in area and an observation angle of 120° (or less) to traffic approaching on a straight with not less than five seconds of travel at the design speed in length. This is illustrated in Figure 13.75.

When the intersection angle is 130° (or more), a left turn island can be provided for a 19.0m semi- trailer as long as the return radius is not greater than 11m. This is illustrated in Figure 13.76.

Observation angles for the above conditions should be checked with criteria shown in Figure 13.33 and Figure 13.69.



Note 1: Approaches to left turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. Treatments to reduce the number of potential conflicts at left turn slip lanes are given in Austroads (1999b).

Note 2: Figure 13.41 details clearances required at a CHL turn treatment where there are no specific cyclist facilities.

Note 3: Refer to Figure 13.50 (CHR turn treatment) for details of the dimensions T, D, S, B, W and W_T.

Figure 13.75 Channelised Left Turn Treatment (CHL) with High Entry Angle on an Urban Road

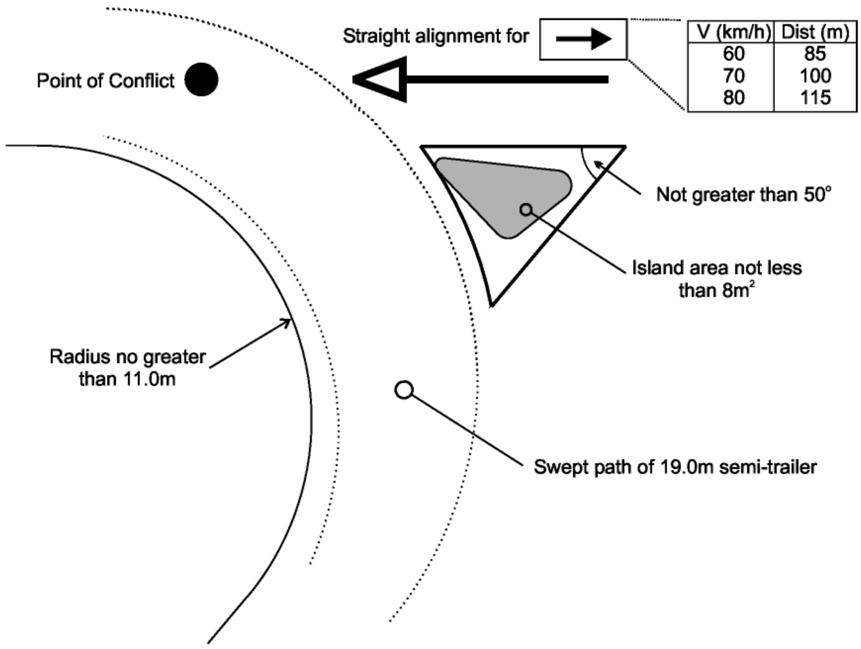


Figure 13.76 Sketch Showing the only Combinations that Meet Island Size and Observation Angle Requirements using a Single Radius Turn for a High Entry Angle Left Turn

The left turn island will assist in reducing pedestrian crossing widths and areas of uncontrolled pavement. If a marked pedestrian crossing is provided in the left turn slip lane, approach sight distance (ASD) shall be provided in approach to the crossing and the pavement markings clearly visible over the entire length of ASD in approach to the crossing.

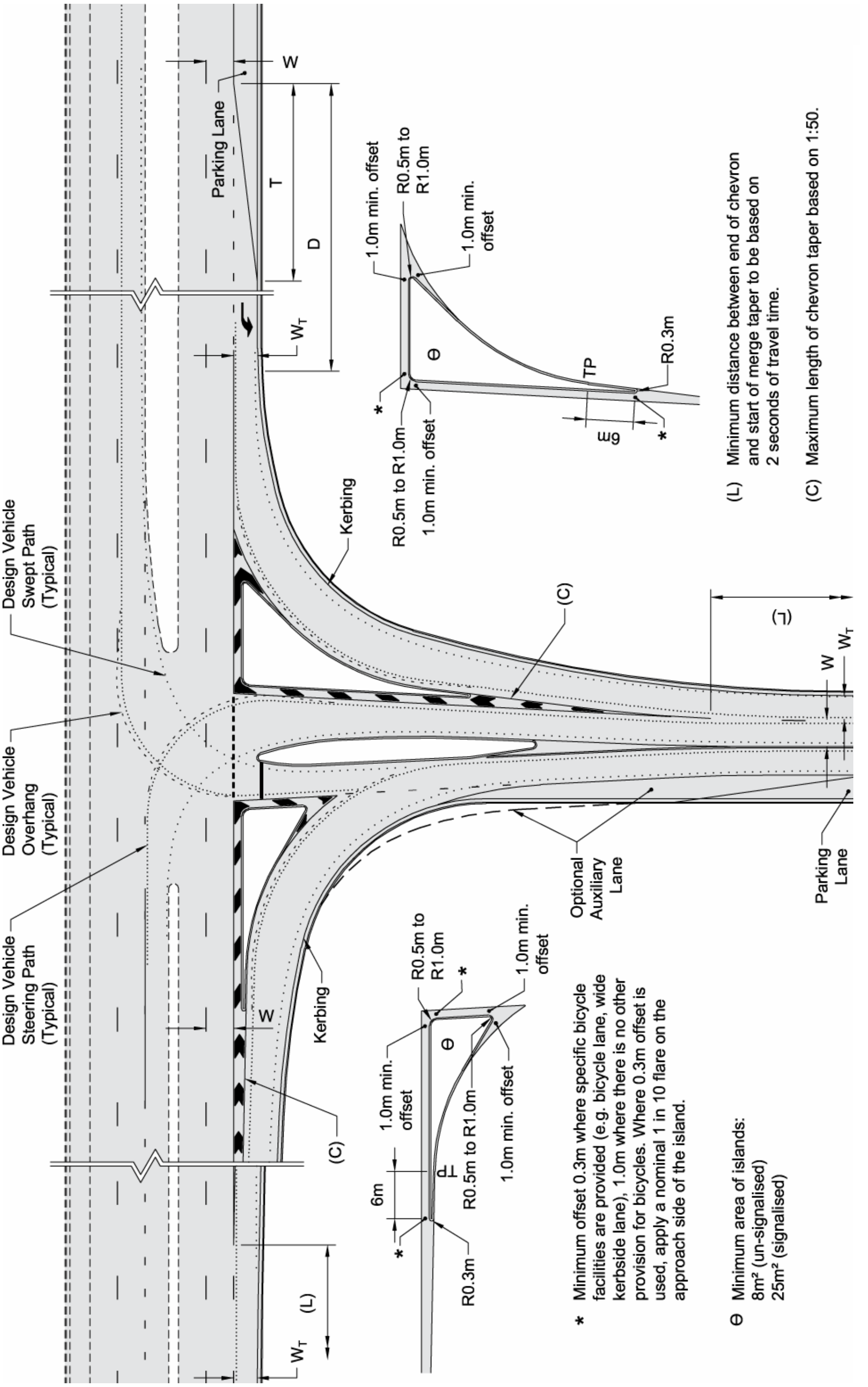
Appropriate bicycle treatments may be required for left turn island provisions. See Austroads (1999b) Section 5.5.1. Such treatments include line marking for bike lanes and warning signs for motorists using the slip lane to watch for cyclists.

13.7.12.5 Channelised Left Turn Treatment (CHL) with Acceleration Lane on an Urban Road

A Channelised Left Turn treatment with an acceleration lane comprises multiple radii returns ie it consists of compound circular arcs having two or three radii in order to best match the swept paths of turning trucks. The acceleration lane is a protected left turn lane. Geometric details of Channelised Left Turn treatments with an acceleration lane are given in Figure 13.77.

Channelised Left Turn treatments with acceleration lanes are useful where:

- the observation angle falls below guideline requirements (e.g. intersection located on the inside of a curve);
- insufficient gaps are available in the major road traffic stream for the left turning movement; and/or
- left turning heavy vehicles will cause excessive slowing of the major road traffic stream.



Note 1: Approaches to left turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. Treatments to reduce the number of potential conflicts at left turn slip lanes are given in Austroads (1999b).
Note 2: Figure 13.41 details clearances required at a CHL turn treatment where there are no specific cyclist facilities.
Note 3: Refer to Figure 13.50 (CHR turn treatment) for details of the dimensions T, D, S, B, W and W_T.

Figure 13.77 Channelised Left Turn Treatment (CHL) with Acceleration Lane on an Urban Road

A three centred curve must not be used for unsignalised left turns without a left turn island that:

- protects the departure lane;
- controls the path of exiting vehicles;
- minimises crossing widths for pedestrians; and
- minimises the area of uncontrolled pavement.

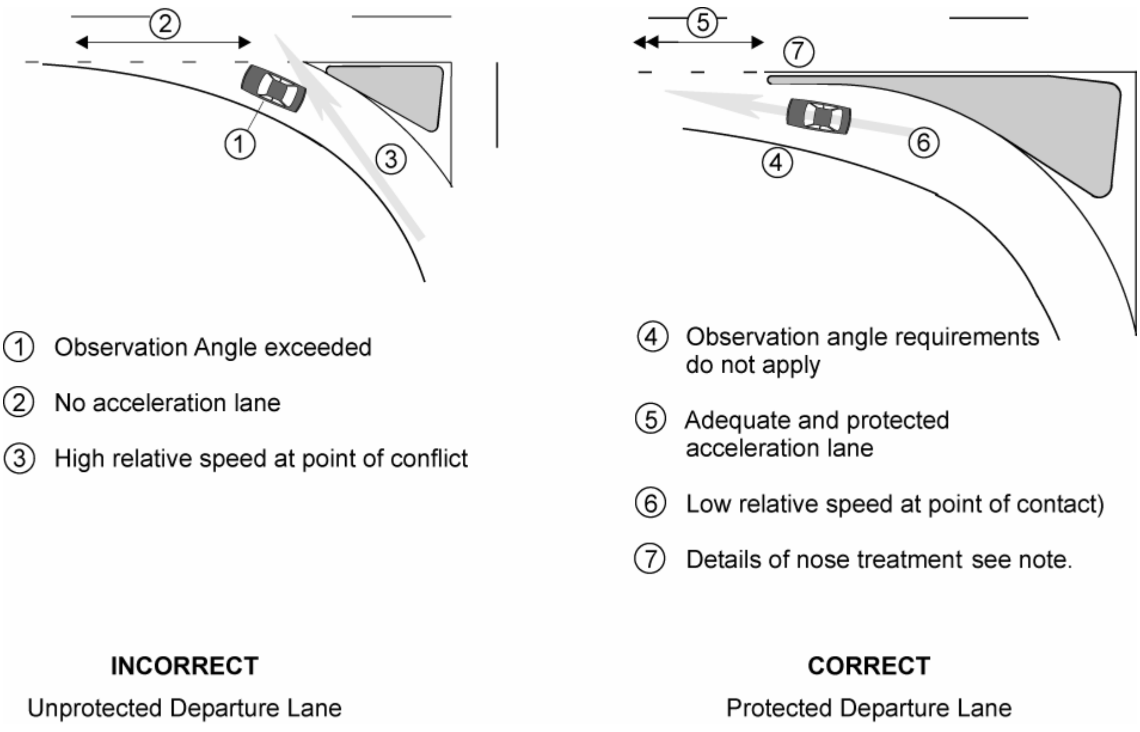
If the path of exiting vehicles is not controlled by the island nose the following will occur:

(a) observation angle to approaching through traffic will be exceeded where the through approach is straight for a distance less than five seconds of travel at the design speed; and

(b) inadequate acceleration taper will result.

These points are illustrated in Figure 13.78.

Where the intersection is used by pedestrians, an island can reduce the width of pavement to be crossed. If a marked pedestrian crossing is provided in the left turn slip lane, approach sight distance (ASD) must be available in approach to the crossing, and pavement markings clearly visible over the entire length of ASD in approach to the crossing.



Note: Refer Figure 13.77.

Figure 13.78 Sketch showing the Incorrect and Correct Treatment of an Unsignalised Three Centred Kerb Return using an Island

13.7.13 Left Turn Treatments for Rural Applications

13.7.13.1 General

At rural locations the return radii are at least designed for semi-trailer turning paths. In some cases returns will need to be designed for B-Doubles, Road Trains or other large vehicles. The VPATH program should be used to determine these turning paths.

Vehicle turning paths for vehicles 19.0 m long, or less, should not cross the centrelines of intersecting roads where the AADT of the side road is greater than 50 vpd or the side road/access is specifically designed for articulated vehicles.

13.7.13.2 Basic Left Turn Treatment (BAL) on a Rural Road

This is the minimum form of treatment that should be applied to a rural left turn. It has a single radius return, auxiliary lanes are not provided, and the layout is not channelised. Geometric details are given in Figure 13.79 (side road AADT less than 50) and Figure 13.80 (side road AADT greater than or equal to 50). It will be noted that the angle of the intersection is between 70°-110°. New or reconstructed intersections must be designed to this requirement even if legs have to be re-aligned.

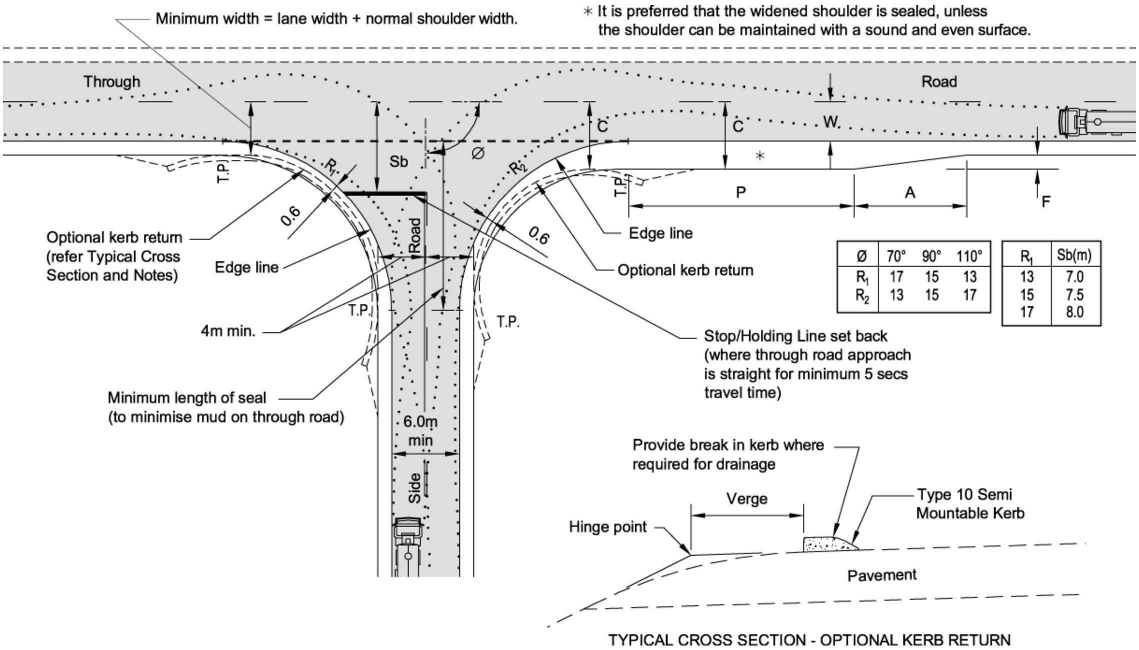
As the return radius exceeds 11m, special measures have been incorporated to allow the observation angle of 120° to be achieved. Where the through approach is a straight, and its length is a minimum of 5 seconds travel at the design speed, the holding line (particularly a STOP line) should be located as shown in the figure. Where roads intersect on the back of a curve the holding line may be located closer to the through road. Conversely, for intersections on the inside of a curve the holding line may need to be located further back but this should be limited to 8 metres from the centreline of a two lane rural road.

Where the holding line set back exceeds 8 m to provide the 120° observation angle, other treatments, such as a high entry angle (as shown in Figure 13.83 in Section 13.7.13.5), or a protected departure lane (as shown in Figure 13.84 in Section 13.7.13.6), should be considered.

Where efforts to relocate (or realign) a side road have failed to achieve an intersection angle in the 70°-110° range, a channelised solution may assist. Such a layout will have to be designed so that sighting angles to approaching traffic will be acceptable, whilst reducing areas of uncontrolled pavement and defining vehicle paths. However, every effort should be made to achieve a desirable angle of intersection in the first place.

Figure 13.79 and Figure 13.80 comprise a widened shoulder for movements from the major to minor road. The widened shoulder is based on a left-turning vehicle having a speed reduction of 30 percent in the through lane, prior to moving onto the shoulder and decelerating. This is based on the assumption that drivers decelerate at a maximum value of $3.5m/s^2$ from the start of the taper to the start of the kerb return. The total width of through lane plus widened shoulder is a minimum of 6m.

Dimensions of rural BAL turn treatments for various design vehicles are given in Appendix 13E.



Design Speed of Major Road Approach (km/h)	Minimum Length of Parallel Widened Shoulder P (m) (1)
50	0
60	5
70	10
80	15
90	20
100	25
110	35
120	45
(1) Adjust for grade using the 'Correction to Grade' table in Figure 13.42.	

W = Nominal through lane width (m) (including widening for curves)

C = On straights - 6.0m minimum
On curves - 6.0m plus curve widening (based on widening for the design turning vehicle plus widening for the design through vehicle)

$$A = \frac{0.33 \times V \times F}{3.6}$$

V = Design speed of major road approach (km/h)

F = Formation/carriageway widening (m)

Note: Layout not to be used when minor road is frequently used by articulated vehicles (more than about one turning articulated vehicle per day). Layout not to be used if associated with other minimum criteria eg sight distance restrictions or tight horizontal curves.

Figure 13.79 Basic Left Turn Treatment (BAL) on a Rural Road where the Side Road AADT is less than 50

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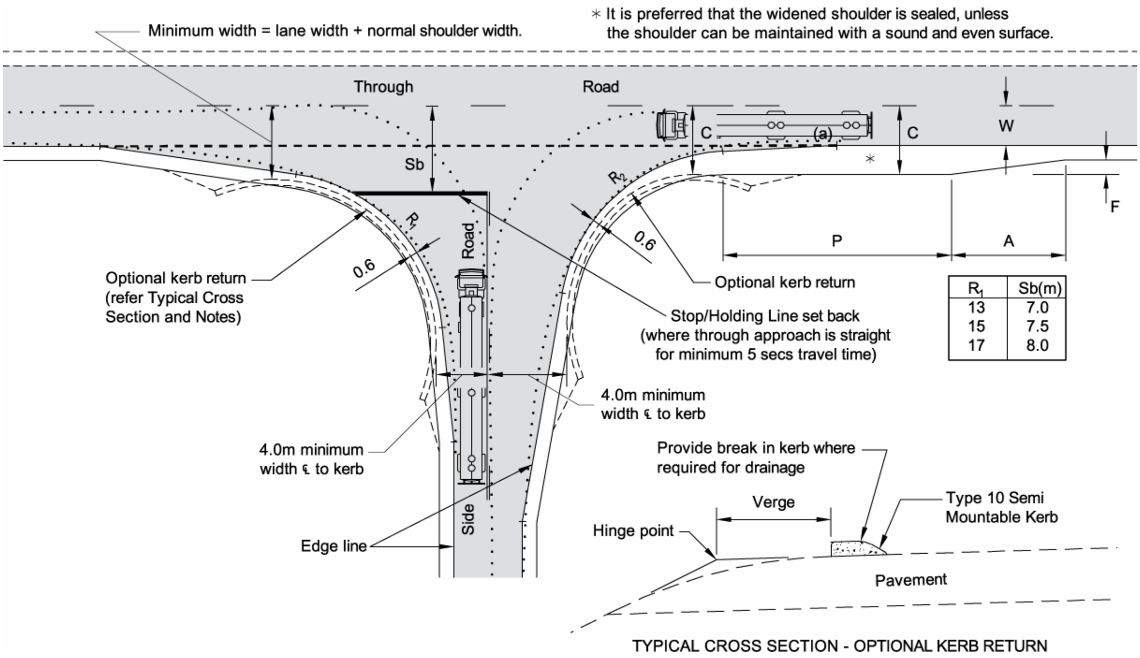

Design Speed of Major Road Approach (km/h)	Minimum Length of Parallel Widened Shoulder P (m) (1)
50	0
60	5
70	10
80	15
90	20
100	25
110	35
120	45
(1) Adjust for grade using the 'Correction to Grade' table in Figure 13.42.	

W = Nominal through lane width (m) (including widening for curves)

C = On straights - 6.0m minimum
On curves - 6.0m plus curve widening (based on widening for the design turning vehicle plus widening for the design through vehicle)

$$A = \frac{0.33 \times V \times F}{3.6}$$

V = Design speed of major road approach (km/h)

F = Formation/carriageway widening (m)

Note: Refer to Appendix 13E for dimensions of BAL layouts to suit various articulated vehicles

Figure 13.80 Basic Left Turn Treatment (BAL) on a Rural Road where the Side Road AADT is greater than or equal to 50 and/or Specifically for Articulated Vehicles

Figure 13.79 and Figure 13.80 show an optional kerb return, which can provide the following advantages:

- Better perception of the intersection, especially for intersections with limited visibility;
- Reduce the amount of 'corner cutting' by drivers; and
- Reduce the amount of scouring in areas of high rainfall, if provided with batter protection for the drainage paths.

13.7.13.3 Auxiliary Left Turn Treatment with a Short Turn Slot [AUL(S)] on the Major Leg of a Rural Road

An AUL(S) turn treatment is shown in Figure 13.81. This treatment is suitable where there are low to moderate through and turning volumes. For higher volume sites, a full length AUL turn treatment is preferred.

13.7.13.4 Auxiliary Left Turn Treatment (AUL) on a Rural Road

A diagram of an AUL turn treatment on the major leg of a rural road is shown in Figure 13.82.

The length of the auxiliary left turn lane should not be restricted to the minimum if there is little difficulty in making it longer and the demand warrants the treatment.

Pavement turn arrows are only necessary if site-specific problems are anticipated.

13.7.13.5 Channelised Left Turn Treatment (CHL) with High Entry Angle on a Rural Road

Provision of a left turn island with a single radius return where edge lines intersect in the range 70° to 110° requires a high entry angle treatment to achieve an island 50m² (or more) in area and the observation sight requirements. Such a Channelised Left Turn Treatment is shown in Figure 13.83.

13.7.13.6 Channelised Left Turn Treatment (CHL) with an Acceleration Lane on a Rural Road

A Channelised Left Turn treatment with an acceleration lane comprises multiple radii returns ie it consists of compound circular arcs having two or three radii. The acceleration lane is a protected left turn lane. A layout of such a Channelised Left Turn treatment is shown in Figure 13.84.

A Channelised Left Turn treatment with an acceleration lane (protected left-turn lane) can be useful where:

- the observation angle falls below guideline requirements (e.g. intersection located on the inside of a curve);
- insufficient gaps are available in the major road traffic stream for the left turning movement; and/or
- left turning heavy vehicles will cause excessive slowing of the major road traffic stream.

The left turn island will help to reduce areas of uncontrolled pavement and define vehicle paths.

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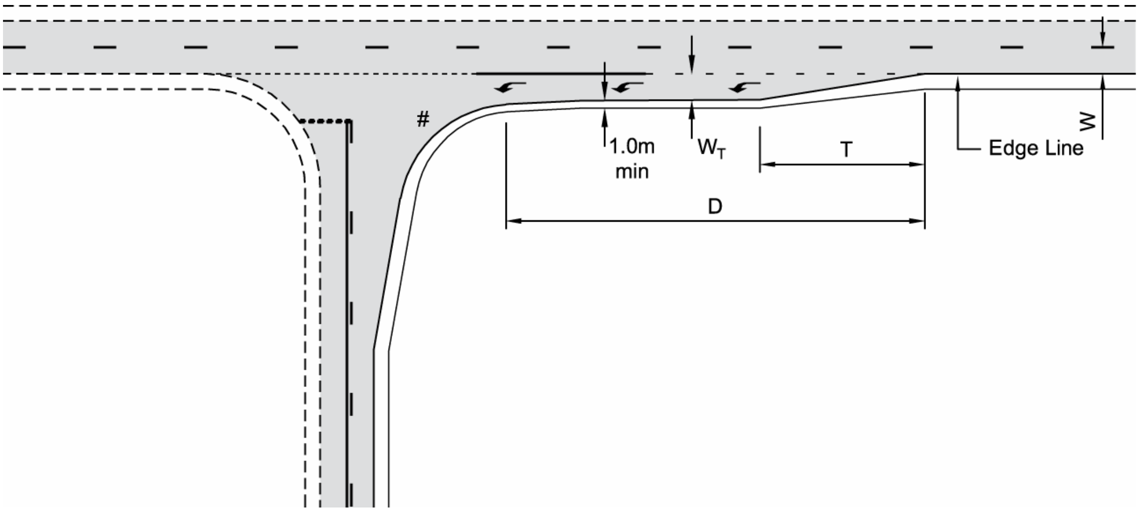


For setting out details of the left turn geometry, refer Figure 13.80 and/or Appendix 13E, as relevant.

Design Speed of Major Road Approach (km/h)	Diverge/Deceleration Length D (m) (1)	Taper Length T (m) (2)
50	15	15
60	25	15
70	35	20
80	45	20
90	55	25
100	70	30
110	85	30
120	100	35
(1) Based on a 20% reduction in through road speed at the start of the taper and a value of deceleration of $3.5 m/s^2$. Adjust for grade using the 'Correction to Grade' table in Figure 13.42. (2) Based on a turn lane width of 3.0m		

W = Nominal through lane width (m) (incl. widening for curves). For a new intersection on an existing road, the width is to be in accordance with the current link strategy.

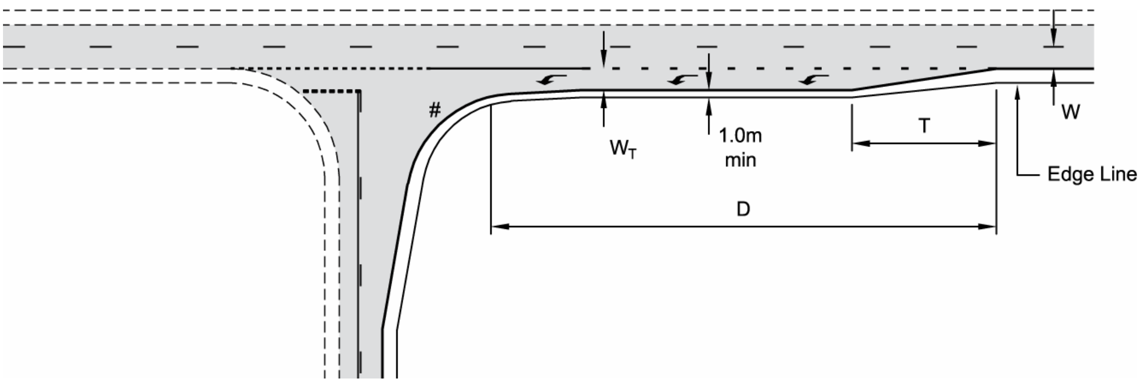
W_T = Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle) = 3.0m minimum

T = Taper length (m) = $\frac{0.33 \times V \times W_T}{3.6}$

V = Design speed of major road approach (km/h)

Note: Approaches to left turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. Treatments to reduce the number of potential conflicts at left turn slip lanes are given in Austroads (1999b).

Figure 13.81 Auxiliary Left Turn Treatment with a Short Left Turn Slot AUL(S) on a Rural Road



For setting out details of the left turn geometry, refer Figure 13.80 and/or Appendix 13E, as relevant.

W = Nominal through lane width (m) (incl. widening for curves). For a new intersection on an existing road, the width is to be in accordance with the current link strategy.

W_T = Nominal width of turn lane (m) (incl. widening for curves based on the design turning vehicle). Desirable minimum = W, absolute minimum = 3.0m.

D = Diverge/deceleration length including taper - refer to Table 13.11 (adjust for grade using the 'Correction to Grade' table in Figure 13.42).

T = Taper length (m) = $\frac{0.33 \times V \times W_T}{3.6}$

V = Design speed of major road approach (km/h)

Note: Approaches to left turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. Treatments to reduce the number of potential conflicts at left turn slip lanes are given in Austroads (1999b).

Figure 13.82 Auxiliary Left Turn Treatment (AUL) on a Rural Road

13.7.14 Left Turn Treatments for Large Vehicles

The extent of roadway required to accommodate large vehicles at BAL turn treatments can become large, creating an undesirable situation for smaller vehicles. The correct path of travel for the smaller vehicles becomes unclear, and the potential for two lane operation is created.

A solution to this problem is to provide the normal turning roadway for a design service truck (or semi-trailer, as relevant) and provide an additional area for the larger vehicles in a different material separated by a white line and diagonal markings. Figure 13.85 to Figure 13.88 illustrate the approach to this issue using a CHL turn treatment with a high entry angle. Whilst these figures show urban intersections, similar layouts are also applicable to rural sites.

Details of BAL turn treatments for large vehicles at rural sites are given in Appendix 13E.

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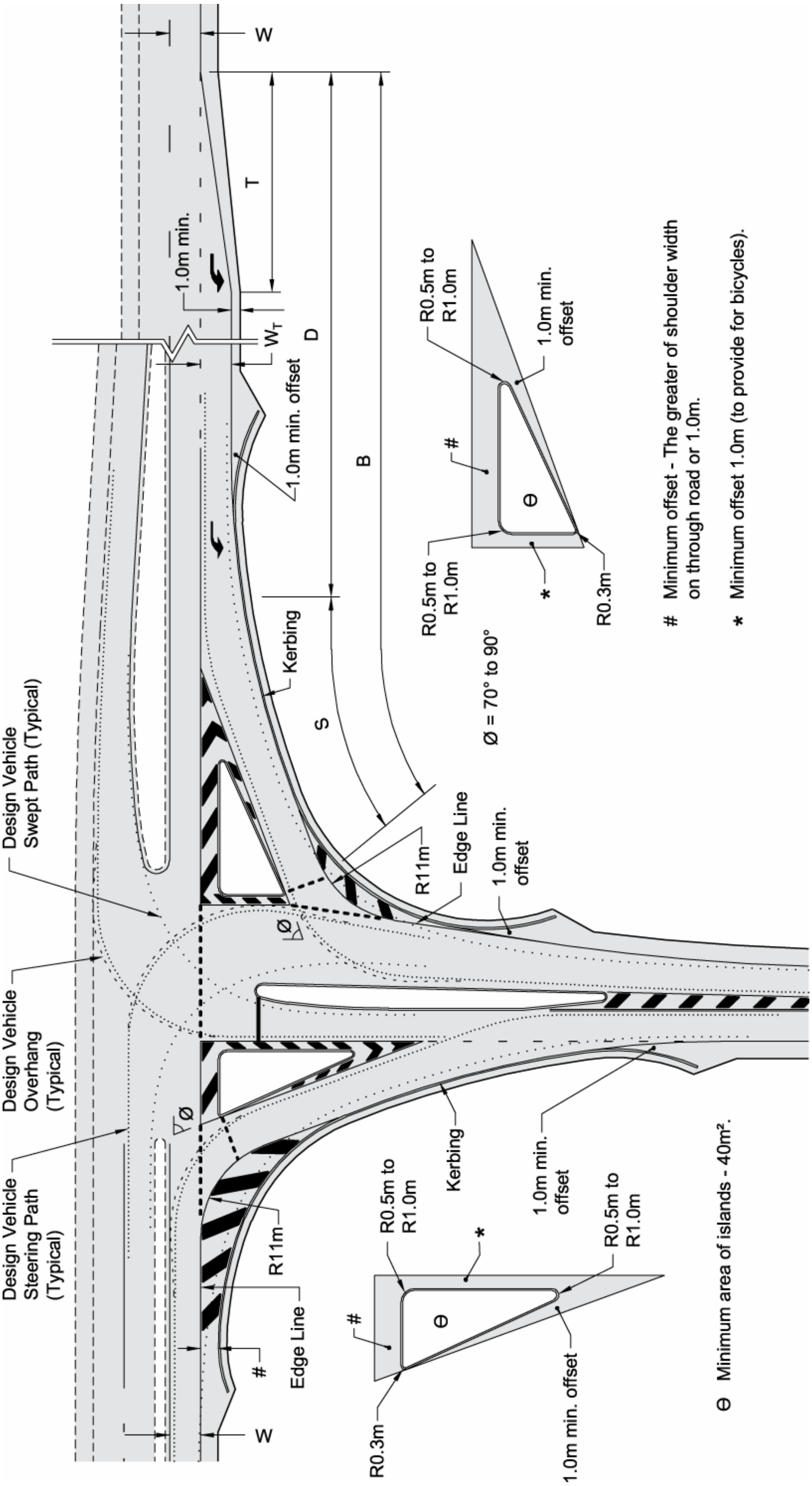
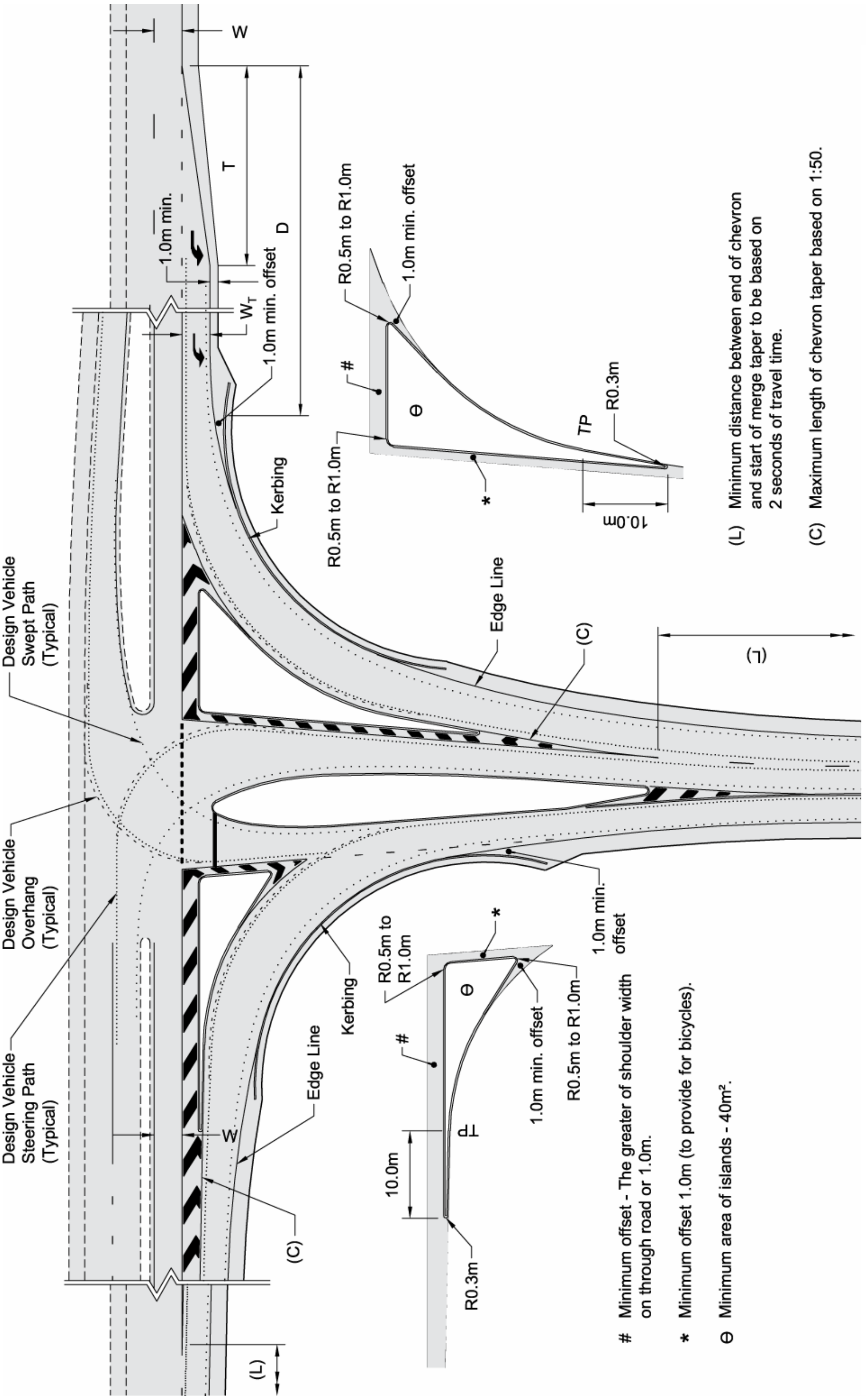


Note 1: Approaches to left turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. Treatments to reduce the number of potential conflicts at left turn slip lanes are given in Austroads (1999b).

Note 2: Figure 13.41 details clearances required at a CHL turn treatment where there are no specific cyclist facilities.

Note 3: Refer to Figure 13.60 (CHR turn treatment) for details of the dimensions T, D, S, B, W and W_T.

Figure 13.83 Channelised Left Turn Treatment (CHL) with a High Angle Entry on a Rural Road



Note 1: Approaches to left turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. Treatments to reduce the number of potential conflicts at left turn slip lanes are given in Austroads (1999b).
Note 2: Figure 13.41 details clearances required at a CHL turn treatment where there are no specific cyclist facilities.
Note 3: Refer to Figure 13.60 (CHR turn treatment) for details of the dimensions T, D, S, B, W and W_T.

Figure 13.84 Channelised Left Turn Treatment (CHL) with Acceleration Lane on a Rural Road

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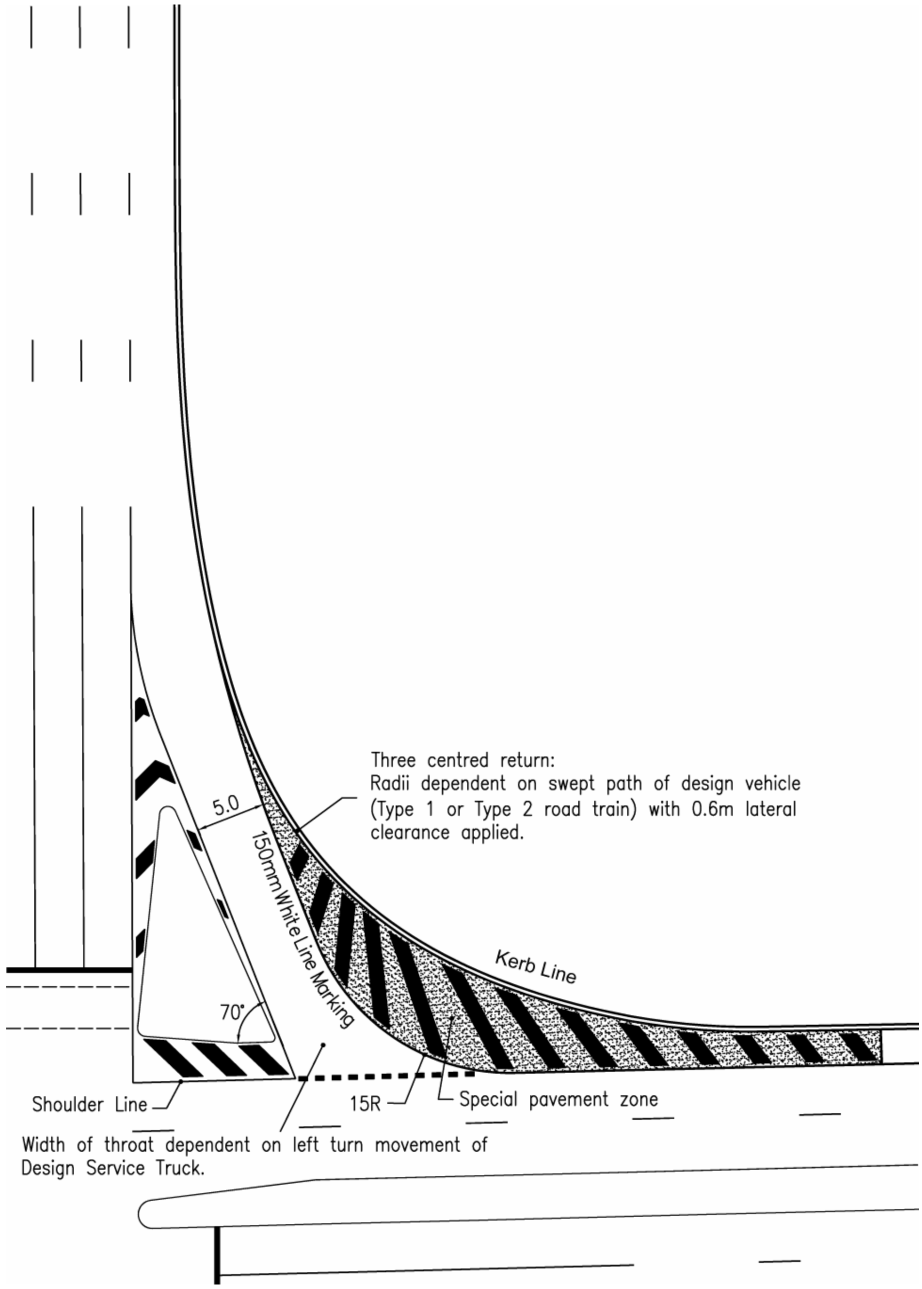


Notes:

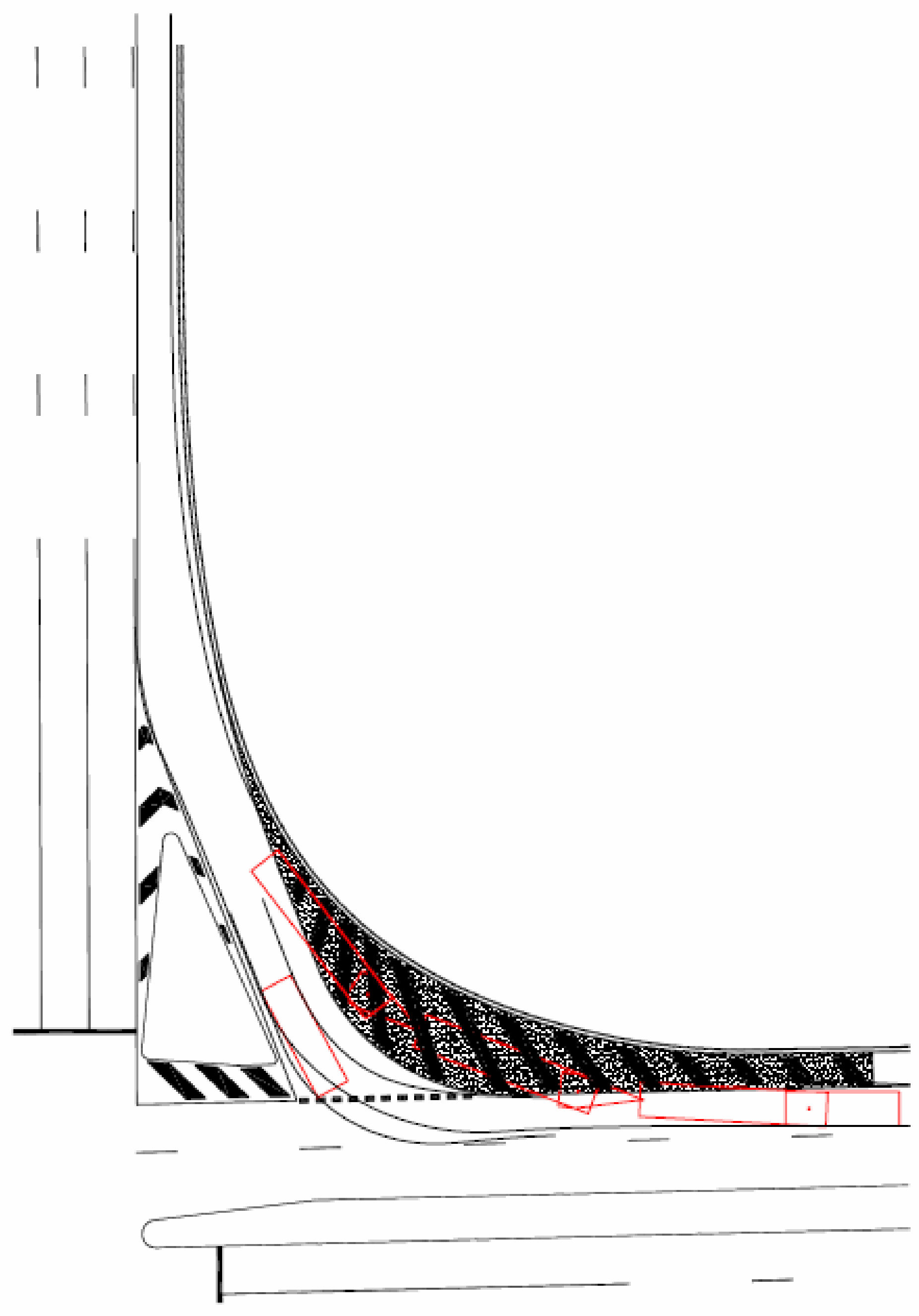
This treatment is shown for an urban site. A similar layout is also applicable to rural sites.

This treatment promotes a desirable observation angle for all vehicle types if drivers minimise any encroachment onto the special pavement zone. However, since large SU trucks, Prime Movers & Semi-Trailer combinations and B-doubles have to encroach onto the special pavement zone, some drivers are likely to exploit this and describe a larger turning radius. These drivers will then have a more difficult observation angle.

Where possible, slight distance requirements should be met at the point prior to the Give Way line where these vehicles have a desirable observation angle.

This intersection treatment assumes that road train operation has been allowed because there is sufficient sight distance to avoid the use of stop signs.

Figure 13.85 Channelised Left Turns (CHL) for Road Trains – Normal Treatment (Basic Setting Out Details)



Note:

This treatment is shown for an urban site. A similar layout is also applicable to rural sites.

Figure 13.86 Channelised Left Turns (CHL) for Road Trains – Normal Treatment (Swept Path Provisions)

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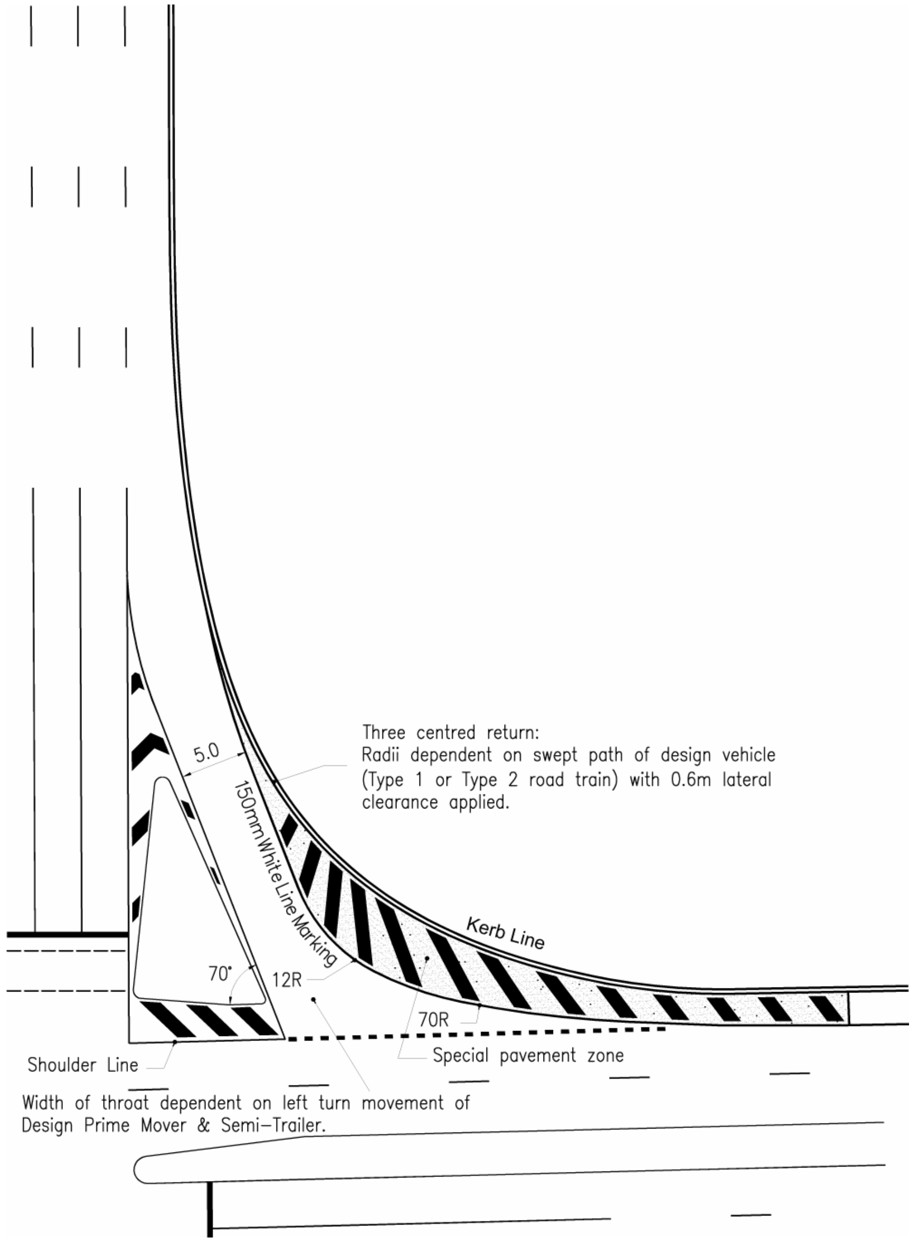


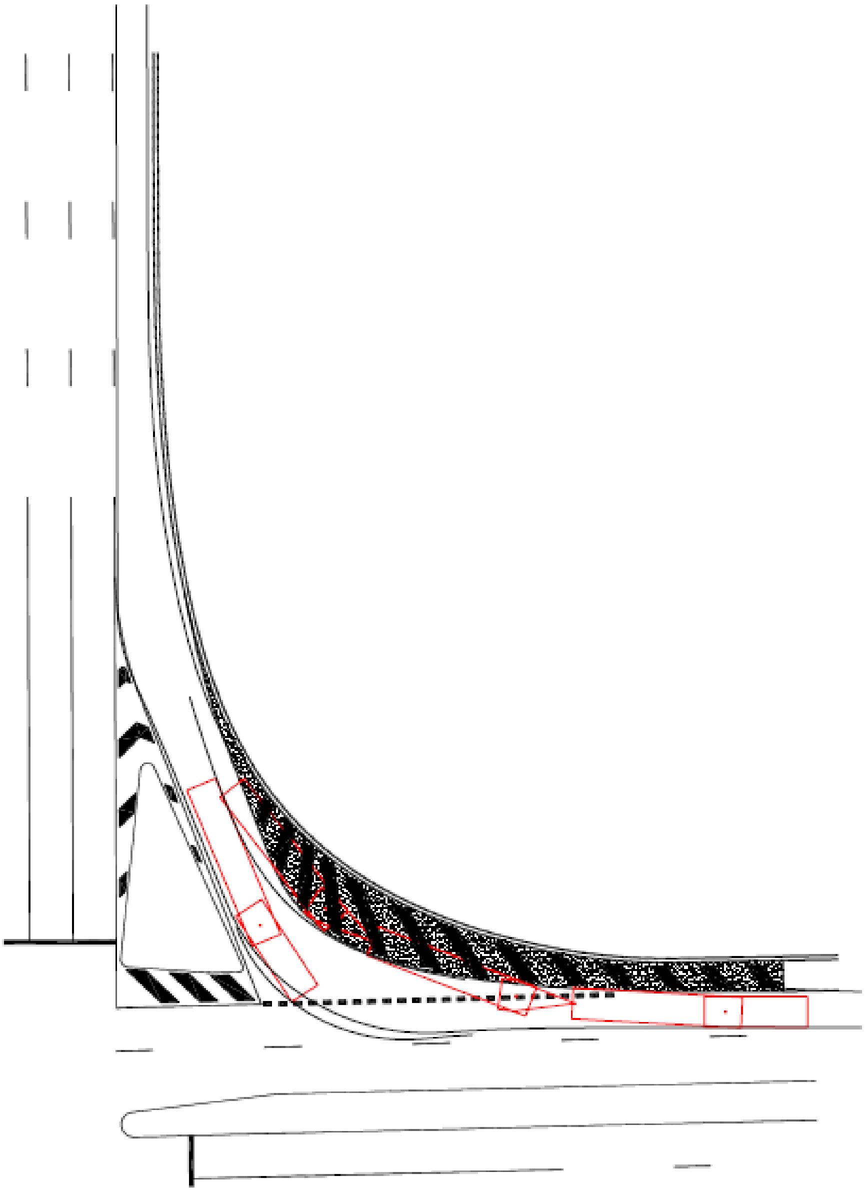
Notes:

This treatment is shown for an urban site. A similar layout is also applicable to rural sites.

This treatment may be used where the volume of large SU trucks and Prime Movers & Semi-Trailer combinations will cause unacceptable maintenance problems for the line marking on the special pavement zone if the normal treatment in Figure 13.85 is used. However, cars and smaller trucks are more likely to describe a turning radius that results in a difficult observation angle if stopped at the Give Way line. Where possible, sight distance requirements should be met at the point prior to the Give Way line where these vehicles have a desirable observation angle.

This intersection treatment assumes that road train operation has been allowed because there is sufficient sight distance to avoid the use of stop signs.

Figure 13.87 Channelised Left Turns (CHL) for Road Trains – Alternative Treatment for Areas where there is a High Volume of Large SU Trucks and Prime Mover & Semi-Trailer Combinations (Basic Setting Out Details)



Note:

This treatment is shown for an urban site. A similar layout is also applicable to rural sites.

Figure 13.88 Channelised Left Turns (CHL) for Road Trains – Alternative Treatment for Areas where there is a High Volume of Large SU Trucks and Prime Mover & Semi-Trailer Combinations (Swept Path Provision)

13.8 Median Cross-overs

13.8.1 General

Principles of intersection design as defined in previous parts of this guide also apply to median crossovers. Sight distance, design vehicle turning paths and interference to through traffic by decelerating and accelerating vehicles should be considered at all median crossovers.

On divided carriageway rural roads, the number of points of conflict can be minimised by incorporating median cross-over facilities with property access points.

13.8.2 Divided carriageways

On motorways, median crossovers are provided primarily for use by emergency services and should be signposted as such. On rural non-motorway type roads they operate as a general U-turn facility allowing access to the opposing carriageway for property owners as well as emergency services.

Median crossovers for emergency or access purposes should be provided at 3-5 km intervals. They should not be within 3km of an interchange or intersection.

The dimensions and treatment of the median crossover will be governed by the median width, the presence of median safety barrier, the type of vehicle using it and whether the site is in a high speed or low speed environment.

In determining the location of a median cross-over the following points should be considered:

- Desirable locations are sags and straights where sight distance is the greatest.
- Undesirable locations are on horizontal curves or crests.
- Maximum spacing should be 5km, but a cross over should not be within 3km of an intersection or interchange.
- Ideal locations are opposite property access points on non-freeway type roads. On motorway type roads, consider locating emergency telephone bays opposite median cross over points where practical – see Figure 13.89
- On new projects the adjacent median planting should be restricted to relatively low vegetation to enhance driver sight distance.
- On existing roads, crossovers should not be installed in heavily planted medians unless corrective measures can be carried out to ensure adequate sight distance is available.
- Crossovers should be located immediately downstream of median gully pits (for gully pits on grade) to eliminate the need for installation of a pipe and associated headwalls which are a potential hazard to errant vehicles.
- On wide medians a desirable slope of 1 on 10 (1 on 6 maximum) between the cross-over and the median invert (measured longitudinally) should be adopted.
- The design through vehicle is also the design vehicle used for the median crossover. Minimum design vehicle for a rural median crossover is a 19m semi trailer. Minimum design vehicle for an urban median crossover is a 12.5m SU truck.

A driver having a need to use a cross-over should be able to recognise that the cross-over exists from at least 10 seconds of travel in either direction.

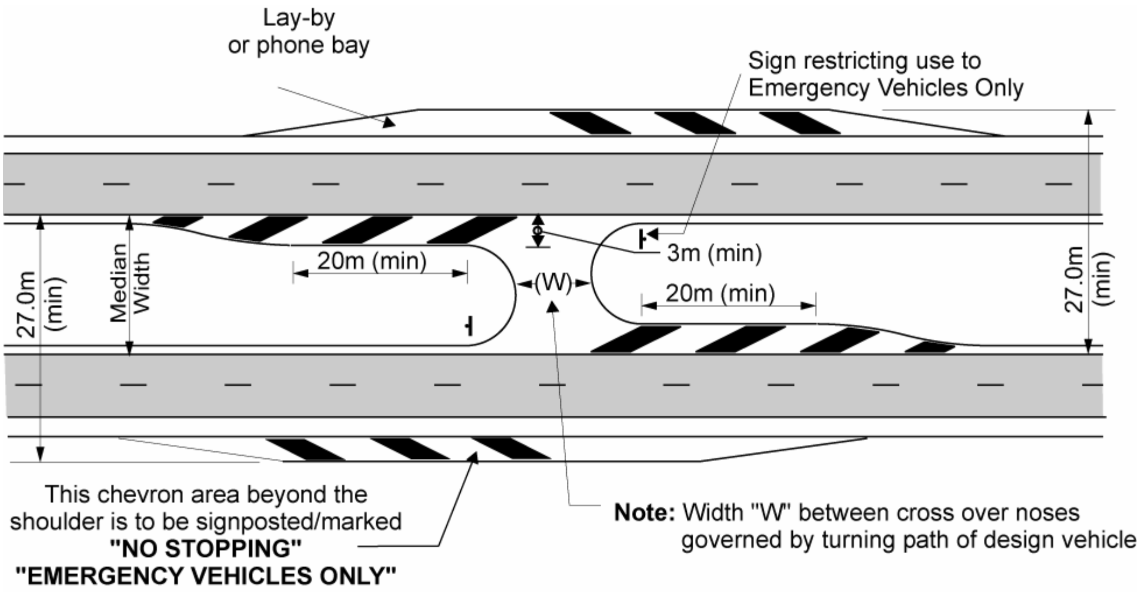


Figure 13.89 Median Cross-over Facilities on Rural Freeways – Emergency Vehicles Use

13.8.2.1 High Speed Rural Roads

High speed divided carriageway roads in rural areas fall into two categories:

- rural motorway; and
- rural arterial road with access control.

Emergency vehicles require access to the adjacent carriageway at rural sites to:

- attend motor vehicle accidents/medical emergencies;
- tow broken down vehicles to nearest town;
- clean/retrieve spilt chemicals; and
- access fire trails.

On rural motorways, the median crossover should be designed for articulated vehicles to allow ready access for emergency vehicles to turn such vehicles around (broken down; retrieving of toxic chemicals from a damaged tanker). The alternative would be to use the nearest multi-directional interchange, which could be up to 20km away.

The median cross-over facility on rural freeways should be located adjacent to emergency telephone bays and/or lay-bys.

On access controlled rural arterial roads, efforts should be made to incorporate the median cross-over with a property access point, “T” intersection, or lay-by.

Generally rural roads with divided carriageways have wide medians. A 27.0m offset covering the median and the adjacent carriageway is the minimum width required to allow an emergency vehicle to turn. This allows the minimum radius required for a 19.0m articulated vehicle turning at speeds of up to 5km/h. Where the median is narrow, a lay-by will need to be constructed on the adjacent carriageway’s nearside shoulder. Figure 13.89 and Figure 13.90 give details of median cross-overs in rural locations.

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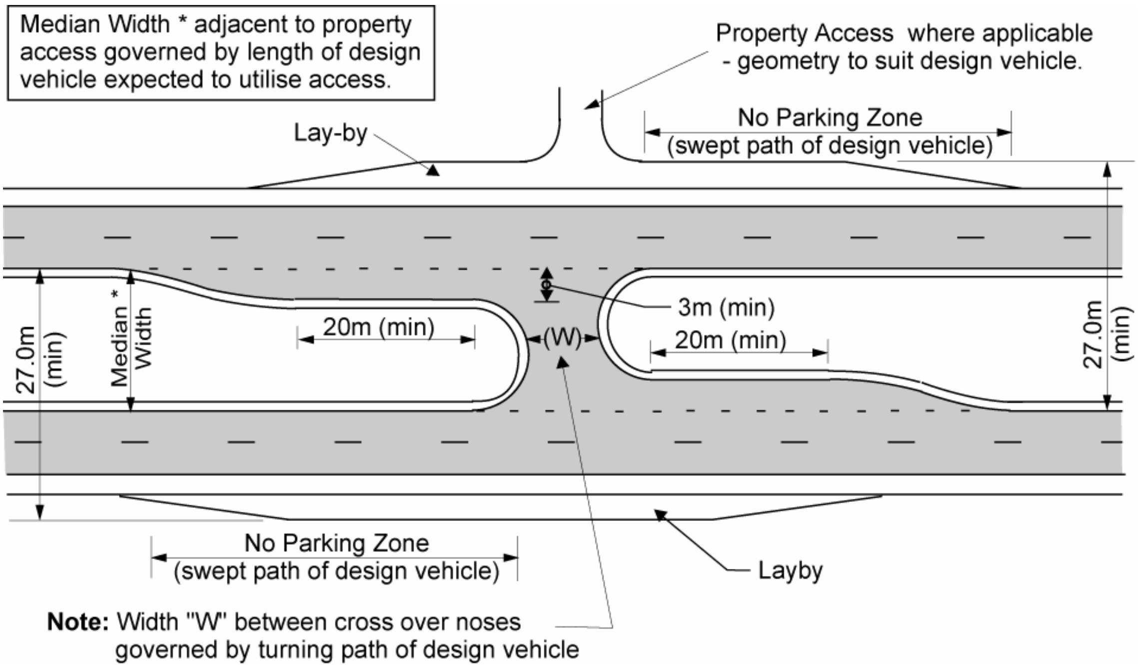

Figure 13.90 Median Cross-over Facilities on Divided Carriageways – General Vehicle Use to Access Properties

13.8.2.2 High Speed Urban Motorways

Emergency vehicles require access to the adjacent carriageway on these roads mainly to:

- attend motor vehicle accidents/medical emergencies;
- tow away broken down vehicles; and
- clean / retrieve spilt chemicals.

Urban freeways have a tendency towards narrower medians than their rural counterparts. The carriageways are usually separated by median safety barrier. The safety barrier can range from rigid concrete in restricted areas to non-rigid steel rail or wire rope systems in wider medians.

The treatment of the break in the safety barrier will need to be considered in the median cross-over design. The break could incorporate an approved crash attenuating device or opening system for concrete barrier (see Chapter 8 for details), or in non rigid systems by positioning the terminals to minimise the gap allowing access to the adjacent carriageway. See Figure 13.91 for details.

Due to the closer proximity of interchanges in the urban area and the desire to restrict the width of median opening, the median cross-over at these sites is designed for a single unit truck (fire engine, ambulance, tow truck). The cross-over should be located adjacent to a layby, bus zone or emergency telephone bay so that the turning vehicle can take advantage of the widened formation.

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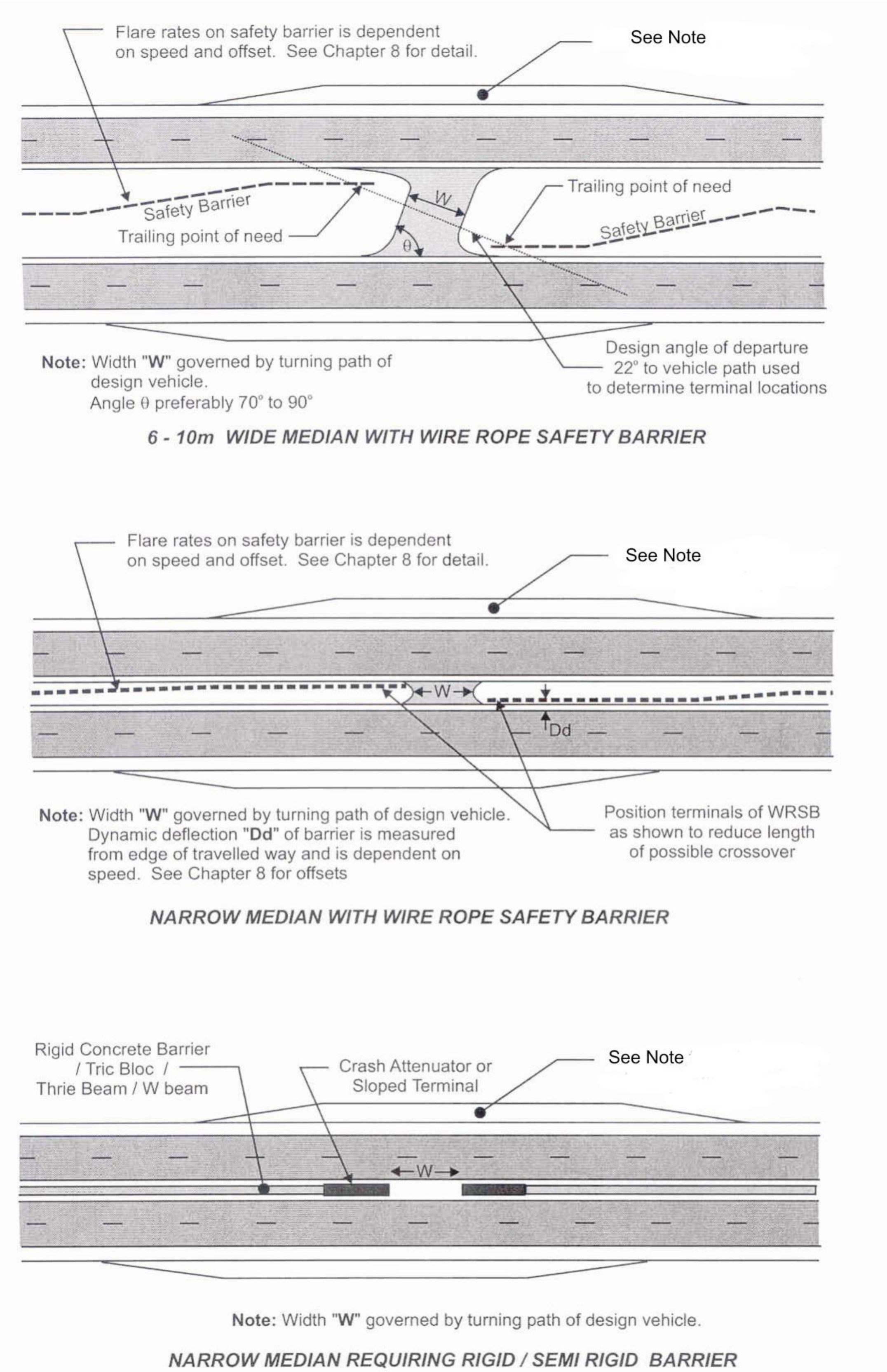

Note: See Figure 13.90 for details where widened formation is required.

Figure 13.91 Narrow Medians with Safety Barrier – Median Cross-overs for Emergency Purposes

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13.8.2.3 High Speed Urban Divided Roads

High speed (70 - 90 km/h) urban arterial roads serve as main connectors between suburban centres. Intersections are widely spaced. Direct property access is controlled and where available usually limited to left turn entry and exit. Where intersections are spaced over 1 km apart, median cross-overs should be provided at 400-800m intervals.

Where the median does not incorporate a physical restraint (eg. safety barrier, pedestrian fence) and the kerb is semi mountable, introduced breaks (low ground covers) can be made within the landscaping at appropriate intervals to allow for access by emergency vehicles to adjacent offside properties.

13.8.2.4 Low Speed Urban Divided Roads

Low speed urban roads (<70km/h) serve residential, commercial and industrial zones. Emergency vehicles require access to the adjacent carriageway to attend motor vehicle accidents, overtake queued traffic and access adjacent properties for medical emergencies, fires, evacuations etc.

Due to the proximity of intersections, formed median crossovers are not required. Except where pedestrian fencing/safety barrier is installed or there is a significant level difference, the median should be traversable with flattened batters, semi-mountable kerb and low growth vegetation.

13.9 Property Accesses

13.9.1 General

The principles of intersection design in the previous parts of this guide also apply to property accesses. Sight distance, design vehicle turning paths and interference to through traffic by decelerating and accelerating vehicles should be considered at all sites.

Property access off major roads should be limited and widely spaced. It is preferable to consolidate multiple access points to a single point of entry and exit. Existing minor side streets or service roads should be utilised as the main point of exiting and entering the major road from a traffic generating development.

13.9.2 Sight Distance at Accesses

Desirably, sight distances at accesses should comply with the sight distance requirements for intersections, as given in Section 13.6.4. Section 13.6.4 requires Approach Sight Distance (ASD), Safe Intersection Sight Distance (SISD), and Minimum Gap Sight Distance (MGSD) to be achieved.

The criteria above often cannot be obtained at accesses on roadways with tighter horizontal and vertical alignments. For new roads comprising such geometry, minimum sight distances at accesses should comply with the following:

- Minimum Gap Sight Distance in Section 13.6.4.4; and
- Safe Intersection Sight Distance using values given under the Extended Design Domain (EDD) Criteria for Sight Distance at Intersections (refer Table 13.17, Table 13.18 and Table 13.19 in Section 13.10.1).

Obtaining ASD at domestic accesses is often not necessary due to the familiarity of their location by the users. At other than domestic accesses, ASD will need to be provided only if adequate perception of the access is not provided through other means.

Care should be taken to ensure that the minimum sight distances are not restricted by the location and height of roadside furniture and vegetation.

13.9.3 Urban Property Access

13.9.3.1 General

Urban property accesses vary from entrances to major developments, such as regional shopping centres, to driveways for individual residential houses. The right of access to the property and the subsequent entry and exit layout is dependent on a number of factors, including:

- type of frontage road;
- land use of the property;
- development approval by the local authority;
- type of vehicle likely to use the access; and
- average daily traffic using the access.

Entrances to major developments such as shopping centres need to be analysed thoroughly and designed to minimise their effect on the through traffic flow. The intersection of this type of access with the major road is to be treated in the same way as the intersection of two public roads. Grade separated right turns, right turn bays and deceleration lanes may need to be provided in some circumstances.

In commercial zones, consolidate access points. By rule of thumb each additional commercial access on a 4 lane urban road adds 5-10 accidents / 1 x 10^8 vehicle km at low access densities (less than 10 access points per kilometre). In high access density areas each additional access can increase accidents by 15-20 accidents / 1 x 10^8 vehicle km.

The width of the entrance or driveway, and the layout of the turnout should:

- provide single manoeuvre turns by the design vehicle;
- provide adequate clearance between the design vehicle's turning path and physical constraints within the property;
- avoid reversing movement into or out of the development (except in the case of individual residential houses);
- provide safety for pedestrians by ensuring adequate sight distance;
- provide adequate room within the development to minimise the risk of traffic being stored or being backed up on the state controlled road network; and
- minimise pedestrian / vehicle conflict areas and control vehicle speed across footways.

Details of the design of the footpath cross over for accesses are provided in Chapter 7. Further discussion on property access is included in Chapter 4.

13.9.3.2 Urban Divided Road

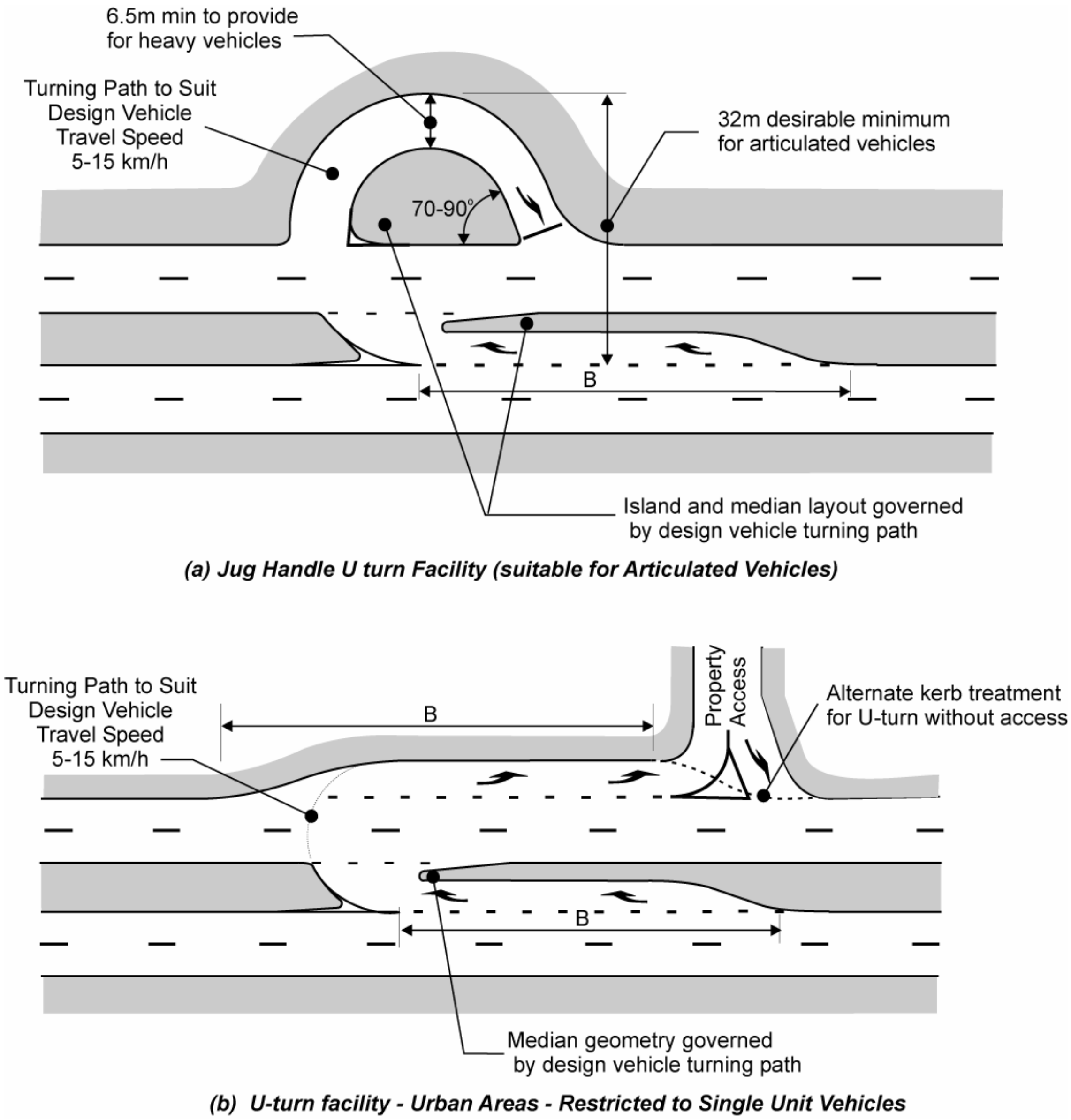
To enhance safety, maintain traffic flow and provide acceptable conditions for pedestrians, the direct access to properties from major urban arterial roads should be limited. Each additional point of access introduces additional area of conflict on the through road.

On arterial routes, new-access points are to be provided on service roads and minor side roads, thereby eliminating multiple conflict points on the major road. A separate service road provides for minimum interference to through traffic on the major road together with safe entry and egress from properties. The provision of a service road is dependent upon an adequate road reserve width.

Major commercial developments, such as shopping centres with mid-block access, may require grade separated right turn movements, deceleration and acceleration lanes or signalised intersections. Intersection analysis will dictate the treatment.

Because the narrow road corridors generally available in urban areas preclude the development of a service road, access to developments should be encouraged via side streets. If the development necessitates direct access onto the major road, the provision of a left turn in / left turn out layout is the preferred treatment. Right turn entry and exit movements should be indirect, requiring detours around the block, sheltered U turn facilities servicing multiple properties or via "U" turns at adjacent roundabouts. Typical sheltered U turn facilities for passenger vehicles and articulated vehicles are shown in Figure 13.92.

Right turn direct access should be limited to situations where the road network layout precludes those measures mentioned above. A separate right turn bay for such an access should be located so that the right turn is a minimum 5 seconds travel distance from the nearest street intersection. An ideal site for this is preceding a signalised intersection where the turning vehicle can take advantage of the gap caused by the intergreen.



Note:

B = Total length of auxiliary lane including taper, diverge / deceleration and storage. Refer to Table 13.11 for diverge and deceleration lengths (adjust for grade using the 'Correction to Grade' table in Figure 13.42).

Figure 13.92 U Turn Facilities for Urban Areas

On secondary arterial roads and collector roads through commercial zones the conversion of raised medians to Two Way Right Turn Lanes (TWRTLs) or Median Turning Lanes can be considered. Median Turning Lanes provide sheltered right turn entry and egress at multiple access points while minimising delay to the through traffic. See Section 13.7.9.6 for more information and layout detail.

13.9.3.3 Urban Undivided Multi-Lane Road

The accident rate for urban undivided four lane roads is high due to the number of conflict points arising from uncontrolled right turn movements at both minor side street intersections and access points.

On arterial roads with high through traffic volumes, there should be limited direct access points for traffic generating developments such as high density strata units or takeaway fast food outlets. Steps should be taken in the strategic planning stage to control the development of access points along the route.

Right turns can be eliminated by using double barrier lines (making it illegal to cross under the 1999 Australian Road Rules) or incorporating a minimum width raised median by narrowing the through lanes and providing a physical barrier. Access via side streets and/or service roads abutting the rear of the properties is preferred. This reduces vehicle conflict points and enhances pedestrian safety in areas of high pedestrian activity e.g. commercial centres and strip shopping zones.

On multi lane roads servicing industrial zones, it may be practicable to narrow or reduce the number of through lanes and incorporate median turning lanes. This type of treatment is also acceptable for arterial roads on the outskirts of rural cities and towns where pedestrian activity is relatively low. The median turning lane gives the turning vehicles shelter with the added safety of increased separation between opposing traffic flows.

13.9.3.4 Urban Two Lane Two Way Road

On Arterial Roads passing through commercial centres, vehicular access to properties is undesirable due to the potential for conflict with pedestrians on crossing footpaths. Local Streets are usually two-lane two-way roads and the preferred location for property access points. Low travel speed and driver expectation of interference reduces the likelihood of conflict. Potential conflict with pedestrian movement must be identified and appropriate solutions adopted.

Restrictions should be placed on right turning movements that comprise insufficient visibility. This can be achieved through:

- linemarking with double barrier lines lines or painted medians;
- providing isolated raised medians as a physical barrier to the turn.

13.9.4 Rural Property Access

13.9.4.1 General

On rural roads, although there are low turning traffic volumes to widely spaced access points to developments, high speed accidents occur due to low driver expectation of turning vehicles. Treatment of access to rural properties is dependent upon several criteria including through traffic volumes, turning volume and vehicle type, single or divided carriageway, land use and general topography.

To enhance safety for the turning vehicle and minimise interference to through traffic, a widened shoulder or short auxiliary lane can be provided for right turning vehicles on dual carriageways. Similarly on two-lane two-way roads shoulder widening as shown for the type BAL and BAR intersections will enhance safety for all movements.

A desirable location for a layby is adjacent to a property access point. It has the additional benefits of acting as a school bus stop and an off-road shelter for mail service providers without the need to provide for additional facilities.

13.9.4.2 Access Location

The location for the point of access will be governed by the following:

- sight distance;
- storage space / median width;
- design vehicle likely to utilise the facility;
- distance to intersection;
- possible confusion with other intersections;
- deceleration / acceleration movements;
- drainage; and
- site restrictions.

Access points off the high speed road should be reduced by either:

- consolidating multiple accesses into one; or
- using existing side roads, right-of-ways and service roads.

13.9.4.3 Access Design

The minimum layout for a rural property access is shown in Figure 13.93. The layout caters for a single unit truck to undertake left turns without crossing the centreline. It will allow for turns by articulated vehicles (providing they use two lanes of a carriageway). This minimum layout is suitable for the following conditions:

- Accesses not used by articulated vehicles on single carriageway roads;
- Left-in, left-out accesses not used by articulated vehicles on dual carriageway roads; and/or
- Dedicated commercial vehicle accesses with infrequent use by articulated vehicles (no more than about one turning articulated vehicle per day) on single carriageway roads with an AADT of less than 2000, provided that the access is **<u>not</u>** associated with other minimum criteria eg sight distance restrictions or tight horizontal curves.

Any dedicated commercial vehicle access used by articulated vehicles that does not meet the conditions in the third dot point above must be designed to allow for a 19m semi-trailer to undertake left turns without crossing the centreline, as given in the layouts shown in Figure 13.94 and Figure 13.95.

At locations where there is high demand for articulated vehicles (eg timber mill, quarry, transport facility etc) a road intersection layout should be adopted.

13.9.4.4 Storage

Where a gate restricts access to a property there should be sufficient length between the edge line and the gate to store one parked design vehicle to allow for the occupants attending the gate.

Storage lengths are:

- 8m - car as design vehicle (only applicable to some rural residential properties);
- 15m - single unit truck as design vehicle; and
- 22m - articulated vehicle as design vehicle.

A stock grid is the preferred control on the boundary as shown in Figure 13.95 and Figure 13.93.

Where an access on a dual carriageway incorporates a median cross-over there should be provision for storage of the design vehicle as shown on Figure 13.94 so that it does not protrude onto the through lanes.

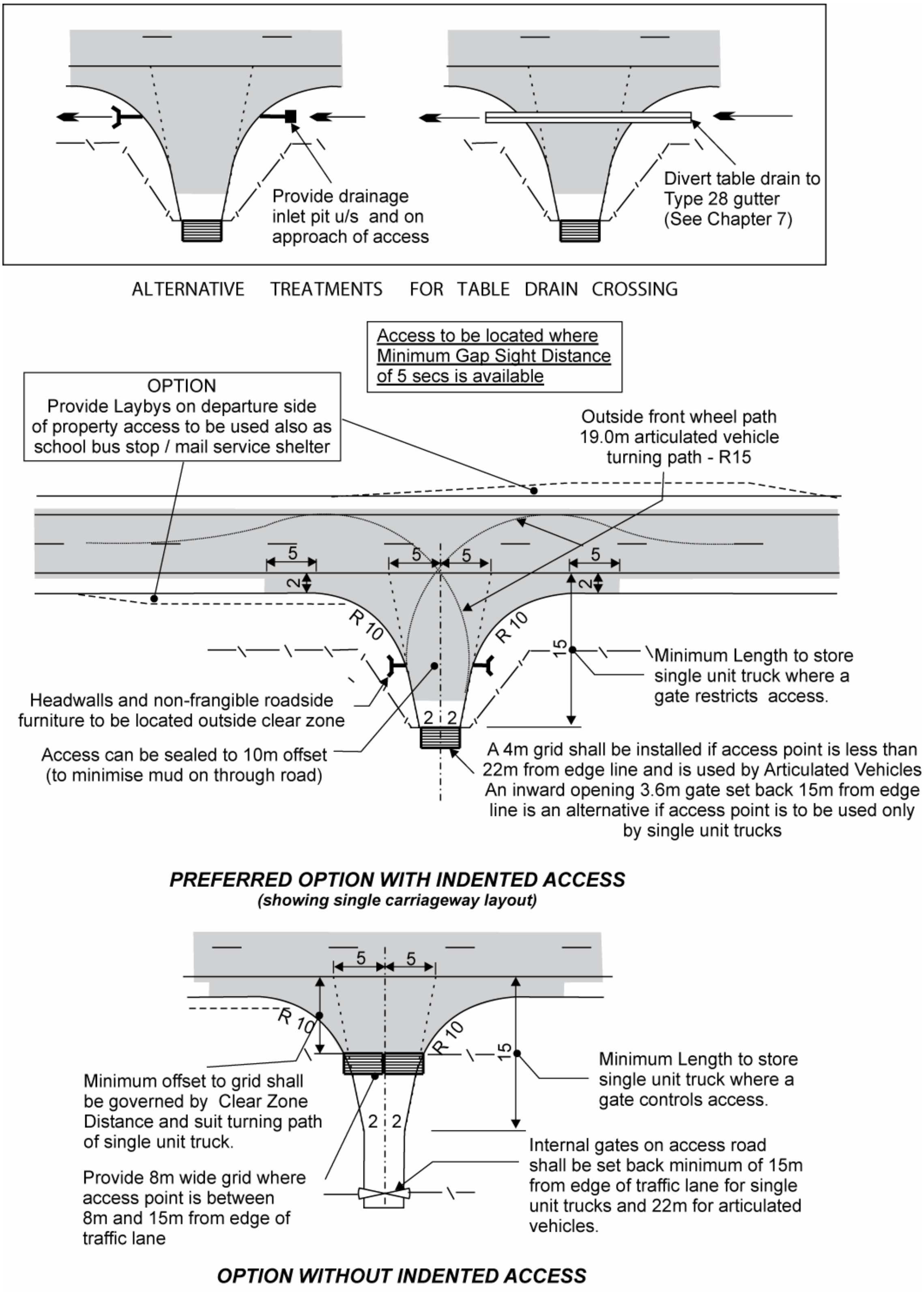


Figure 13.93 Rural Property Access that is not used by Articulated Vehicles (on Single and Dual Carriageway Roads) or has Infrequent use by Articulated Vehicles on Single Carriageway Roads with an AADT <2000.

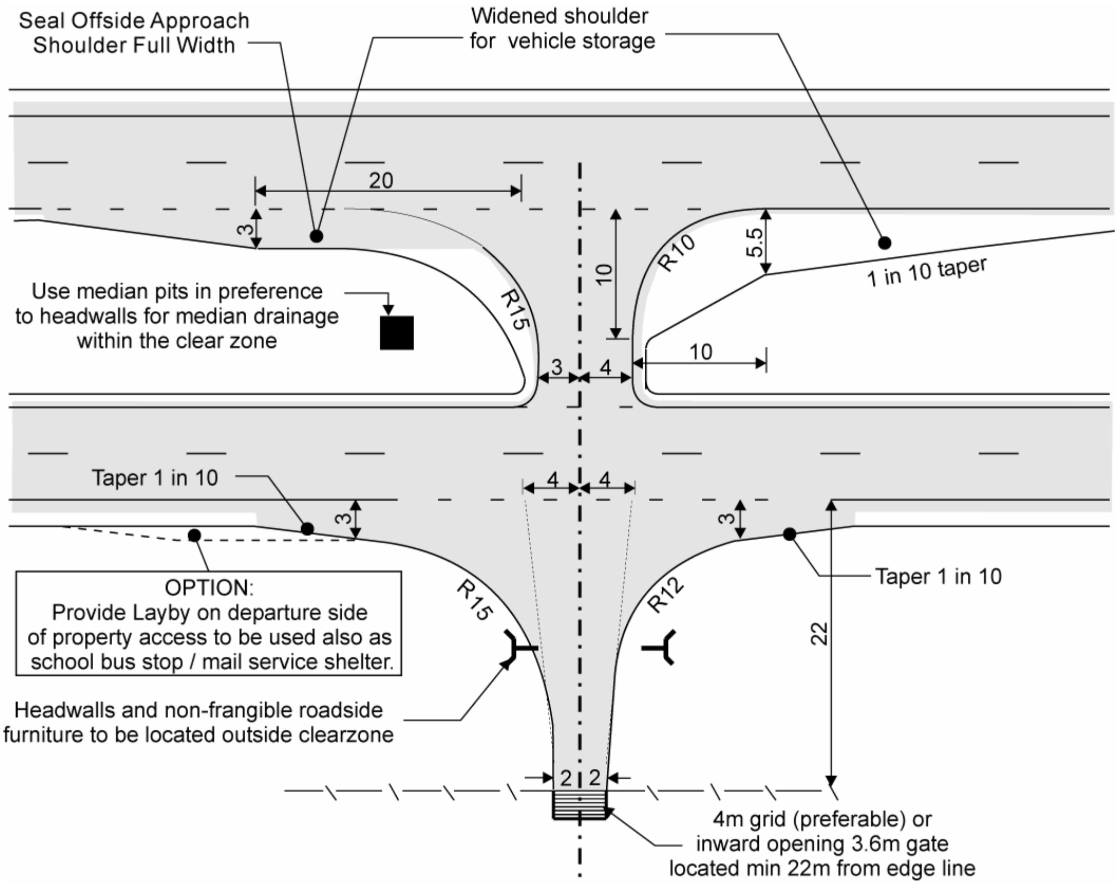


Figure 13.94 Rural Property Access Specifically Designed for Articulated Vehicles on a High Speed Dual Carriageway Road

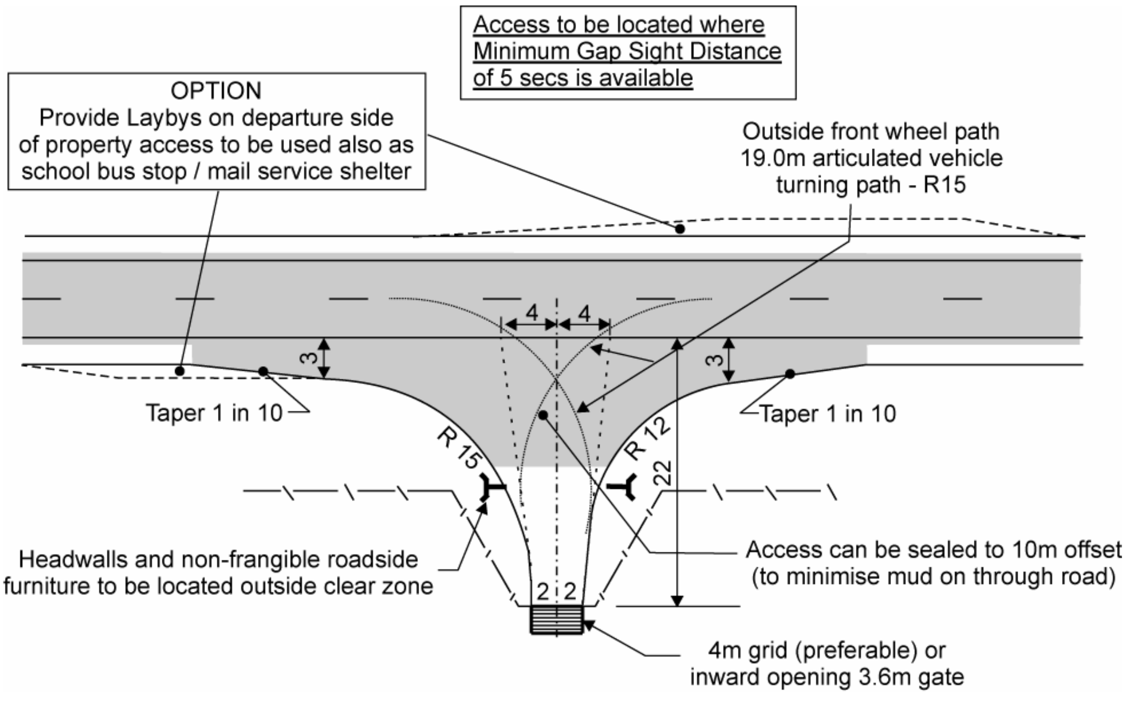


Figure 13.95 Rural Property Access Specifically Designed for Articulated Vehicles on a Single Carriageway Road

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13.9.4.5 High Speed Divided Road

A “left in / left out” layout is the preferred option for property access on a dual carriageway road. Connection to the opposing carriageway is accomplished via a U-turn at adjacent median cross-overs or other facilities. Ideally the location of median cross-overs is based on the side roads and accesses with higher demand so the need for additional cross median accesses should be minimal.

Direct right turn access should only be provided at access points where the daily trip demand is high or there is an expectation of regular articulated vehicles, eg a dairy farm or a right of way serving several properties. They can be considered where safety factors justify an additional access within 3km of a U-turn facility. Safety will be improved by the easier manoeuvrability of articulated vehicles at a direct access in contrast to the 180° turn on a minimum 12.5m radius turning path required at a U-turn facility.

Where there is high demand for articulated vehicles, a road intersection layout incorporating a CHR for the right turn movement and a BAL for the left turn should be adopted. See Figure 13.60 and Figure 13.80 for details.

13.9.4.6 High Speed Single Carriageway

Figure 13.95 and Figure 13.93 show layouts for single carriageway roads. To enhance safety or cater for higher volumes consideration can be given to the inclusion of BAR and BAL layouts.

13.10 Extended Design Domain (EDD) Criteria at Intersections

This section provides intersection design criteria that are outside the bounds of the normal design domain (ie within the extended design domain). The criteria are particularly relevant when reviewing the geometry of existing intersections. Refer to Chapter 4 for further guidance on the appropriate use of these criteria. A summary is given in Cox and Arndt (2005).

Application of the Extended Design Domain involves identification and documentation of capability. Ultimately, the capabilities that are accepted may have to pass the test of what is reasonable capability (the capability that a court decides a motorist can reasonably expect when they are taking reasonable care for their own safety.)

13.10.1 EDD for Sight Distance at Intersections

13.10.1.1 Intent of using EDD Sight Distance at Intersections

In line with the intent of the sight distance requirements for a new road, application of the Extended Design Domain for sight distance at intersections is about ensuring that a reasonable and defendable sight distance capability is provided for all approaching, entering, crossing and turning vehicles. For the Extended Design Domain stopping capabilities on all roads, including through intersections, refer to the "Guide for the Extended Design Domain for stopping sight distance" in Appendix 4B.

13.10.1.2 Application of EDD Sight Distance at Intersections

The Extended Design Domain for sight distance at intersections is primarily for assessing the sight distance capability at existing intersections. However it can be applied to special cases of new work, for example:

- upgrading sight distance at existing intersections; or
- where a new intersection must be installed on an existing road and it is not possible to achieve all normal design domain criteria

What is a reasonable sight distance capability at intersections depends upon the traffic volume, traffic characteristics and road function. For example, a traffic stream with a high proportion of heavy vehicles will warrant the provision of a greater sight distance capability.

The use of Extended Design Domain allows and requires a detailed assessment of sight distance capabilities at an intersection, in line with the predicted operating (i.e. 85^{th} percentile) speeds at all points along the road. It is essential to gain a full understanding of the sight distance capabilities and the extent to which the intents of normal sight distance criteria are achieved. The effects of grade and coincident horizontal curves are more critical. It also involves a careful assessment for cases of possible driver distraction or deception since these may preclude the use of the Extended Design Domain at these locations, or require increased stopping capability.

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13.10.1.3 Use of EDD Sight Distance at Intersections

Using Extended Design Domain, the minimum capability that can reasonably be defended should satisfy ALL of the following conditions:

- It must be at least equal to the maximum value derived from the following:
  - o Approach Sight Distance (ASD) base cases given in Table 13.16 for any one of the following circumstances:
    - important intersections;
    - complex and non-standard intersection layouts;
    - in situations where drivers may be distracted by other features; or
    - where adequate perception of the intersection is not provided through means other than ASD.
  - o Minimum Gap Sight Distance (MGSD) as per Section 13.6.4.4.
  - o Safe Intersection Sight Distance (SISD) base cases given in Table 13.17.
- When a Decision Time (D_T) of less than four (4) seconds is used for the SISD base cases (i.e. Table 13.17), the SISD cases in Table 13.18 should also be checked for satisfactory performance. For borderline cases, checks can also be undertaken using Table 13.19.
- The criteria given in the "Guide for the Extended Design Domain for stopping sight distance" in Appendix 4B.
- The criteria given in Section 13.10.1.4 "General Considerations" and Section 13.10.1.5 "Notes for Horizontal Curves" (given below).

Note that for convenience of documentation of the use of the Extended Design Domain within project reports etc, Table 13.16, Table 13.17, Table 13.18 and Table 13.19 provide the nomenclature for concisely defining the available stopping capability.

13.10.1.4 General Considerations

The following must be considered whenever EDD is applied to sight distance at intersections:

- Zones clear of obstructions, defined by "sight triangles" for each of the appropriate sight distance models, are required at intersections and must be maintained (e.g. resumptions, grass mowing practice/schedule, vegetation maintenance strategy, interference by signage, effect of installation of noise and safety barriers, etc are all taken into account).
- **The minimum stopping capability calculated from the criteria given above can only be justified provided it meets the following conditions:**
  - o **Crash data indicates that there are no sight distance related crashes.**
  - o **It is not combined with any other lower order value for the same element (i.e. only one lower order value per element is justifiable, e.g. if an element only meets minimum standard horizontal curvature in conjunction with minimum standard vertical crest radius it can not be justified).**
  - o Future arrangements/planning must be satisfied. (For example allow for future planning layouts, fencing, safety barriers, noise barriers, etc as

appropriate. This includes taking account of their effects on sight lines/distances [e.g. a noise barrier may reduce horizontal visibility]).

- o Geometric features and other features of the road do not distract drivers.
- o The "Guide for the Extended Design Domain for stopping sight distance", contained in this Appendix 4B, must be used in conjunction with this guide to determine the minimum acceptable standard.

- Horizontal curves and vertical curves should not be considered in isolation. Check sight distances/lines in both the vertical and horizontal planes taking into account both the horizontal and vertical curvature.
- Particular attention must be given to checking truck requirements on routes with high proportions of heavy vehicles. Some capability for trucks should be provided on any road.

Designers and planners should note that for drivers travelling at the operating (85th percentile) speed it is reasonable to assume that they are conscious of their speed (commonly around 10km/h above the speed limit when not constrained by horizontal curvature). Consequently it is likely they will be prepared to brake harder in emergencies.

Designers and planners should also note that providing stopping capability for drivers travelling at the operating (85th percentile) speed will usually cater for less capable drivers (e.g. mean-day). This should not be assumed however; calculations and checks should always be undertaken for all (relevant) cases.

Finally, it should be noted that only stopping for wet conditions is included in this guide. This is because at intersections there is increased exposure and therefore an increased probability that a hazard will be encountered during wet conditions.

13.10.1.5 Notes for Horizontal Curves

Whenever the Extended Design Domain is applied to roads and a horizontal curve exists, whether in isolation or in combination with a vertical curve, the following must be considered:

- Where sight distance is only restricted in the horizontal plane, the height of the object has no effect.
- For any horizontal curve with a side friction factor greater than the desirable maximum (i.e. $f_{des\ max}$ given in Chapter 11 using the operating speed), the co-efficient of deceleration used to calculate any of the stopping sight distance cases (for ASD and SISD) should be reduced by 0.05.
- Consider the likely maintenance strategy and make allowance for it. (For example, if grass is allowed to grow up to the edge of seal the available offset for horizontal sight distance may be only about 3m. Allowance must be made for the height of vegetation, including grass, on benches, in table drains, etc that results from the maintenance strategy.)
- Cars and trucks require different offset values for horizontal sight distance due to differences in:
  - o stopping sight distances; and
  - o the eye position of drivers.

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- The offset required for trucks will be affected by the direction of the curve (i.e. a left turning curve requires a different offset to a right turning curve).
- The adequacy of horizontal curves should also be assessed as discussed in Chapters 6 and 11 (e.g. in terms of side friction demand, geometric consistency, etc).

13.10.1.6 Spreadsheet Tool

A spreadsheet tool has also been developed to assist in the assessment of geometry against Extended Design Domain standards. A copy may be obtained by contacting the Principal Engineer (Road Design Standards).

13.10.1.7 Formulae

Chapter 11 provides formulae for the calculation of offsets required to obtain stopping sight distance around horizontal curves. It also has a graph that can be used to determine this offset.

Chapter 12 provides formulae for the calculation of vertical curve radii required to obtain stopping sight distance for a crest or sag curve.

Section 13.6.4 provides formulae for the calculation of stopping sight distance required at intersections, and accesses (where appropriate).

Where horizontal and vertical curves overlap or coincide it is usually necessary for the designer to determine and check stopping sight distance via plots or Computer Aided Drafting and Design (CADD) packages (rather than formulae).

Table 13.16 Approach Sight Distance Base Cases for the Extended Design Domain

Case Code	Case description	Speed, "V" (km/h)	Reaction Times "R_T" (s)	[1]Coefficient of deceleration, "d"	"h_1" (m)	"h_2" (m)
ASD.	Normal car driver travelling at the operating (85^{th} percentile) speed in daylight hours.	Operating (85^{th} percentile) car speed.	2.5 - isolated element where V >70km/h. 2 - normal cases for roads with V >70km/h. 1.5 - normal cases for roads with V ≤70km/h [2]1.5 - roads with alert driving conditions and V >70km/h	0.61 - predominantly dry area with low traffic volumes. 0.46 -all other cases.	1.15 - eye height of passenger car.	0.0- road surface.

<u>Notes to Table 13.16:</u>

1. Refer to Section 13.10.1.5 "Notes for Horizontal Curves" for the effect of horizontal curves.
2. Must only be used where Designer/Engineer is confident that drivers will be constantly/continuously alert and that there is nothing unusual present that will deceive or distract the driver. For this case the physical and/or built environment should increase drivers' expectation that they will have to react quickly/stop. For example:
   a. a road in a rural area with a horizontal alignment that requires the driver to maintain a high level of awareness due to the presence of a continuous series of curves with a side friction demand > $f_{des\ max}$; or
   b. a road in a heavily built up urban area with many direct accesses and intersections.
3. For convenience, an ASD case (or capability) can be described in terms of the following nomenclature: Case Code-R_T-wet/dry (For example, ASD-2-wet describes the capability corresponding to a normal car driver travelling at the operating [85^{th} percentile] speed in daylight hours with an R_T of 2s when stopping in wet conditions for a 0m high object.)

Table 13.17 Safe Intersection Sight Distance Base Cases for the Extended Design Domain

Case Code	Case description	Speed, "V" (km/h)	[3]Decision Times "D_T" (s)	[2]Coefficient of deceleration, "d"	"h_1" (m)	"h_2" (m)
Norm-day.	Normal car driver travelling at the operating (85^{th} percentile) speed in daylight hours.	Operating (85^{th} percentile) car speed.	4.0 - isolated element where V >70km/h. 3.5 - normal cases for roads with V >70km/h. 3.0 - normal cases for roads with V ≤70km/h [4]3.0 - roads with alert driving conditions and V >70km/h	0.46 - all cases.	1.15 - eye height for driver of passenger car.	1.15 - top of car.
[1]Truck-day.	Truck in daylight hours.	Use speed of average laden design prime-mover and semi-trailer in free flowing conditions.	3.5 - all cases.	0.28 - type 1 road trains 0.26 - type 2 road trains [1]0.29 - all other cases	2.4 - eye height for driver of truck.	1.15 - top of car.

Notes to Table 13.17:

1. These cases cover design single unit trucks, semi-trailers and B-doubles. The deceleration rates given above allow for the brake delay times associated with the air braking systems used on these vehicles.
2. Refer to Section 13.10.1.5 "Notes for Horizontal Curves" for the effect of horizontal curves.
3. D_T = decision time (s) = observation time (s) + reaction time (s).
4. Must only be used where Designer/Engineer is confident that drivers will be constantly/continuously alert and that there is nothing unusual present that will deceive or distract the driver. For this case the physical and/or built environment should increase drivers' expectation that they will have to react quickly/stop. For example:
    a. a road in a rural area with a horizontal alignment that requires the driver to maintain a high level of awareness due to the presence of a continuous series of curves with a side friction demand > $f_{des\ max}$; or
    b. a road in a heavily built up urban area with many direct accesses and intersections.
5. For convenience, a stopping distance case (or capability) can be described in terms of the following nomenclature: Case Code-D_T (For example, Norm-day-4 describes the capability corresponding to a normal car driver travelling at the operating [85^{th} percentile] speed in daylight hours with an D_T of 4s when stopping in wet conditions for a 1.15m high object.)

Table 13.18 Safe Intersection Sight Distance Checks for the Extended Design Domain

Case Code	Case description	Speed, "V" (km/h)	[3]Decision Times "D_T" (s)	[2]Coefficient of deceleration, "d"	"h_1" (m)	"h_2" (m)
Norm-night	Normal car driver travelling at the operating (85th percentile) speed at night	Operating (85th percentile) car speed	[4]2.5 - all cases	0.46 - all cases	[7]0.75 - height of car headlight	[7]1.15 - top of car
[1]Truck-night	Truck travelling at night	Use speed of average laden design prime-mover and semi-trailer in free flowing conditions.	[4]2.5 - all cases	0.28 - type 1 road trains 0.26 - type 2 road trains [1]0.29 - all other cases	1.1 - height of truck headlight	1.15 - top of car
Mean-day	Car driver travelling at the mean free speed in daylight hours	Mean car speed ≈ 0.85 times operating (85th percentile) speed	4.0 - isolated element where V >70km/h. 3.5 - normal cases for roads with V >70km/h. 3.0 - normal cases for roads with V ≤70km/h [5]3.0 - roads with alert driving conditions and V >70km/h	0.41 - all cases	1.15 - eye height for driver of passenger car	1.15 - top of car
Mean-night	Car driver travelling at the mean free speed at night	Mean car speed ≈ 0.85 times operating (85th percentile) speed	3.5 - all cases	0.41 - all cases	[7]0.75 - height of car headlight	[7]1.15 - top of car

Notes to Error! Reference source not found.**:**

1. These cases cover design single unit trucks, semi-trailers and B-doubles. The deceleration rates given above allow for the brake delay times associated with the air braking systems used on these vehicles.
2. Refer to Section 13.10.1.5 "Notes for Horizontal Curves" for the effect of horizontal curves.
3. D_T = decision time (s) = observation time (s) + reaction time (s).
4. In reality, this is a check to ensure that a vehicle on the through road can see and stop for a stalled vehicle.
5. Must only be used where Designer/Engineer is confident that drivers will be constantly/continuously alert and that there is nothing unusual present that will deceive or distract the driver. For this case the physical and/or built environment should increase drivers' expectation that they will have to react quickly/stop. For example:
   a. a road in a rural area with a horizontal alignment that requires the driver to maintain a high level of awareness due to the presence of a continuous series of curves with a side friction demand > $f_{des\ max}$; or
   b. a road in a heavily built up urban area with many direct accesses and intersections.
6. For convenience, a stopping distance case (or capability) can be described in terms of the following nomenclature: Case Code-D_T (For example, Mean-day-4 describes the capability corresponding to a car driver travelling at the mean free speed in daylight hours with an D_T of 4s when stopping in wet conditions for a 1.15m high object.)
7. This also achieves stopping for an eye height of 1.15m (the eye height for driver of passenger car) to an object height of 0.75m (height of car headlight).

Table 13.19 Safe Intersection Sight Distance, Optional Checks for Borderline Cases for the Extended Design Domain

Case Code	Case description	Speed, "V" (km/h)	[2]Decision Times "D_T" (s)	[1]Coefficient of deceleration, "d"	"h_1" (m)	"h_2" (m)
Skill-day	Skilled car driver travelling at the operating (85th percentile) speed in daylight hours	Operating (85th percentile) car speed	3.0 - all cases	0.56 - all cases	1.15 - passenger car	1.15 - top of car
Skill-night	Skilled car driver travelling at the operating (85th percentile) speed at night	Operating (85th percentile) car speed	3.0 - all cases	0.56 - all cases	[4]0.75 - height of car headlight	[4]1.15 - top of car

Notes to Table 13.19:

1. Refer to Section 13.10.1.5 "Notes for Horizontal Curves" for the effect of horizontal curves.
2. D_T = decision time (s) = observation time (s) + reaction time (s).
3. For convenience, a stopping distance case (or capability) can be described in terms of the following nomenclature: Case Code-D_T (For example, Skill-day-3 describes the capability corresponding to a skilled car driver travelling at the operating [85th percentile] speed in daylight hours with an D_T of 3s when stopping in wet conditions for a 1.15m high object.)
4. This also achieves stopping for an eye height of 1.15m (the eye height for driver of passenger car) to an object height of 0.75m (height of car headlight).

13.10.2 EDD for Sight Distance at Domestic Accesses

13.10.2.1 Intent of using EDD Sight Distance at Domestic Accesses

In line with the intent of the sight distance requirements for a new road, application of the Extended Design Domain for sight distance at domestic accesses is about ensuring that a reasonable and defendable sight distance capability is provided for vehicles entering and exiting a domestic access. A domestic assess is one that services three or less domestic units.

13.10.2.2 Application of EDD Sight Distance at Domestic Accesses

The Extended Design Domain for sight distance at domestic accesses is primarily for assessing the sight distance capability at existing accesses. However it can be applied to special cases of new work, for example:

- upgrading sight distance at existing accesses; or
- where a new access must be installed on an existing road and it is not possible to achieve all normal design domain criteria.

13.10.2.3 Use of EDD Sight Distance at Domestic Accesses

Using Extended Design Domain, the minimum capability that can reasonably be defended should satisfy ALL of the following conditions:

- It must be at least equal to the maximum value derived from the following:
  - o Minimum Gap Sight Distance (MGSD) as per Section 13.6.4.4, using both an eye height and object height equal to 1.15m.
  - o Safe Intersection Sight Distance (SISD) base cases given in Table 13.17 (SISD for intersections using EDD), using a Decision Time (D_T) of 0.5 seconds less than those given in the table.
- When a Decision Time of less than 3.5 seconds is used in Table 13.17, the SISD cases in Table 13.18 should also be checked for satisfactory performance. For borderline cases, checks can also be undertaken using Table 13.19. In all cases, it is permissible to use a decision time of 0.5 seconds less than those given in the tables.
- The criteria given in the "Guide for the Extended Design Domain for stopping sight distance" in Appendix 4B.
- The criteria given in Section 13.10.1.4 "General Considerations" and Section 13.10.1.5 "Notes for Horizontal Curves".

Normally, the provision of ASD at domestic accesses is not necessary due to the familiarity of their location by the users.

13.10.2.4 Domestic Accesses with less than EDD Sight Distance

On road upgrade projects, it is preferable that at least EDD sight distance is provided at all accesses. Sometimes, financial and/or practical constraints may dictate that this cannot be achieved at a particular access/s. Where this occurs, it forms a 'Design Exception' and must be justified and documented as discussed in Chapter 2.

When design exceptions are used, the installation of mitigating treatments must be considered in order to offset any potential

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safety problems. At accesses with less than EDD sight distance, the following may be appropriate mitigating treatments in some instances:

- Reduction of the speed limit;
- Introduction of local area traffic management devices;
- Installation of signs warning of the limited sight distance;
- Banning of particular movements at the access by the provision of medians/linemarking;
- Installation of a higher order turn treatment into/out of the access; and/or
- Installation of a mirror/s opposite the access.

13.10.3 EDD for Intersection Turn Treatments

13.10.3.1 Intent of EDD Turn Treatments

The intent of using the EDD turn treatments in this section is to maximise the use of Channelised Right-turn (CHR) and Auxiliary Left-turn (AUL) treatments at existing intersections in order to improve safety. Arndt (2004) has shown that these turn types are considerably safer than other types of turn treatments, namely BAR, AUR and BAL. This is especially true for the right-turn treatments.

In some situations, the EDD turn treatments may be used to justify retaining existing geometry.

13.10.3.2 Use of EDD Turn Treatments

This guide presents Extended Design Domain dimensions for CHR and AUL turn treatments that are smaller than the minimums used for the Normal Design Domain (i.e. those used for a new intersection in a greenfield site). In general, these treatments are intended to replace lower order turn types (e.g. linemarking an existing AUR turn treatment to form a CHR turn treatment). The Extended Design Domain dimensions have been found to operate effectively in practice, providing a higher level of safety than any of the lower order treatments.

The treatments shown in this guide are predominantly for application to existing intersections, where sufficient area of pavement exists for them to be incorporated. Sometimes, they may be applied as new construction at existing intersections, where insufficient length is available to introduce a turn-slot with dimensions as per the Normal Design Domain.

13.10.3.3 General Considerations

The use of the Extended Design Domain turn treatments can only be justified provided they meet the following conditions:

- **They are not combined with other minima e.g.**
    - o **very tight horizontal curves;**
    - o **limited visibility to the intersection ie Normal Design Domain ASD not achieved; or**
    - o **major road on a steep downgrade.**
- Future arrangements/planning must be satisfied (e.g. allow for future traffic growth, which may well affect storage lengths);
- Geometric features and other features of the road do not distract drivers;
- For existing layouts meeting the EDD criteria, the accident data indicates that there is not a high accident rate related

to the use of the shorter dimensions eg not a high rear-end accident rate at the start of the turn slots; and

- The length of left and right turn bays should not be restricted to the minimum length if there is little difficulty in making them longer and the demand warrants the treatment.

13.10.3.4 Minimum Extended Design Domain Channelised Right-turn Treatment for Two-lane, Two-way Roadways without Medians

Figure 13.96 shows a minimum Extended Design Domain Channelised Right-turn treatment for two-lane, two-way roadways without medians.

The primary intent of this treatment is to enable an AUR turn treatment to be linemarked as a CHR turn treatment. This is only possible if full depth pavement exists under the original auxiliary lane and, if required, the shoulder. In this treatment, the through road deviates by the width of the turn lane. The dimensions of the lateral movement length "A" are deemed suitable for horizontal straights and larger radius horizontal curves. On smaller curves, "A" will need to be increased above the lengths given in Figure 13.96 so that the resulting alignment of the through lane means that minimal decrease in speed is required for through drivers.

13.10.3.5 "S" Lane Turn Treatment on an Urban Road

An "S" lane treatment on a multi-lane undivided road converts three through lanes into two through lanes with a right turn bay (ie to form a Channelised Right-turn treatment). This is shown in Figure 13.97.

"S" lanes can be installed on an existing three lane carriageway where it is not possible to add a right turn bay.

As with any proposed intersection treatment, an "S" lane should be evaluated by examining capacity, safety, economic and environmental issues for comparison of existing conditions with any proposal.

The advantages and disadvantages of "S" lanes are:

Advantages

- lane changing by through vehicles reduced the incidence of rear-end collisions involving right turning vehicles;
- free flowing conditions are provided for vehicles in the offside through lane (adjacent to centreline or median); and
- travel times may be reduced.

Disadvantages

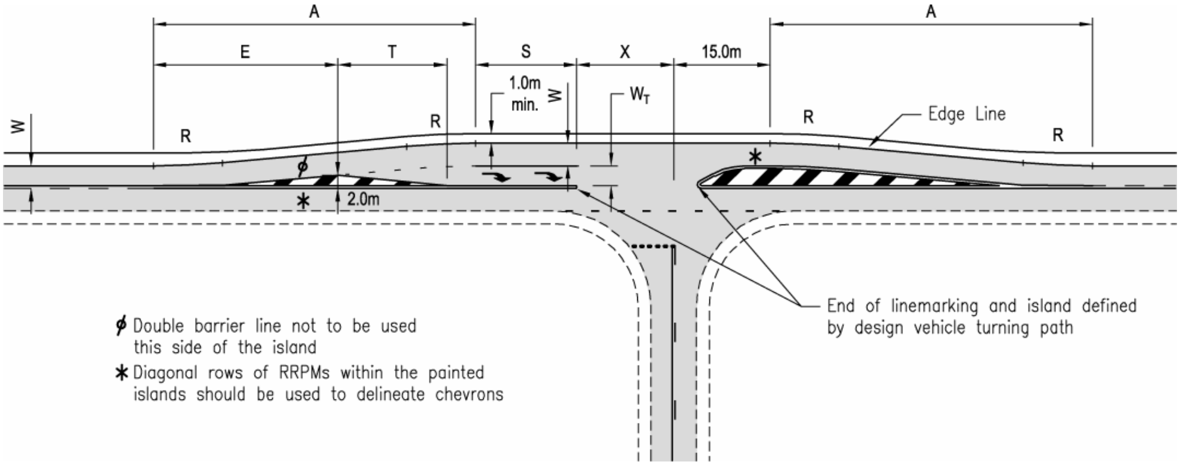
- reduces kerb side parking opportunities near intersections with consequent problems for delivery vehicles and customer parking (loss of trade objections);
- moves through traffic adjacent to kerb at intersections on a permanent basis with possible noise, vibration, and pedestrian (young, aged, disabled) problems;
- creates merge conflict for kerb side lane where three through lanes reduce to two through lanes;
- creates problems for cyclists where three through lanes reduce to two (for a possible solution, refer Austroads 1999 - Sections 5.4.2.6 and 5.4.2.7);
- may require relocation of bus stops, taxi ranks, mail collection points; and

- rigid kerb side objects (poles, trees, signposts etc.) may have to be moved where three through lanes merge into two.

13.10.3.6 Minimum Extended Design Domain Channelised Right-turn Treatment for Roadways with Medians

Figure 13.98 shows a minimum Extended Design Domain Channelised Right-turn treatment for roadways with medians.

This treatment can be used at intersections on existing roads where sufficient area of pavement already exists to introduce a right-turn slot. Alternatively, the treatment may be applied as new construction at existing intersections where insufficient length is available to introduce a right-turn slot with dimensions as per the Normal Design Domain.



Design Speed of Major Road Approach (km/h)	Minimum Lateral Movement Length "A" (m) (1)	Desirable Radius "R" (m)	Taper Length, "T" (m) (2)
60	40 (3)	175	10
70	50 (3)	240	15
80	55 (3)	280	15
90	60	350	15
100	70	425	20
110	75	500	20
120	80	600	20

(1) Based on a diverge rate of 1.25m/s and a turn lane width of 3.0m. Increase lateral movement length if turn lane width >3m. If the through road is on a tight horizontal curve, increase lateral movement length so that a minimal decrease in speed is required for the through movement.
(2) Based on a turn lane width of 3.0m.
(3) Where Type 2 road trains are required, minimum A = 60m.

W = Nominal through lane width (m), including widening for curves.

W_T = Nominal width of turn lane (m), including widening for curves based on the design turning vehicle = 2.8m minimum.

E = Distance from start of taper to 2.0m width (m) = $(A/W_T) \times 2$

S = Storage length (m), greater of:
1. The length of one design turning vehicle
2. (calculated car spaces –1) x 8m (refer to Appendix 13A or use computer program eg. aaSIDRA)

T = Taper length (m) = $\dfrac{0.2 \times V \times W_T}{3.6}$

V = Design speed of major road approach (km/h).

X = Distance based on design vehicle turning path, typically 10m to 15m.

Note: Diagram shown for a rural intersection layout. The dimensions shown are also suitable for an urban intersection layout.

Figure 13.96 Minimum Extended Design Domain Channelised Right-turn Treatment for Two-lane, Two-way Roadways without Medians

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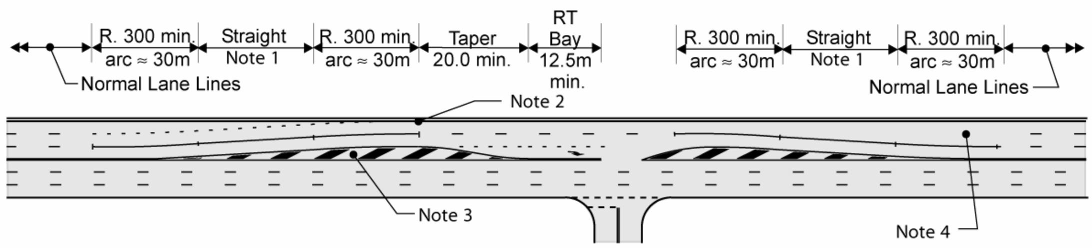
13.10.3.7 Minimum Extended Design Domain Auxiliary Left-turn Treatment

Figure 13.99 shows a minimum Extended Design Domain Auxiliary Left-turn treatment.

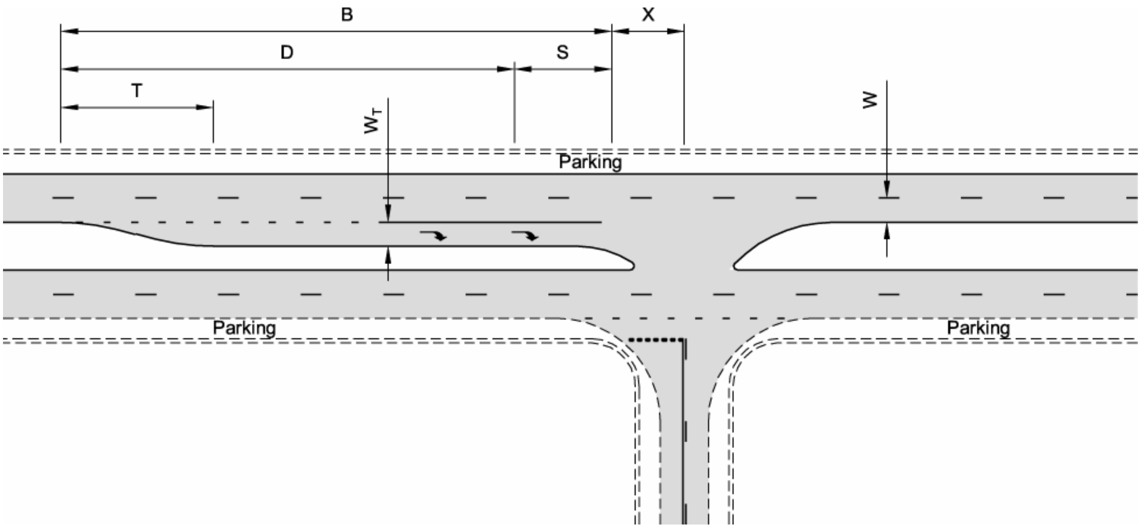
This treatment can be used at intersections on existing roads where sufficient area of pavement already exists to introduce a left-turn slot. Alternatively, the treatment may be applied as new construction at existing intersections where insufficient length is available to introduce a left-turn slot with dimensions as per the Normal Design Domain.



Notes:

1. For 60 and 70km/h design speeds, use 25m and 30m lengths respectively. This allows approximately 1.5 seconds of travel for through vehicles to reverse steer.
2. Provision for cyclists is to be incorporated into the design, particularly at this "squeeze point".
3. Median may be painted or raised.
4. Continuity lines to be used where nearside lane is used for parking (including public transport stops) or dedicated left turn lane.
5. Should only be adopted after consideration of the advantages and disadvantages listed in Section 13.10.3.5.

Figure 13.97 "S" Lane Treatment on an Urban Road



Design Speed of Major Road Approach (km/h)	Minimum diverge / deceleration length "D" (m) (1)	Taper length "T" (m) (2)
50	15	10
60	20	10
70	25	15
80	35	15
90	45	15
100	55	20
110	65	20
120	80	20
(1) Based on a 30% reduction in through road speed at the start of the taper to a stopped condition using a value of deceleration of 3.5m/s^2 (adjust for grade using the Correction to Grade' table in Figure 13.42). (2) Based on a turn lane width of 3.0m		

W = Nominal through lane width (m), including widening for curves.

W_T = Nominal width of turn lane (m), including widening for curves based on the design turning vehicle = 2.8m minimum.

B = Total length of auxiliary lane (m), including taper, diverge/deceleration and storage.

S = Storage length (m), greater of: 1. The length of one design turning vehicle
2. (calculated car spaces –1) x 8m (refer to Appendix 13A or use computer program eg. aaSIDRA)

T = Taper length (m) = $\frac{0.2 \times V \times W_T}{3.6}$

V = Design speed of major road approach (km/h).

X = Distance based on design vehicle turning path, typically 10m to 15m.

Note: Diagram shown for an urban intersection layout. The dimensions shown are also suitable for a rural intersection layout.

Figure 13.98 Minimum Extended Design Domain Channelised Right-turn Treatment for Roadways with Medians

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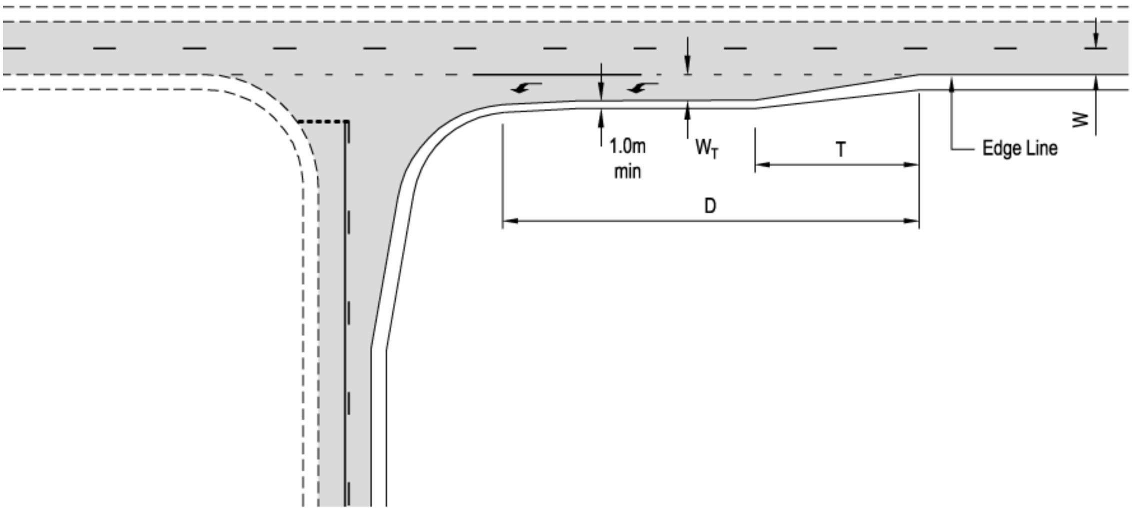

Design Speed of Major Road Approach (km/h)	Minimum Diverge / Deceleration Length "D" (m) (1)	Taper Length "T" (m) (2)
50	15	10
60	20	10
70	25	15
80	35	15
90	45	15
100	55	20
110	65	20
120	80	20
(1) Based on a 30% reduction in through road speed at the start of the taper to a stopped condition using a value of deceleration of $3.5 m/s^2$ (adjust for grade using the Correction to Grade' table in Figure 13.42). (2) Based on a turn lane width of 3.0m		

W = Nominal through lane width (m), including widening for curves.

W_T = Nominal width of turn lane (m), including widening for curves based on the design turning vehicle = 2.8m minimum.

T = Taper length (m) = $\frac{0.2 \times V \times W_T}{3.6}$

V = Design speed of major road approach (km/h).

Note 1: Diagram shown for a rural intersection layout. The dimensions shown are also suitable for an urban intersection layout, except that the shoulder width criterion does not apply.
Note 2: Approaches to left turn slip lanes can create hazardous situations between cyclists and left-turning motor vehicles. Treatments to reduce the number of potential conflicts at left turn slip lanes are given in Austroads (1999b).

Figure 13.99 Minimum Extended Design Domain Auxiliary Left-turn Treatment

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Relationship to Other Chapters

- Chapters 13 and 14 are complementary;
- Chapter 18 deals with signalisation of intersections;
- This chapter relies on details in chapters 7, 9, 11, 12 and 17;
- Aspects of Chapters 4, 15, 16 and 20 are dependent on this chapter; and
- Close relationship with Chapters 5 and 6.

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Appendix 13A: Computation Analyses for Non Signalised Intersections

The following analysis can be used to compute various capacities, delays and storage requirements (eg right turn bays at type CHR intersections) for unsignalised sites. In this analysis, the terms 'Minor Stream' refer to the traffic stream that is giving way and finding acceptable gaps in the 'Major Stream'. The term 'Minor Stream' does not correlate with the term 'Minor Road'. For example, the Minor Stream may be the right-turn traffic stream from the major road.

13A.1 Practical Absorption Capacity C_p

- Determine major stream volume Q.
- Using Table 13A.1 select suitable critical acceptance gap t_a and follow up headway t_f.
- Using Figure 13A.1 determine practical absorption capacity C_p.

13A.2 Average Delay Wm

- Using Figure 13A.1 determine practical absorption capacity C_p.
- Determine minor stream volume Q_m.
- Determine the number of minor stream approach lanes n required. where $n = Q_m / C_p$
- Using Figure 13A.2 (a) to (h) determine Average Delay W_m.

13A.3 Storage Requirements

- Determine minor stream volume Q_m.
- Determine minor stream service rate Q_s (maximum number of vehicles that can be absorbed considering all conditions).
- Generally $Q_s = C = C_p / 0.8$
- Determine Utilisation Ratio, ie ratio of arrival rate to service rate ρ

where $\rho = Q_m / Q_s = Q_m / C$

- Decide on a probability that the design queue length will not be exceeded (normally 95%).
- Using Figure 13A.3 determine the storage spaces required such that the probability of the queue not being exceeded is greater than the desired probability.
- Adopt a storage length of 8m for each storage space selected.

13A.4 Auxiliary Lane Dimensions

- Diverge/deceleration length D can be calculated using Table 13.11.
- Length of taper T within the diverge / deceleration distance is calculated as given in Section 13.7.4.1.
- As the calculations take into account one stored vehicle, the length of storage S will equal the calculated car spaces minus 1 x 8m.
- Total length of auxiliary lane B equals D + S.

13A.5 Example Calculation

On a two-lane two-way road at a T junction there is a right turning movement of 250 vph parallel to a through movement 1100 vehicles. The right turn is opposed by 1100vph (ie straight ahead and left turns). The intersection site is in a 100 km/h speed zone (operating speed on the major road is 110km/h).

1. Determine the length of storage required at the CHR intersection.

2. Determine all dimensions of the auxiliary right turn lane for a type CHR intersection as shown in Figure 13.60.

Practical Absorption Capacity Cp

- Major stream volume Q_p = 1100 vph.
- Using Table 13A.1 t_a = 4 secs, t_f = 2 secs.
- Using Figure 13A.1, C_p =560 vph.

Average Delay W_m

- From above C_p = 560 vph.
- Minor stream volume Q_m = 250vph.
- Number of Minor stream approach lanes required n = Q_m / C_p = 250/560 < 1.
- Next higher integer is 1.
- Using Figure 13A.2(b) W_m = 6.4 sec.

Storage Requirements

- Minor stream volume Q_m = 250 vph.
- Minor stream service rate Q_s = C = C_p / 0.8

Q_s = 560/0.8 = 700

- Utilisation ratio $\rho = Q_m / Q_s$
- ρ = 250/700 = 0.357
- Using Figure 13A.3 queue length = 3 vehicles (95% probability).
- Provide for a queue length of 4 x 8m = 32 m.

Auxiliary Lane Dimensions

- Diverge/Deceleration D from Table 13.11 for comfortable deceleration from 110km/h = 185m.
- Length of taper T from Equation 13.7 in Section 13.7.4.1 (assuming a 3.5m wide turn lane) = 35m.
- Storage Length S = (Calculated Car Spaces minus 1) x 8m, S = (4-1) x 8 = 24m.
- Using Figure 13.51, total length of Auxiliary Lane B = D + S = 185m + 24m = 209m.

Note: The likely growth of traffic should be taken into account in deciding on the storage required. In this case, the queue length from the graph is close to the next step. It would be prudent to take the next level and design for that i.e. take the queue length to be 3 vehicles as shown above.

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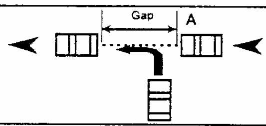
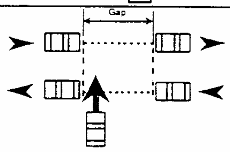
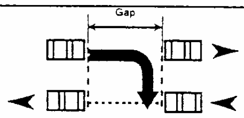
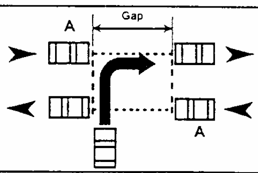
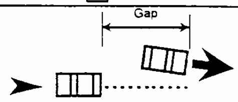
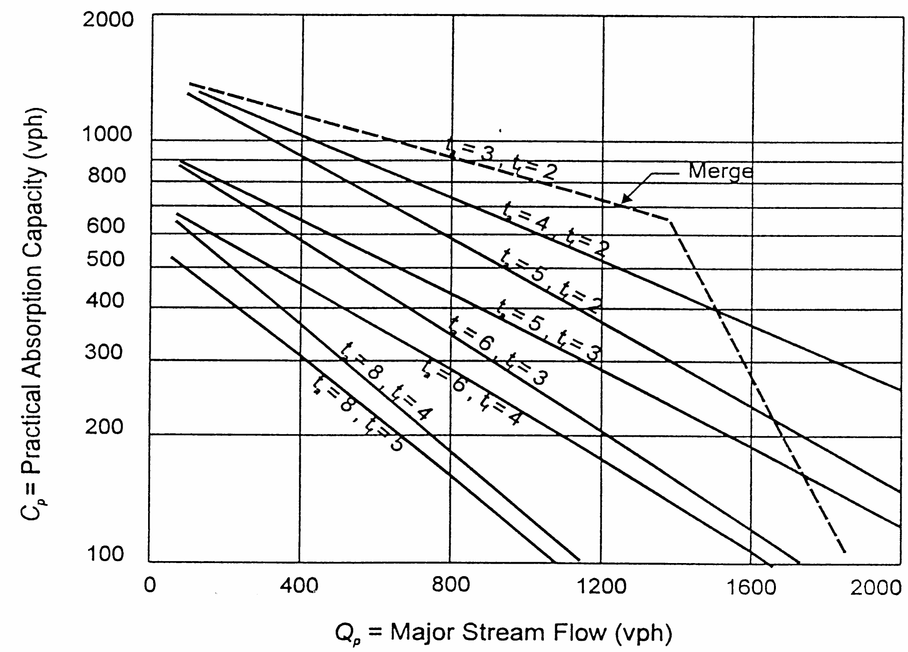
Table 13A.1 Gap Acceptance Time

Movement	Diagram	Description	t_a	t_f
Left Hand Turn		Not interfering with A Requiring A to slow	14-40 sec 5 sec	2-3 sec 2-3 sec
Crossing		Two lane / one way Three lane / one way Four lane / one way Two lane / two way Four lane / two way Six lane / two way	4 sec 6 sec 8 sec 5 sec 8 sec 8 sec	2 sec 3 sec 4 sec 3 sec 5 sec 5 sec
Right Hand Turn from major road		Across 1 lane Across 2 lanes Across 3 lanes	4sec 5sec 6sec	2 sec 3 sec 4 sec
Right Hand Turn from minor road		Not interfering with A One way Two lane / two way Four lane / two way Six lane / two way	14-40 sec 3 sec 5 sec 8 sec 8 sec	3 sec 3 sec 3 sec 5 sec 5 sec
Merge		Acceleration Lane	3 secs	2 secs

Note: *$t_{(a)}$ = Critical acceptance gap,* *$t_{(f)}$ = follow up headway*



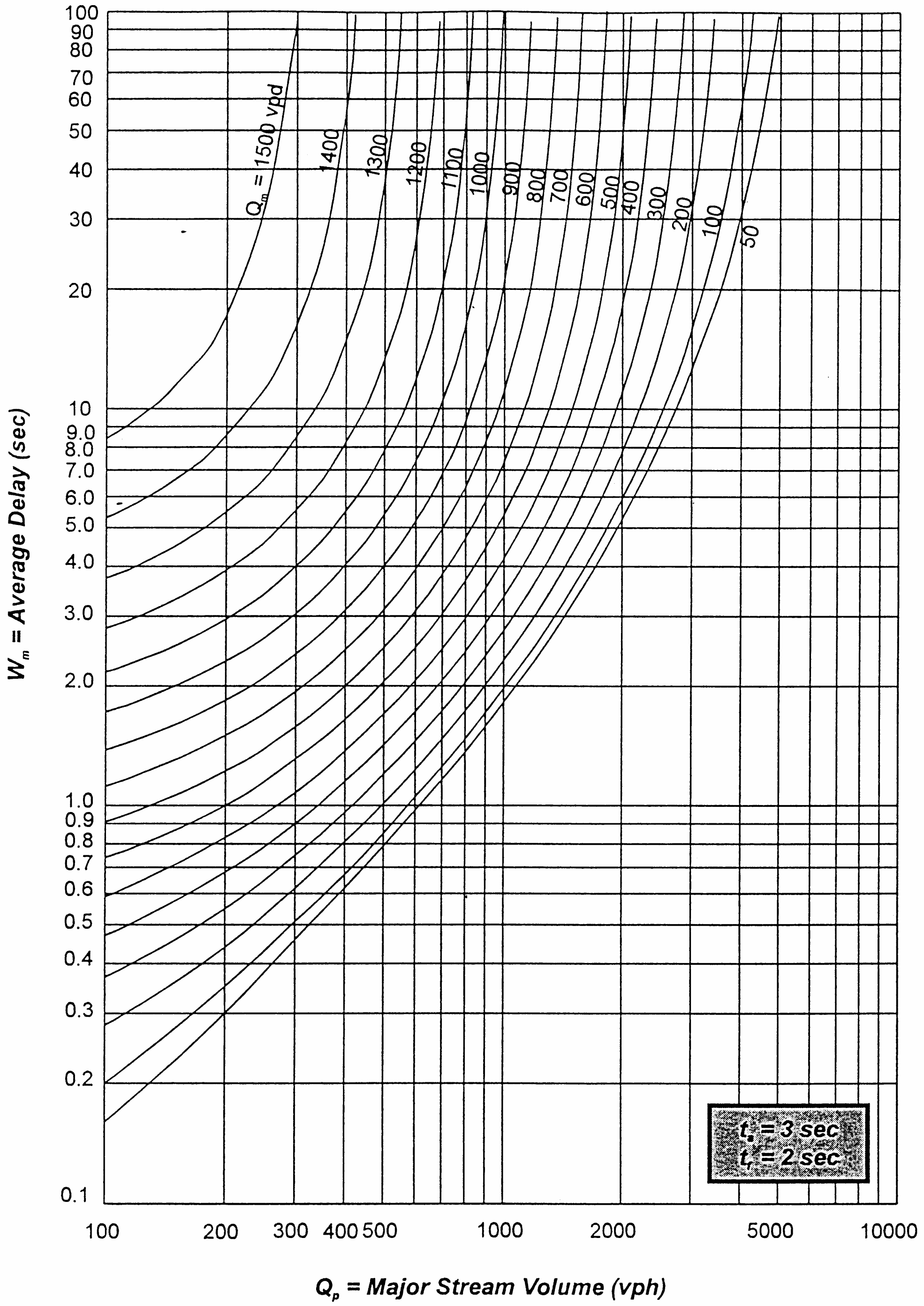
Formula:

$$C = \frac{q_p e^{-q_p t_a} \times 3{,}600}{1 - e^{-q_p t_f}}$$

$Cp = 0.8\,C$

C = absorption capacity (vph)
C_p = practical absorption capacity (vph)
q_p = major (priority) stream flow rate (vps)
t_a = critical acceptance gap (sec)
t_f = follow up headway (sec)
e = constant (2.7183)

Figure 13A.1 Practical Absorption Capacity at Unsignalised Intersections



Formulae:

$$W_m = \frac{q_p e^{q_p t_f}\left[e^{q_p t_a} - q_p t_a - 1\right] + q_m e^{q_p t_a}\left[e^{q_p t_f} - q_p t_f - 1\right]}{q_p\left[q_p e^{q_p t_f} - q_m e^{q_p t_a}\left(e^{q_p t_f} - 1\right)\right]}$$

q_p = major (priority) stream flow rate (veh/s)
q_m = minor stream flow rate per lane (veh/s)
W_m = average delay to minor stream vehicles per vehicle (sec)
t_a = critical acceptance gap (sec)
t_f = follow-up headway (sec)
e = constant (2.7183)

Figure 13A.2(a) Average Delay to Minor Stream Vehicles at Unsignalised Intersections

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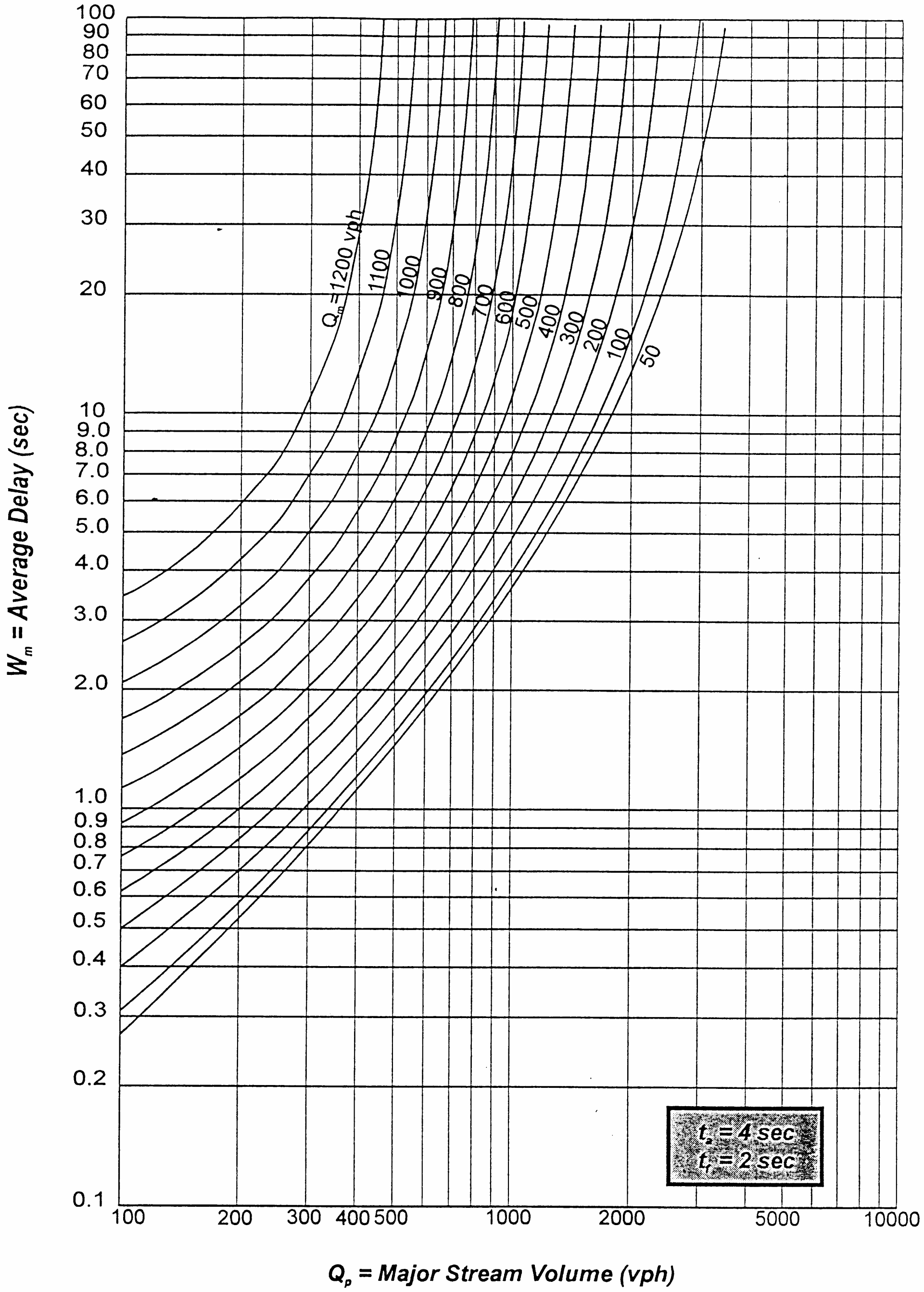
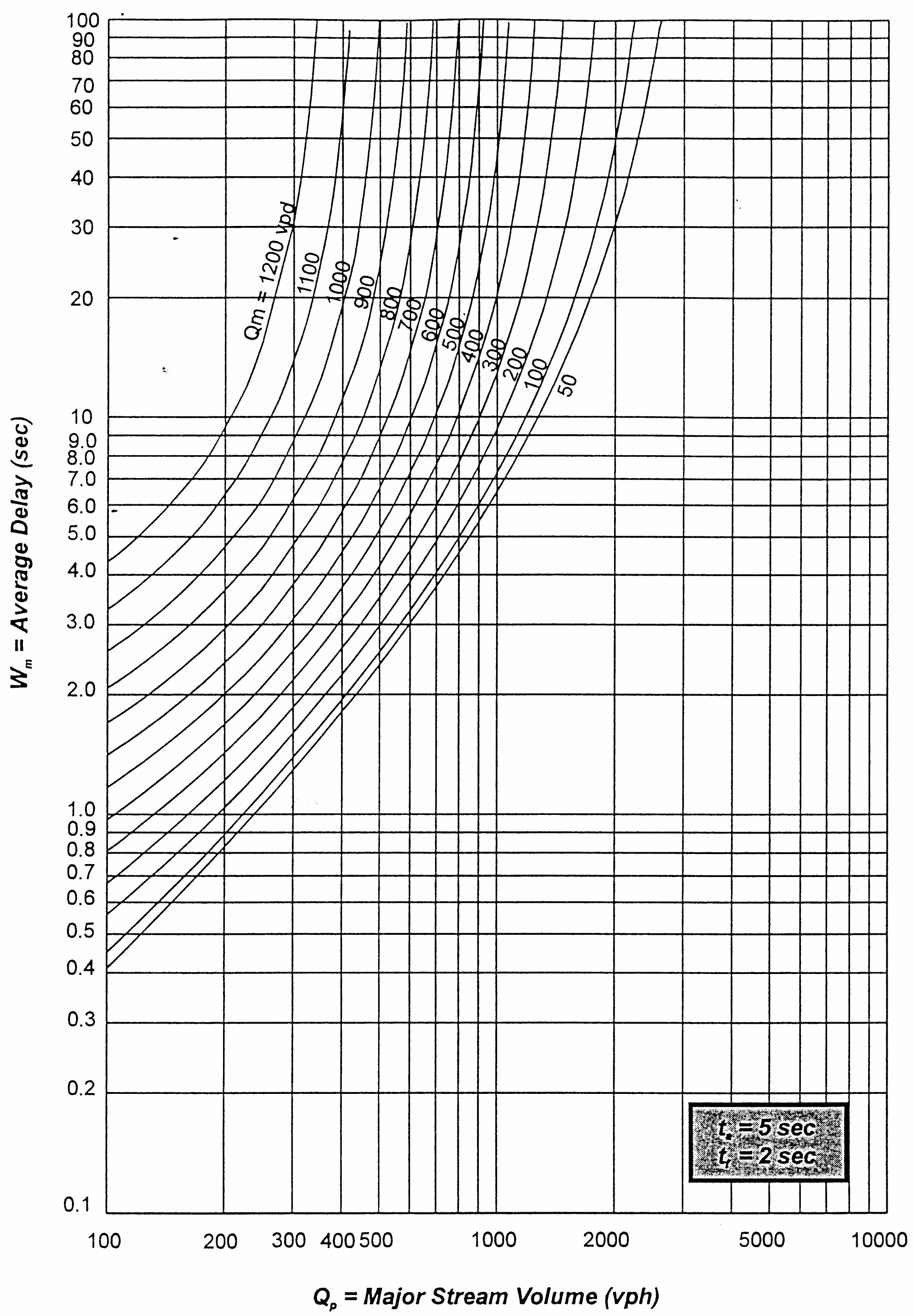


Formulae:

$$W_m = \frac{q_p e^{q_p t_f}\left[e^{q_p t_a} - q_p t_a - 1\right] + q_m e^{q_p t_a}\left[e^{q_p t_f} - q_p t_f - 1\right]}{q_p\left[q_p e^{q_p t_f} - q_m e^{q_p t_a}\left(e^{q_p t_f} - 1\right)\right]}$$

q_p = major (priority) stream flow rate (veh/s)
q_m = minor stream flow rate per lane (veh/s)
W_m = average delay to minor stream vehicles per vehicle (sec)
t_a = critical acceptance gap (sec)
t_f = follow-up headway (sec)
e = constant (2.7183)

Figure 13A.2(b) Average Delay to Minor Stream Vehicles at Unsignalised Intersections

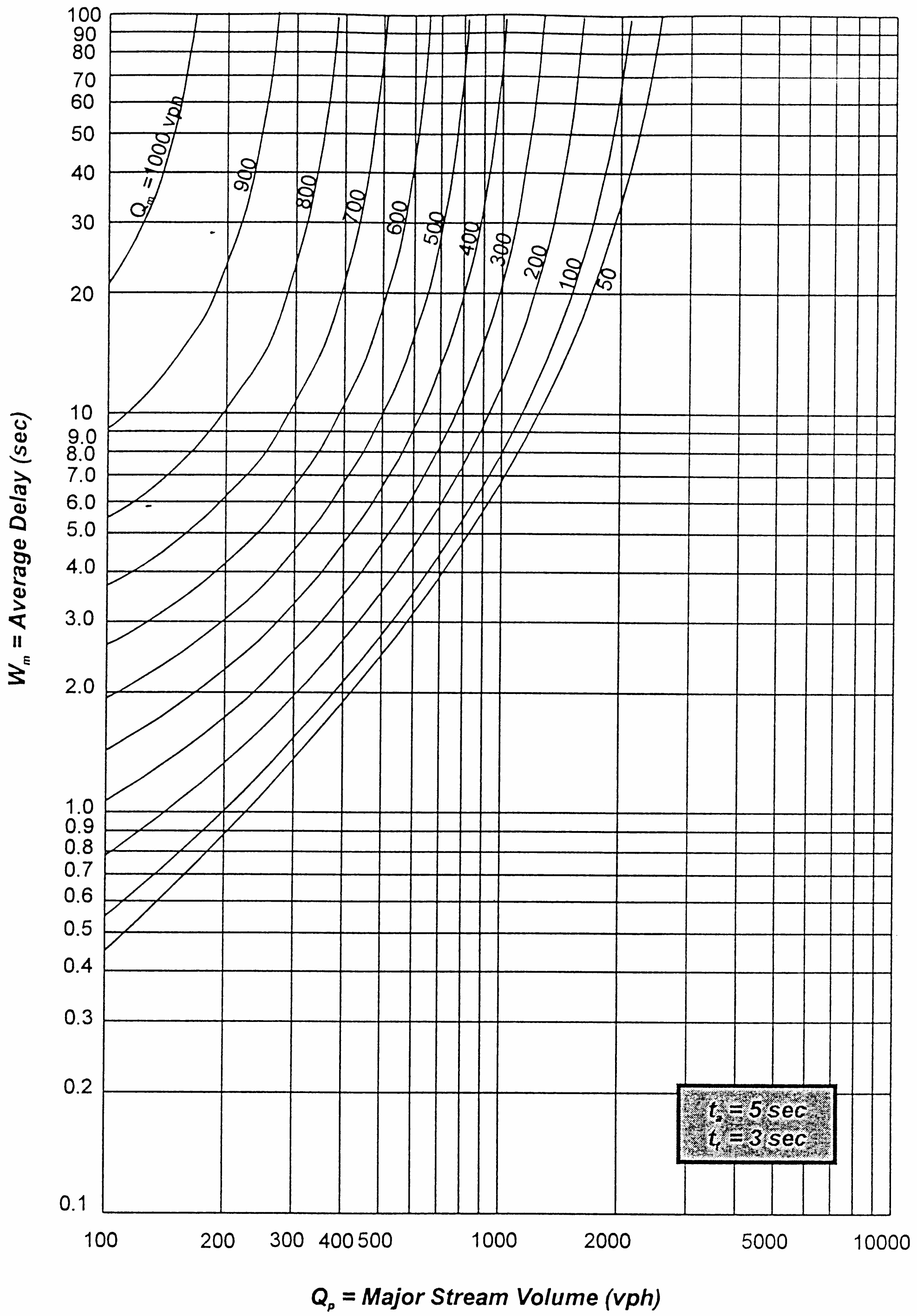


Formulae:

$$W_m = \frac{q_p e^{q_p t_f}\left[e^{q_p t_a} - q_p t_a - 1\right] + q_m e^{q_p t_a}\left[e^{q_p t_f} - q_p t_f - 1\right]}{q_p\left[q_p e^{q_p t_f} - q_m e^{q_p t_a}\left(e^{q_p t_f} - 1\right)\right]}$$

q_p = major (priority) stream flow rate (veh/s)
q_m = minor stream flow rate per lane (veh/s)
W_m = average delay to minor stream vehicles per vehicle (sec)
t_a = critical acceptance gap (sec)
t_f = follow-up headway (sec)
e = constant (2.7183)

Figure 13A.2(c) Average Delay to Minor Stream Vehicles at Unsignalised Intersections

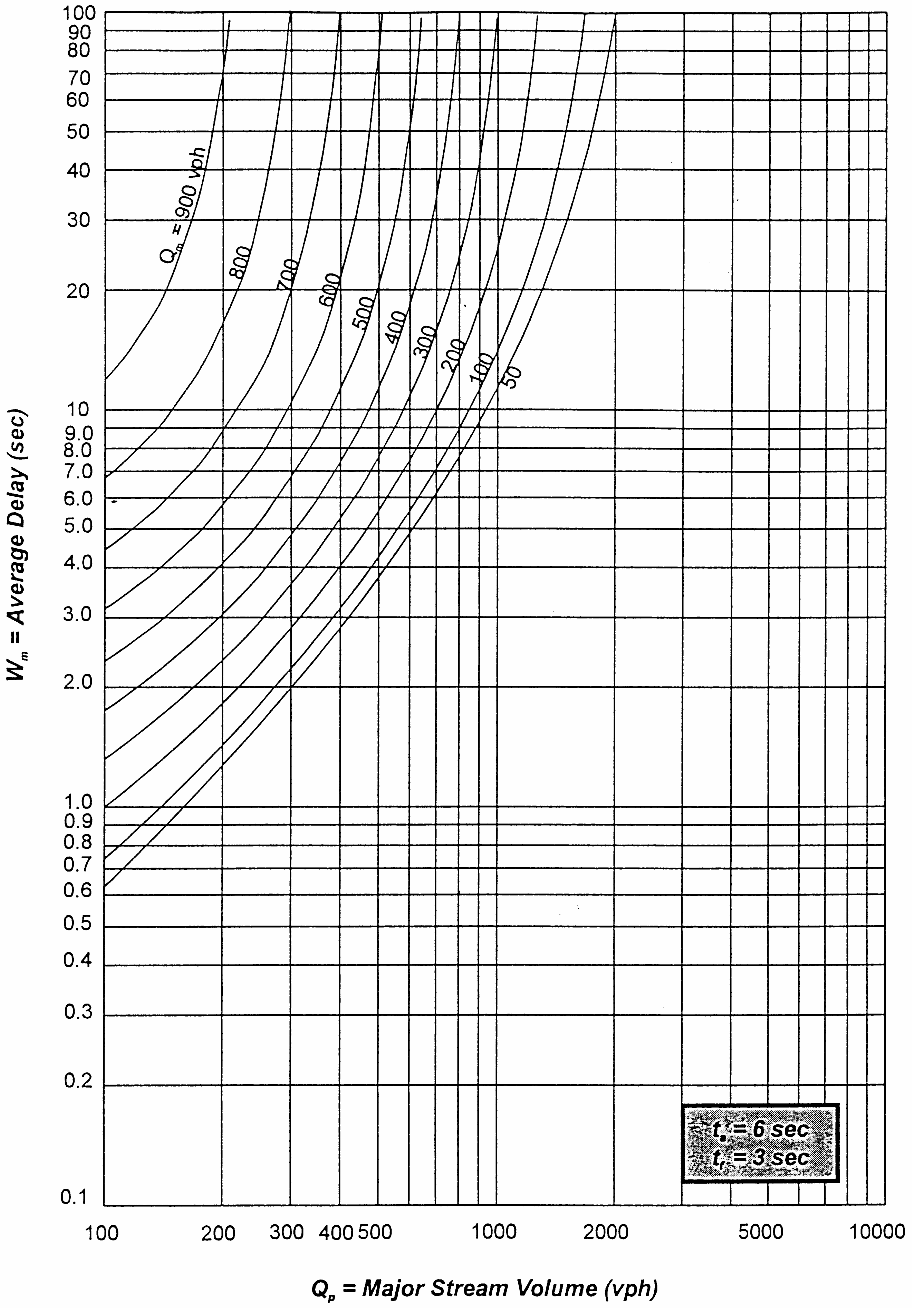


Formulae:

$$W_m = \frac{q_p e^{q_p t_f}\left[e^{q_p t_a} - q_p t_a - 1\right] + q_m e^{q_p t_a}\left[e^{q_p t_f} - q_p t_f - 1\right]}{q_p\left[q_p e^{q_p t_f} - q_m e^{q_p t_a}\left(e^{q_p t_f} - 1\right)\right]}$$

q_p = major (priority) stream flow rate (veh/s)
q_m = minor stream flow rate per lane (veh/s)
W_m = average delay to minor stream vehicles per vehicle (sec)
t_a = critical acceptance gap (sec)
t_f = follow-up headway (sec)
e = constant (2.7183)

Figure 13A.2(d) Average Delay to Minor Stream Vehicles at Unsignalised Intersections

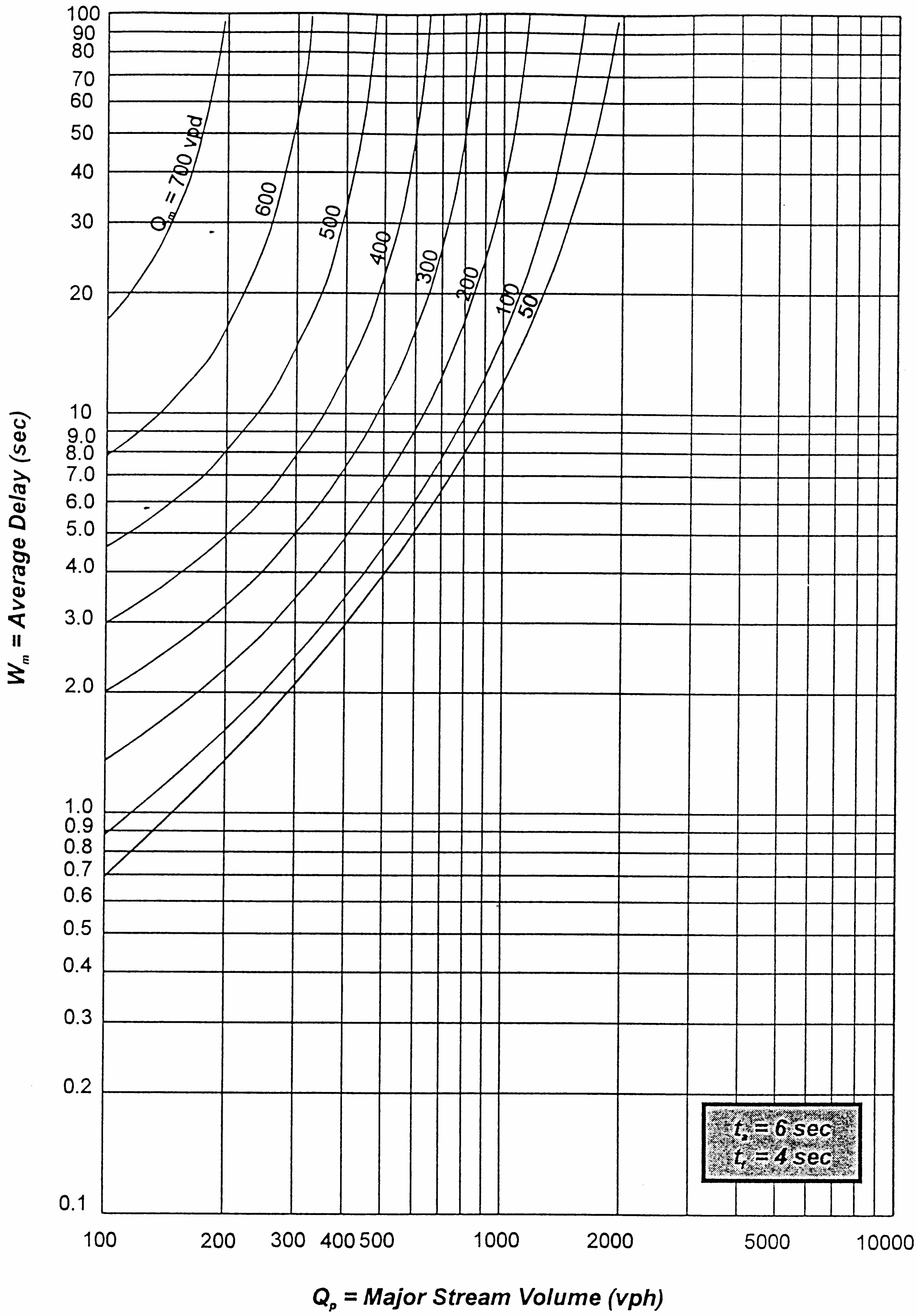


Formulae:

$$W_m = \frac{q_p e^{q_p t_f}\left[e^{q_p t_a} - q_p t_a - 1\right] + q_m e^{q_p t_a}\left[e^{q_p t_f} - q_p t_f - 1\right]}{q_p\left[q_p e^{q_p t_f} - q_m e^{q_p t_a}\left(e^{q_p t_f} - 1\right)\right]}$$

q_p = major (priority) stream flow rate (veh/s)
q_m = minor stream flow rate per lane (veh/s)
W_m = average delay to minor stream vehicles per vehicle (sec)
t_a = critical acceptance gap (sec)
t_f = follow-up headway (sec)
e = constant (2.7183)

Figure 13A.2(e) Average Delay to Minor Stream Vehicles at Unsignalised Intersections

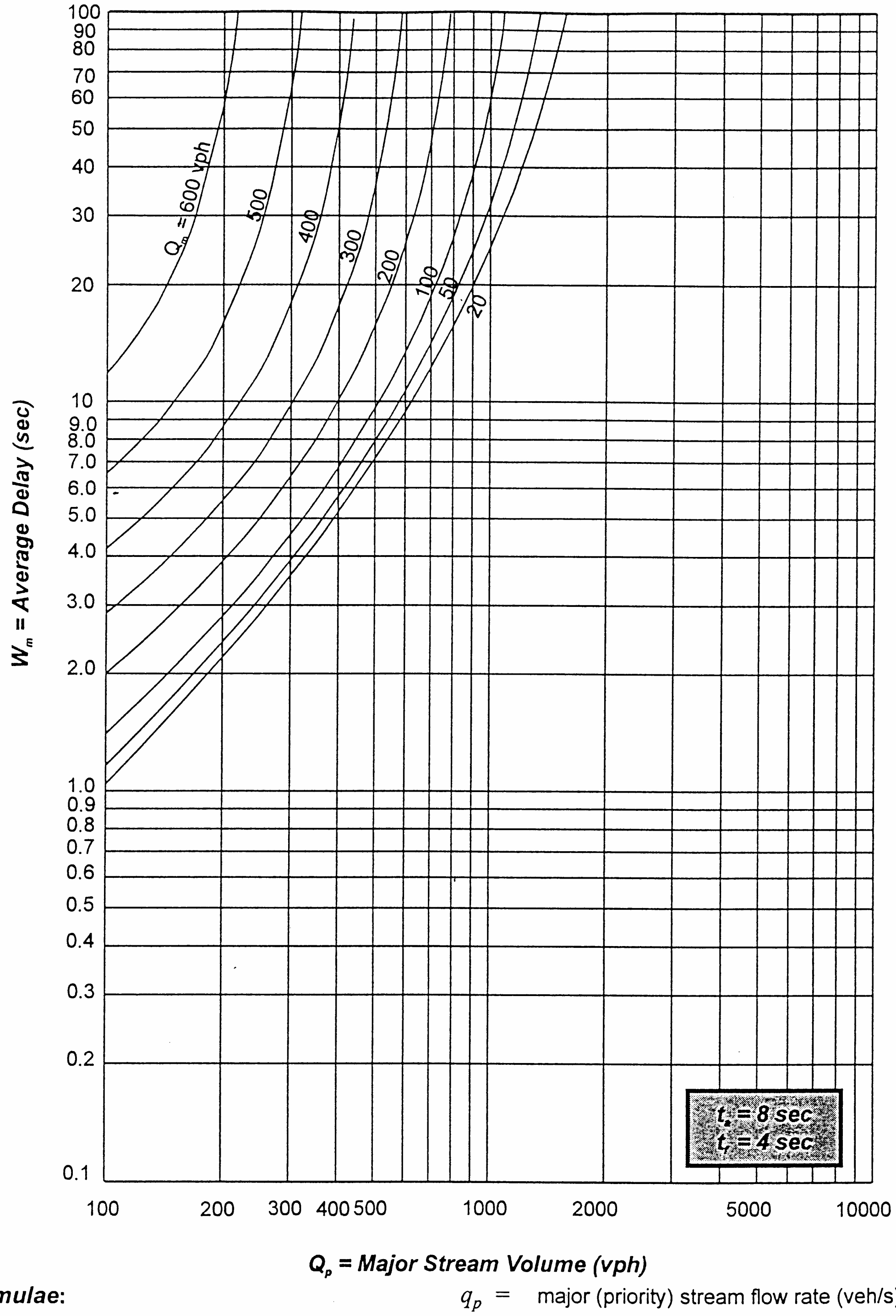


Formulae:

$$W_m = \frac{q_p e^{q_p t_f}\left[e^{q_p t_a} - q_p t_a - 1\right] + q_m e^{q_p t_a}\left[e^{q_p t_f} - q_p t_f - 1\right]}{q_p\left[q_p e^{q_p t_f} - q_m e^{q_p t_a}\left(e^{q_p t_f} - 1\right)\right]}$$

q_p = major (priority) stream flow rate (veh/s)
q_m = minor stream flow rate per lane (veh/s)
W_m = average delay to minor stream vehicles per vehicle (sec)
t_a = critical acceptance gap (sec)
t_f = follow-up headway (sec)
e = constant (2.7183)

Figure 13A.2(f) Average Delay to Minor Stream Vehicles at Unsignalised Intersections

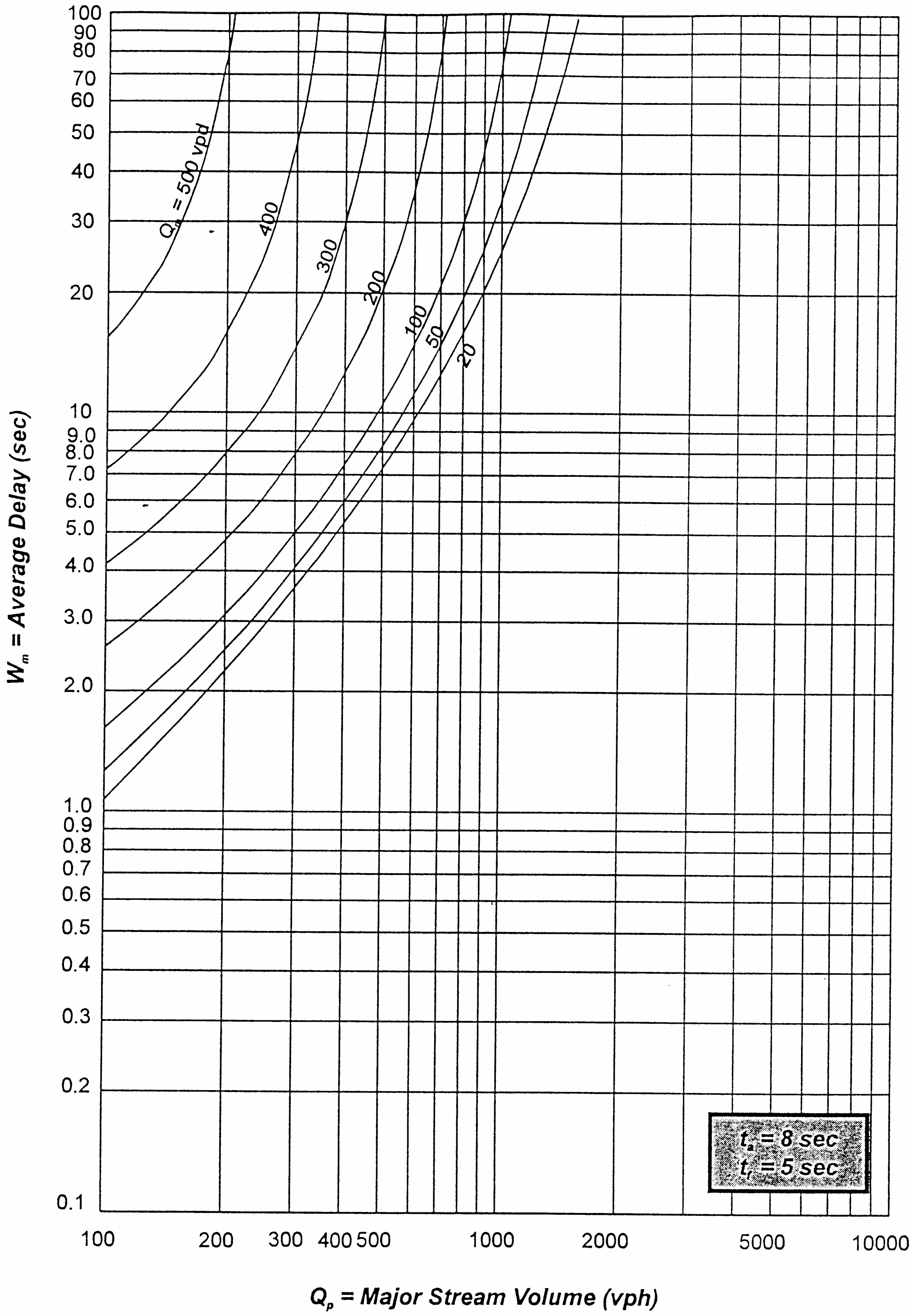


Formulae:

$$W_m = \frac{q_p e^{q_p t_f}\left[e^{q_p t_a} - q_p t_a - 1\right] + q_m e^{q_p t_a}\left[e^{q_p t_f} - q_p t_f - 1\right]}{q_p\left[q_p e^{q_p t_f} - q_m e^{q_p t_a}\left(e^{q_p t_f} - 1\right)\right]}$$

q_p = major (priority) stream flow rate (veh/s)
q_m = minor stream flow rate per lane (veh/s)
W_m = average delay to minor stream vehicles per vehicle (sec)
t_a = critical acceptance gap (sec)
t_f = follow-up headway (sec)
e = constant (2.7183)

Figure 13A.2(g) Average Delay to Minor Stream Vehicles at Unsignalised Intersections



Formulae:

$$W_m = \frac{q_p e^{q_p t_f}\left[e^{q_p t_a} - q_p t_a - 1\right] + q_m e^{q_p t_a}\left[e^{q_p t_f} - q_p t_f - 1\right]}{q_p\left[q_p e^{q_p t_f} - q_m e^{q_p t_a}\left(e^{q_p t_f} - 1\right)\right]}$$

q_p = major (priority) stream flow rate (veh/s)
q_m = minor stream flow rate per lane (veh/s)
W_m = average delay to minor stream vehicles per vehicle (sec)
t_a = critical acceptance gap (sec)
t_f = follow-up headway (sec)
e = constant (2.7183)

Figure 13A.2(h) Average Delay to Minor Stream Vehicles at Unsignalised Intersections

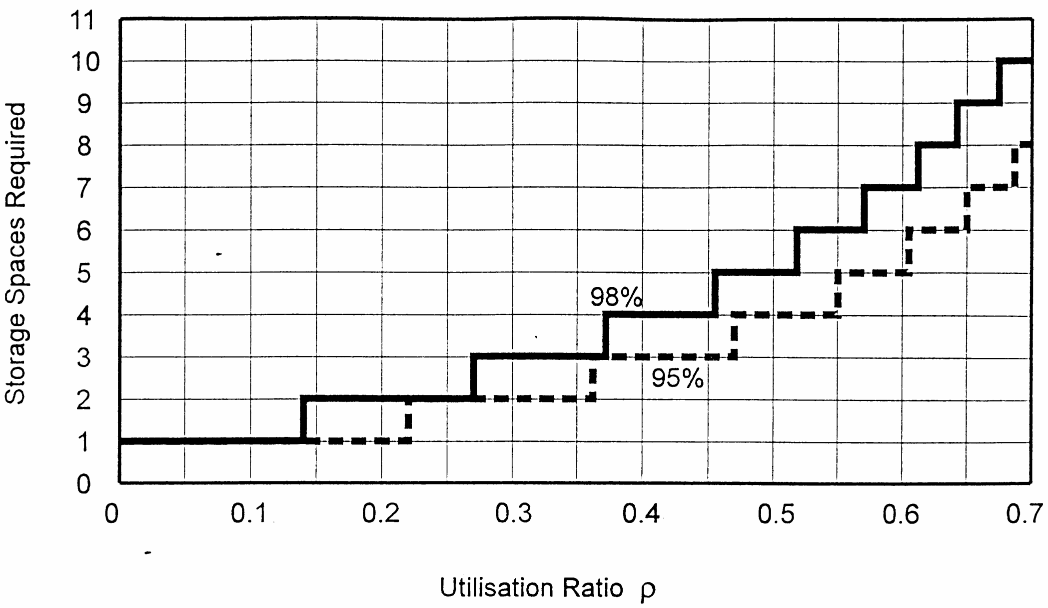


Figure 13A.3 Vehicle Storage Requirements at Unsignalised Intersections

(Source: Austroads (1988d) "Guide to Traffic Engineering Practice" Part 5 Intersections at Grade)

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Appendix 13B: Definitions

13B.1 Purpose

The purpose of this Appendix is to provide a more detailed explanation of some important terminology which relates to intersections. It is arranged in alphabetical order.

13B.2 Points of Conflict

A point of conflict occurs where road space desired by one traffic movement, is simultaneously required by another. The basic forms of conflict are shown in Figure 13B.1. The analysis of an intersection to identify the points of conflict is illustrated in Figure 13B.2.

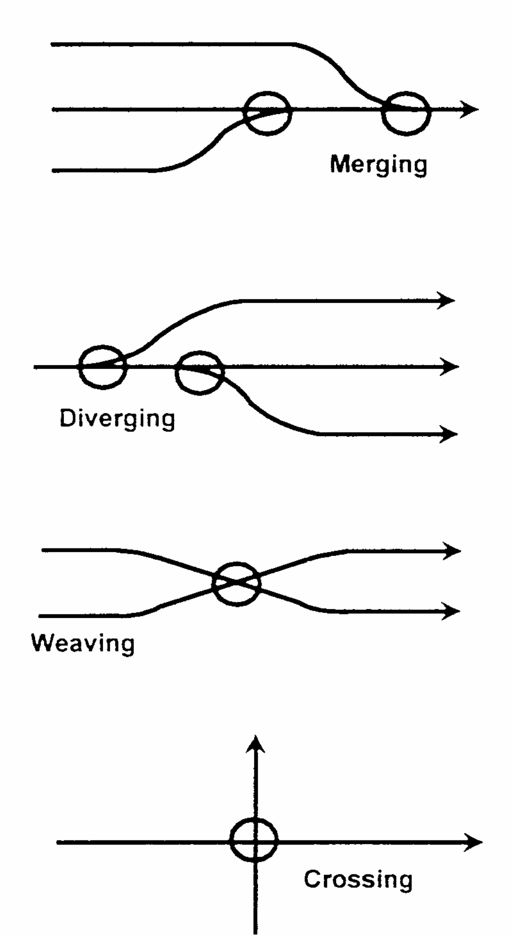

Figure 13B.1 Basic Forms of Point of Conflict

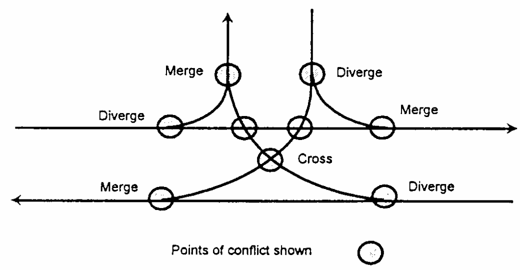

Figure 13B.2 Points of Conflict for a 'T' Intersection

13B.3 Relative Speed

Relative speed is the resultant vector determined from the velocities of individual vehicles at a point of conflict. Refer to Figure 13B.3.

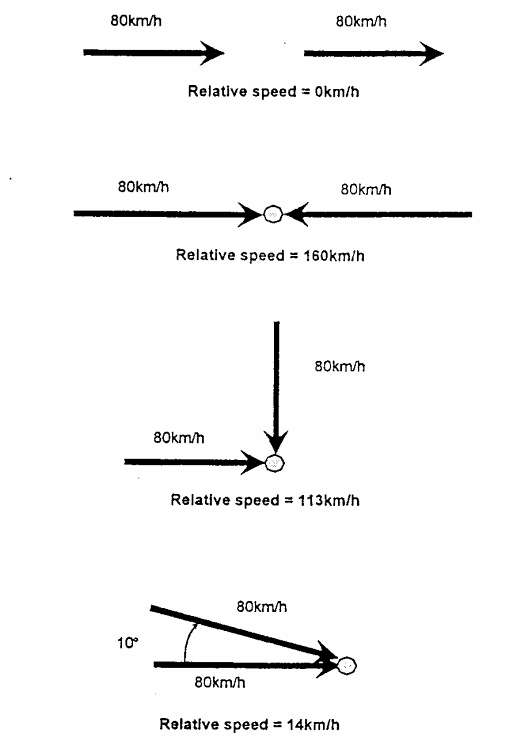

Figure 13B.3 Determination of Relative Speed

13B.4 "Y" Values

(i) General

An intersection should be designed to cope with traffic flows predicted over its design life. Generally, this is 20 years.

The basic objective in providing adequate capacity at an intersection is to absorb and disperse traffic flows with the minimum of delays with a layout that is physically possible, economically justifiable, and as safe as possible.

To achieve these objectives, comparison of the capacities of various intersection treatments may be required. However, it is important to remember that capacity considerations are only part of an overall evaluation procedure. Refer to Appendix 13G.

The "Y" value of an intersection is a useful preliminary guide in assessing the operational characteristics of an intersection, assuming that it would be signalised. Computer simulations should be used for final capacity analysis.

Care must be taken in capacity calculations to determine whether an intersection acts in isolation, or is affected by the capacity and operation of an adjoining intersection(s). Mid-block capacity flow should not be confused with saturation flow at an intersection. They should not be substituted for satflows in computer programs. Information on mid-block capacity is given in Austroads (1988d).

(ii) Definition

The "Y" value of an intersection is the sum of critical movement flow ratios for the whole of the intersection.

$$Y = \sum y \qquad (13B.1)$$

Where:

Y = Sum of "representative" y values for each signal phase

The movement flow ratio (y) is the ratio of arrival flow (q) to saturation flow (s).

$$y = \frac{q}{s} \qquad (13B.2)$$

Where:

y = Ratio of arrival flow to saturation flow for an approach.

q = Flow rate or arrival rate (veh/h).

s = Saturation flow (veh/h).

Satisfactory operating conditions prevail when Y< about 0.7.

(iii) Saturation Flow (s)

At an intersection, the saturation flow can be defined as the maximum rate of flow of vehicles across a stop line at a signalised approach during the effective green time if there is a continuous queue of vehicles waiting to move during that time.

Saturation flow (s) for various flow conditions and lane types are shown in Table 13B.1.

Table 13B.1 Saturation Flows (Through Car Units Per Hour)

Environment Class	Lane Type		
	1	2	3
A	1850	1810	1700
B	1700	1670	1570
C	1580	1550	1270

Where:

- Environment Class A:- ideal flow conditions, such as residential areas.
- Environment Class B:- average, partly restricted conditions, e.g. shopping centres.
- Environment Class C:- poor, heavily restricted conditions, e.g. CBD

and

- Lane Type 1:- through vehicles only.
- Lane Type 2:- turning vehicles in lane (exclusive or shared with through).

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- Lane Type 3:- restricted turning vehicles (e.g. pedestrian interference or small turning radius).

(iv) Arrival Flows (q)

Arrival flows should be adjusted to through car equivalent units (tcu's). Adjustment factors for cars and heavy vehicles, through and turning movements are shown in Table 13B.2.

Table 13B.2 Adjustment Factors to Convert All Arrivals to "through car units" (tcu)

	Thro-ugh Lane	Unopposed turn		Oppos-ed turn
		Normal	Restrict	
Car	1	1	1.25	E_0
Heavy Vehicle	2	2	2.5	E_0+1

Although E_0 can vary depending upon the opposing flow, a value of 3.0 is acceptable for preliminary evaluation purposes.

(v) Calculation of "Y" value

Step 1 Draw phasing diagram.

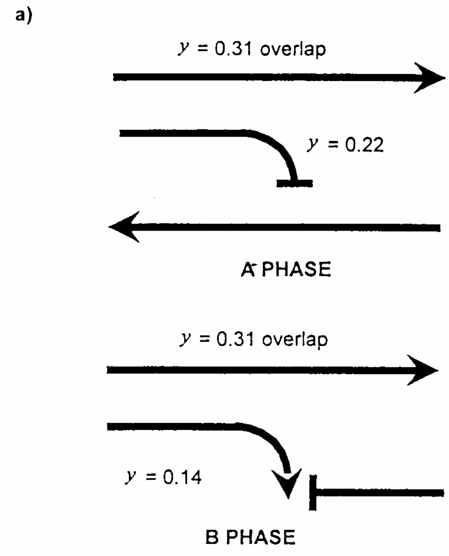
Step 2 Calculate "y" values in accordance with movements in phase diagram, environment, lane type, turning conditions, and the number of lanes.

Step 3 Determine critical "y" values for each phase.

Step 4 Add critical "y" values to determine "Y" value.

A critical "y" value is determined by locating the movement with the highest "y" value within each phase, unless that movement occurs in two or more phases (i.e. overlap movements).

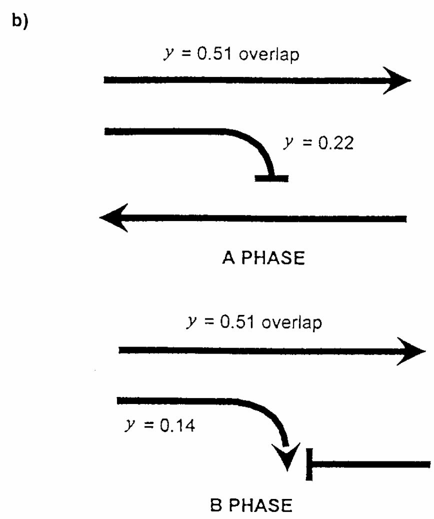
The overlap movement becomes the critical value when the sum of the next highest "y" values in each of the phases it operates is less than the overlap "y" value. Figure 13B.4 provides an example.



Overlap $y = 0.31 < 0.22 + 0.14 = 0.36$

$\therefore$ critical $y = 0.36$ for A and B Phases

Figure 13B.4(a) Example of "Y" value Calculations



Overlap $y = 0.51 > 0.22 + 0.14 = 0.36$

$\therefore$ $y = 0.51$ for A and B Phases

Figure 13B.4(b) Example of "Y" value Calculations

Note: Although the above example has been shown for a through movement, overlaps can occur for turning movements.

Appendix 13C: Computer Software

Several computer programs are available to assist the designer with the capacity analysis of intersections.

The programs most frequently used by Queensland Main Roads are as follows:

13C.1 aaSIDRA

This program, originally developed by ARRB (now maintained and distributed by Akcelik and Associates), can be used as an aid for design and evaluation of the following signalised and unsignalised intersection types:

(a) signalised intersections (fixed time / pre-timed and actuated);

(b) roundabouts;

(c) two-way stop sign control;

(d) all-way stop sign control; and

(e) give way sign control.

Future versions of this program will allow staggered “T” and paired intersections to be analysed.

The program is very flexible allowing up to 8 approaches at an intersection, upstream and downstream short lanes and variable heavy vehicle ratios for each individual leg. Graphical representation of the intersection layout, volumes and results can be copied into word processing documents.

Output information includes vehicle delay, stops, average speed, fuel consumption, emissions, queue lengths, level of service, spare capacity and lane utilisation. This is in both graphical and table format. At signalised intersections optimum cycle times and phase times are included.

For further details refer to the aaSIDRA User Manual.

13C.2 VPATH

VEHICLE/PATH, or VPATH as it is frequently known, is a program which calculates and plots swept path details for turning vehicles. It may be used for the production of standard templates or the design or checking of the turning requirements for vehicles in operation on specific road segments, e.g. turning paths at intersections, roundabouts and so on.

VEHICLE/PATH uses a mathematical swept path model which incorporates the effects of tyre operating conditions. The model was originally developed for the Australian Road Research Board and was later expanded to handle complex turning manoeuvres.

The program can handle vehicles of any dimension having up to 11 units. Each unit may have 1, 2, 3 or 4 fixed (i.e. trailing or non-steering) axles. A steering path can consist of up to 20 segments, each consisting of a circular arc followed, if required, by a straight. A circular arc may turn either clockwise or anti-clockwise through any angle up to 360°.

This program can be run interactively within current design systems (MX, 12Dmodel, AutoCAD).

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Appendix 13D: Summary of Quick and Easy Approximations Related to Intersection Design

Acknowledgment

The items detailed below are based on Ogden et al (1996), Chapter 8.

13D.1 Traffic Flow Characteristics

Capacities

1. The saturation flow for a through lane on an approach lane to a signalised intersection is 1800 - 2000 veh/h of green time.

2. The typical capacity of a through lane at a signalised intersection, where the intersecting roads have approximately equal flows, is between 800 veh/h and 900 veh/h. (45% of saturation flow).

3. The maximum flow of an urban arterial road is 1000 veh/h/lane, the achievement of which generally requires at least 60% green time. However, maximum flow is a function of green time and intersections with minor side roads may achieve higher flows.

4. In a simple gap acceptance situation with single lane minor flow, capacity is achieved with the sum of the major and minor flows being approximately 1500 veh/h.

Traffic Growth

5. The doubling of traffic volume over a period of x years is equivalent to a linear growth rate of approximately (72/x)% per annum.

6. Traffic growth on a major rural road is typically around 3-5% per annum, but may be more or less than this depending on the regional economic growth.

7. On many major roads, traffic growth is closer to linear than it is to exponential.

Directional Splits

8. On radial arterials in outer suburbs, directional splits are 75/25 to 80/20 on the AM peak and 65/35 to 70/30 in the PM peak.

9. On inner suburban radial arterials and on circumferential roads, directional splits are 50/50 to 60/40 in either peak period.

Volume Ratios

10. A typical peak hour/24 hour volume ratio on a rural road is 15%.

11. On urban arterials, peak hour/24 hour volume ratios are 10-12% for uncongested conditions and 7-10% for congested conditions.

12. On urban residential streets, peak hour/24 hour volume ratios around 10-12% are typical; a ratio in excess of 14% suggests that the street may have been used as a rat run by significant volumes of non-local traffic during peak periods.

13. 24 hour/12 hour volume ratios are typically 1.20 to 1.25 for rural roads and 1.25 to 1.30 for urban roads.

14. The 30th highest hourly volume of the year is around 15% of AADT for rural roads and 12% of AADT for urban roads.

Commercial Vehicles

15. Peak volumes of commercial vehicles often occur between 10 am and 12 noon on urban roads.

16. Averaged over the day, the proportion of commercial vehicles on urban arterials is approximately 10% of all traffic. During the peak period the proportion of commercial vehicles is about 5%. It must be noted that these values vary depending on location.

17. On rural highways, commercial vehicles typically comprise 15-25% of all traffic, but this depends on location.

13D.2 Intersections General Principles

18. The fundamental rule for safe intersection design in rural areas is to ensure that effective priority is maintained. This means that priority should be simple and obvious (eg vehicles on the minor road slowed down by a physical means) and intersection control should be appropriate to the volumes.

19. At a cross, T or Y intersection, the desirable minimum angle of intersection is 70°. Every attempt should be made to align the intersection at close to 90°.

20. The travel time between two adjacent intersections should not be less than 5 seconds (equivalent to a distance of 1.4V where V is the travel speed in km/h).

21. The legal definition of a cross or T-intersection is the area between prolongations of the property lines abutting the intersecting roads.

Safety at Intersections

22. In rural areas some 20-30% of accidents occur at intersections. In urban areas the proportion is around 50-60% and about 50% of these are at minor-major intersections.

23. Because of lower traffic volumes, the accident rates (per veh) at rural intersections are usually higher than at urban intersections, even though the number of accidents are lower.

24. New intersections in rural areas should be T intersections or possibly, in some cases, roundabouts. Cross and Y intersections have a much poorer accident record.

Appendix 13E: Basic Left-Turn (BAL) Layouts at Rural Intersections

This appendix provides set-out details to cater for various design vehicles at rural BAL turn treatments.

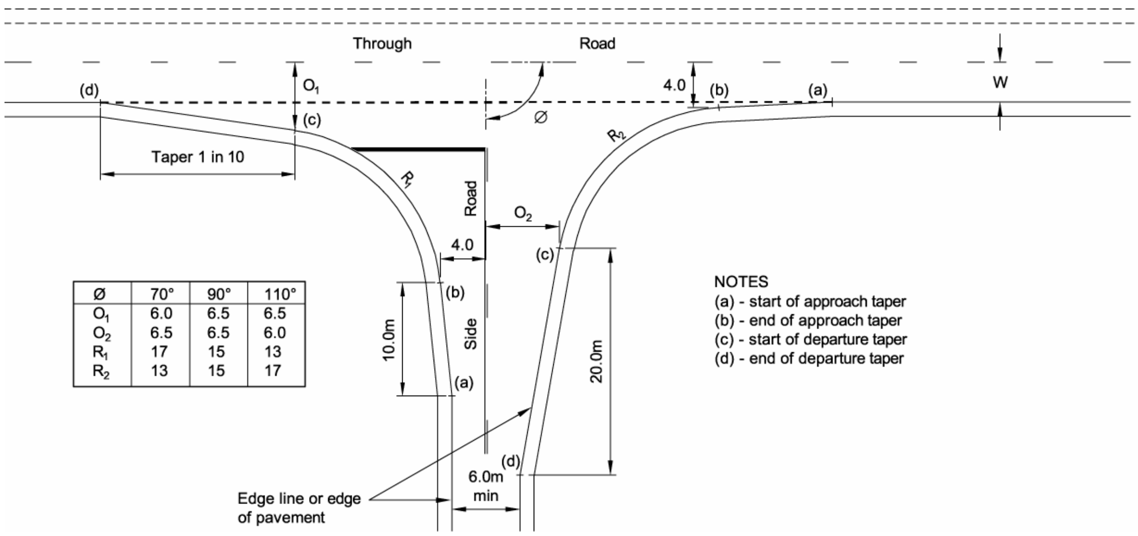

Figure 13E.1 Details of Type "BAL" Layout for Rural Sites to suit 19m Semi-trailer Operation

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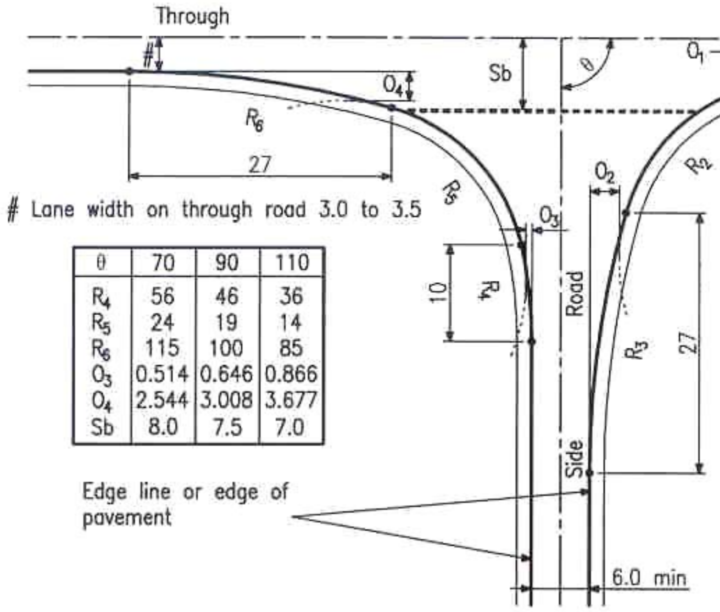
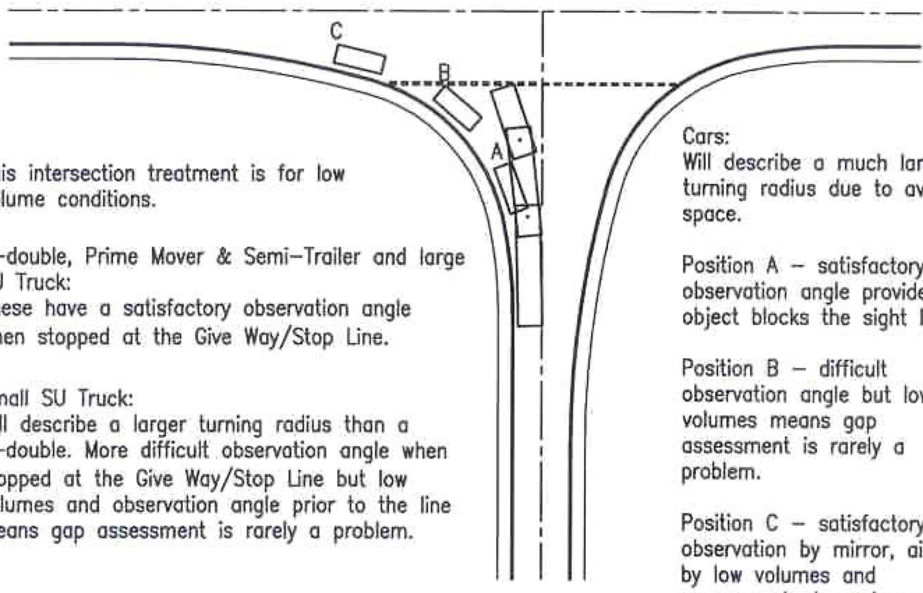


θ	70	90	110
R_4	56	46	36
R_5	24	19	14
R_6	115	100	85
O_3	0.514	0.646	0.866
O_4	2.544	3.008	3.677
Sb	8.0	7.5	7.0

θ	70	90	110
R_1	36	46	56
R_2	14	19	24
R_3	85	100	115
O_1	0.866	0.646	0.514
O_2	3.677	3.008	2.544

Setting Out Details



This intersection treatment is for low volume conditions.

B–double, Prime Mover & Semi–Trailer and large SU Truck:
These have a satisfactory observation angle when stopped at the Give Way/Stop Line.

Small SU Truck:
Will describe a larger turning radius than a B–double. More difficult observation angle when stopped at the Give Way/Stop Line but low volumes and observation angle prior to the line means gap assessment is rarely a problem.

Cars:
Will describe a much larger turning radius due to available space.

Position A – satisfactory observation angle provided no object blocks the sight line.

Position B – difficult observation angle but low volumes means gap assessment is rarely a problem.

Position C – satisfactory observation by mirror, aided by low volumes and assessment at previous positions.

Observation Angle Assumptions

Figure 13E.2 Details of Type "BAL" Layout for Rural Sites to suit B-double Operation

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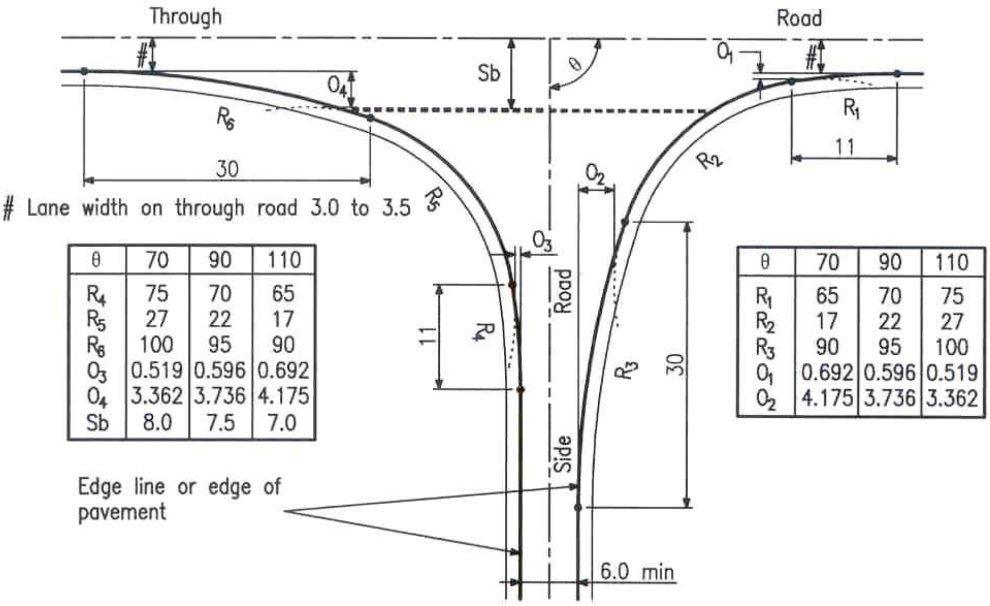
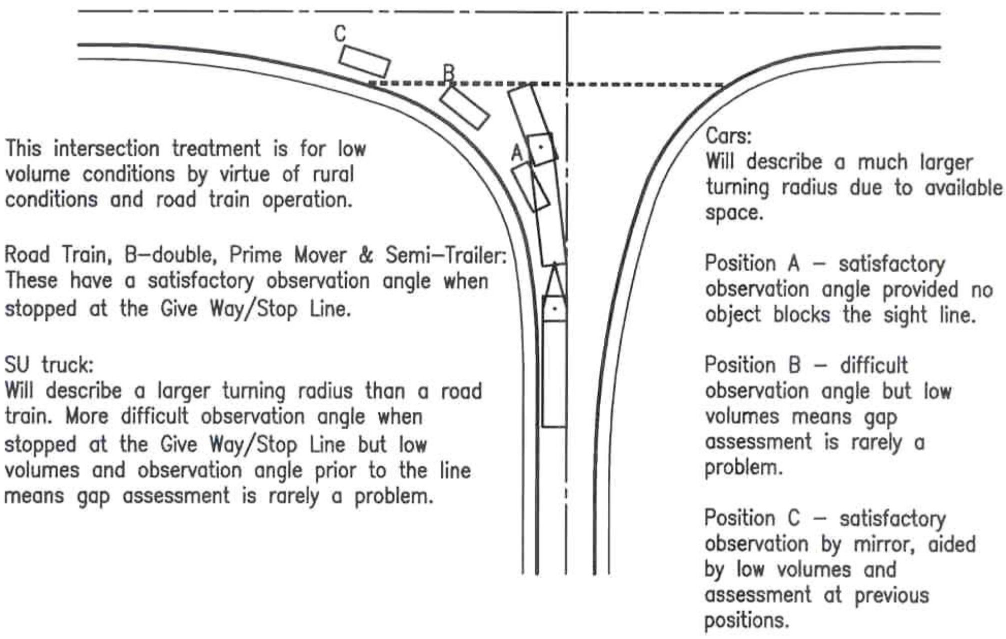


θ	70	90	110
R_4	75	70	65
R_5	27	22	17
R_6	100	95	90
O_3	0.519	0.596	0.692
O_4	3.362	3.736	4.175
Sb	8.0	7.5	7.0

θ	70	90	110
R_1	65	70	75
R_2	17	22	27
R_3	90	95	100
O_1	0.692	0.596	0.519
O_2	4.175	3.736	3.362

Setting Out Details



Observation Angle Assumptions

Figure 13E.3 Details of Type "BAL" Layout for Rural Sites to suit Type 1 (Double) Road Train Operation

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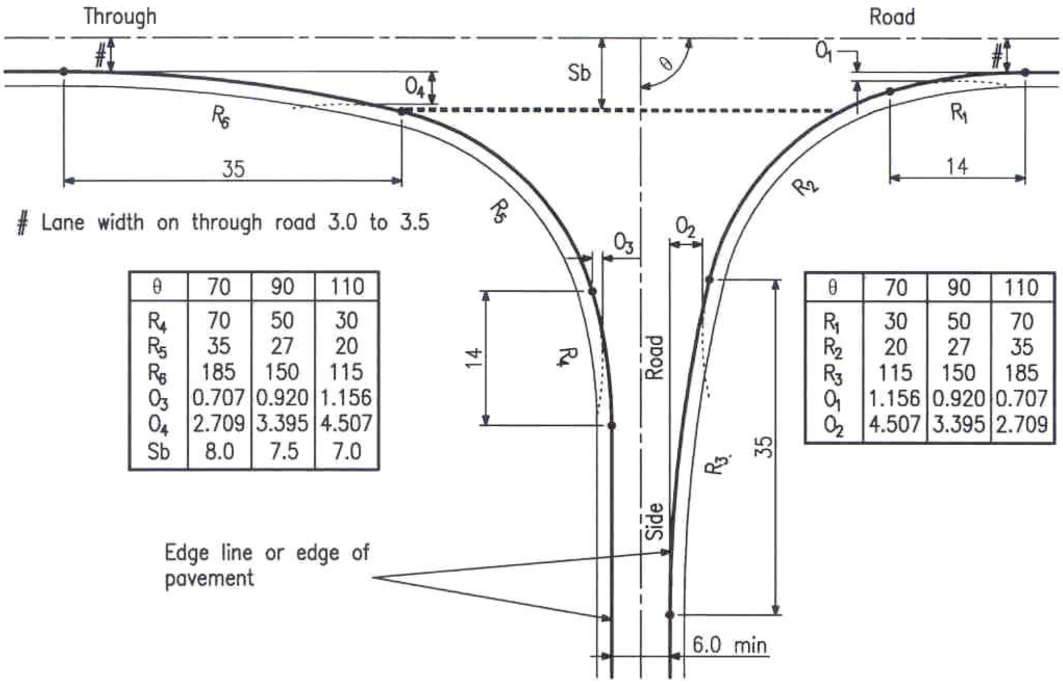
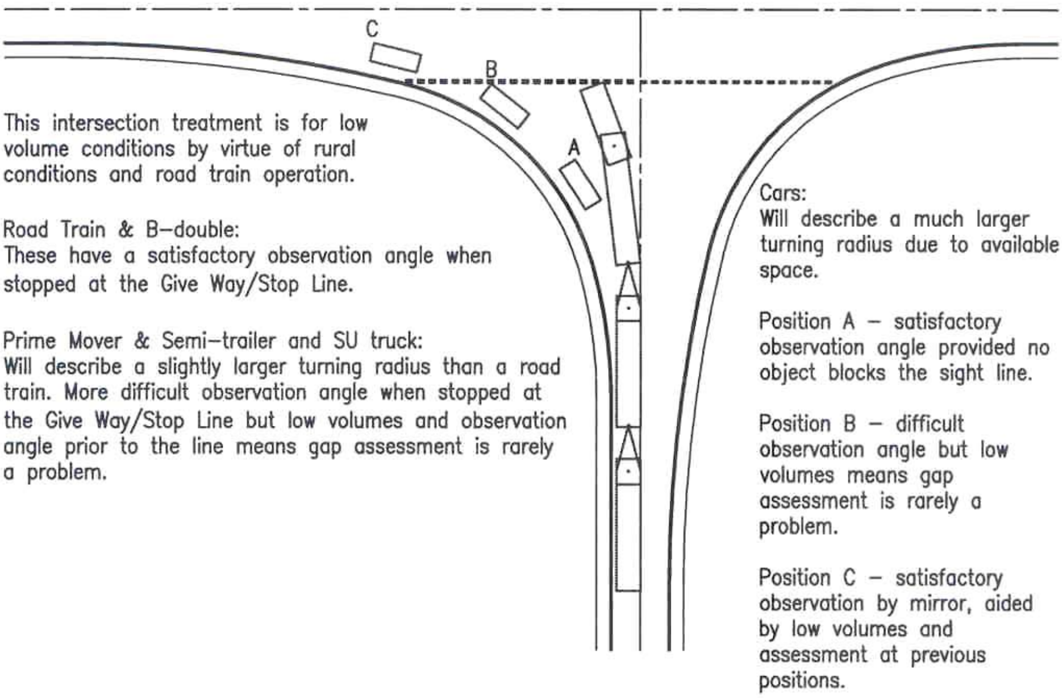


θ	70	90	110
R_4	70	50	30
R_5	35	27	20
R_6	185	150	115
O_3	0.707	0.920	1.156
O_4	2.709	3.395	4.507
Sb	8.0	7.5	7.0

θ	70	90	110
R_1	30	50	70
R_2	20	27	35
R_3	115	150	185
O_1	1.156	0.920	0.707
O_2	4.507	3.395	2.709

Setting Out Details



Observation Angle Assumptions

Figure 13E.4 Details of Type "BAL" Layout for Rural Sites to suit Type 2 (Triple) Road Train Operation

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Appendix 13F: Accident Types in Arndt (2004)

Arndt (2004) is a study of the effect of unsignalised intersection geometry on accident rates. This appendix lists the various accident types used in the study, which are referenced in the body of the chapter.

A total of 1091 accidents were recorded for the 206 intersections in the study. An analysis period of 5 years was selected for the majority of the intersections, with a period of 10 years for several lower volume intersections.

The sample generally includes accidents within 200m of each intersection but excludes accidents at nearby intersections or other features. The accidents were classified as shown in Table 13F.1. Diagrams of the High Frequency and Low Frequency Intersection accident types in Table 13F.1 are shown in Figure 13F.1 and Figure 13F.2 respectively.

'Intersection' accidents are those where the physical presence of the intersection directly influenced the accident. 'Through' accidents are those where the intersection did not directly influence the accident.

Table 13F.1 Accident Categories in Arndt (2004)

Broad Accident Category	Major Accident Type	General Accident Description	No.	Total
High Frequency Intersection Accidents	Angle-Minor	A vehicle on the minor road fails to give way and collides with a vehicle on the major road.	466	694
	Rear-End-Major	A through vehicle on the major road collides with a turning vehicle on the major road.	121	
	Angle-Major	A right-turning vehicle on the major road collides with an oncoming major road vehicle.	107	
Low Frequency Intersection Accidents	Rear-End-Minor	A vehicle on the minor road runs into another vehicle on the minor road.	27	109
	Single-Minor-Turn	A vehicle turning from the minor road loses control.	23	
	Single-Major-Turn	A vehicle turning from the major road loses control.	17	
	Incorrect Turn	A vehicle undertakes an incorrect turn at the intersection and collides with another vehicle.	17	
	Overtaking-Intersection	An overtaking major road vehicle collides with a right-turning major road vehicle.	13	
	Sideswipe-Major-Auxiliary	A major road vehicle moves from a deceleration lane and collides with another major road vehicle.	4	
	Other		8	
High Frequency Through Accidents	Single-Through	A through vehicle loses control and is involved in a single vehicle accident.	167	167
Low Frequency Through Accidents	Head-on	A through vehicle loses control and collides with an oncoming vehicle.	40	121
	Pedestrian	A vehicle collides with a pedestrian or cyclist crossing the road.	39	
	U-turn	A vehicle undertaking a U-turn (not at the intersection) collides with another vehicle.	33	
	Changed Lanes	An accident resulting from an unsafe lane change.	16	
	Single-Object	A vehicle collides with or avoids an object or animal.	16	
	Overtaking	An accident resulting from unsafe overtaking.	7	
	Other		10	
Total				1091

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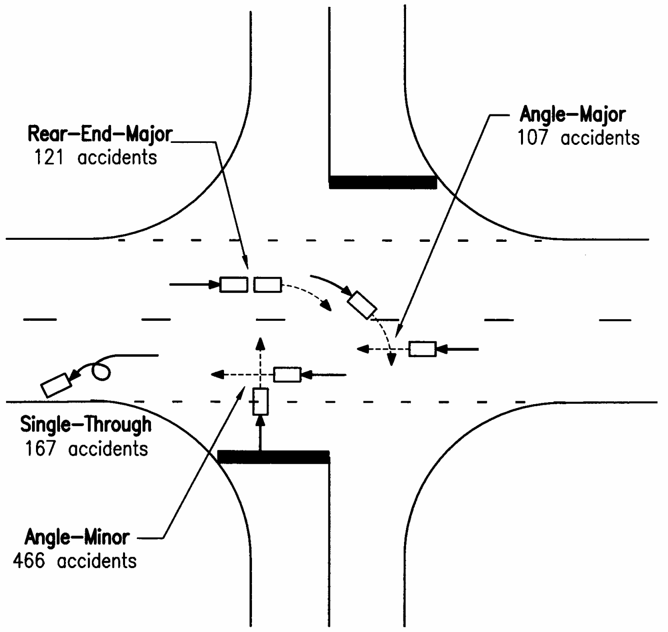

Figure 13F.1 High Frequency Intersection and Through Accident Types

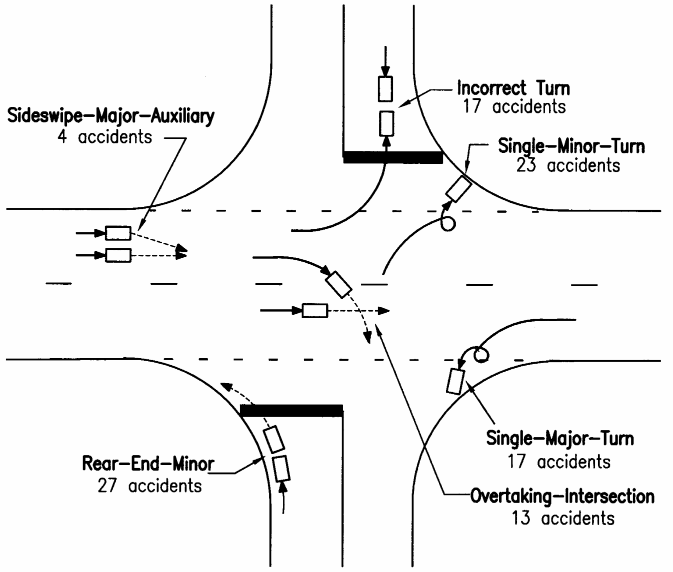

Figure 13F.2 Low Frequency Intersection Accident Types

Appendix 13G: Evaluation of Options

This appendix details a method of evaluating and selecting the most appropriate intersection layout.

13G.1 General

The resulting selection of feasible options identified in Sections 13.4.3 and 13.4.4 has to be evaluated to decide on the preferred option. This evaluation is required for all cases whether at new sites or for the upgrading of existing intersections, and must be undertaken in conjunction with the Public Consultation process. This consultation should identify all of the concerns of the people involved (all types of users and people living nearby) and establish a suitable evaluation framework (including weightings) acceptable to all stakeholders.

The evaluation matrix shown in Table 13G.7 provides an example of the issues to be covered and a possible weighting for the subjective items. A further process of applying suitable weighting to the cost factors is also required to provide an appropriately objective approach to the analysis.

The process may not identify a single preferred option, showing that there is more than one suitable solution. This allows other factors to be applied to decide on the preferred option. In addition, the process may identify or highlight elements of a particular solution which cause that option to perform badly, allowing refinement of that option for further consideration.

The process involves a direct comparison between a base case and each option, using selected performance criteria. These criteria will vary from site to site. The four criteria which should be addressed as a minimum requirement are:

- Safety;
- Delay;
- Site suitability; and
- Financial considerations.

Discussion of these four criteria follows in Sections 13G.2 to 13G.5.

Where appropriate, data from the existing intersection should be available to form the “base” case. Obviously, where there are site specific problems, details of performance in the areas which are the cause of concern must be known so that the complaint(s) can be positively addressed.

Modelling is an important part of the evaluation process. Modelling involves some type of artificial representation of the real world. It always involves assumptions which should be known and understood from the start. For example, many models of traffic assume a random distribution. In many networks this may not be the case as upstream events may tend to cause traffic to arrive in platoons.

In all areas examined, the sensitivity to changes in the numbers used in the analysis should be tested. For instance, in determining issues relating to delay, the sensitivity of the solution should be checked using both pessimistic and optimistic values of demand on each side of the design volume. Similarly, in checking economic performance, the consequences of variations in the estimated cost and in interest rates should be analysed.

The process also requires constant questioning of any constraints adopted to ensure they are real and remain appropriate to the solution selected.

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13G.2 Safety

Comparison of intersection proposals from the viewpoint of safety is difficult because it is difficult to isolate specific features which cause accidents. Whilst the principles of safe design given in Section 13.3.3 are helpful in the development of a scheme, they tend to favour the more elaborate (and expensive) arrangements.

The primary objective in considering safety is to reduce the community loss from crashes involving motor vehicles, pedestrians and cyclists. At existing intersections this means analysing the layout and form of control to determine whether there is an inherently high risk at the site being considered and an economically justifiable opportunity to reduce this risk. At new sites the analysis is to ensure that treatments that will create high risk situations are avoided.

An accident is a randomly occurring, rare (in statistical terms) event. As such, statistical principles can be applied. Many researchers have concluded that a "Poisson" type distribution for accidents at one location, during one period, is well based. Given that this is true, the mean of the statistical distribution of a number of accidents in a given time interval is:

- the time-based risk times the length of the interval; and
- the exposure-based risk times the exposure occurring at the location during the same time interval.

This means there are two problems when using existing accident data to define sites with a high risk. The first is that a few incidents may have biased the results and the risk at the location is, in reality, normal. The converse applies when a site with a high risk is masked by the short analysis period used. Existing accident data MUST be critically reviewed to ensure that accident experience exceeds that reasonably expected by a considerable margin.

Accident information on the whole of the QLD road network is gathered by Qld Transport, with a statement being published every calendar year.

Table 13G.1 shows the safety advantages of separating carriageways and controlling intersections (both at formal junctions and at points of access).

The effect of traffic volumes (or exposure) has been taken into account in this table by quoting the rates per 10^6 vehicle kilometres travelled (vkt). This process is known as normalising. To normalise exposure (E) at a cross intersection, Austroads (1988b) suggests the expression:

$$E = 2 \times \sqrt{\frac{(V_1 + V_3)}{2} \times \frac{(V_2 + V_4)}{2}} \qquad (13G.1)$$

where V_1 and V_3 are the two way traffic volumes (AADT) on opposite legs, as are V_2 and V_4 (V_4 is omitted for a three way junction). These volumes are defined in Figure 13G.1.

At a three way intersection (T or Y) the expression becomes:

$$E = 2 \times \sqrt{\frac{(V_1 + V_3 - V_2)}{2} \times V_2} \qquad (13G.2)$$

These expressions are considered to give a good mathematical representation of the conflicts on the various legs with differences in demand.

The rates obtained are not directly comparable with those given in Table 13.G.1. See Section 4.2.3 of Austroads (1988b) for further details.

Table 13G.1 Normalised Accident Rates for Various Types of Road (typical values)

Road Type	Accident Rates** per 100 Mvk		
	Fatal Accidents	**Serious Casualty Accidents***	**Total Accidents**
Urban			
Freeway	0.87	5.28	39.3
Divided Road (≥4L)	1.42	12.54	152.1
Undivided Road (≥4L)	3.07	25.23	256.7
Undivided Road (<4L)	3.96	18.55	131.9
Rural			
Freeway	1.26	2.29	21.9
Divided Road (≥4L)	1.59	8.05	32.2
Undivided Road (<4L)	2.11	11.52	49.3
* Serious Casualty Accidents are the sum of Fatal Accidents and serious Injury Accidents.			
** The rates are averages with a large standard deviation.			

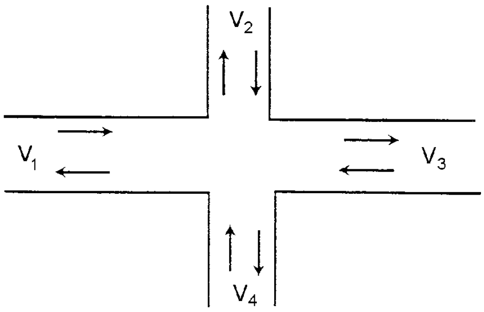

Figure 13G.1 Sketch Showing the Terms in the Exposure Calculation Formula

There have been many attempts to identify the relationship between accidents, exposure, intersection layout, and the form of control. A general relationship between exposure and layout at rural sites is given in Figure 13G.2. This shows the rapid decline in the performance of "Y" and "X" type layouts as exposure increases.

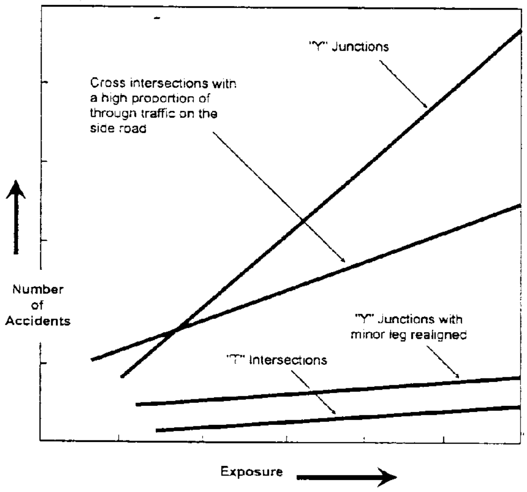

Figure 13G.2 Safety Performance of Various Rural Layouts (in terms of exposure): Source: O'Brien (1976)

In 1988, approximately 47% of reported accidents in QLD occurred at intersections. Of the accidents which caused one or more fatalities, 21% were at intersections, whilst intersections had 34% of the accidents which resulted in serious injury to vehicle users or pedestrians. Therefore, intersection accidents are generally less severe although over represented in numbers, than between intersection accidents. However there is a great deal of scope for safety improvement.

Table 13G.2 gives brief details. From this table the following features can be seen:

- multiple vehicle accidents dominate at intersections;
- whilst the number of day time accidents is high at intersections, nearly 30% of

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the accidents occur at night. When the lower exposure is taken into account the night time intersection accident rate is probably poor; and

- during poor conditions, the percentage of accidents at intersections is similar to the rest of the network. However, when account is taken of the number of intersections compared to the length of the rest of the network, it is likely that intersections again perform relatively poorly.

Table 13G.2 Subdivison of reported accidents according to various criteria

	All Locations (%)	Non-Inter-sections (%)	Inter-sections (%)
Single Vehicle Accidents	33	49	15
Multiple Vehicle Accidents	67	51	85
Wet Accidents	18	20	16
Dry Accidents	82	80	84
Night-time Accidents	29	32	26
Day-time Accidents	71	68	74

Notes:
1. Each item shown is independent of the others given.
2. An intersection accident is where the first impact occurred at, or within 10m of an intersection. This definition reduces the number of accidents reported as “intersection accidents”, and means that the figures quoted are conservative.

This figure shows that some layouts perform badly as traffic volumes increase. This graph is roughly to scale. The form of control at the sites investigated was NOT traffic signals or a roundabout.

The road user movements (RUM) which dominate intersection accidents are those involving cross movements, right-hand turn movements, and rear-end collisions with a vehicle turning right. Table 13G.3 gives details. From this table it is noticeable that where the relative speed at the point of impact is low, the probability of serious/fatal accidents is also low; conversely, where the relative speed is high there is a higher probability that a fatality will occur.

A discussion on relative speed and methods for minimising it is given in Appendix 13B.

A suggested method for comparing layouts, and forms of control, from the viewpoint of safety is to carry out a comparison in four areas as follows:

- Item S1

(a) establish the average accident cost from Queensland Transport data.

(b) determine the exposure using formulae 13G.1 and 13G.2 above, and estimate the cost of accidents for the various layouts, over the design period, using the rates from Table 13G.4.

This result is entered into the evaluation matrix (See Table 13G.7).

- Item S2

In an attempt to address the common crash types (cross, right hand turn and rear end) each layout is to be critically reviewed with respect to these conflicts for each leg and given a score on a scale of ten for each (ie. for an intersection with three legs there would be 12 rankings). These are shown in the matrix under item S2.

- Item S3

Where appropriate, a third safety check relates to pedestrians and cyclists. All options are to ensure that these users are clearly visible, particularly at night, and relative speeds at points of conflict between these users and motor vehicles are minimised. Again, a score out of 10 is considered appropriate. Obviously, where pedestrians or cyclists are not an issue this item is left blank.

- Item S4

The final safety rating is to rank schemes in terms of network safety. The objective is to penalise arrangements which merely divert traffic to other intersections in the network (with a net loss in system safety), and reward those which promote safe driver behaviour. A rating out of 10 is appropriate.

These safety points are summed and the final score adjusted to be out of 40.

The following general comments on safety should be noted:

- arrangements with the greatest potential to reduce delay and operating costs are often those with the greatest cost effectiveness in accident reduction. (Clark and Ogden, 1973).
- multi-leg intersections should be avoided.
- on rural roads staggered "T" intersections are preferred over cross intersections.
- some improvements to safety at a specific site may result in a decline in network safety performance if traffic diverts.
- accident severity increases where speeds are higher.
- "Y" junctions have poor safety records.

Traffic signals change the accident patterns; they may reduce the number of accidents at sites which already have a poor record but may increase the number of accidents at sites where current rates are low. (AIR 394-10 (Cairney, 1988).

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Table 13G.3 Reported Accident Statistics (Source: RTA , NSW)

DETAILS OF REPORTED ACCIDENTS, showing the total number of accidents and details of those where serious injuries or fatalities occurred. (58/625 means 58 fatal accidents 625 serious injury accidents; the number under is the sum of both). There may be more than one fatality, or person injured per accident. Note: Due to the definition used for an intersection accident, the number of rear end collisions is probably understated.

ROAD USER MOVEMENT (RUM)	TOTAL ACCIDENTS	FATAL AND SERIOUS INJURY ACCIDENTS								
		METROPOLITAN			COUNTRY URBAN			RURAL		
		DAY	NIGHT	TOTAL	DAY	NIGHT	TOTAL	DAY	NIGHT	TOTAL
1. CROSS	20,362	58/625 683	27/355 382	1,065	14/386 400	8/150 158	558	8/37 45	1/8 9	54
2. RIGHT/FAR	1,504	2/25 27	1/28 29	56	0/17 17	0/6 6	23	2/8 10	0/3 3	13
3. LEFT/FAR	643	0/14 14	0/9 9	23	0/4 4	0/4 4	8	0/2 2	0/0 0	2
4. RIGHT/NEAR	12,040	21/382 403	11/178 189	592	7/134 141	3/54 57	198	7/62 69	0/20 20	89
5. LEFT/NEAR	1,302	3/28 31	1/9 10	41	0/13 13	0/1 1	14	0/3 3	0/0 0	3
6. RIGHT/THROUGH	18,574	29/595 624	25/408 433	1,057	16/169 185	4/82 86	271	5/49 54	1/11 12	66
7 LEFT/REAR	1,283	0/22 22	0/16 16	38	0/7 7	0/1 1	8	0/3 3	0/1 1	4
8. RIGHT/REAR	6,255	3/100 103	2/36 38	141	2/80 82	2/23 25	107	2/34 36	2/12 14	50
9. RIGHT/SIDE SWIPE	1,103	0/22 22	0/12 12	34	1/14 15	0/6 6	21	4/12 16	0/0 0	16
10. LEFT/SIDE SWIPE	1,366	3/37 40	0/9 9	49	0/13 13	1/1 2	15	0/2 2	0/2 2	4
11. U TURN	3,739	10/112 122	3/49 52	174	2/77 79	2/28 30	109	0/19 19	0/19 19	38

Table 13G.4 Casualty Accident Rates per Million Vehicles Entering for Various Intersection Layouts and Forms of Control

(Source: Ogden and Bennett, 1989 and RACV)

		Basic		Auxiliary		Channelised		
		R/S	TS	R/S	TS	R/S	TS	R'bout
U	T	0.15	0.14	0.14	0.13	0.12	0.11	
U	Y							
U	X	0.24	0.17	0.22(8)	0.15	0.19	0.14	0.09
U	Multi		0.32					
R	T	0.33	0.21	(0.30)	(0.20)	(0.27)	(0.20)(7)	(0.14)(6)
R	Y							
R(8)	X	0.52	0.25	(0.47)(8)	(0.20)	(0.43)	(0.33)(7)	(0.24)(6)
R	X	0.35	0.25	0.29(3)	(0.20)	(0.43)	0.25	(0.24)(6)

NOTES:

(1) These rates are preliminary and will be revised; research is under way to provide the missing numbers.

(2) R = rural, U = urban, R / S = regulations and / or signs, TS = traffic signals.

(3) Staggered T layout.

(4) Numbers given are averages which have a large standard deviation.

(5) Numbers bracketed are estimates.

(6) Not recommended unless speed control assured.

(7) Usually not Practical.

(8) Not staggered.

13G.3 Delay

At an intersection, delay has three basic components:

(a) reduction in speed and/or additional distance travelled due to the layout;

(b) reduction in speed due to the volume of traffic travelling along the same route; and

(c) the availability of gaps at points of conflict with users on other legs.

In computer programs these are known as geometric, traffic and queue delays respectively. Delay due to (a) is sensibly constant; delays due to (b) and (c) vary with volume.

Evaluation of schemes on the basis of delay involves both objective and subjective assessment. In the matrix (Table 13G.7), it has three items as follows:

- Item D1

The delay caused by (a) can be estimated for all user movements, and the cost calculated over the life of the project using the rates given in the Cost Benefit Cost Analysis Manual for Road Infrastructure Investment. In this calculation it is assumed that traffic signals (where encountered) are at green.

- Item D2

The delay due to (b) and (c) relates to the capacity provided for each movement. A typical relationship between delay and volume of entering traffic is given in Figure 13G.3. The rapid increase in delay at certain volumes should be noted. Such volumes define the nominal capacity for the intersection with Table 13G.5 giving some typical figures. Table 13G.6 shows the resulting service volumes on an approach leg that can be supported.

Preliminary analysis of the likely delay at an intersection can be determined using "Y" values. Details of this procedure are given in Appendix 13B. Detailed calculations are best carried out using computer programs such as aaSIDRA. Brief details of these procedures are given in Appendix 13C.

It is not uncommon for the capacity of an intersection to be substantially reduced by the controls operating at nearby intersections. Checks to ensure that the network performance is maintained can be important. Depending on the function of the intersection, priority may be given to various movements during certain times of the day. On local, or collector roads the capacity may be deliberately restricted for environmental reasons.

The delay due to (b) and (c) is estimated using the design volumes for the am peak, pm peak, interpeak, and other periods where volumes cause significant delay. The output is given in hours per hour. The cost is calculated for each period over the design life using the rates given in Cost Benefit Cost Analysis Manual for Road Infrastructure Investment.

- Item D3

The evaluation matrix not only includes the costs of delay determined above, but also a subjective assessment of delay for legs where the demand is small. This is to consider how long this small group of users will have to wait. For minor vehicle movements at traffic signals a period of approximately 120 seconds is the maximum time usually adopted for servicing such a call. A shorter time (say 60 seconds) might apply at unsignalised sites. The delay to minor pedestrian and bicycle movements should also be of this order. A scoring of options on a scale of ten will address this issue satisfactorily.

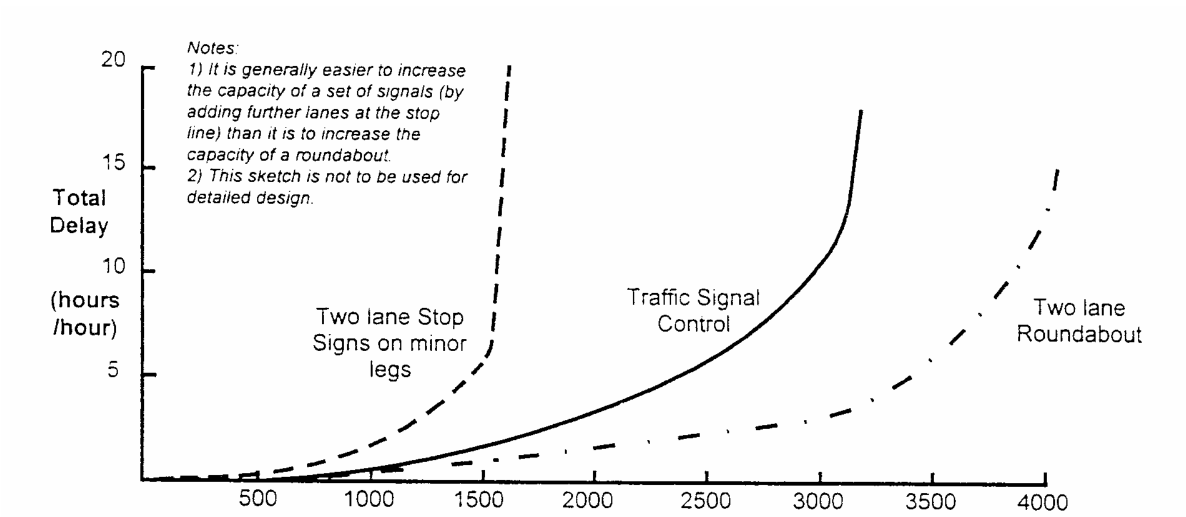

Figure 13G.3 Typical Relationship between the Volume of Traffic Entering an Intersection and the Total Delay for all Users of the Intersection

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Table 13G.5 Typical Limits of Intersection Capacity (veh/h) (four way intersection with equal demand for all movements)

Approach Width	Type of Control		
	No Signals (a)	Round-about (b)	Signals (c)
1 lane	1500	2600	1500
2 lanes	1500	4560	3000
3 lanes	NA	6000	4500
4 lanes	NA	N/A	6000

Notes:
(a) Based on Practical Absorption Capacities of Unsignalised Intersections.
(b) Based on Gap Acceptance Criteria.
(c) Based on Four Split Phases (120s cycle).

13G.4 Site Suitability

A site specific check list should be developed. Options should be scored in terms of the items on this list and may include:

- expectations of drivers, based on previous intersections encountered;
- types of vehicle expected, and approach speed;
- public transport operations;
- service provided to pedestrians and pedal cyclists;
- adjacent land use (proximity to schools, homes for elderly citizens, community facilities, etc.) and points of access (driveways, lanes, etc.);
- parking;
- compatibility with adjacent intersections and overall traffic management strategy;
- available road reserve width;
- local amenity (noise, fumes, etc.);
- effect of possible traffic growth and redistribution;
- street scape effects;
- topographic suitability;
- method of traffic control proposed; and
- community acceptance of the option.

The choice of items to be evaluated should be made in conjunction with the Public Consultation process (see 13G.1). Relative weightings for the items selected should also be assigned in this process.

Table 13G.6 Typical Maximum Service Volumes on the Major Leg of an Intersection

Type of Road	OMC* (veh/h)
Median or inner lane - divided road - undivided	 1000 900
Outer or kerb lane - adjacent parking lane - clearway conditions - occasional parked vehicles	 900 900 600
4 lane undivided	1500
4 lane undivided - clearway conditions	1800
4 lane divided - clearway conditions	1900
6 lane undivided	2400
6 lane divided - clearway conditions	2900

OMC* - One-way Mid-block Capacity
These are volumes which can be handled by most intersection designs on the basis that they are the major movement and approximately 35 minutes of effective green time can be provided each hour.

13G.5 Financial Considerations

The cost of constructing and operating an intersection needs careful analysis, particularly in urban areas. In addition to the total cost of the option being considered, careful attention has to be paid to the elements making up this cost and their sensitivity to changes in the layout or location.

Items that can be very expensive and which are often sensitive to relatively small changes in design are the alterations to Public Utility Plant. Designers should assess the impact of the design on these services and identify means of reducing that impact and consequently, cost.

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The ease of construction and/or the ease with which the construction can be staged over time to meet the expected growth in traffic volumes often determines the viability of an option. Designers should prepare the construction staging and sequencing plans for the options and estimate the cost of each stage. Inability to stage the construction may be a significant factor in deciding to accept or reject an option. This will be important if the available funding falls short of the whole amount required to build the complete design.

If future expansion of the intersection is expected, the costs of doing this have to be assessed and the options compared on this basis. Easily expanded options will score better in the evaluation.

The following factors are to be evaluated using a score out of 10 and the combination of these factors summed to reflect an overall weighting of 20%:

- Total cost;
- Benefit Cost Ratio;
- Cost of PUP adjustments;
- Maintenance cost;
- Operating cost;
- Constructability;
- Capacity to stage construct; and
- Cost of future expansion.

Omit any items not applicable to the project being analysed.

Details of the likely funding must also be known as some solutions may have to be ruled out on the grounds that they cannot be funded.

13G.6 Summary

The evaluation matrix (Table 13G.7) can now be completed and the final score computed. This overall score should only be used as a guide to the final decision since subjective issues may override a strictly numerical approach to the decision.

The final report on the process should address all of the issues described in this section and must include all options considered unsuitable. Reasons for all recommendations must be given.

Table 13G.7 Suggested Matrix used to Compare Options

AREA	DETAILS	OPTIONS			
		OPTION 1	OPTION 2	OPTION 3	OPTION 4
LAYOUT	Layout Type				
	Number of Legs				
	Form of Control				
(1) Safety performance	(S1) Estimated accident cost[2]	$	$	$	$
	(S2) Rating of option with regards to	Traffic from leg	Traffic from leg	Traffic from leg	Traffic from leg
		1 2 3 4 5	1 2 3 4 5	1 2 3 4 5	1 2 3 4 5
	Cross RUM (No 1)[5] (10 each)				
	Right / Near RUM (No.4)[5] (10 each)				
	Right/Through RUM (No.6)[5] (10 ea)				
	Right / Rear RUM (No.8)[5] (10 each)				
	(S3) Pedestrian Safety				
	Bicycle Safety				
	(S4) Network Safety				
	Total [ ]				
	Pro rata rating out of 40				
(2) Delay performance	(D1) Cost[2] of geometric delay	$	$	$	$
	(D2) Cost[2] of traffic and queue delay	$	$	$	$
	Total delay cost	$	$	$	$
	(D3) Rating of individual user delay	Traffic from leg	Traffic from leg	Traffic from leg	Traffic from leg
	(10 each)	1 2 3 4 5	1 2 3 4 5	1 2 3 4 5	1 2 3 4 5
	Isolated vehicles				
	Pedestrians				
	Bicyclists				
	Total [ ]				
	Pro rata rating out of 40				
(3) Site suitability	Rating for Issue No:	Traffic from leg	Traffic from leg	Traffic from leg	Traffic from leg
		1 2 3 4 5	1 2 3 4 5	1 2 3 4 5	1 2 3 4 5
	1				
	2				
	3				
	4				
	5				
	Total [ ]				
	Pro rata rating out of 20				
(4) Financial	(F1) Cost of construction[3]	$	$	$	$
	Cost of land	$	$	$	$
	Cost of utility adjustments	$	$	$	$
	Cost[2] of future expansion	$	$	$	$
	(F2) Cost[2] of maintenance	$	$	$	$
	(F3) Total capital cost	$	$	$	$
(5) Other criteria					
(6) Summary	Economic BCR[4] at given rate of funding				
	Total subjective score out of 100				

Notes for the use of Table 13.G7

(1) Ratings are out of 10 with each considered subjectively. Scores for the same item across options should reflect the relative performance of each. It is suggested that excellent solutions are given 0, good solutions 2, fair solutions 5, poor solutions 7 and very poor 10.

(2) All costs are discounted to "present worth"

(3) When construction must be financed over several years (due to budget or other factors) the deferral of benefits must be taken into account.

(4) Select the option with the highest estimates accident costs as the base case (this could be the current or "do nothing option) to calculate the BCR.

(5) The numbers are from Table 13G.3.

(6) Add additional columns (as necessary) for further options.

(7) Where an issue is not appropriated at a site, the matrix is left bank.